RDG
2023

GHANA ROAD DESIGN GUIDE 2023

MINISTRY OF ROADS AND HIGHWAYS

JICA
JAPAN INTERNATIONAL COOPERATION AGENCY

MESSAGE FROM THE MINISTER OF ROADS AND HIGHWAYS

The Government of Ghana (GoG) has embarked on various development strategies to solve the challenges of unemployment. These include the Ghana Poverty Reduction Strategy, the Ghana Shared Growth Development Agenda (GSGDA) and our current Medium-Term Development Agenda which is in line with the Sustainable Development Goals (SDG).

To solve the challenges of poverty, the government has prioritized infrastructure development, human development and employment among others in its development agenda. The Ministry of Roads and Highways is a vital ministry in achieving Ghana's Developmental Agenda.

It is with this motivation that the Ministry of Roads and Highways (MRH) agreed to collaborate with the Japan International Cooperation Agency (JICA) under the "PROJECT ON CAPACITY BUILDING FOR ROAD AND BRIDGE MANAGEMENT (CBRB)" with the intent to establish uniform procedures and standards for planning and designing of roads in Ghana.

The maiden revised edition of the Ghana Highway Authority Road Design Guide (GHA RDG), 1991 addresses the original edition's limitations and brings it up to date with international best practices. The revised edition has harmonised the design standards for all classes of roads in Ghana with a focus on enhancing safety and efficiency. This harmonization can simplify the design process, ensure consistency, and help achieve better road infrastructure quality and performance outcomes.

The Ministry stands by its vision to provide and maintain an integrated, cost-effective, safe and sustainable road transport network responsive to the needs of users, supporting growth and poverty reduction hence will not let this Design Guide waste on the shelves.

I would like on behalf of the Ministry of Roads and Highways to thank JICA and all organisations and technical staff for their relentless efforts in making this Guide a reality.

Hon. Kwasi Amoako Attah

(Minister for Roads and Highways)

PREFACE

The Ghana Road Design Guide (RDG) hereinafter called "Guide" has been developed by the Ministry of Roads and Highways (MRH) and its Agencies [Ghana Highway Authority (GHA), Department of Feeder Roads (DFR), Department of Urban Roads (DUR) and Koforidua Training Centre (KTC)] in collaboration with Japan International Cooperation Agency (JICA) under "THE PROJECT ON CAPACITY BUILDING FOR ROAD AND BRIDGE MANAGEMENT (CBRB)" with the intent to establish uniform procedures and standards for planning and designing of roads in Ghana. The technical standards prescribed in this Guide are applicable to all classes of roads defined in this Guide.

It is the first revised edition of the Ghana Highway Authority Road Design Guide (GHA RDG), March 1991.

The revision of the GHA RDG, 1991, has become necessary in view of its limitation in the design of grade separated intersections and tunnels, culverts and bridge hydraulics, traffic calming measures, Intelligent Transportation System (ITS) amongst others. These limitations resulted in road engineers and other road-related professionals resorting to the use of several other manuals as references for road designs in Ghana. Thus, making it difficult to ensure uniformity of designs across the road network.

This revision has been carried out taking cognisance of best international practices, and feedback from seasoned road engineers, road users and other road-related professional bodies. To enhance safety and efficiency in the design process, road safety audit and other road safety tools have been included. In addition, to standardize road surveying in the road industry, road surveying procedures and requirements have been captured.

The revision was carried out through a series of workshops from March 2021 to September 2023 involving the following Counterpart Personnel (C/P) from GHA, DFR, DUR and KTC, JICA Expert Team (JET) from Eight-Japan Engineering Consultants Inc. (EJEC) and CTI Engineering International Co. Ltd (CTII) and Local Staff on the CBRB Project.

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It is recommended that this Guide is revised every ten (10) years.

Proposed future amendments to the Guide for consideration should be forwarded to the Chief Director, Ministry of Roads and Highways, P. O. Box M57, Ministries, Accra, Ghana or info@mrh.gov.gh.

ACKNOWLEDGEMENT

The Ghana Road Design Guide (RDG) is a comprehensive guide provided with technical standards, guidelines and recommendations for road infrastructure design in Ghana. It is a revised edition of the Ghana Highway Authority Road Design Guide (GHA RDG), March 1991 edition.

We gratefully recognize the contribution of Ing. Dr. Abass M. Awolu, Chief Director, MRH; Ing. Edmund Offei-Annor, former Chief Director, MRH; Ing. Mrs. Rita Ohene Sarfo, Director, Policy and Planning, MRH; and Ing. Mrs. Efua Effah, Director, Public Investment Unit, MRH, for their guidance and support during the preparation of this Guide.

We also greatly appreciate the contribution of Ing. Paul Y. A. P. Duah, Director of Survey and Design at GHA, whose MSc thesis on the Revision of the GHA Road Design Guide 1991 served as the blueprint for the organization of this Guide.

We acknowledge the contribution of professionals who gave comments and suggestions during the preparation of this Guide. The comprehensive input from contributors within MRH, JICA, GHA, DFR, DUR, KTC, Private Practitioners, retired seasoned Engineers and other stakeholders during the Technical Seminars and Joint Coordinating Committee (JCC) meetings is highly appreciated.

The immense assistance and support provided by both the technical and non-technical staff at the MRH JICA office is greatly appreciated.

The Guide draws extensively on the experience of best practice road design guides/manuals around the world. Key sources are listed below, and the authors' contributions are highly acknowledged.

- Ghana Highway Authority Road Design Guide, 1991.
- Japanese Road Structure Ordinance, April 2021.
- Explanation and Operation of the Road Structure Ordinance.
- Geometric Design Manual of Uganda, (2005).
- A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.
- Austroads, 2021.
- Asian Highway Design Standard for Road Safety, 2017.

The warm and gracious support of the staff of KTC during the numerous workshops on the revision of the GHA RDG is also appreciated.

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LIST OF ABBREVIATIONS

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
ADT	Average Daily Traffic
AEP	Annual Exceedance Probability
AGPS	Assisted Global Positioning System
ARI	Average Recurrence Interval
ASCII	American Standard Code for Information Interchange
ASI	Accident Severity Index
ATS	Average Travel Speed
AWSC	All-Way Stop Control
B/C	Benefit/Cost Ratio
BC	Beginning Of Curve
BEAT-SSCC	Bursting Energy Absorbing Terminal–Single Sided Crash Cushion
BFFS	Base Free-Flow Speed
BRT	Bus Rapid Transit
BS	British Standard
BSEN	British Standard European Norm
C/P	Counterpart Personnel
CAD	Computer-Aided Design
CALC	Calculate
CBD	Central Business Districts
CBR	California Bearing Ratio
CBRB	Capacity Building for Road and Bridge
CCT	CASS Cable Terminal
CDOT	Colorado Department of Transportation
CDR	Collector Distributor Road
CH	Chainage
CIE	Commission Internationale De L’Eclairage
CIL	Coefficient Of Luminous Intensity
CN	Curve Number
COORD	Coordinate
CORS	Continuously Operating Reference Stations
CSD	Context-Sensitive Design
CT	Curve to Tangent
CTII	CTI Engineering International Company Limited
CWZ	Construction Work Zone
DDI	Diverging Diamond Interchanges
DEM	Digital Elevation Model
DFR	Department of Feeder Roads
DHV	Design Hour Volume
DOT	Department of Transportation
DSM	Digital Surface Model
DTM	Digital Terrain Model

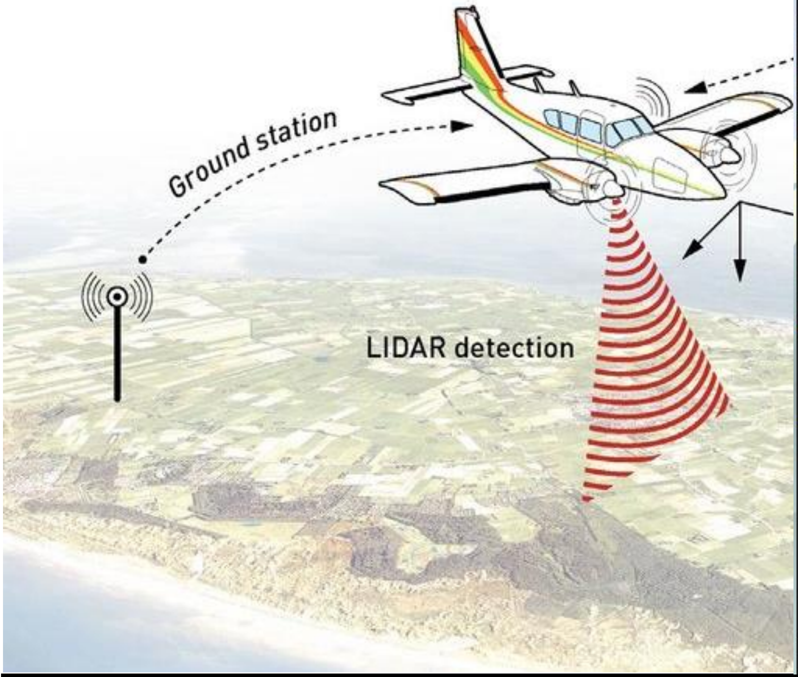
DUR	Department of Urban Roads
EBL	Eastbound Left-Turn
EBR	Eastbound Right-Turn
EBT	Eastbound Through
EBU	Eastbound U-Turn
EC	End of Curve
ECDP	Eastern Corridor Development Project
ECG	Electricity Company of Ghana
ECOWAS	Economic Community of West African States
EDD	Extended Design Domain
EDM	Electronic Distance Meter
EJEC	Eight-Japan Engineering Consultants
EN	Entry
ESC	Escape
ESIA	Environmental and Social Impact Assessments
EU	European Union
EX	Exit
FFS	Free-Flow Speed
FHWA	Federal Highway Administration
FV	Future Value
GHA	Ghana Highway Authority
GIS	Geographic Information System
GLONASS	Globalnaya Navigatsionnaya Sputnikovaya Sistema
GNGS	Ghana National Geodetic System
GNSS	Global Navigation Satellite System
GoG	Government of Ghana
GPS	Global Positioning System
GSGDA	Ghana Shared Growth Development Agenda
HAR	Horizontal Angle
HCM	Highway Capacity Manual
HDS	Hydraulic Design Series
HEC	Hydrologic Engineering Centre
HID	High-Intensity Discharge
HOV	High-Occupancy Vehicle
HSWIM	High-Speed Weigh-In-Motion
HV	Hourly Volume
HVIS	Heavy Vehicle Interception Site
HW	Head Water
IEEE	Institute of Electrical and Electronics Engineers
IES	Illuminating Engineering Society
IMU	Inertia Measurement Unit
IR	Infrared Radiation
IRE	Institute of Radio Engineers
IRR	Internal Rate of Return

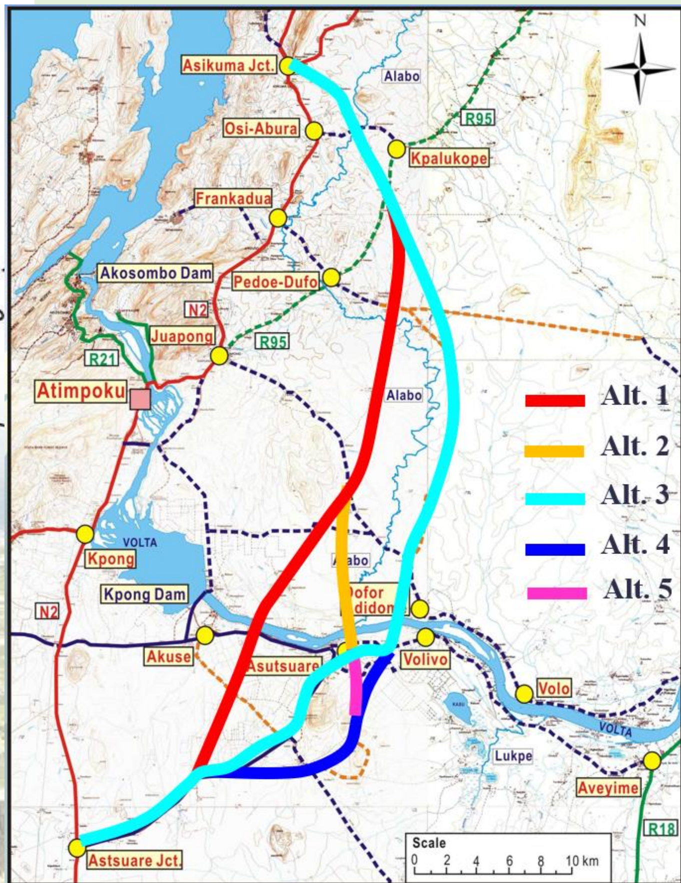
ISA	Intelligent Speed Adaptation
ISBN	International Standard Book Number
ITS	Intelligent Transportation System
IU	In-Car Unit
JCC	Joint Coordinating Committee
JET	JICA Expert Team
JICA	Japan International Cooperation Agency
JPEG	Joint Photographic Experts Group
KTC	Koforidua Training Centre
LAS	LiDAR Aerial Survey
LCC	Lay-by Control Centre
LCV	Light Commercial Vehicle
LED	Light Emitting Diode
LiDAR	Light Detection and Ranging
LMC	Low-Maintenance Cartridge
LOS	Level of Service
LRFD	Load and Resistance Factor Design
LSWIM	Low-Speed Weigh-In-Motion
LTZ	Limited Traffic Zone
MAAP	Modular Accident Analysis Program
MAS	Median Attenuator System
MASH	Manual for Assessing Safety Hardware
MB	Mega Byte
MCV	Multi-Combination Vehicle
MELT	Modified Eccentric Loader Terminal
MEM	Memory
MLPU	Machine Learning Processing Unit
MPEG	Moving Picture Experts Group
MRH	Ministry of Roads and Highways
MSL	Mean Sea Level
MT	Median Terminal
MTLS	Mobile Terrestrial Laser Scanning
MUTCD	Manual on Uniform Traffic Control Devices
NA	Not Applicable
NBU	Northbound U-Turn
NCHRP	National Cooperative Highway Research Program
NCIAS	Narrow Connecticut Impact Attenuation System
NEMA	National Electrical Manufacturers Association
NMT	Non-Motorized Transport
NPV	Net Present Value
NRCS	Natural Resources Conservation Service
NRSS	National Road Safety Strategy
NSW	New South Wales
OBS	Observation

OSOM	Oversize and Over-Mass
PC	Point of Curvature
PCMS	Portable Changeable Message Signs
PDO	Property-Damage Only
PDOP	Positional Dilution of Precision
PFFS	Per Cent Free-Flow Speed
PHF	Peak Hour Factor
PI	Point of Intersection
PIARC	Permanent International Association of Road Congresses
PIN	Personal Identification Number
PMF	Probable Maximum Flood
POC	Point On Curve
PRF	Pulse Repetition Frequency
PT	Point of Tangency
PTSF	Per Cent Time-Spent-Following
PV	Present Value
PVC	Point of Vertical Curvature
PVT	Point of Vertical Tangency
RAM	Random Access Memory
RDG	Road Design Guide
RGB	Red, Green and Blue
RIRO	Right-In Right-Out
ROW	Right Of Way
RSA	Road Safety Audit
RSI	Road Safety Inspection
RSIA	Road Safety Impact Assessment
RTPI	Real Time Information to Public Transport Passengers
RUC	Road User Charging
SBI	Straight Back-In
SBL	Southbound Left-Turn
SBT	Southbound Through
SBU	Southbound U-Turn
SCAT	Sydney Co-Ordinated Adaptive Traffic
SCI	Smart Cushion Innovations
SCOOT	Split, Cycle and Offset Optimisation Technique
SCS	Soil Conservation Service
SD	Secure Digital
SDG	Sustainable Development Goals
SFR	Saturation Flow Rate
SMS	Short Message Service
SORTA	Southwest Ohio Regional Transit Authority
SPDI	Single-Point Diamond Interchanges
SPUI	Single Point Urban Interchange
SSD	Stopping Sight Distance

SUV	Sport Utility Vehicle
TAA	Technical Approval Authority
TBM	Temporary Benchmark
TC	Tangent to Curve
TCC	Traffic Control Centre
TCD	Traffic Control Devices
TDOT	Tennessee Department of Transportation
TMA	Truck- and Trailer-Mounted Attenuators
TMP	Traffic Management Plan
TRACC	Trinity Attenuating Crash Cushion
TRB	Transportation Research Board
TRL	Technology Readiness Level
TV	Television
TW	Tail Water
TWSC	Two-Way Stop Control
UAV	Unmanned Aerial Vehicle
UK	United Kingdom
UN	United Nations
US	United States
USA	United States of America
USB	Universal Serial Bus
USGS	United States Geological Survey
UTM	Universal Transverse Mercator
UTS	Ultimate Tensile Strength
UV	Ultraviolet
VCR	Volume-to-Capacity Ratio
VIP	Vertical Intersection Point
VMS	Variable Message Signs
VOC	Vehicle Operating Costs
VPC	Vertical Point of Curvature
VPD	Volume or Vehicles Per Day
VPI	Vertical Point of Intersection
VPT	Vertical Point of Tangency
WAP	Wireless Application Protocol
WBL	Westbound Left-Turn
WBU	Westbound U-Turn
WIM	Weigh-In-Motion
WSDOT	Washington State Department of Transportation
ZA	Zenith Angle
ZOI	Zone Of Intrusion

Volume I





GHANA ROAD DESIGN GUIDE 2023

CHAPTER 1

INTRODUCTION

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CHAPTER 1 INTRODUCTION

1.1 OBJECTIVES OF THIS GUIDE

The Ghana Road Design Guide (RDG) hereinafter called "Guide" is intended to provide road engineers and other road related professionals with technical standards, guidelines and recommendations for the design of road infrastructure in Ghana.

The key objectives of the Guide are:

i. To maintain uniformity in cross section, alignments, intersections, drainage, and other road related infrastructure.
ii. To ensure optimum level of safety and comfort for road users.
iii. To arrive at an economic design.
iv. To achieve a road infrastructure that is environmentally friendly, sustainable and climate resilient.

1.2 SCOPE OF THE GUIDE

The contents of the Guide are partly guidelines and recommendations to be considered, and partly standards which as a rule should be adhered to. In some instances, special conditions may demand modifications to these standards, in which case special consideration should be given in consultation with the Agency Head.

It is vitally important to understand that guidelines provide broad design principles in both urban and rural settings, as well as technical details, but do not provide full details on design for every situation. Safety principles and technical details need to be adhered to in order to achieve required outcomes, including a provision for safety. However, every solution is a unique combination of standard elements that requires expert knowledge and local understanding to apply correctly.

There are no 'off the shelf' solutions that will fully address all situations encountered, and the rigid and unthinking application of charts, tables, and figures is unlikely to lead to a successful design outcome. Good design requires creative input based on experience and a sound understanding of the principles. However, every situation is different, and therefore design requirements will also differ. This applies to all elements of design, and particularly to safety. Thus, designing and constructing roads according to guidelines will not necessarily produce safe outcomes. Based on the outcomes of design and knowledge of safety performance, this has unfortunately proven to be true in many situations.

Safe road design is not like following a recipe, but rather considerable expertise is required to safely design roads for all road users. Because of the complexities of road design, additional checks and tools have been developed to help identify safety risk and maximize the safety potential through design. These tools include road safety audit, road safety inspection and crash investigation and remedial measures.

Road design guides have always considered road safety. Issues such as sight distance and design speed dictate much of the design process, and these are based fundamentally on trying to achieve safe outcomes for road users. However, roads are still designed and constructed with inherent risks that result in death and serious injury. This lack of safety may be because there is a "trade-off" between safety and efficiency or mobility due to project constraints such as cost, inconsistency in road design, or simply lack of consideration for vulnerable road users. Designs must follow Safe System principles, and as far as practical to eliminate death and serious injury.

It is anticipated that users of this Guide will contribute by putting forward any proposals for further development and revision of this Guide stemming from the actual field experience and practice.

1.3 ORGANIZATION OF THE GUIDE

The Guide consists of 13 chapters and appendixes grouped into five (6) volumes:

Volume I Chapters 1 – 5

Chapter 1 Introduction
Chapter 1 deals with the objective and scope of the Guide.

Chapter 2 Road Classification
Chapter 2 covers road classification. It classifies roads based on Road Administration, Function and Design Standard.

Chapter 3 Road Design and Corridor Selection
The Chapter 3 handles the concept of road design, roles and functions of road and routes selection.

Chapter 4 Road Survey Procedure and Requirements
Chapter 4 deals with road survey procedure and requirements.

Chapter 5 Road Safety, Design Control and Criteria
Chapter 5 discusses elements of safe road and roadside designs for road networks that can provide safe mobility to all road users. It also deals with design control and criteria affecting the selection of geometric design values.

Volume II Chapters 6 and 7

Chapter 6 Cross Section Elements
Chapter 6 focuses on cross sectional elements of roads by defining standard widths of lane, shoulder, median, lay-by, turn out, stopping lane, walkway, bicycle lane etc., for the

different road design classes.

Chapter 7 Elements of Design

Chapter 7 discusses the positioning of the physical element of the roadway (horizontal and vertical alignment) according to standards and constraints.

Volume III Chapters 8 and 9

Chapter 8 At-grade Intersections

Chapter 8 deals with design standard for at-grade intersections. The considerations for planning and design of at grade intersection is also described. Recommended design standard value such as cross section composition, auxiliary lanes, channel, island, pedestrian and bicycle crossing etc., are also provided.

Chapter 9 Grade Separated Intersections

Chapter 9 deals with warrants for grade separated intersections. It also discusses the planning and design of grade separated intersections.

Volume IV Chapters 10 - 12

Chapter 10 Traffic Calming Measures

This chapter describes the different traffic calming devices that can be used on roads, where they can be used, and how they should be designed.

Chapter 11 Accessories to Road

Chapter 11 deals with accessories to roads. This includes recommended design standard for Intelligent Transportation System (ITS), Traffic Signals, Parking, Rest Areas, Heavy Vehicle Inspection Sites, Toll Plaza, Emergency Ramps, Road Protection Facilities, Pedestrian Facilities and Protective Screening at Overpasses.

Chapter 12 Road Furniture

Chapter 12 discusses the various types of road furniture which encompasses all fixtures on the road, above the road or within the roadside used for safety and control of traffic.

Volume V Chapter 13

Chapter 13 Road Drainage

Chapter 13 deals with the determination of the quantity of runoff for the design of adequate and appropriate road drainage structures.

Volume VI Appendix A-H

Appendix-A Details of Horizontal Curve
Appendix-B Details of Vertical Curve
Appendix-C Rainfall Intensity

Appendix-D Toll Gates
Appendix-E Presentation of Drawings and Design Reports
Appendix-F Procedure for Determining Geometric Structures of At-Grade Intersections
Appendix-G Road Design Check Sheet
Appendix-H Glossary

1.4 REFERENCES

1. Ghana Highway Authority Road Design Guide March 1991.
2. Austroads. 2015. Guide to Road Design Part 1, AGRD01-15, Austroads, Sydney, Australia. Austroads. 2019.
3. Guide to Road Safety Part 6: Road Safety Audit, AGRS06-19, Austroads, Sydney, Australia.
4. Geometric Design Manual of Uganda, (2005).

CHAPTER 2

ROAD CLASSIFICATION

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CHAPTER 2 ROAD CLASSIFICATION

2.1 CONCEPT OF ROAD CLASSIFICATION

Functional classification of roads defines the role of each roadway in serving motor-vehicle movements within the overall transportation system. The functional classification of a roadway suggests its position within the transportation network and its general role in serving car, truck, and transit vehicles.

The classification of roads into different operational systems, functional classes or geometric

types are necessary for communication between engineers, administrators, and the general public. Road classification is a tool by which a complex network of roads can be subdivided into groups having similar characteristics.

Roads are classified according to the function of the road, design traffic volume, adjoining features of the road and terrain. The concept of road classification is shown in **Figure 2.1**.

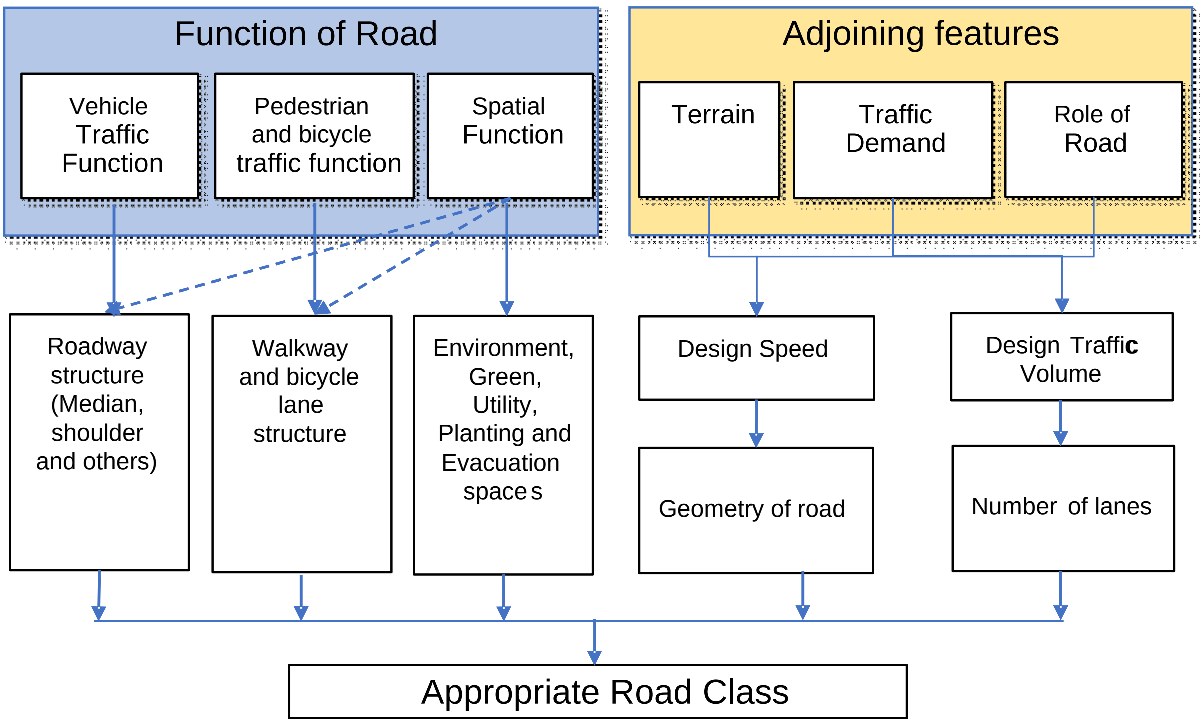

Figure 2.1 Concept of road classification

The two major considerations in classifying highway and street networks functionally are access and mobility. The conflict between providing mobility for through-traffic movements and providing access to a dispersed pattern of trip origins and destinations leads to the differences and gradations in the various functional types.

Figure 2.2 illustrates the general balance between mobility and access for each functional class. **Figure 2.2** is conceptual in nature because the extent and context of development along each road and street varies widely, and some arterials may have more extensive access needs than many collector or local roads.

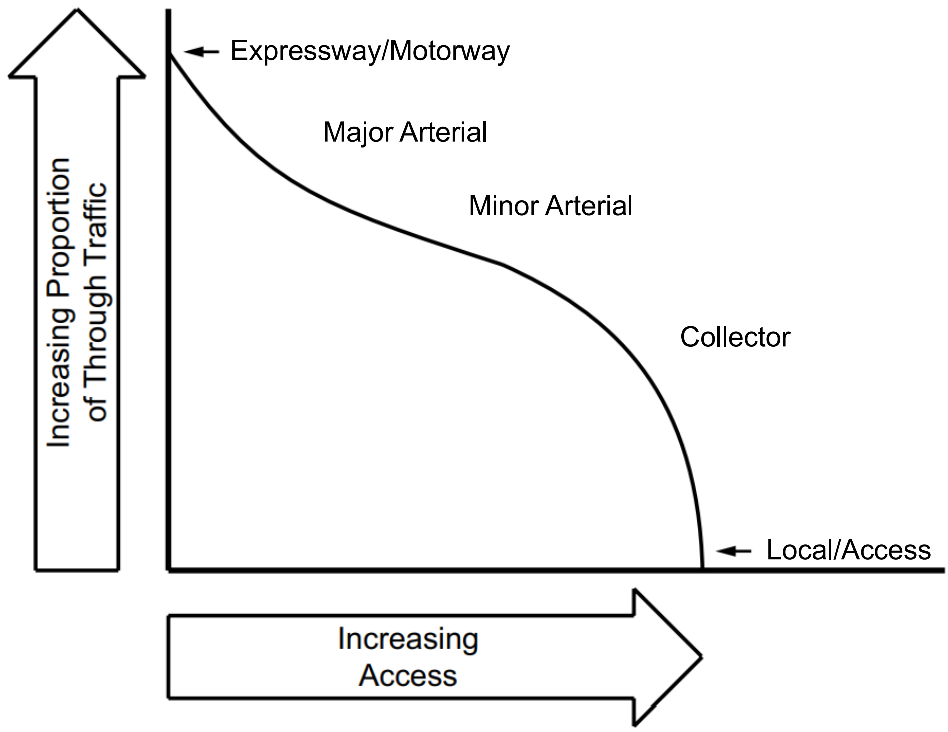

Figure 2.2 Relationship of Functionally Classified Systems Serving Traffic Mobility and Land Access for Motor-Vehicle Traffic

Although numerous classification criteria are used on road networks world-wide, in Ghana, there are basically three criteria used to classify road types. These are:

i. Administrative classification
ii. Functional classification
iii. Design standard classification

This document uses the third type for design purposes.

2.2 CLASSIFICATION ACCORDING TO THE ROAD ADMINISTRATION

In Ghana, three organizations have jurisdiction over roads: Ghana Highway Authority (GHA), Department of Feeder Roads (DFR) and Department of Urban Roads (DUR). Depending on the jurisdiction, roads are classified as Trunk Road (GHA), Feeder Road (DFR) or Urban Road (DUR).

2.3 CLASSIFICATION BASED ON FUNCTION

Table 2.1 shows the various road classes based on their functions.

Table 2.1 Road Classification based on Function

Road Administration	Road Classification	Function
GHA	National (N)	National roads link the national capital and regional capitals. They are roads of strategic importance such as main trunk roads to neighbouring countries.
	Inter-regional (IR)	Inter-regional roads are defined as roads of inter-regional importance. They connect one region to another region.
	Regional (R)	The Regional roads connect district capitals to their respective regional capitals in addition to nearest district capital, major industrial, trade or tourist centres.
DFR	Inter-District (I)	A feeder road which connects to other roads with the start and end in two administrative districts.
	Connector (C)	A feeder road which connects other roads within a single administrative district.
	Access (A)	Roads that start from trunk of high-class feeder road and ends in a community.
DUR	Major Arterial (A)	A high-capacity urban road, primarily to move traffic with very limited access and interruptions. Designed for mobility and ensures facilitated movement of people and good along primary corridors only. Accommodates traffic volume generally above 4,000 veh/day at peak periods.
	Minor Arterial (B)	The minor arterial predominantly provides mobility and as such they carry traffic between industrial, commercial, and residential areas within the urban area. Similar as above, except for relatively lower capacities and volumes of traffic between 2000veh/day and 4000veh/day.
	Collector/ Distributor (C)	A low to moderate capacity road, primarily to move traffic from local/access roads to arterial roads. They are designed to provide access to residential properties. Responsible for exchange of traffic between high function arterials and low function accesses. Expected traffic volumes less than 2000veh/day.
	Local/Access (D)	A low-capacity road, primarily to offer access to properties and also pedestrians in a neighbourhood. Low function road designed primarily to allow access (for varied land use types). Traffic volumes are less than 500veh/day.

2.4 CLASSIFICATION BASED ON DESIGN STANDARD

Roads can be classified into five (5) classes based on their design standards. They are:

i. Design Class A (Expressway/ Motorway)
ii. Design Class B (Major Arterial)
iii. Design Class C (Minor Arterial)
iv. Design Class D (Collector)
v. Design Class E (Local/Access)

Road design classification is based on the road functions, design traffic volume (traffic capacity), and terrain. Based on the classification, the horizontal and vertical alignment, design traffic volume, composition and width of the road cross section, etc. are decided. The main design elements according to each design classification are shown in **Table 2.2**.

2.4.1 ROAD DESIGN CLASS

2.4.1.1 DESIGN CLASS A (EXPRESSWAY/MOTORWAY)

These highest capacity roads, primarily provide mobility with limited access and interruptions. Grade separations are to be considered for access control design. They have a design speed of more than 100km/h with large design traffic volume and high mixing rate of large vehicles. A standard lane width of 3.65m with median is primarily adopted for flat and hilly terrain.

The Rural Expressway/Motorway is classified as A1; Urban Expressway/Motorway is classified as A2.

2.4.1.2 DESIGN CLASS B (MAJOR ARTERIAL)

These high-capacity roads, predominantly provide mobility with partially limited access and interruptions. Semi-access control is to be considered by providing grade and at-grade intersections as well. They have a design speed of 100km/h with relatively large design traffic volume and large vehicles. A standard lane width of 3.65m with median is adopted for flat and hilly terrain.

Rural Major Arterial is classified as B1; Urban Major Arterial is classified as B2.

2.4.1.3 DESIGN CLASS C (MINOR ARTERIAL)

These roads, predominantly provide traffic with partially limited access. At-grade intersection are normally considered to allow traffic access to the road. They have a design speed of 80km/h with relatively large design traffic volume and less large vehicles. A standard lane width of 3.65m without median is adopted for flat and hilly terrain.

The Rural Minor Arterial is classified as C1; Urban Minor Arterial is classified as C2.

2.4.1.4 DESIGN CLASS D (COLLECTOR)

The roads, predominantly to provide access to traffic. At-grade interchanges are normally designed. They have a design speed of 60km/h with low design traffic volume with mainly passenger vehicles. A standard lane width of 3.50m without median is adopted for flat and hilly terrain. In mountainous areas, since the road design speed is 40km/h, lane width of 3.00m can be adopted from economic aspects of construction.

The Rural Collector is classified as D1; Urban Collector is classified as D2.

2.4.1.5 DESIGN CLASS E (LOCAL/ACCESS)

These are roads with low to moderate capacity to primarily provide accessibility without access control. They have a design speed of 40km/h or less. A standard lane width of 3.50m and a minimum width of 3.0m is adopted for flat and hilly terrain.

2.4.2 TERRAIN TYPE

Terrain is divided into Flat, Rolling/Hilly or Mountainous.

i. **Level or flat terrain** is that condition where carriageway sight distances, as governed by both horizontal and vertical restrictions, are generally long or could be made so without construction difficulty or major expense. The slope of the existing terrain is from zero (0) percent to and including five (5) percent.

ii. **Rolling/hilly terrain** is that condition where the natural slopes rise above and fall below the carriageway grade line consistently. Normal carriageway alignment is restricted by occasional steep slopes. The slope of the existing terrain is from five (5) percent to and including fifteen (15) percent.

iii. **Mountainous terrain** is that condition where longitudinal and transverse changes in the elevation of the ground with respect to the carriageway are abrupt and where the roadbed is obtained by frequent benching or side hill excavation. The slope of the existing terrain exceeds fifteen (15) percent.

2.4.3 DESIGN TRAFFIC VOLUME

The design traffic volume is an important element when planning and designing roads. Based on the design traffic volume, the road design classification, number of lanes, horizontal design, vertical alignment, longitudinal composition, pavement composition, etc. are determined. The design traffic volume refers to the average traffic volume of vehicles per day on the road in the design target year.

This is generally given as 10 years for minor roads and 20 years for major roads from the start of service.

Table 2.2 Road design Classification

<table>
<tr><th colspan="2" rowspan="2">Design Class</th><th rowspan="2">Right-of-Way (ROW) (m)</th><th rowspan="2">Functional Class</th><th rowspan="2">Administrative Class</th><th rowspan="2">Access Control</th><th rowspan="2">Design traffic volume (AADT)</th><th rowspan="2">Terrain</th><th colspan="2">Design Speed (km/h)</th></tr>
<tr><th>(Desirable)</th><th>(Absolute)</th></tr>
<tr><td rowspan="3">A</td><td rowspan="2">A1 - Rural Expressway/Motorway</td><td rowspan="3">90</td><td rowspan="3">National</td><td rowspan="3">Trunk Road</td><td rowspan="3">Full</td><td rowspan="3">>10,000</td><td>Flat</td><td>120</td><td>100</td></tr>
<tr><td>Hilly</td><td>100</td><td>80</td></tr>
<tr><td>A2 - Urban Expressway/Motorway</td><td>Mountainous</td><td>80</td><td>60</td></tr>
<tr><td rowspan="3">B</td><td>B1 - Rural Major Arterial</td><td rowspan="2">90</td><td rowspan="2">National</td><td rowspan="2">Trunk Road</td><td rowspan="3">Partial</td><td rowspan="3">>3,000</td><td>Flat</td><td>100</td><td>80</td></tr>
<tr><td rowspan="2">B2 - Urban Major Arterial</td><td>Hilly</td><td>80</td><td>60</td></tr>
<tr><td>60</td><td>Major Arterial</td><td>Urban Road</td><td>Mountainous</td><td>60</td><td>50</td></tr>
<tr><td rowspan="3">C</td><td>C1 - Rural Minor Arterial</td><td rowspan="3">60</td><td rowspan="2">Inter-Regional & Regional</td><td rowspan="2">Trunk Road</td><td rowspan="3">Partial/ No</td><td rowspan="3">>1,000</td><td>Flat</td><td>80</td><td>60</td></tr>
<tr><td rowspan="2">C2 - Urban Minor Arterial</td><td>Hilly</td><td>60</td><td>40</td></tr>
<tr><td>Minor Arterial</td><td>Urban Road</td><td>Mountainous</td><td>50</td><td>30</td></tr>
<tr><td rowspan="3">D</td><td rowspan="2">D1 - Rural Collector</td><td rowspan="3">45</td><td rowspan="2">Inter-District Connector</td><td rowspan="2">Feeder Road</td><td rowspan="3">No</td><td rowspan="3">300-1000</td><td>Flat</td><td>60</td><td>40</td></tr>
<tr><td>Hilly</td><td>50</td><td>30</td></tr>
<tr><td>D2 - Urban Collector</td><td>Collector/Distributor</td><td>Urban Road</td><td>Mountainous</td><td>40</td><td>20</td></tr>
<tr><td>E</td><td>Local/Access</td><td>30</td><td>Local/Access</td><td>Urban Road
Feeder Road</td><td>No</td><td><300</td><td>-</td><td>40</td><td>20</td></tr>
</table>

2.5 REFERENCES

1. Ghana Highway Authority Road Design Guide, 1991.
2. Japanese Road Structure Ordinance, April 2021
3. Geometric Design Manual of Uganda, (2005).
4. A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.
5. Road Maintenance Operation Manual, GHA, 2003.
6. Road Reservation Management (Manual for Coordination), first edition.

CHAPTER 3

ROAD DESIGN CONCEPT AND ROUTE SELECTION

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CHAPTER 3 ROAD DESIGN CONCEPT AND ROUTE SELECTION

3.1 CONCEPT OF ROAD DESIGN

The design concept presented in this guide is intended to promote flexibility in the design of roads in Ghana. Designers should be flexible in decision-making concerning the design decisions made about each project. The context of the project and the identified performance issues need to be considered. Design decisions should consider safety, economy, mobility, and preserving environmental (scenic, aesthetic, historic etc.), community and social values.

Non-compliance with geometric design criteria is not, by itself, a performance issue for a project on an existing road. If some aspects of the existing roadway geometry are out of the design criteria ranges shown in this guide, and the road is performing satisfactorily, there is no need for a new project to modify those aspects of the existing roadway.

The most important concept to keep in mind throughout the road design process is that every project is unique. The setting and character of an area, the values of the surrounding community, the needs of the road users and the associated physical challenges and opportunities are unique factors that road designers must consider with each project. For each potential project, designers are faced with the task of balancing the need for improvement of the road with the need to safely integrate the design into the surrounding natural and human environments.

There are a number of options available to aid in achieving a balanced road design and to resolve design issues. Among these are the following:

i. Use the flexibility available within the design standards.
ii. Recognise that design exceptions may be required where environmental impact consequences are great.
iii. Be prepared to re-evaluate decisions made earlier in the project planning and environmental impact assessment phase.
iv. Conduct road safety audit at the conceptual, preliminary and detailed design stages.
v. Adopt a lower design speed where appropriate.
vi. Maintain the road's existing horizontal and vertical geometry and cross section where possible.
vii. Consider developing alternative design standards, especially for scenic or historic roads.
viii. Consider adopting harmonised standards for international routes.
ix. Recognise the safety and operational impacts of various design features and

modifications.

3.1.1 STAKEHOLDER INVOLVEMENT

In addition to exercising flexibility, a successful road design process should involve the public. To be effective, the public view should be canvassed at the onset, even before the need for the project has been defined. If the primary purpose and need for the improvement has not been agreed on, it would be extremely difficult to reach consensus on alternative design solutions later in the process. Public input can also help to assess the characteristics of the area and to determine what physical features are most valued by the community and, thus, have the greatest potential for impact. Awareness of these valued characteristics at an early stage will help designers to avoid changing them during the project, reducing the need for mitigation and the likelihood of controversy.

After working with the community to define the basic project need and to assess the physical character of the area, public involvement is necessary to obtain input on design alternatives.

Working with the affected community to solve design challenges as they arise is far more effective than bringing the public into the process only after major design decisions have been made. The public needs to be involved at all stages in the project where there are the greatest opportunities for changes to be made in the design.

3.1.2 FUNDAMENTAL DESIGN CONTROLS

There are a number of other fundamental design controls that must be balanced against one another. These include the:

i. Design speed of the facility.
ii. Design-year peak-hour level of service of the facility.
iii. Physical characteristics of the design vehicle.
iv. Performance characteristics of the design vehicle.
v. Capabilities of the typical driver (first time users) on the facility.
vi. Existing and future traffic demands likely to be placed on the facility.

3.2 ROLES AND FUNCTIONS OF ROADS

Roads constitute basic infrastructure that is essential for the movement of people and transportation of goods and services. As such, roads play a major role in ensuring socio-economic development and improving living standards for citizens. Roads also play a role in the formation of urban areas, disaster prevention space, and formation of environmental spaces. Road cross sections based on the existing conditions endowed with multi-purpose functions are therefore required.

Roads have two major functions as depicted in **Figure 3.1**. These are traffic and spatial functions.

i. Traffic functions

The traffic functions are primary functions of the road with the following contents:

- A mobility function that allows motorized and non-motorized traffic to pass safely, smoothly and comfortably.
- An access function that allows easy access to roadside facilities.
- Stopping and parking function for allowing vehicles to stop and park and for holding pedestrians.

ii. Spatial functions

- Urban area formation functions such as composing urban framework and promotion of roadside land use.
- Disaster prevention space for fire and earthquake, greening and landscape formation.
- Environmental space for roadside environment protection.
- Installation space for facilities and utilities.

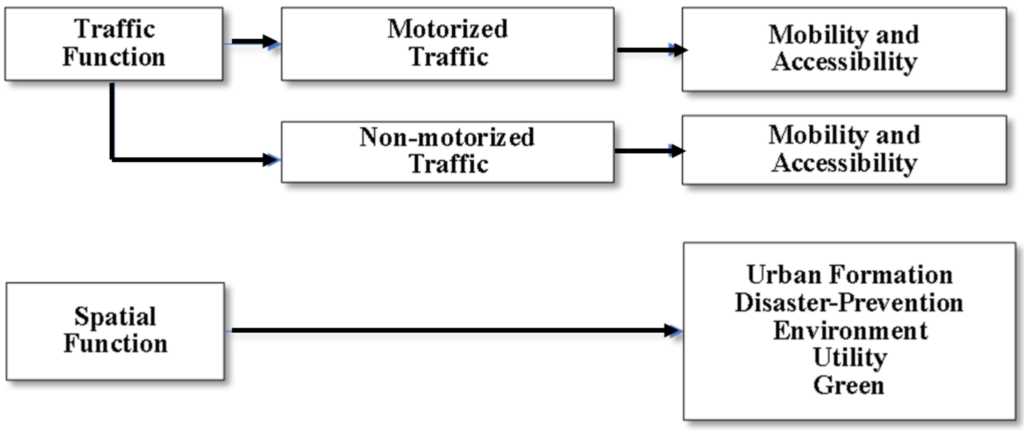


Figure 3.1 Functions of roads

3.2.1 TRAFFIC FUNCTIONS AND ROAD GEOMETRY

3.2.1.1 MOTORISED TRAFFIC FUNCTIONS AND ROAD GEOMETRY

While ensuring the mobility and access functions on each road, vehicle spaces having stopping and parking functions should also be provided according to the need.

3.2.1.1.1 MOBILITY FUNCTIONS AND ROAD GEOMETRY

To secure smoothness and safety of vehicular traffic, the road geometry that permits the operating speed corresponding to each road should be adopted and congestion countermeasures, traffic safety measures, etc. should be considered. Moreover, to ensure reliability and comfort, it is desirable to adopt a road geometry that also considers the impact on traffic at times of disasters or accidents, scenic views from road users, etc.

i. **Smoothness condition**

To maintain the required smoothness condition, care should be taken to provide the operating speed required on the road and ensure that congestion does not arise. In particular, on sections where a uniform operating speed needs to be continuously provided, it is desirable to secure continuity of the road geometry too.

ii. **Cross sectional composition for ensuring smoothness**

a. **Number of lanes corresponding to traffic characteristics**

The number of lanes is basically decided based on the design standard traffic volume, however, on suburban roads, it is desirable to take the traffic characteristics of each road such as peak hourly traffic volume during the morning and evening rush hours into consideration.

b. **Addition of lanes in consideration of the impact of slow vehicles**

To mitigate reduction in speed arising from slow vehicles, additional overtaking lanes and mixed vehicle lanes are provided according to necessity.

iii. **Alignment, intersections, and road geometry that considers congestion**

a. **Lane balancing**

On diverging and merging vehicular roads that have heavy traffic volume in urban areas, it is desirable to secure a balanced number of lanes according to the traffic volume. Details of lane balancing is discussed under **Section 9.4.3.7**.

b. **Toll gate**

On toll roads, adequate number of toll gates should be provided to accommodate traffic. Electronic Toll Collection System (ETC) may be installed as required.

c. **Intersection**

At intersections, the road cross-section should be adequate to prevent congestions. Auxiliary lanes should be provided to secure adequate capacity. Grade separated intersection may be studied for high traffic roads. The cross section should be designed not to allow drivers park vehicles in the vicinity of intersections.

d. **Tunnel**

At the entrance and exit of tunnels, drivers may reduce speed due to sudden change of visibility. The road designer should consider gradual luminance of lighting and shape of tunnel portal.

iv. Safety Consideration

The basic methods for ensuring safety entails adopting appropriate design speeds, cross-section composition, visibility and sight distance in consideration of the road functions. To achieve this, consistent design must be adopted over a uniform section of the road structure comprising number of lanes, alignment, interval between intersections on the level plane, entry and exit restrictions.

v. Reliability Consideration

To ensure reliability, it is necessary to adopt a road geometry that guarantees road functions even at times of accidents and disasters.

vi. Comfort Considerations

To ensure comfort, design should be conducted in consideration of the landscape from the viewpoint of drivers and passengers.

3.2.1.1.2 ACCESS FUNCTIONS AND ROAD GEOMETRY

To ensure access functions, the road geometry should consider the ease of entering and exiting intersecting roads and roadside facilities according to need as shown in **Figure 3.2**.

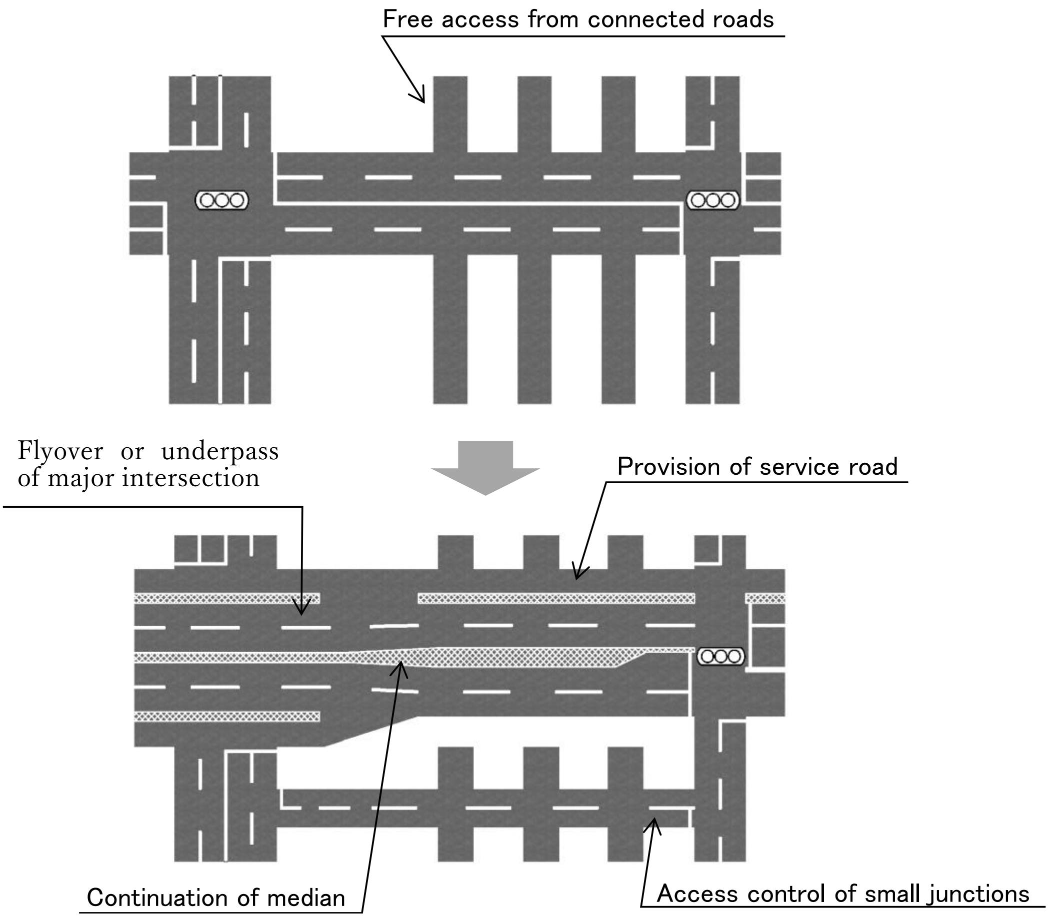


Figure 3.2 Typical layout of access control

3.2.1.1.3 STOPPING AND PARKING FUNCTIONS

If stopping and parking functions are required, spaces for the parking of vehicles and resting by vehicle users should be ensured according to need as shown in **Figure 3.3**.

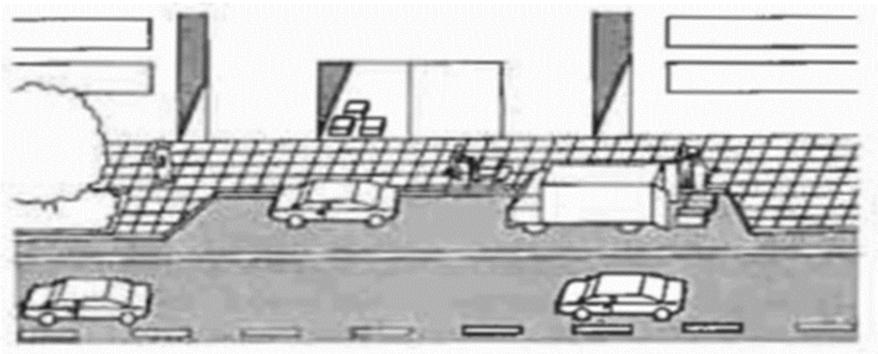


Figure 3.3 Typical stopping and parking layout

Motorised traffic functions and road geometry are summarized in **Table 3.1**.

Table 3.1 Motorised traffic functions and road geometry

Traffic Function	Geometry Consideration	Description
Mobility	Smoothness	Road geometry with high specifications regarding entry and exit
		Setting of number of lanes according to traffic characteristics
		Consideration of capacity not only on road sections of uninterrupted flow, but also on traffic bottlenecks
	Safety	Phasing of horizontal and vertical alignments
		Visibility at intersections, diverging and merging areas
	Reliability	Preservation of road geometry during accidents and disasters
	Comfort	Geometry in consideration of aesthetics
		Provision of information on traffic signs and at roadside facilities, etc.
Access	Access	Geometry that allows direct entry to and exit from the main road
		Minimum access distance
		Appropriate intervals between intersections
		Provision of parking spaces
		Provision of bus parking zones
Stopping and Parking	Stopping and Parking Space	Parking lots, bus parking spaces
		Rest Facilities, roadside stations

3.2.1.2 NON-MOTORISED TRAFFIC FUNCTIONS AND ROAD GEOMETRY

While ensuring the motorised traffic functions of roads, pedestrian and bicycle spaces with stopping and parking functions should also be considered according to the need.

3.2.1.2.1 MOBILITY FUNCTION AND ROAD GEOMETRY

To ensure the continuity and safety of traffic spaces for pedestrians and bicycles, it is necessary to provide continuous traffic spaces and form a separate network from vehicles. Moreover, traffic spaces for pedestrians and bicycles should be designed as separate or co-existing road features. When adopting a road geometry that has traffic space for pedestrians, it is necessary to adopt an accessible structure. Moreover, to ensure comfort for wheelchair users, various kinds of pedestrians and bicycle users, it is also desirable to consider the type of surface, road landscaping, provision of information etc.

3.2.1.2.2 ACCESS FUNCTIONS AND ROAD GEOMETRY

To ensure access functions, the road geometry should take into consideration the ease of entering from and exiting to roadside facilities for pedestrians and ease of using roadside facilities for cyclists according to the need.

3.2.1.2.3 STOPPING AND PARKING FUNCTIONS AND ROAD GEOMETRY

If stopping and parking functions are required, the road geometry should take into consideration the stopping and parking spaces of pedestrians, cyclists and wheelchair users. Non-motorised traffic functions and road geometry are summarized in **Table 3.2.**

Table 3.2 Non-motorised traffic function and road geometry

<table>
<tr><th>Non-Motorised Function</th><th>Geometry Consideration</th><th>Description</th></tr>
<tr><td rowspan="7">Mobility</td><td>Smoothness</td><td>Continuous establishment of walkways and pedestrian space co-existing with other road features</td></tr>
<tr><td rowspan="2">Safety</td><td>Separation of pedestrians and vehicles space</td></tr>
<tr><td>Construction of pedestrian co-existing road features</td></tr>
<tr><td rowspan="2">Reliability</td><td>Effective width, flatness, and gradient of walkways considering wheelchair users and the vulnerable.</td></tr>
<tr><td>Installation of elevators and other equipment for assisting pedestrians</td></tr>
<tr><td rowspan="2">Comfort</td><td>Type and texture of pavement surface</td></tr>
<tr><td>Shape and colour of road signs and other road facilities</td></tr>
<tr><td>Access</td><td>Access</td><td>Ease of entering from and exiting to roadside facilities</td></tr>
<tr><td>Stopping and parking</td><td>Stopping and parking</td><td>Securing of stopping and parking spaces such as bus stops, road crossings, benches, terminals etc.</td></tr>
</table>

3.2.2 SPATIAL FUNCTIONS AND ROAD GEOMETRY

3.2.2.1 ROAD STRUCTURE FOR URBAN FORMATION

Roads have the function of forming urban areas and therefore it is necessary to consider forming the framework of cities and districts to attract the appropriate roadside land uses.

3.2.2.1.1 ROAD GEOMETRY FOR FORMING URBAN FRAMEWORK

Roads are important in the formation of pleasant townscape. Accordingly, it is necessary to decide the width upon considering the balance with roadside buildings according to the existing conditions.

In cases of ceremonial roads in central business districts that have transport terminals, it is desirable to adopt a broad width to secure adequate spatial functions.

3.2.2.1.2 GEOMETRY FOR ROADSIDE LAND USE

It is necessary to make roads into disaster prevention spaces that are resilient to disasters. Accordingly, it is basically desirable to ensure the total road width and necessary space in consideration of preventing the spread of disaster and ensuring traffic flow.

3.2.2.2 ROAD GEOMETRY AS ENVIRONMENTAL SPACE

As environmental space, it is necessary to secure greening, form pleasant scenery and preserve the roadside environment.

3.2.2.2.1 ROAD GREENING

While securing road traffic functions, effort should also be made to promote greening, which is effective for forming scenery, securing green shade, mitigating noise, cleansing the atmosphere etc. On urban roads, it is necessary to actively conduct road greening by planting vegetation along walkways and in median strips. **Figure 3.4** shows a cross section with planting along walkway and park.

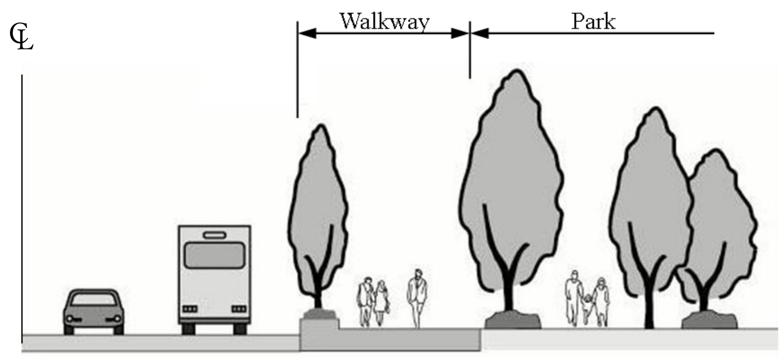


Figure 3.4 Road cross section with planting along walkway and park

3.2.2.2.2 ROAD GEOMETRY FOR LANDSCAPE FORMATION

As important landscape component elements, roads should be harmonized with the surrounding scenery. Accordingly, it is necessary to pay attention to the selection of pavement raw materials, tree types and the shape and colour of road furniture that harmonize with the surrounding scenery.

3.2.2.2.3 ROAD GEOMETRY FOR PRESERVING THE ROADSIDE ENVIRONMENT

To preserve the roadside environment, care should be taken to secure the necessary space to avert and mitigate noise, air pollution, etc. as shown in **Figure 3.5**.

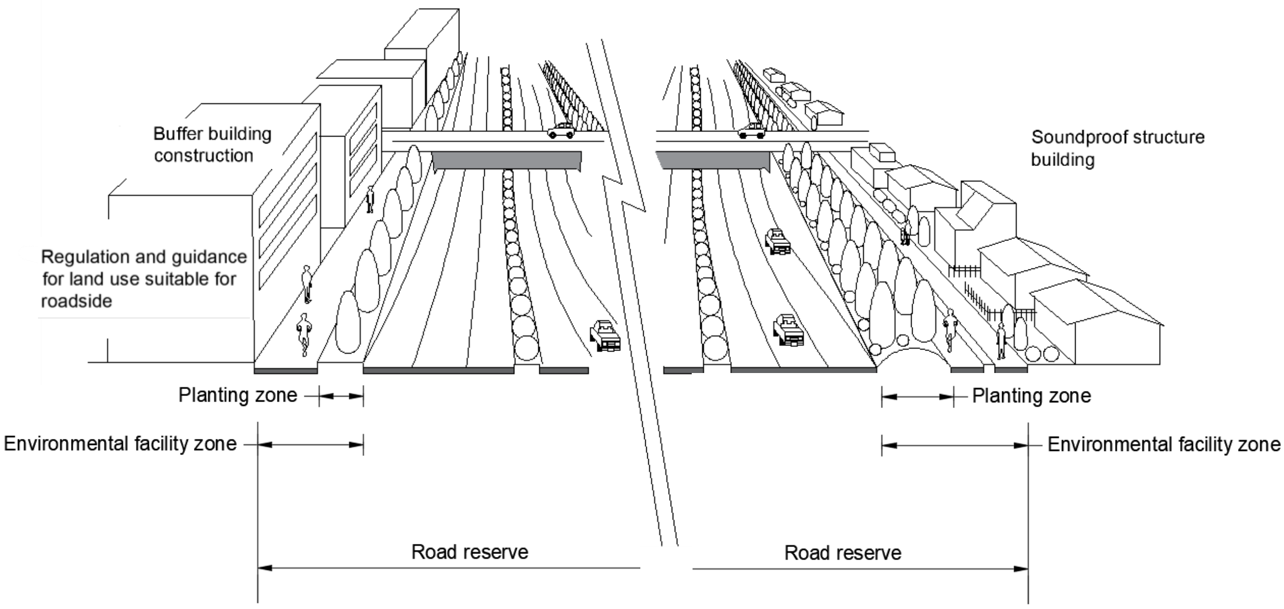


Figure 3.5 Schematic view for preservation of roadside environment

3.2.2.3 ROAD GEOMETRY AS UTILITY SPACE

It is necessary for roads to have spaces for holding public transportation facilities such as urban monorails, subways, etc. and utility lines such as water, electricity, telecommunication, sewerage, gas etc. according to need. A summary of Spatial functions and road structure is shown in **Table 3.3**.

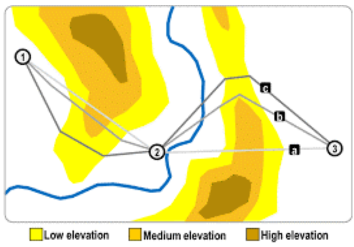
Table 3.3 Spatial functions and road structure

Spatial Function	Geometry Consideration	Description
Urban formation	Urban framework	Good balanced road width with roadside building for forming pleasant townscapes.
	Stimulating of roadside land use	Road geometry for attracting appropriate roadside land use.
Disaster prevention space	Disaster prevention space	Road width and necessary space in consideration of preventing the spread of disaster and ensuring traffic flow.
Environmental space	Road greening	Providing greening for effective scenery, green shade, mitigating noise, cleansing the atmosphere etc.
		Road greening by planting vegetation along walkways and median strips
	Landscape formation	Road facilities' selection of pavement raw materials, tree types and the shape and colour of road furniture that harmonize with the surrounding scenery
	Preservation of roadside environment	Ensuring necessary space to avert and mitigate noise, air pollution, etc.
Utility space	Utilities and other facilities	Space for public transportation facilities such as urban monorails, subways, etc.
		Space for utility lines such as water, electricity, telecommunication, sewerage, gas etc.

3.3 ROUTE SELECTION

Between any two points to be connected by road, there exist an infinite number of alternative routes. Before implementing road planning and design, it is necessary to decide the outline road route. This entails determining the rough position of the route and the basic road structure. **Table 3.4** summarises the road design stages.

Table 3.4 Three different design stages

Master Plan	Route Selection	Design
		
The master plan is a schematic plan based on engineering surveys and studies to achieve the Road Ministry's vison for a set target year. The plan should be prepared by considering other modes of transport based on social and economic development.	Route selection is the study to find a path minimizing cost (C) and maximizing efficiency (E) from multiple alternative routes considering adjacent conditions. The study results in a proposed corridor for the most optimal road development. Conceptual design study is carried out in this phase.	Based on the selected route, road design shall be carried out. There are several levels of design depending on purposes. Preliminary design can be carried out to determine the centreline of the road alignment, sometimes called, definitive plan. Detailed design is carried out based on the preliminary design to prepare detailed bill of quantities and define construction limit.

This section describes initial stages of road alignment selection that include desk study, preliminary identification of potential corridors and comparison and necessary site visits.

3.3.1 DESK STUDY FOR IDENTIFICATION AND FEASIBILITY

Road Design, Construction and Maintenance is highly influenced by the terrain of the area through which the road traverses. The shortest road alignment is not necessarily the easiest, quickest, safe, or most economical option for construction and maintenance. Frequently, topography, slope stability, flood hazard and erosion potential are likely to be the most significant controls in the choice of the most suitable alignment and design of cross-section.

Variations in geology and slope greatly influence road design and hence the cost of construction and these variations can occur over very short lengths of alignment. Geology, geomorphology and hydrology, therefore, are key factors in the route corridor selection during the feasibility study, design, construction and maintenance of roads. Road geometry, earthworks, retaining structures and drainage measures must be designed in such a manner as to cause the least impact on the stability of the surrounding slopes and natural drainage systems.

Excessive blasting, cutting, side tipping of spoil and concentrated or uncontrolled surface water runoff negatively affect the environment and can lead to instability and erosion. Although many of these effects are often unavoidable, the design and the construction method adopted should aim to minimize them. This Sub-section describes the methodology for analysing possible corridors and selecting the optimum route from technical, economic, social, and environmental considerations.

In determining the outline route, multiple proposals should be compared and evaluated from a general viewpoint including project impact, environment, safety, cost, etc. This is usually carried out in the office using mainly secondary data such as:

i. Published literature covering a range of topics including road construction and maintenance case histories and geological, economic and environmental reviews
ii. Topographic maps
iii. Digital Elevation Model (DEM)
iv. Geological maps, agricultural soil maps and other natural resource maps
v. Aerial photographs
vi. Satellite images
vii. Hydrological maps
viii. Soil map
ix. Seismic map
x. Land use and land cover maps etc.

Before commencing with selection of the route corridors, the controlling requirements of the route need to be defined. These may include the following:

i. What are the constraints in regard to the beginning and ending points of the road? Must these be at existing junctions in settlements? Do economic considerations such as volume of earthworks limit the alternatives?
ii. Through which settlement(s) must the route pass? Must the route pass directly through these settlement(s), or can linking roads connect the settlement(s)? If so, what are the implications to the settlement(s) in terms of lost trade?
iii. Planning particulars (rough length, number of lanes, design speed, etc.). For example,

what is the desired design speed and design standard requirement based on, among other criteria, the traffic that would use the road? How does this standard fit the terrain in terms of geometric parameters such as gradients, horizontal and vertical curves?

iv. Envisaged route corridor (rough determination of the planned route position in a corridor of certain width on a scale of roughly 1/25,000-1/50,000). This differs from the alignment plan.

v. Main connecting roads.

vi. Main structures (according to cuts and fills, tunnels, bridges or elevated sections, and other structures)

vii. Geotechnical consideration

viii. Drainage consideration. If major rivers are to be crossed, what are the possible crossing locations, given constraints of topography and geology? What are the economics of the alternative bridge sites with the corresponding road geometries?

ix. Other necessary considerations

x. Rough project cost (using cost per unit length according to each main structure)

3.3.2 PRELIMINARY IDENTIFICATION OF POTENTIAL CORRIDORS AND COMPARISON

The basic requirements of an ideal alignment between two terminal stations are that it should be short, easy to construct and maintain, safe in terms of stability of natural hill and embankment slope and economical in terms of initial cost, minimum environmental impact, maintenance cost and operational cost.

Using the 1:50,000 scale hardcopy maps and/or digital maps, it is possible to trace out alternative alignments. This is readily accomplished by referring especially to the vertical geometric design criteria for maximum grade and plotting possibilities through correlation with the contour lines shown on the map.

For instance, assume that the road classification and terrain are such that a 10% maximum grade is permissible and that the contour interval on the 1:50,000 maps are 20m. A preliminary alignment needs to be selected in such a way that a distance not less than 200m (0.4cm on the map) is used to achieve the 20m interval, giving a 10% grade.

For each of the possible alternative alignment corridors, the existing maps should be studied and aerial photographs examined. From this study, it will be possible to assess the positive or negative influence of the following local factors:

i. Topographic, geologic, and physical characteristics
ii. Number, type and characteristics of water courses

iii. Potential risk of slides, slope instability or floods
iv. Human settlements affected by the road and
v. Environmental impact of the selected route

The proposed alternative alignments corridors are next studied, evaluated and compared based on the criteria below and best alternatives are to be selected for further studies and field assessment.

i. What are the relative lengths of the alternatives? Normally the shortest distance is preferable.
ii. What are the average and mean gradients of the alternatives? Normally the least severe grade alternative is preferred.
iii. Which alternative more closely follows an existing road or track? This makes survey and construction easier and may indicate the route of least earthworks.
iv. Which alternative follows the least severe terrain type? An alignment through, for instance, rolling terrain should be less costly to construct, have lower vehicle operating costs and maintenance costs, and less severe horizontal/vertical curves than a route through mountainous terrain.
v. Which route remains for a longer period on the crest of the terrain? Such an alignment minimizes the need for drainage structures.
vi. Which alignment minimizes the need for land acquisition? The amount of farmland to be taken by the road.
vii. Which alignment minimizes the need to demolish buildings and houses less resettlement?
viii. What is the total number of bridges and their respective estimated span required for each alternative? What is the total aggregate length of these bridges?
ix. Which route results in the least environmental disturbance to the surrounding area?

3.3.3 SITE VISIT AND SURVEY

After the preliminary office work, a site visit must be carried out on the proposed routes. Where terrain constraints make such a visit problematic, a flight can be made over the terrain and all potential routes can be directly examined from the air.

When potential route corridors have been identified from the desk study analysis, then a reconnaissance survey is usually employed to verify, modify and update the desk study and interpretations, to further assess the selected corridors during the desk study, to help determine the preferred corridor, and to identify factors that will influence the feasibility design concept and cost comparisons.

A team consisting of the following personnel should conduct the site visit:

i. Highway Engineer

ii. Soils & Materials Engineer
iii. Hydrologist
iv. Geodetic/Geomatic Engineer
v. Bridge/Structural Engineer
vi. Environmentalist/Sociologist
vii. Transport Economist/Planner
viii. Local Government Personnel
ix. Other Relevant Stakeholders

In most cases, the information obtained from the reconnaissance survey will require significant modification of the desk study interpretations. During the reconnaissance survey, in addition to the data collected in respect of the evaluation criteria, the following information should be determined:

i. Topographic and geometric characteristics
ii. The location of topographical constrains, such as cliffs, gorges, ravines, rock outcrops, and any other features not identified by the desk study
iii. Slope steepness and limiting slope angles identified from natural and artificial slopes (cutting for paths, agricultural terraces and existing roads in the area)
iv. Slope stability and the location of pre-existing landslides
v. Geology, tectonics, rock types, geological structures, dip orientations, rock strength and rippability
vi. Approximate percentage of rock in excavations
vii. Availability of construction materials sources and their distribution
viii. Soil types and depth (a simple classification between residual soil and colluvium is useful at this stage)
ix. Soil erosion and soil erodibility
x. Slope drainage and groundwater conditions
xi. Hydrology, drainage stability and the location of shifting channels and bank erosion
xii. Land use, land cover and their likely effect on drainage
xiii. Likely foundation conditions for major structures
xiv. Approximate bridge spans and the sizing and frequency of culverts
xv. Flood levels and river training/protection requirements
xvi. Environmental considerations, including forest resources, land use impacts and socio-economic considerations
xvii. Verify the accuracy of the information collected during the desk study
xviii. The possibility of using any existing road alignments including local re-alignment improvements
xix. Information on the physical accessibility to bridge sites and the proposed corridors,

including the geomorphology of drainage basins, soil characteristics, slopes, vegetation, erosion and scouring

xx. Economic settings

During the site inspection the team should examine all alternatives. This information can be combined with the results of the desk study to determine the most appropriate alignment alternative. Appropriate field assessment report of each alternative by each discipline will have to be prepared. Cost estimates of each alternative shall be determined for comparison purposes.

3.3.4 ENVIRONMENTAL CONSIDERATIONS

No road project is without both positive and negative effects on the environment. The location and design of the road should aim at maximizing the positive effects and minimizing the negative effects. A positive effect could be to remove undesirable traffic from environmentally vulnerable areas, while at the same time minimizing the adverse effects of the project as much as possible.

3.3.4.1 EFFECTS RELATED TO THE ROAD AS A PHYSICAL FEATURE

The following factors, related to the road as a physical feature in the environment, should be considered in the location and design of the road projects:

i. The preservation of the natural beauty of the countryside and the adaptation to the conditions and architecture of the surrounding features.
ii. The preservation of areas and land use of particular value, including:
    - national parks and other recreational areas
    - wildlife and bird sanctuaries
    - forests and other important natural resources
    - cemeteries
    - land of high agricultural value or potential
    - other land use of great economic or employment importance
    - historic sites and other man-made features of outstanding value
    - the prevention of soil erosion and sedimentation
    - the prevention of health hazards by ponding of water leading to the formation of swamps
    - the avoidance or reduction of visual intrusion
    - the prevention of undesirable roadside development

3.3.4.2 EFFECTS RELATED TO THE TRAFFIC

Negative effects related to the traffic can often be quantified, for example noise levels and air quality. The effects which should be considered are:

i. Noise pollution

ii. Air pollution
iii. Ground water pollution
iv. Vibrations
v. Severance of areas (barrier effect)
vi. Accident

Among the solution to avoid the problems is to locate the road outside trading centres and towns. If this is not possible the best way to reduce the problems is to lower the speed and provide safe crossings for local traffic, pedestrians and cyclists. However, it is appropriate and necessary to seek the advice and service of relevant agencies to properly evaluate the impacts and establish proper and adequate mitigation measures.

3.3.5 ECONOMIC EVALUATION

Decisions as to the exact location and details of geometric design must be based on cost-benefit analysis that takes into account all factors concerned. The purpose of the analysis should be to determine whether or not the maximum benefits to be provided by the road are consistent with the costs involved.

The most economic design will often not involve the shortest route or the use of minimum standards.

Savings in road maintenance costs, Vehicle Operating Costs (VOC), travel time costs and accident costs etc. may offset the extra construction costs for a road with higher design standards. The economic outcome of the design will depend upon both in the route selection and in the geometric design of the chosen route.

The designer is required to establish the costs of the project, as well as its benefits so that he/she can then compare the two. In working out the costs, the designer must recognise the economic resources that are to be used up by the project and at the same time identify economic costs separated from financial costs. The benefits of a project are worked out on the basis of the contribution the project will make towards improving the country's public welfare.

When comparing the costs and benefits of a project, it is essential that both are brought to a common base year using a discounting formula, as shown in **Equation 3.1**.

$$PV = \frac{FV}{(1+i)^n} \tag{3.1}$$

Where,

PV= Present Value (Gh¢)
FV= Future Value (Gh¢)
i = discount rate (%)
n = number of years between present and future values (years)

In an economic appraisal of a road project, the following techniques should be used for evaluating costs and benefits, and for determining whether a project is economically viable or not:

- Net Present Value (NPV)
- Internal Rate of Return (IRR)
- Benefit/Cost Ratio (B/C)

The NPV involves discounting benefits and costs to a common year during the project life. If the NPV is positive, the project is economically viable.

The IRR involves calculating the rate of return at which the NPV is zero. The project is considered to be acceptable if the calculated rate of return is greater than opportunity cost of capital.

The B/C involves calculating the ratio between the present values of all benefits and the present value of all costs. The project is acceptable if the B/C is greater than one.

3.3.6 IMPORTANT KEY PLANNING ISSUES

The key points arising in road planning are:

i. Planning and appraisal procedures should consider a wide range of external factors, many of them of a non-technical nature, that affect the planning process if long-term sustainability of the investment is to be achieved.
ii. Stakeholder consultations are critical in the planning process for which there are a number of techniques which shall be undertaken as appropriately and as transparently as possible.
iii. The traditional methods of investment appraisal are generally not adequate for capturing the full range of benefits (often of a social rather than economic nature) arising from the provision of roads. More recently developed models shall be used for appraising the roads.
iv. The implications of adopting cost-reducing measures, such as the use of more appropriate pavement and geometric design methods and wider use of natural gravels rather than crushed stone.
v. Environmental and Social issues are of great importance in Ghana. Environmental and Social Impact Assessments (ESIA) should be an integral aspect of all road projects. The effectiveness of the ESIA will depend on the extent to which it is actively used and incorporated into different stages of the project planning process.

3.3.7 PROCESS OF ROUTE SELECTION

The “Road examination procedure”, which covers work from the proposal of plan review based on the National (Master) Road Network Plan to decision of the policy of countermeasures, is composed of six steps. None of these processes are independent of the other; rather, in each stage of the “Road examination procedure”, “Technical and specialized review” is conducted and opinions are coordinated with related Administrative Agencies, and other stakeholders. Based on these opinions, it may be desirable to review the affected stage(s) of the “Road examination procedure”. **Figure 3.6** indicates the flowchart of route selection.

Step 1: Confirmation of Related Master Plans

↓

Step 2: Issues Identification and Goals Setting

↓

Step 3: Confirmation of Need of the Route

↓

Step 4: Setting of Multiple Routes and Evaluation Criteria

↓

Step 5: Comparative Evaluation of Multiple Routes

↓

Step 6: Selection of Recommended Route

Figure 3.6 Flowchart of route selection

Step 1: Confirmation of related Master Plans

Mater Road network plans, such as overall road network plans on the National, Regional, Municipal, District levels constitute the basis for the road planning and review procedure. However, if a comprehensive planning framework is not established in the local area, consistency should be confirmed with wide area regional plans, infrastructure priority development policies, local vision plans or any other related plan/policy.

Step 2: Issues Identification and Goals Setting

Based on **Step 1**, current or future issues that need to be resolved should be identified and shared with related parties as early as possible, and the resolution of such issues should be set as goals of the said road plan.

Step 3: Confirmation of the Need of the Route

The need of the route should be confirmed through ascertaining that the goals set in **Step 2** can be efficiently and effectively realized through the route and making sure that the said goals cannot be attained only by methods other than road construction.

Step 4: Setting of Multiple Routes and Evaluation Criteria

i. **Setting of multiple routes**

Multiple realistic and rational plans should be set considering the clarified goals. In addition, the “Do nothing Option” should be set as the baseline for conducting comparative evaluation.

ii. **Setting of evaluation criteria**

Evaluation criteria for the multiple routes used in the comparative evaluation should be set from the viewpoints of degree of achievement of goals and impacts. The standard evaluation criteria should be (a) to (f), with other criteria added according to local conditions and the planned goals (g).

a. Traffic (effects and impacts on wide area trunk flows, intra-regional traffic and pedestrian traffic, disaster vulnerability, access to emergency medical care facilities, etc.)
b. Natural Environment (effects and impacts on the wide area environment and roadside environment)
c. Social Environment {Impact on Project Affected Persons (PAPs)}
d. Land use and urban area development (effects and impacts on urban structure, roadside land use and communities)
e. Society and local economy (effects and impacts on wide area society and local industry, tourism promotion, resident livelihoods, etc.)
f. Project feasibility (project costs, technical constraints, etc.)
g. Others (items that need to be added according to local conditions and the planned goals

An example of an evaluation criteria is shown in **Table 3.5**.

Table 3.5 Examples of evaluation criteria

Evaluation Criterion	Examples of Evaluation Criteria
Traffic	Reduction of travel time, congestion mitigation, reduction of traffic accidents, mobility and safety of pedestrians and bicycles, access to emergency medical care facilities, maintaining road functions and safety during disaster, connectivity with existing networks, accessibility from/to traffic attraction, road density in urban area etc.
Natural Environment	Air and water quality, noise, flora and fauna, ecosystem, etc.
Social Environment	Impact on Project Affected Persons (PAPs)
Land use and urban area development	Impact on agricultural land use, disaster prevention characteristics of urban area, degree of contribution to urban area development (accessibility, etc.), impacts on roadside commercial facilities, etc.
Society and local economy	Reinvigoration of the local economy, urban regeneration, tourism promotion, access to schools, hospitals, and other social amenities.
Project feasibility	Construction and maintenance costs, construction duration, impacts during construction, land acquisition risks, flexibility of plan in relation to unexpected situations.
Others	Items that need to be added according to local conditions and the planned goals

Step 5: Comparative Evaluation of Multiple Routes

Concerning the multiple routes that are set, ranking should be based on technical and specialist review of the set evaluation criteria. Moreover, when explaining the ranking of multiple routes to related agencies and stakeholders, it is important to indicate objectively based on accurate data. It is desirable to categorize data using numerical values and objective expressions. Clarify differences while showing easy to understand figures and comparative evaluation tables, so that the results of comparative evaluation can be easily understood. A sample evaluation sheet is shown in **Table 3.6**.

Table 3.6 Example of evaluation sheet

Criterion	Evaluation Criteria	Road Construction			Baseline
		Route A	Route B	Route C	
Traffic					
Natural Environment					
Social Environment					
Land use and urban area development					
Society and local economy					
Project feasibility					
Others					

Step 6: Selection of Recommended Route

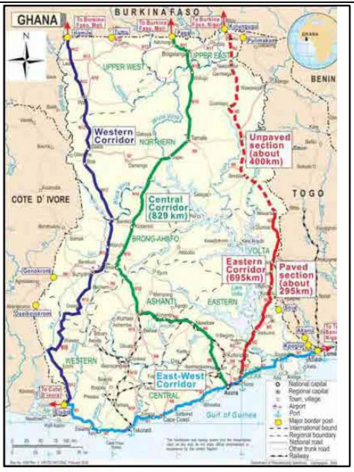
The best route should be selected based on the evaluation conducted at **Steps 4** and **5** upon feedback from related administrative agencies, experts, and other stakeholders. Review should be repeated in response to the feedback.

3.3.7.1 CASE STUDY: JICA PREPARATORY SURVEY ON EASTERN CORRIDOR DEVELOPMENT PROJECT

As a case study of a corridor selection study, actual survey and studies conducted for the Eastern Corridor Development Project (ECDP) by Japan International Corporation Agency (JICA) are summarized in **Table 3.7** and **Table 3.8**.

This study was conducted in the year 2013 to select the optimum route, with a new bridge across the Volta River, among alternative routes between Asutsuare Junction and Asikuma Junction on the Eastern Corridor (N2), and to confirm the viability of the road and bridge development project between Aveyime and Asutsuare.

Table 3.7 Selection of desirable routes for ECDP

Step 1: Confirmation of Related Master Plans i. National Development Plan ii. Regional Development Plan (ECOWAS coordination programme) iii. Potential of Growth Sectors in the Study Area iv. Collection of relevant concept design	 Source: Study Team Major Transport Corridors and Border Posts
Step 2: Issues Identification and Goals Setting i. Present situation of the Road Sub-sector ii. Road network and inventory iii. Socio-economic condition survey iv. Land use survey v. Demography survey vi. Economic condition including neighbouring countries vii. Natural condition survey viii. Development potential survey ix. Railway sub-sector x. Air Transport sub-sector	 Road Network in the Study Area
Step 3: Confirmation of Need of the Route i. Traffic survey ii. Origin and Destination survey iii. Traffic demand analysis iv. Route assessment	 Desired Line Diagram of Future Traffic

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Step 4: Setting of Multiple Routes and Evaluation Criteria

i. Strategies for future development of the road network in the study area

ii. Design standards applied for the study roads and bridges

iii. Environmental and social assessment

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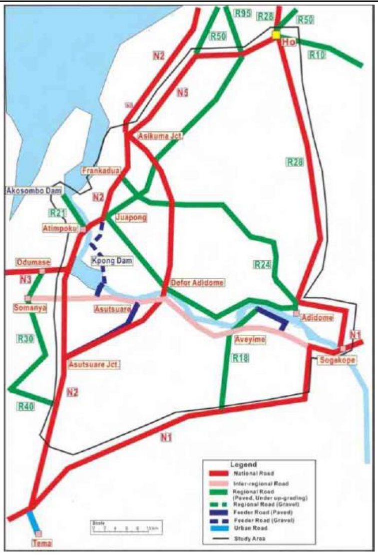

Road network development options

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Step 5: Comparative Evaluation of Multiple Routes

i. Road Alignment setting

ii. Road Alignments study

iii. Bridge and Drainage Structure Study

iv. Preliminary Design

- Alignment study
- Pavement design
- Drainage design
- Intersection design
- Bridge/ structure design
- Road safety audit

v. Cost estimation

vi. Construction plan

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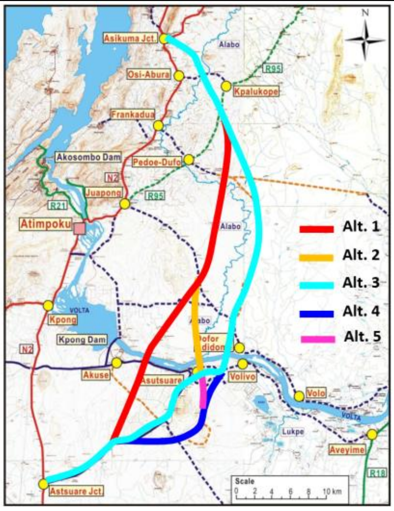

Alternative study

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Step 6: Selection of Recommended Route

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Table 3.8 Example of evaluation table of route selection

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5
Length of Road	60.0km	62.0km	65.1km	65.8km	65.3km
Minimum Radius (new construction section)	2,000m +	1,000m	1,500m	1,800m +	2,000m +
Maximum longitudinal gradient	Less 2%	Less 2%	Less 2%	3% (2km)	3% (2km)
Number of bridges	5	3 +	2 ++	2 ++	3 +
Length of required soil improvement for Black Cotton Soil	8.4km +	7.5km ++	19.1km	26.9km	9.1km ++
Area of affected agricultural land	29 ha.	6 ha. ++	15 ha. +	18 ha. +	22 ha.
Disturbance for agricultural development scheme	No +	Yes	No +	Yes	No +
Length of passing roadside communities	No +	5.0 km	5.0 km	No +	No +
Environmental and social consideration	Desirable ++	Not desirable	Not desirable	Desirable ++	Desirable ++
Contribution for development of arable land without road access	Medium +	Medium +	High ++	High ++	Medium +
Total score	7 +	6 +	6 +	9 +	9 +

Source: JICA Preparatory survey on Eastern Corridor Development Project

3.4 REFERENCES

1. Ghana Highway Authority Road Design Guide, 1991.
2. Japanese Road Structure Ordinance, April 2021.
3. Explanation and Operation of the Road Structure Ordinance.
4. Geometric Design Manual of Uganda, (2005).
5. A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.
6. Preparatory Survey on Eastern Corridor Development Project (ECDP) by Japan International Corporation Agency (JICA), 2013

CHAPTER 4

ROAD SURVEY PROCEDURE AND REQUIREMENTS

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CHAPTER 4 ROAD SURVEY PROCEDURE AND REQUIREMENTS

4.1 INTRODUCTION

The road project may consist of either construction of a new road or improvements to an existing one. In either case, the working drawings have to be prepared after detailed surveys, design and investigations. The manner in which surveys are conducted has vital influence on designs, on production of quantities and cost estimates and finally on execution of the work. Thus, high responsibility rests upon those organizing the surveys and investigation.

This chapter presents requirements on performing surveys associated with the road design process.

Technical requirements for planning and design of roads are outlined so that survey services are uniform and standardized.

The project's Geodetic/Geomatic Engineer is responsible for identifying the appropriate survey data requirements (type of data, area of coverage and level of accuracy), selecting the method of data collection and obtaining the survey data.

Factors that should be considered when determining and deciding the survey data collection method are:

i. Size and scope of the project
ii. Time requirements to move from data collection to the start of design
iii. Estimated data collection cost
iv. Level of accuracy and detail needed.

4.1.1 UNITS OF SURVEY MEASUREMENTS

Metric system of measurement shall be used and all distances and heights shall be in metres following the best engineering and construction practices. Angular measurement shall be in degrees, minutes and seconds.

4.1.2 DATUM AND DISTANCE MEASUREMENT

Coordinates shall be based on the Ghana National Geodetic System (GNGS) unless otherwise authorized. Levels shall be referred to Mean Sea Level (MSL) and related to a Local National Benchmark unless otherwise authorized. All staked distance must be horizontal.

4.1.2.1 SURVEYING CONTROL POINTS (PILLARS)

When starting any road construction, the surveyor's first job on site is to establish accurate surveying control points (primary and secondary) that can be used throughout the project.

A surveying control point is a mark or monument that has an established **horizontal** and **vertical** position by way of being surveyed and is identifiable on the ground. They are very important and once established, need to be protected from being damaged or disturbed. If they do get disturbed and moved from their original surveyed position, a new survey will need to be completed to re-coordinate its new position.

It is important that these controls are accurately established, since they are likely to be used as basis for other surveying on site.

4.1.2.2 HORIZONTAL CONTROL PILLARS

This type of control survey is used to measure the position of the points in the horizontal plane. The surveyor will measure and record the coordinates of the points in the x and y directions. Typically, horizontal control surveys are used to establish the horizontal position of points in a survey network and also used to measure distances and angles between points and to establish the relative positions of points in a plane.

A system of horizontal control pillars (monuments or beacons) originating from and closing upon existing Ghana National Geodetic Control shall be established at an appropriate location.

Standard survey control establishment, coordination procedure and standard accuracy are to be followed with the following limitations to be adhered to:

i. Pairs of inter-visible primary survey control pillars for optical equipment shall be established at a maximum interval of 1.0km, at stable and secure locations, preferably close to the boundary of the road reserve. The primary control pillars can be established by Static/Real-Time Kinematic (RTK) GNSS equipment surveys and/or traverse, triangulation, or trilateration surveys from established national horizontal control monuments.
ii. The secondary control pillars should generally not be more than 300m apart along the baseline of the road and shall be inter-visible for optical equipment.
iii. Standard Monuments shall be constructed as shown in **Figure 4.1**.
iv. Every survey control pillar shall be surveyed by appropriate self-checking survey techniques and principle of “working from the whole to the part” adhered to.

4.1.2.3 VERTICAL CONTROL MONUMENTS/BENCHMARKS

This type of control survey is used to measure the elevation of the points in the survey area. The surveyor will measure and record the coordinates of the points in the z-direction.

Vertical control pillars should consist of levels run in circuits originating from and closing upon the National primary levelling benchmarks.

Standard benchmark establishment and vertical elevation difference closure accuracy are to be adhered to with the following limitations to be maintained:

i. The elevation of each primary and secondary control monument shall be determined using the appropriate methods and to standard closure accuracy.
ii. Other benchmarks as deemed necessary shall be established at an interval of maximum 150m, depending on the terrain along the line close to the road reserve boundary, and at all major structures (e.g. bridges, box culverts etc). Standard Benchmark is shown in **Figure 4.1**.
iii. Every Benchmark is to be checked, levelled by a forward run and a subsequent backward run forming a closed "loop".
iv. The accuracy must meet the standard level of accuracy given in **Table 4.1**. The maximum permissible error of closure is determined by **Equation 4.1**.

Table 4.1 Required accuracy for levelling

K (km)	0.30	0.40	0.50	0.60	0.70
C (cm)	± 0.55	± 0.63	± 0.71	± 0.77	± 0.84

$$C = \pm\sqrt{K} \tag{4.1}$$

Where;

C = maximum permissible error of closure in centimetres,
K= distance between benchmarks in kilometres

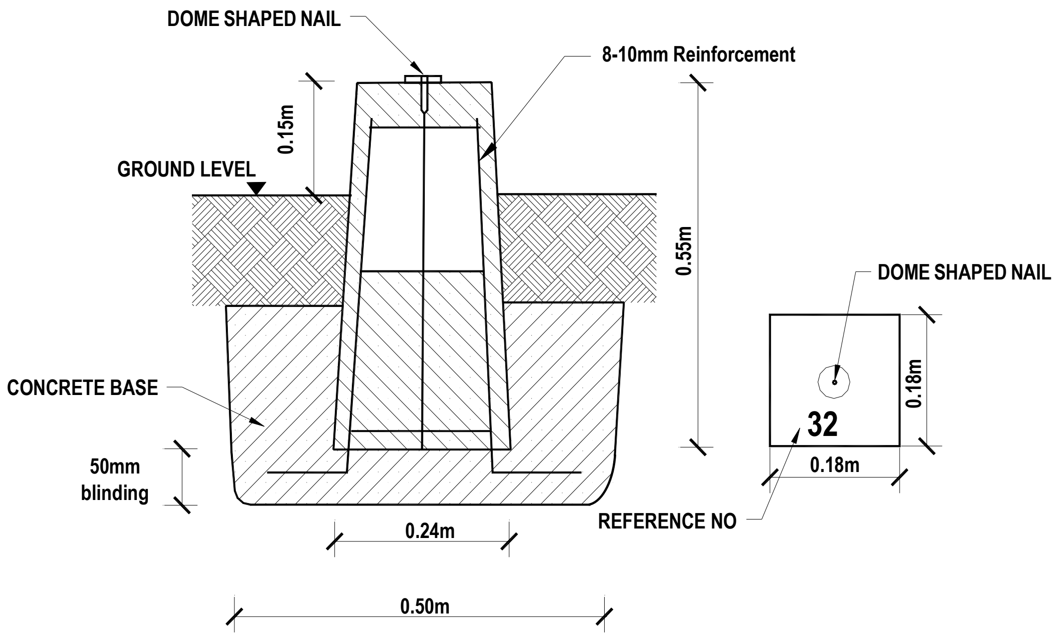


Figure 4.1 Typical Benchmark (Concrete beacon)

4.1.3 TYPES OF SURVEY

The type of survey, map scale, and contour interval shall be selected in each case to interpret the character of the terrain most suitably for the purpose, and the tolerance or permissible error shall be prescribed in each instance. Detailed requirements will be presented in the terms of reference for each project. The minimum width of the survey corridor should be the road reserve width plus 2.5m on each side.

The road surveys are classified in terms of their purpose into three general groups, namely Reconnaissance Survey, Preliminary Survey, Final Location Survey and Detail Survey.

4.1.3.1 RECONNAISSANCE SURVEY

The objective of this survey is to examine the general characteristics of the area with a view to determining the possible alternative routes (refer to **Section 3.3**) which might serve the purposes for which the road is intended.

4.1.3.2 PRELIMINARY SURVEY

The preliminary survey is conducted for the purpose of collecting all the physical information which affect the proposed route profile/or improvements to a road.

The survey results is a paper/digital location and alignment that defines the line for the subsequent final location survey. The paper/digital location and alignment should show enough ties of the existing topography to permit the survey team to peg (set-out) the centreline.

In many cases field details for final design may also be obtained economically during the preliminary survey phase.

Two approaches are available for preliminary survey mapping; aerial surveys and ground surveys, either separately or in various combinations.

A preliminary map is prepared on the basis of the data collected by ground survey and plotted on a paper and a plot of the baseline and all planimetric detail. On this map, the Horizontal alignment is plotted.

4.1.3.3 FINAL LOCATION SURVEY

The final location survey serves the dual purpose of permanently establishing the final centreline of the road in the field and collection of necessary information for road design and preparation of detailed working drawings.

All beginning and end of circular and transition curves should be fixed and referenced.

The final centreline of the road should be suitably staked. Stakes should be fixed at 25m intervals in plain and shorter intervals for rolling/hilly and mountainous terrain. In the case of existing roads, point marks or nails should be used instead of stakes.

Levels along the final centreline should be taken at all staked stations and at all breaks in the ground.

Cross sections should be taken at 25m intervals in the case of flat and shorter intervals in rolling/hilly and mountainous terrain.

4.1.4 SURVEY DATA

The use of proper field procedures are essential in order to prevent confusion in generating the final site plan map. Collection of survey points is a meaningful pattern that aids in identifying map features.

Survey data for road design purposes shall include:

i. Planimetric features (roads, buildings, etc.),
ii. Ground elevation data points needed to fully define the topography,
iii. Utilities
iv. Detailed sketches of facilities and other features that cannot be easily developed (or sketched) in a data collector.
v. A field book sketch, photographs or video of features within the survey corridor are essential elements for proper translation of field data.

Topographic survey data shall be saved in acceptable electronic formats for future reference and actions.

4.1.5 BASIS FOR DEVELOPING MAPS AND DIGITAL TERRAIN MODEL

A variety of survey methods are used to develop maps and the terrain models for projects. The technique employed is a function of the following:

i. type of survey equipment,
ii. the detail required, and
iii. specified elevation accuracy for all physical features such as bridge crossings, streamlines etc.

The Global Navigation Satellite System (GNSS) equipment (which uses either GPS, GLONASS, BEIDOU, GALILEO etc or a combination of these systems), LiDAR (Terrestrial and Airborne), Total Station, Unmanned Aerial Vehicle (UAV), among others whose outputs should be compatible with a CAD software are used.

4.2 METHODS OF SURVEYING DATA COLLECTION

There are primarily three (3) methods of surveying data collection depending on project type and scope namely:

i. Aerial Surveying
   - Aerial Photography
   - Light Detection and Ranging (LiDAR) - Airborne and Terrestrial
ii. Ground Surveying using:
   - Global Navigational Satellite System (GNSS) equipment
   - Total Station
   - Level Instrument
iii. Combination of (i) and (ii).

In most cases, a Computer-Aided Design (CAD) software may be required for design and visualization.

4.2.1 AERIAL SURVEYING

Aerial surveys utilize photographic, LiDAR, electronic, digital, or other data obtained from an airborne platform. Photographic data processed by means of photogrammetry and LiDAR processing using Assisted Global Positioning System (AGPS) and Inertia Measurement Unit (IMU) data represent the principal applications of aerial surveys to be discussed under this section. Aerial survey data combined with field survey data will produce high precision mapping and meet the required accuracy standards.

Aerial photogrammetry is the science of deducing the physical dimensions of objects on or above the surface of the Earth from measurements on aerial photographs of the objects. The result produces the coordinate (X, Y, and Z) position of a point, a planimetric feature, and a graphic representation of the terrain from a Digital Terrain Model (DTM).

The processes of detailed survey, alignment design and setting out are time consuming, especially if changes to the alignment are made later owing to unforeseen ground conditions or changing design criteria. The use of photogrammetry can speed up these procedures and provide the flexibility to allow additional off-site engineering works such as access to borrow pits, spoil disposal sites and slope drainage works to be designed at a later date.

Photogrammetry requires the establishment of a base line traverse and the commissioning of aerial photography at a scale between 1:5,000 and 1:30,000. This method of data collection is preferable for mapping and provision of DTM.

It is more cost effective for huge project sizes and for mapping of urban and big cities. Photogrammetry is also sufficiently accurate for most applications which do not need greater accuracy. Aerial photographs of scale 1:25,000 from photogrammetry can yield uncontrolled contour mapping at a maximum scale of 1:5,000, with contours at 5mr intervals. However, if a project is small, has dense foliage, or requires only mapping of limited features, a field survey with a GNSS equipment or Total Station is the logical choice. Moreover, some fieldwork like fixing wing points, primary control pillars, and GNSS equipment or Total Station surveying will be required in order to accomplish aerial photography.

Elevations of photogrammetric DTM points on hard surfaces are accurate to within ±60mm. If more precise vertical accuracy is required, such as for road pavement elevations or if obstructed views occur, photogrammetric data should be supplemented with ground survey elevations, otherwise use ground survey data only.

The scale of photography is an important factor to consider in the reliability and ground resolution of the interpretation. **Table 4.2** gives guidance and indicates the optimum scales of photography required to perform various desk study and design tasks.

Table 4.2 Aerial photo scales for various project tasks

Task Activity	Optimum Air Photo Scale
Feasibility Study:	
Route corridor identification	1: 20,000 - 1: 30,000
Terrain classification	1: 15,000 - 1: 25,000
Drainage/Drainage Area mapping	1: 20,000 - 1: 30,000
Landslide hazard mapping	1: 10,000 - 1: 20,000
Contour Mapping for preliminary estimation of quantities	1: 15,000 - 1: 25,000
Preliminary design:	
Detailed interpretation of chosen corridor(s) for geotechnical purposes. Ground model (Contour) for preliminary alignment design	1: 10,000 - 1: 15,000
Detailed design:	
Ground model (Contour) for detailed alignment design	1:5,000 – 1:10,000

4.2.1.1 AERIAL PHOTOGRAPHY (PHOTOGRAMMETRY)

This is a low flight photographic mission along the route corridor of the project area to produce large scale aerial photographs, usually scales varying from 1:5,000 to 1:30,000. One way of carrying out this activity is performing some necessary ground works ahead of the photographic mission. These ground works are discussed hereunder.

i. First, the centre line, left and right-wing points of the selected route corridor will be marked on available small scale topographic map.
ii. The next step will be to navigate using a topographic map, on which the selected route

corridor is clearly marked, and establish on the ground, the centre and wing points with white painted 1.0m by 1.0m square concrete monument with their serial number clearly denoted.

iii. Establish primary control pillars (benchmarks) at an approximate interval of 5 km maximum with two secondary control pillars close to each of the primary control pillars and pre-marking of same for low flight. Every effort should be made to obtain information for any existing control pillar in the project area. If any primary control pillar exists from the GNGS database or project control pillars (benchmarks) from previous Road Agency projects, it is highly recommended that these control pillars (benchmarks) be used for further densification of the project. Recovered control monuments must be evaluated before being used as a basis for new control surveys.

iv. Along an existing road, control pillars are also to be established on head walls of structures (bridge or culvert). This is done by chiselling a cross and marking a square or a circle around the cross on the headwall of structures with a white paint.

These control pillars and the benchmarks should be pre-marked in such a way that they can easily be detected on the low flight photographs and their horizontal and vertical positioning will be determined by ground survey and can be identified on the photographs.

The ground control crew will follow and take observations by using Total Station or GNSS equipment on the already established centre points, wing points, primary control pillars, secondary control pillars (by static method in the case of GNSS equipment) and determine the X, Y, Z co-ordinates, thereby relating the coordinates with the Ghana National Grid System. Wherever difficulties arise for observations by a GNSS equipment, the survey crew can use the Total Station. Due to the required accuracy, the vertical coordinate “Z” is to be determined by a levelling crew using a level instrument. Differential levelling by run and check back will be conducted between consecutive benchmarks.

4.2.1.1.1 ADVANTAGES OF AERIAL PHOTOGRAPHY (PHOTOGRAMMETRY)

Photogrammetric method of data collection and mapping has some advantages over the ground survey. Some of the advantages are:

i. It covers a wider area allowing for comparison and selection of the best route.

ii. Helps to avoid unnecessary disturbance of private or public properties when acquiring the Right-of-Way (ROW) for the final location of the route.

iii. Provides an inventory of surface features showing the land usage, land cover, drainage patterns, sources of possible construction materials etc., and it is most effective for urban surveying.

iv. The processes of detailed survey, alignment design and setting out are time consuming, especially if changes to the alignment are made later owing to unforeseen ground conditions or changing design criteria. The use of photogrammetry can speed up these procedures and provide the flexibility to allow additional off-site engineering works such as access to borrow pits, spoil disposal sites and slope drainage works.

v. Photos can be used to convey information to the general public, and other local agencies.
vi. Photogrammetry can be used in locations that are difficult or impossible to access from the ground.

4.2.1.1.2 LIMITATIONS OF AERIAL PHOTOGRAPHY (PHOTOGRAMMETRY)

The photogrammetric method of data collection has some limitations, which restrict the data acquisition and compel the use of ground survey method. These conditions happen when:

i. Smoke and mist occur over the surveyed areas during aerial photographing.
ii. Survey details are covered by dense growth of evergreen forests and other features within the right-of-way.
iii. The method is relatively expensive for small scale work.
iv. The level of accuracy is not high as compared to ground survey.
v. Identification of planimetric features is difficult or impossible (e.g., type of kerb and gutter, size of culverts, type of fences, and information on signs).
vi. Underground utilities cannot be located, measured, or identified.

4.2.1.2 LIDAR

LiDAR stands for Light Detection and Ranging. It is a remote sensing technology that uses laser beams to measure distances and create high-resolution 3D maps of the environment. LiDAR works by sending out laser pulses that bounce off objects and return to the sensor, allowing for the measurement of distance, shape, and orientation of objects. LiDAR data can be collected through Aerial and Terrestrial techniques and can be used for a variety of applications, including topographic mapping, urban planning, agriculture, forestry, and even self-driving cars. LiDAR has become an increasingly popular tool for data collection due to its high accuracy, precision, and ability to capture data in real-time.

4.2.1.2.1 AERIAL LIDAR DATA COLLECTION

Aerial LiDAR is an optical remote sensing technology that measures properties of scattered light to find range and/or other information of a distant target.

Aerial LiDAR data is collected on board the aircraft during flight. Base station information must be collected on the ground during the flight mission. These data provide the input necessary to provide initial geo-referencing. Swath to swath calibration is then performed to refine the relative accuracy of the resulting point cloud. To achieve high levels of accuracy and quality control, application of ground control is applied in the data calibration process.

Elevation data is converted from ellipsoid to orthometric values, completing the process.

A classification process follows which identifies the type of return, (for example bare-earth, water, vegetation, structure, etc.). The classification process typically includes automated and a final manual editing process. The automated classification routines are best accomplished with highly sophisticated software that provide for user inputs that modify algorithms for different return densities and land cover types. A final manual editing process is necessary to assure the required quality level of the data point classification. The result produces the coordinate (X, Y, and Z) position for each return, called a point cloud. Point clouds can be used to generate a DTM, Digital Elevation Model (DEM), vegetation clouds or may be used as a source from which to extract planimetric map features.

LiDAR is best for fast and detailed collection of 3-Dimensional (3D) point clouds of the earth for the production of true orthophotos and DTMs. Base Resolution LiDAR is helpful in planning level documents. Traditional orthophoto generation does not use a Digital Surface Model (DSM) and often suffers from obscured areas and indistinct edges. By improving the surface modelling, more accurate, true orthophotos can be produced. LiDAR is best used for increased accuracy on a project. The use of Aerial LiDAR and/or photogrammetry will still require limited field survey methods, especially for items not visible from the sky (i.e. manhole depth, underside of bridge/culvert, etc.)

4.2.1.2.2 TERRESTRIAL LIDAR DATA COLLECTION

Terrestrial LiDAR uses lasers to make measurements from a tripod or other stationary mount, or a mobile surface vehicle. The term LiDAR is sometimes used interchangeably with laser scanning but is more often associated with the airborne method (performed from an airplane, helicopter, or other aircraft).

Terrestrial LiDAR (a specialty survey) has the potential to acquire millions of survey points in a short time, especially in dangerous areas that are not conducive to traditional methods of data collection. Every detail of even a bridge that is visible can be picked up from one short field trip with Stationary Scanning or Mobile Scanning without inverted survey rods, negative sign errors, requisite field sketches, or climbing on and around pilings or bents. While the scanning collects the data very quickly, the post-processing can be very difficult and time consuming.

Mobile Scanning also known as Mobile Terrestrial Laser Scanning (MTLS) is an emerging technology that uses laser scanner technology in combination with GNSS and other sensors to produce accurate and precise geospatial data from a moving vehicle. MTLS platforms may include sport utility vehicles, pick-up trucks, hi-rail vehicles, boats, and other types of vehicles. Data collection on 33km of road per day is achievable.

Stationary Terrestrial Laser Scanning (STLS) is commonly referred to as Tripod Scanning.

Stationary Scanning has many of the same advantages of Mobile Mapping but is generally applied to smaller projects or spot improvements.

Data collected using LiDAR have both advantages and disadvantages when compared with ground survey methods.

4.2.1.2.3 ADVANTAGES OF LIDAR

i. Like aerial photography, LiDAR data sets provide a permanent record of the existing terrain conditions at the time of aerial survey.
ii. The information extraction may be limited to bare-earth terrain or extend to other data on an as needed basis. It is possible to extract planimetric data from LiDAR as well. Vegetation may be extracted as 3D points, (or cloud data), defining the vegetation extents in 3D space. The vegetation classed points can also be sub-classified based on height which may be useful for identifying line of sight issues.
iii. The information extracted from a LiDAR point cloud provides more detailed information to designers and environmental personnel with respect to topography. It can also offer 3D point cloud visualization opportunities that could prove useful for line of sight analysis and alignment study.
iv. Data points in a LiDAR point cloud are geographically referenced by means of GNSS/IMU technology. Vertical values for points are initially derived on the ellipsoid. The most current geoid information is then applied to the elevations to arrive at orthometric values. There is no initial least squares adjustment application as required for the relative orientation of photographs. Secondly, there is no interpolation of positional values by visual means; therefore, the relative accuracy is higher than that which can be achieved photogrammetrically. It should be noted that final achievable absolute accuracy is dependent on the application of aerial project control that has been tied to the primary control network.
v. Aerial LiDAR can be used in locations that are difficult or impossible to access from the ground.
vi. If collected as a stand-alone data set, (without aerial photography), it can be collected at any time of day or night.
vii. Faster data collection: LiDAR systems can cover large areas quickly and efficiently, making them ideal for mapping projects that require fast data acquisition.
viii. Safe and non-invasive: LiDAR is safe, non-invasive, and doesn't emit radiation, making it suitable for mapping sensitive areas like historical sites and archaeological ruins.

4.2.1.2.4 DISADVANTAGES OF LIDAR

i. LiDAR data is dependent on Airborne GNSS technology, and therefore accuracy is limited to the accuracy of the Airborne GNSS solution and applied geoid model until calibrated to the project ground survey control. This makes it more dependent on low satellite Positional Dilution of Precision (PDOP) levels than aerial photography. It should be noted that conventional ground survey methods using appropriate elevation procedures, still provide the most accurate measurements.

ii. LiDAR must be collected in appropriate weather conditions. While not as demanding as aerial photography, there must be no rain, fog or smoke between the sensor and the ground. While LiDAR has more opportunity to provide ground data than photogrammetry in wooded areas, it does not penetrate full cover. Heavy vegetation canopy may completely obscure the ground.

iii. Processed LiDAR data sets are very large. Point clouds delivered in LAS or ASCII format as project source data must be tiled to manageable file sizes.

iv. The type of material used for construction of fences, buildings, or other man-made features is not interpretable from aerial LiDAR.

v. ROW and property boundary monuments cannot be located, measured, or identified.

4.2.1.3 GROUND CONTROL FOR AERIAL SURVEYS

Aerial survey data must be referenced to ground control pillars in order to maximize the absolute accuracy achievable for the aerial data. This is achieved by survey crews establishing photo ground control pillars within the project area. Targets are placed over ground control so that the location of the point is easily identified on the imagery. The field measurement of the horizontal and vertical elevation (X, Y and Z) of the control pillars will be used in the downstream processes of photogrammetry and/or point cloud calibration to register the data sets to field survey values. Elevations (Z), must be provided at surface grade. If a target is laid over a monument that is below grade, the offset elevation must be applied to the elevation since the aerial control target will be measured at surface grade. **Figure 4.2** shows monument targeted below grade.

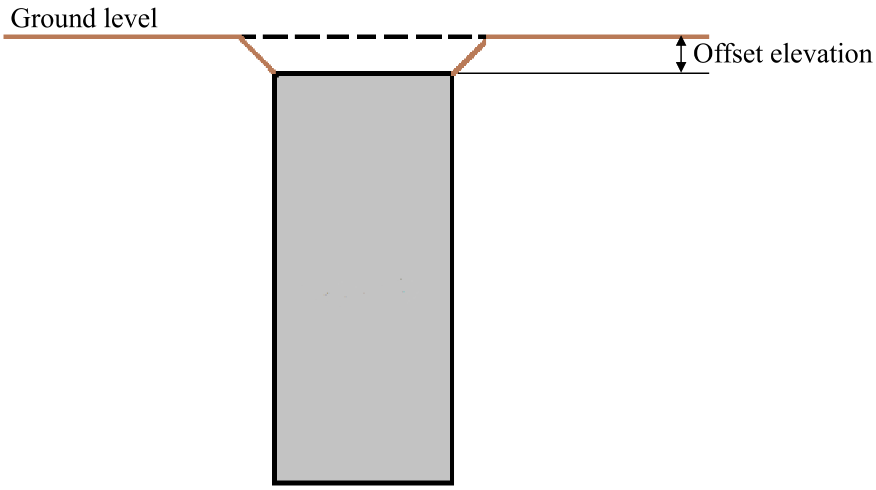


Figure 4.2 Monument below grade

4.2.1.4 GROUND CONTROL MONUMENTATION FOR AERIAL PHOTOGRAPHY

Survey crews establish ground control pillars for aerial photography. Targets are placed over the control pillars on the ground so that the location of the point is easily identified in the aerial imagery. The aerial crew will be responsible for the targeting of control pillars to ensure identification in the aerial imagery.

Photo control pillars typically consist of the following:

i. Centre point control
ii. Wing point control

4.2.1.4.1 CENTRE POINT CONTROL

Centre (i.e., flight line) point control is established as close to the centre of the flight line as possible. Their location and configuration are dependent upon the flight height. For highway work, the closest to the flight line centre that is most often achievable on the ground is on the shoulder of the highway. Whenever possible, Road Agency's primary control monuments that have been previously established on the ground by a primary control survey shall be used for all photo centre control monuments. This allows the aerial control survey to be horizontally and vertically referenced and tied directly to the primary control established on the ground as the framework for the survey control network without having to install additional monuments. This also greatly reduces the amount of field surveying needed to establish photo ground control since the primary control monuments need only to be targeted.

For projects where no Road Agency primary control monuments have been previously established on the ground, the aerial centre control pillar shall be monumented with a concrete pillar and stamped with the appropriate aerial control pillar number or name. In areas where concrete pillar monument is not suitable or desired, the monument shall consist of a material that when set solidly into the ground will prove to hold the required minimum horizontal and vertical accuracy tolerance for the aerial control survey.

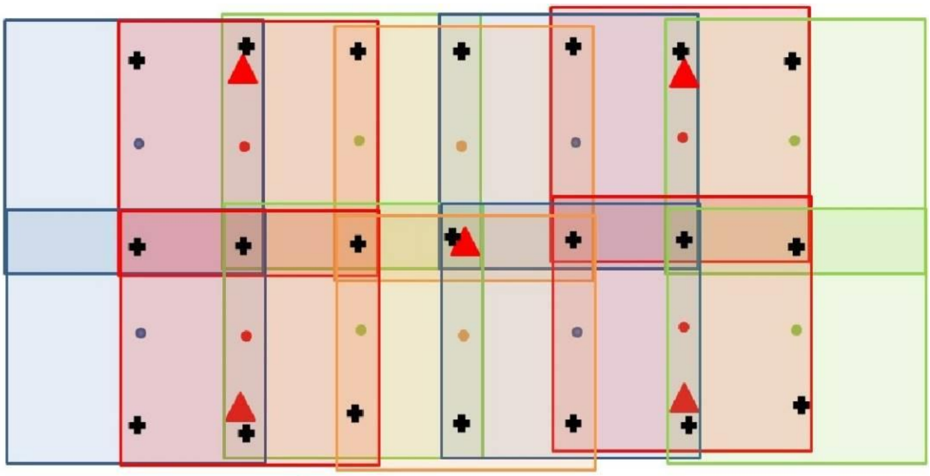
4.2.1.4.2 WING POINT CONTROL

Wing point control is established at the right or left outer edge of the flight lines. These points become more critical for flight plans that include multiple flight strips run parallel to one another. Their location and configuration are dependent upon the flight plan.

Wing point control shall be monumented and stamped with the appropriate aerial control pillar number or name. In areas where a concrete pillar monument is not suitable or desired, the monument shall consist of a material that when set solidly into the ground will prove to hold the required minimum horizontal and vertical accuracy tolerance for the photo control survey.

4.2.1.5 GROUND CONTROL FOR PHOTOGRAMMETRY

The control pillars must be visible from a minimum of two overlapping photographs. To apply the basic principle of photogrammetry, at least three photo ground control pillars are needed for any single stereo model, (one overlapping pair of photographs) or block of adjoining stereo models. This establishes the spatial relationship between the ground and the model coordinates. One or more additional points are required to determine the accuracy of the model based residual error and to identify any data entry errors. When controlling multiple models or blocks of photography, aerial triangulation is applied serving to bridge control across multiple stereo models by combining their relative orientations with the ground control measurements. **Figure 4.3** illustrates the aerial triangulation concept for a small block of photographs.



Ground control point

Centre points

Photogrammetric tie point

Figure 4.3 Aerial triangulation block concept

The photo coordinates of identifiable points on the ground (i.e. photo ground control pillars) are measured on multiple photographs, (at least two), along with other image locations, or tie points, common to multiple photographs to begin the aerial triangulation process. From these measurements and the camera calibration data, a trigonometric calculation determines the camera (focal point) location and sensor attitude for each exposure. Finally, a least squares adjustment is applied to the entire block, refining relative orientations of each image and registering the block to ground control for absolute orientation.

The aerial triangulation output allows analysis of stereo models using a digital softcopy workstation to produce photogrammetric mapping and terrain modelling. Digital workstations allow the operator to accurately compile and record data in 3D. The aerial triangulation data can also be used in combination with the camera calibration data and DTM to produce orthophotography.

More modern aerial survey acquisitions apply AGPS or a combination of AGPS and IMU technology. This is supported by collection of data at static ground base stations during the aerial survey. AGPS provides additional control to aerial photography by establishing a coordinate value for each photo centre. In addition to AGPS, aerial imagery may be combined with IMU data to provide a more accurate photo centre along with the camera attitude and heading (tip, tilt, swing), also known as direct geo-referencing. For photogrammetry, the direct geo-referencing provides additional input to aerial triangulation process, facilitating more automation. Modern aerial triangulation software automates the selection of photogrammetry tie points. This allows a much larger number of tie points to be incorporated into the aerial triangulation solution improving overall results.

4.2.1.6 AERIAL LIDAR CONTROL PILLARS

LiDAR requires both AGPS and IMU data. These inputs provide a relative positioning solution. A minimum of two base stations should be used to provide a basis of comparison for the repeatability of the solution and redundancy in case of equipment failure. While not necessarily within the mapping boundaries, base stations must be tied to the ground survey network associated with the mapping project. To achieve a 7.6cm vertical accuracy, it is recommended that base stations should not be more than 40km from the sensor at any time during the data acquisition (CDOT, 2021). Application of this technology can reduce the number of targeted control pillars required. This may be helpful by reducing the necessity for Wing Point Control as these points tend to fall beyond the right-of-way and may be difficult to place.

The Geomatic Engineer is responsible for determining control requirements for the aerial survey. Final photo control monument locations, spacing, and configurations for the survey may be influenced by conditions. It is important that the aerial mapping crew and the field survey team work in close coordination to ensure control requirements for the project are met. Additional considerations include the type of sensor employed, the technology applied, and the required positional accuracy of the data. The designee (e.g. project manager, head of survey etc.) shall work closely with the aerial mapping crew when determining how many and where each photo control pillar shall be located.

4.2.2 GROUND SURVEYING

Ground surveying is based on survey methods-specifically, geo-referenced observations taken from survey instruments setup on tripods over fixed control pillars or benchmarks. These methods usually provide the highest accuracy for engineering surveys and are necessary when surface and subsurface utilities must be definitively located and identified.

Topographical ground surveys should use appropriate surveying equipment such as Total Station or GNSS equipment to collect data in respect of road alignment and cross sections and all bridges, culverts and other features that are considered necessary to complete the detailed design and the estimation of quantities.

Topographical ground survey has the capacity of achieving greater accuracy as compared to photogrammetry, hence it is preferred for works which need greater accuracy. It is also more cost effective for small size projects and is appropriate for projects which have dense forests.

An approximate alignment should first be drawn on a topographic base map and/or aerial photographs (hard or soft copy) in the office prior to embarking on detailed fieldwork as indicated in **Section 3.3**. Having the approved route, the Geomatic Engineer, with a survey team, will peg and flag the approximate centreline.

The pegged/flagged road centreline should be identified and staked every 25m maximum depending on the type of terrain. The coordinates are recorded automatically using the Total Station or GNSS equipment. Planimetric features such as road edges, cuts, ditch edges, culverts, hilltops, water crossings, embankments etc. are taken. Topographic survey information is also collected for an adequate distance on each side of the centreline and cross sections at appropriate intervals, depending on the type of terrain.

Each cross section comprises such numbers of points as to enable it to properly define the existing road and such other spots as are required to define the ground shape for an adequate distance beyond the existing construction width. The data are used to generate a Digital Terrain Model (DTM) for the whole road. All pertinent features including buildings, drainage structures details, built up areas, etc. should be recorded for inclusion on the design drawings.

Detailed site investigation and surveys should be carried out for areas susceptible to flooding or landslide and at all recommended new or replacement drainage structure locations including a sufficient length upstream and downstream to the structure. The full requirements for survey data for drainage structures are provided in **Chapter 13**.

New alignments are recommended where inadequate horizontal sight distances and sharp curves exist and wherever the existing route is not to the standards. Therefore, the vertical and horizontal alignments should be given due attention with respect to sight distance, maximum grade, maximum length of grade criteria, and safety. In introducing new alignments, major bridges and drainage structures should be retained as control pillars or as node points on the new centreline wherever they are in good condition. Should there be a need for realignment of the existing road, topographic surveys along the chosen realignment should be established. The centreline of the road is defined at every 25m interval. Topographical cross-sections, extending to the ROW (maximum) are taken.

4.2.2.1 GNSS SURVEYING

Surveying has changed substantially over the years, what used to take months of observation, measurement and geometrical calculations now takes a few hours or days thanks to the introduction of GNSS technology. In fact, the surveying industry was one of the first to utilize GNSS technology, recognizing the potential benefits of the technology. Today, surveying professionals rely on GNSS to provide accurate and reliable data for clients across a wide range of industries and applications. Despite the widespread usage of GNSS technology in surveying, however, it’s not a topic many know about. This section explains the GNSS surveying basics.

GNSS equipment uses a network of satellites, which communicate with receivers on the ground. When a receiver requests data to calculate its location, four or more GNSS satellites will communicate with the receiver, sending the position of the satellite, the time the data was transmitted and the distance between the satellite and the receiver. The information collected from these satellites then calculates the latitude, longitude and height of the receiver. If the receiver is moving, continuous data collection can be used to calculate the changing position of the receiver over time, which can be used to calculate speed. No matter the weather conditions or time, GNSS equipment can triangulate the signal and provide a location.

While most people are familiar with GPS (an example of GNSS) and have used it to some degree on their smartphones or car navigation systems, GPS is a powerful tool for commercial applications. It is particularly useful for the surveying industry. Surveying was one of the first commercial adaptations for GPS for its ability to obtain latitudes and longitudes without the need for measuring distances and angles between points.

In combination with other surveying equipment, like the Total Station, GNSS technology provides valuable information for surveyors to help develop plans and models for client projects.

4.2.2.1.1 HOW TO CONDUCT GNSS SURVEYING

GNSS surveying uses similar technology to nearly any other GNSS application — however, how surveyors use GNSS differs significantly. The primary differences are in two areas, namely, technology and usage.

- **Technology:** Surveyors use more sophisticated technology than typical GNSS applications to increase the accuracy of the data they collect. The receivers used for surveying are significantly more complex and expensive than those you would find in a typical car navigation system, with high-quality antennas and more sophisticated calculation technology.
- **Data Usage:** The data surveyors collect from the GNSS technology is used differently than in a typical navigation system, instead of using location data for navigation, the data is used for measuring distance between two points. These measurements are collected then stored, manipulated and displayed in a Geographic Information System (GIS) for use in a survey model.

The specifics of how to use GNSS to collect data come down to the GNSS surveying techniques that surveyors use. While the basics of GNSS are simple to understand, there are several techniques that surveyors use to make the most of the GNSS measurements they collect. There are three primary methods of GNSS measurement that surveyors use, namely:

i. Static GNSS baseline observation

ii. Real-Time Kinematic (RTK) GNSS observation

iii. Continuously operating reference stations (CORS)

A. STATIC GNSS BASELINE OBSERVATION

A Static GNSS data collection is a technique used to determine accurate coordinates for survey points. Baseline measurements achieve this by recording GNSS observations over time, then processing that data to provide the most accurate result.

The technique works by using two GNSS receivers. These receivers are placed at each end of a line to be measured. The receivers then collect GNSS data simultaneously for at least 20 minutes, the exact duration of the observation period varies based on how long the line is and how accurate the measurements need to be. Once all of the data is collected, a special type of software is used to calculate the difference in position between the two receivers.

This GNSS surveying technique is basic but highly useful and accurate, especially when measuring particularly long distances. Because the GNSS data is collected over a long period of time, and the observations are collected at the same time at each end of the baseline, the natural distortions that occur in GNSS signals cancel each other out. Generally speaking, the accuracy of Static GNSS Baseline measurements are one part per million, meaning that a 30km distance can be measured with about 30 mm of uncertainty.

B. REAL-TIME KINEMATIC GNSS OBSERVATION

Real-Time Kinematic (RTK) observations are similar to baseline methods in that they are used to measure distances between a base station and a second receiver. The difference, however, is that instead of measuring the location of two points over a long period of time, RTK Observations use multiple points in quick succession.

Like the baseline method, the RTK method uses two receivers, one being a static base station. The other receiver is the Rover Station, which moves to multiple positions during the measurement period. The position of the Rover Station is collected within a few seconds and stored. Once the measurement period is complete, this data is stored and used as survey data.

RTK observations are nearly as accurate as the baseline technique, though they are limited to a range of about 20km. This method maintains a high level of accuracy by collecting data at the Base Station and the Rover Station simultaneously and correcting data in real time. The exact position of the Base Station is known, so any variations can be used to correct the position of the Rover Station in real time. This method, therefore, can quickly gather survey data for smaller areas.

C. CONTINUOUSLY OPERATING REFERENCE STATIONS

Continuously Operating Reference Stations (CORS) operate using the same principles as the other measurement techniques described. The primary difference is that the base station is installed in a permanently known location. This allows measurements to be taken at any point in the area using the permanent base station as a starting point.

With a CORS-based system, receivers can be placed anywhere in the local area to collect data. When data collection is complete, the surveyors can combine the collected data with data from the CORS to calculate positions, correcting any anomalies to obtain an accurate position. In some cases, if multiple CORS are available, receiver data may be compared to the data of multiple CORS to achieve even more accurate results.

CORS are commonly used for major engineering projects that require continuous surveying over a long period of time, some examples include local government projects, mining sites and tectonic plate studies for scientific organizations. Some countries even have CORS systems that cover their entire nation, allowing for more accurate and reliable GNSS positioning anywhere in the country for both commercial and consumer applications.

4.2.2.2 TOTAL STATION SURVEYING

A total station (**Figure 4.4**, **Figure 4.5** and **Figure 4.6**) is an electronic/optical instrument used in modern surveying and building construction that uses electronic transit theodolite in conjunction with Electronic Distance Meter (EDM).It is also integrated with microprocessor, electronic data collector and storage system.

The instrument is used to measure sloping distance of object to the instrument, horizontal angles and vertical angles. This Microprocessor unit enables the computation of data collected to further calculate the horizontal distance, coordinates of a point and reduced level of point.

Data collected from the total station can be downloaded into computer/laptops for further processing of information.

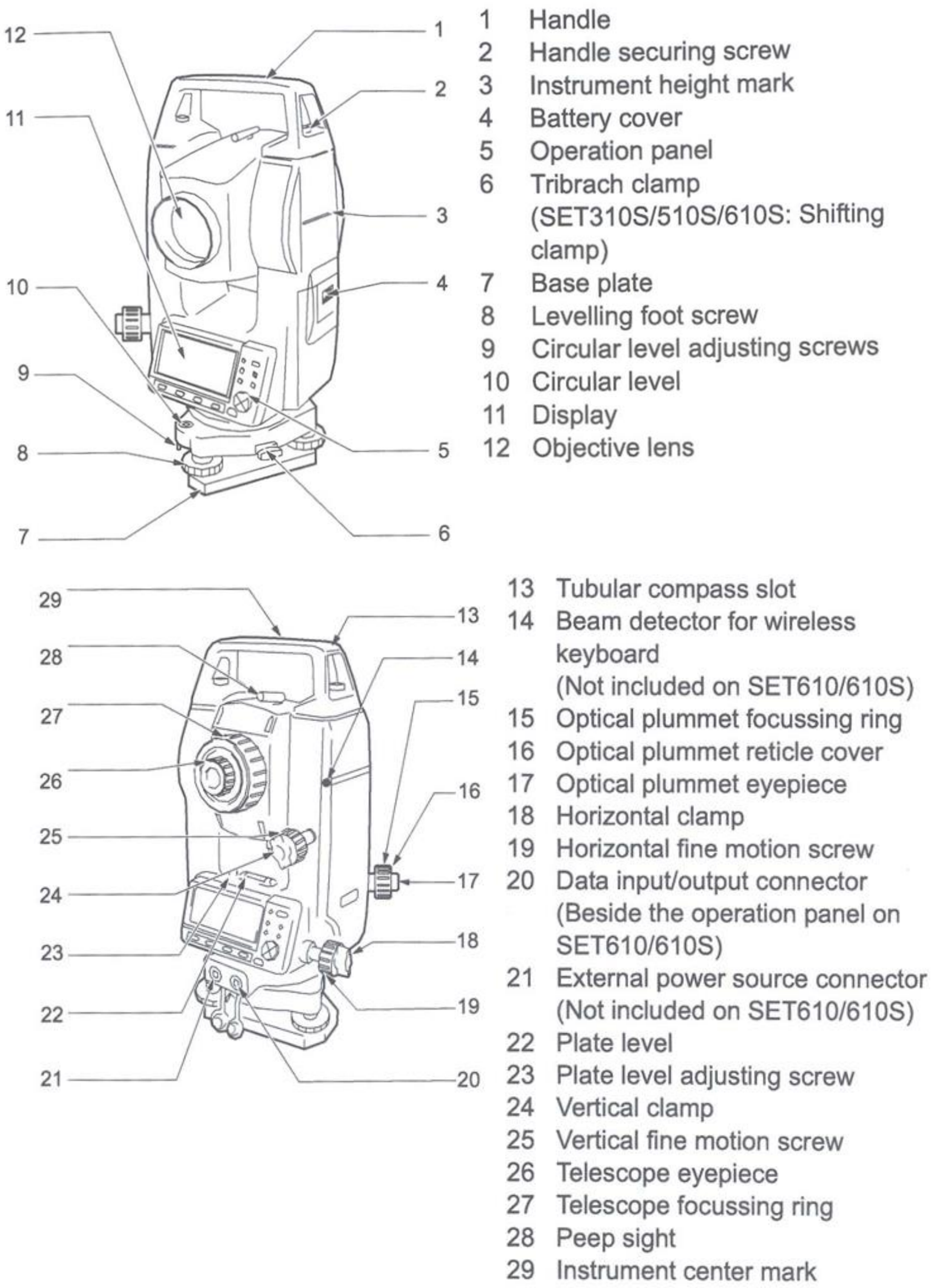


Figure 4.4 Parts of the Sokkia SET 550 Total Station

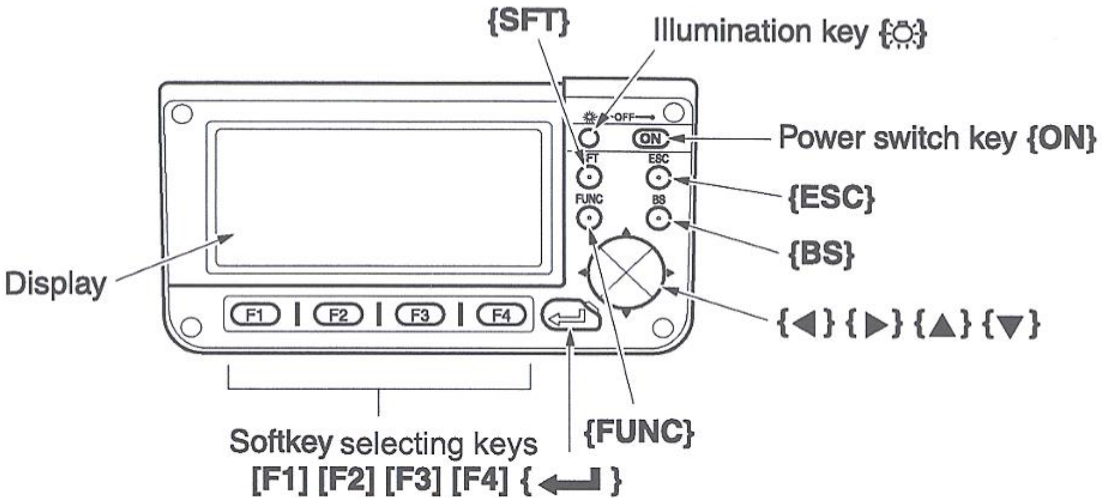


Figure 4.5 Sokkia SET 550 Total Station Keys/Screen

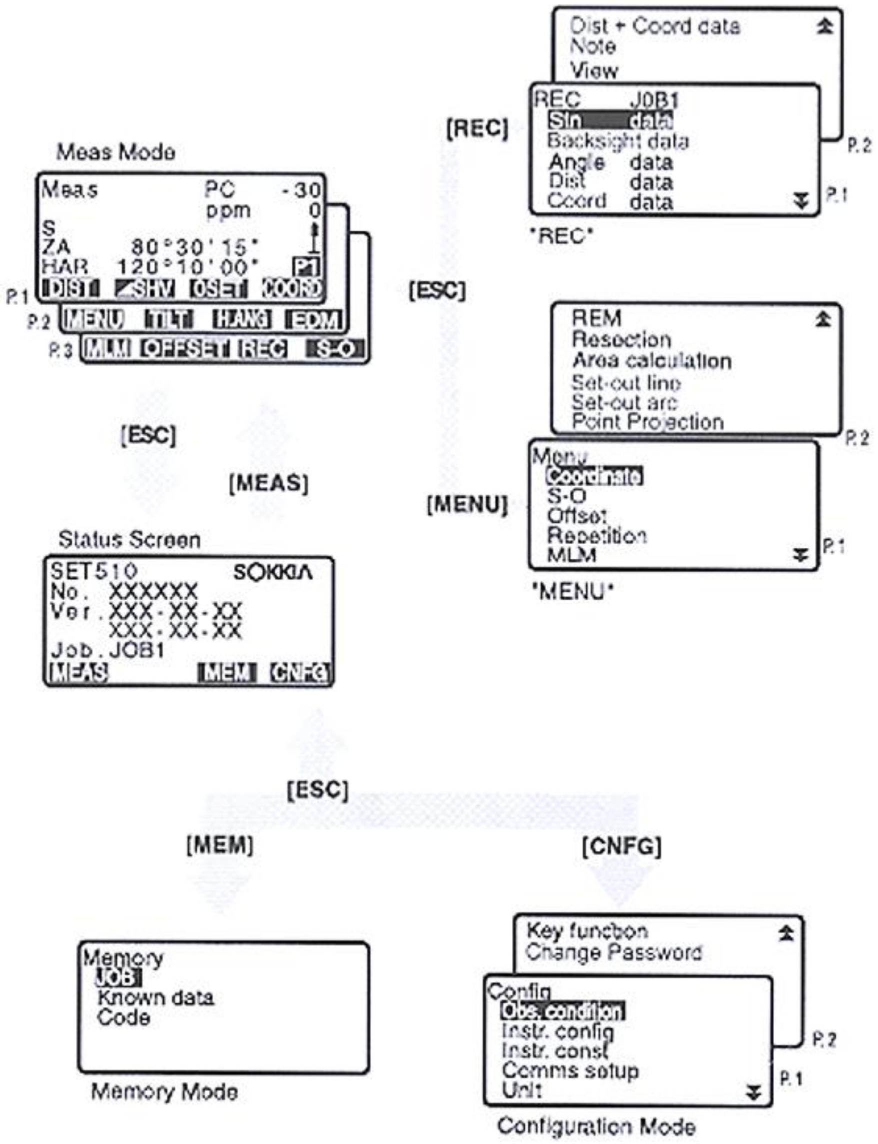


Figure 4.6 Sokkia SET 550 Menu Pages

4.2.2.2.1 LEVELLING OF THE TOTAL STATION

Levelling the Total Station must be accomplished to sufficient accuracy otherwise the instrument will not report results.

Make sure you can see all targets from the instrument station before going through the process.

Step 1: Tripod Setup (Figure 4.7)

- Tripod legs should be equally spaced.
- Tripod head should be approximately level.
- Head should be directly over survey point.

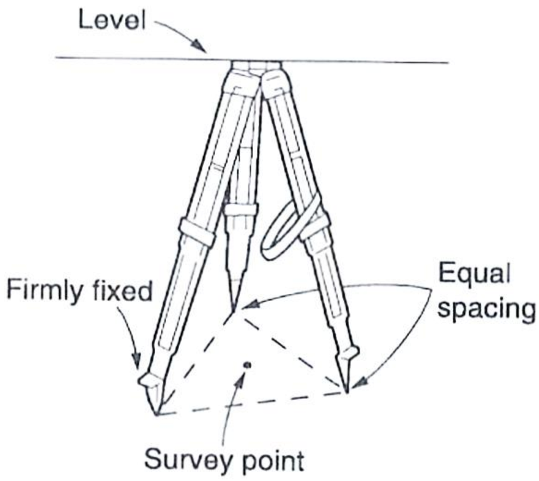


Figure 4.7 Step 1: Levelled Tripod on a Survey point

Step 2: Mount Instrument on Tripod (Figure 4.8)

- Place Instrument on Tripod
- Secure with centring screw while bracing the instrument with the other hand.
- Insert battery in instrument before levelling.

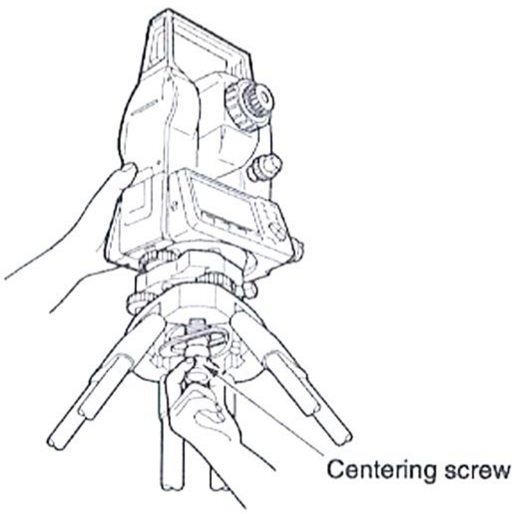


Figure 4.8 Mounted Instrument on a Tripod

Step 3: Focus on Survey Point (Figure 4.9)

- Focus the optical plummet on the survey point.

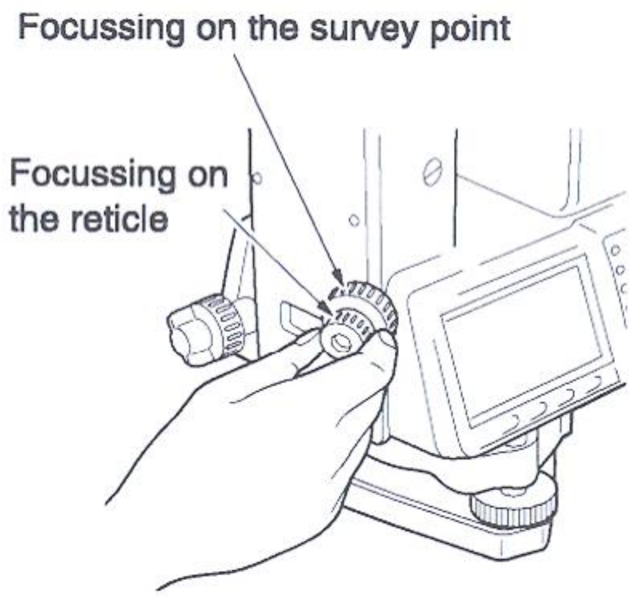


Figure 4.9 Focusing on the Survey Point

Step 4: Levelling the Instrument

- Adjust the levelling foot screws to centre the survey point in the optical plummet reticle.
- Centre the bubble in the circular level by adjusting the tripod legs as shown in **Figure 4.10**.

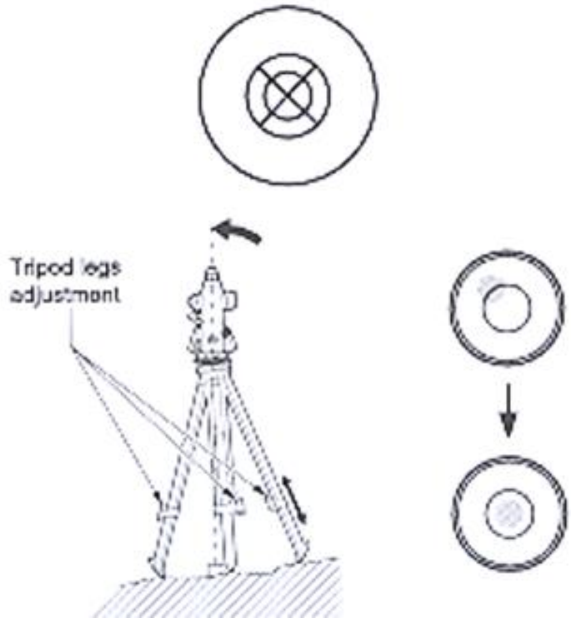


Figure 4.10 Adjusting the Tripod legs to canter the survey points in optical plummet reticle

- Loosen the horizontal clamp and turn instrument until plate level is parallel to 2 of the levelling foot screws.
- Centre the bubble using the levelling screws- the bubble moves toward the screw that is turned clockwise.
- Rotate the instrument 90 degrees and level using the 3rd levelling screw (**Figure 4.11**).

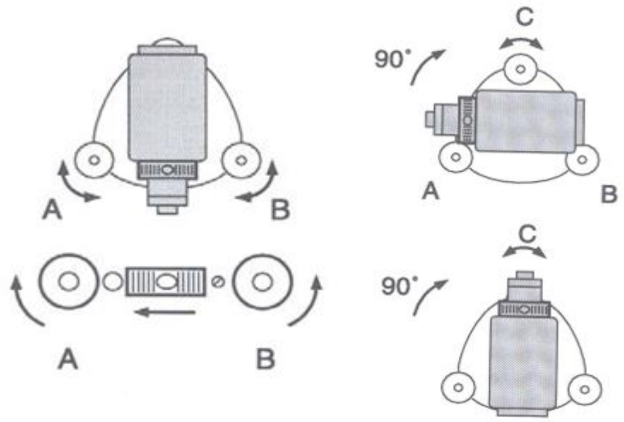


Figure 4.11 Rotating the instrument 90^0

- Observe the survey point in the optical plummet and centre the point by loosening the cantering screw and sliding the entire instrument.
- After re-tightening the centring screw check to make sure the plate level bubble is level in several directions.

Step 5: Electronically Verify Levelling (Figure 4.12)

- Turn on the instrument by pressing and holding the “on” button (you should hear an audible beep)
- The opening screen will be the “MEAS” screen. Select the [Tilt] function.
- Adjust the foot level screws to exactly centre the electronic “bubble”.
- Rotate the instrument 90 degrees and repeat.

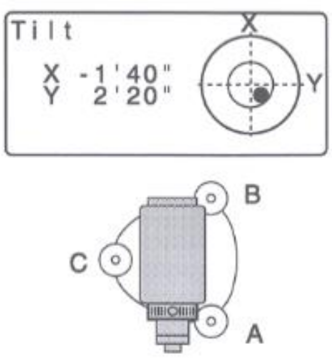


Figure 4.12 Centring Electronical bubble

Step 6: Adjust Image & Reticle Focus

- Release the horizontal & vertical clamps and point telescope to a featureless light background.
- Adjust the reticle (i.e. cross-hair) focus adjustment until reticle image is sharply focused.
- Point telescope to target and adjust the focus ring until target is focused.
- Move your head from side-to-side to test for image shift (i.e., parallax).
- Repeat the reticle focus step if parallax is significant. Refer to **Figure 4.13**.

NOTE: When the instrument operator changes the reticle focus may need to be adjusted

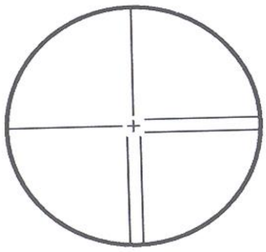


Figure 4.13 Focused reticle (Cross-hair)

4.2.2.2.2 OPERATION OF THE TOTAL STATION

i. Measuring the Height of an Object (Figure 4.14)

- Level the instrument at a site where the target can be viewed through the telescope and the mirror target can be setup directly below the target.
- After **powering on** the instrument, select “REM” from “MEAS” > “Menu”
- Ht = h1 + h2
- h2 = S (sin θz1) (cot θz2) – S (cos θz1)

NOTE: Instrument height does not affect this calculation

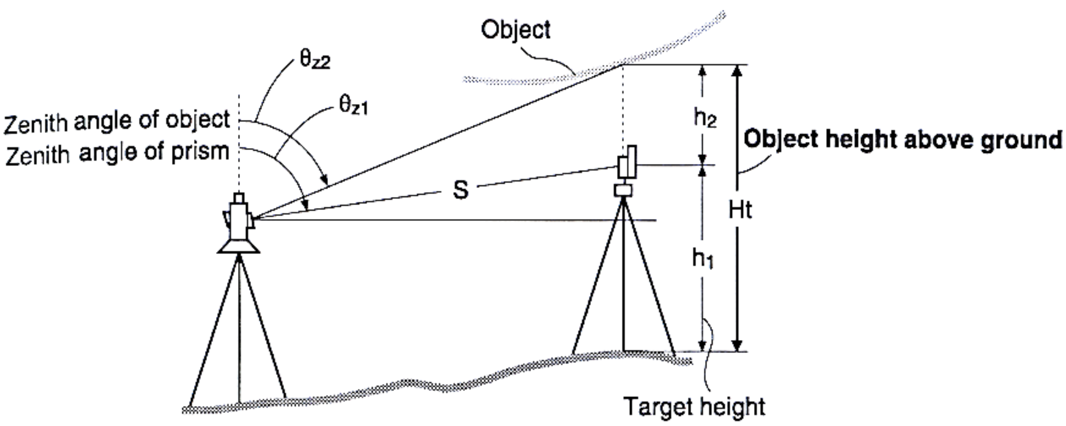


Figure 4.14 Object Height Measuring Process

ii. Measurement of Target Height (Figure 4.15)

- Set the Target Height from “MEAS” > “Menu” > “Coordinate” > “Station Orientation” > “Station Coordinate”
- Set the target height to the measured height of the mirror target. Make sure you use the metric side of the tape measure if working with metric units. You do not have to fill out the other fields for a REM measurement, however, it is good practice to measure and enter the instrument height. After entering the TH and IH make sure you press “OK” (F4) to accept new values.
- Press “ESC” to return to the “MEAS” menu
- Select the “MEAS” > “Menu” > “REM”, sight the mirror target, press [OBS] to measure “S”, then [STOP]
- Sight the object above the target for height measurement
- Select [REM] after sighting the top of the height target, and then [STOP] to stop the calculations.

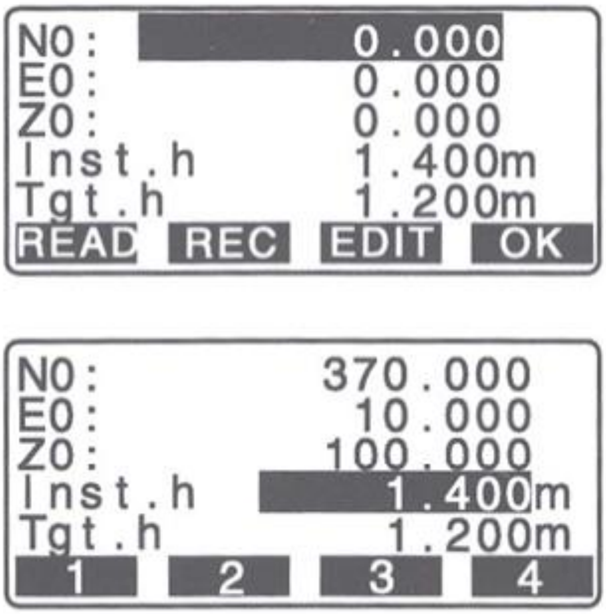


Figure 4.15 Before and After Measurement Reading

iii. REM Screen Results (Figure 4.16)

- To re-shoot the mirror target use the [OBS] on the REM screen.
- Select “STOP” to terminate calculations on the REM command.

Note that after selecting REM the instrument continues to make calculations in case you need to adjust the vertical angle on the height target.

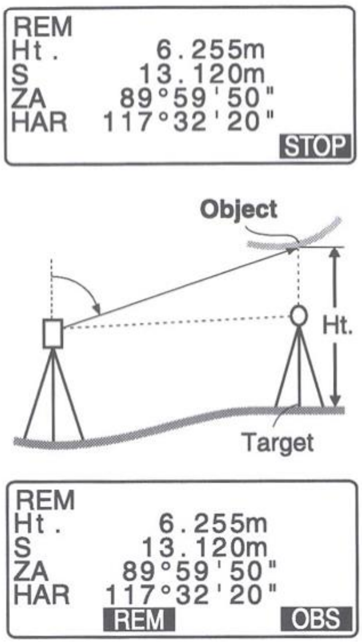


Figure 4.16 Sample REM Screen Results

iv. Trouble-Shooting the REM Measurement

- The only numerical input is the target height so make sure that is entered correctly. When TH is changed make sure you hit the "OK" function key.
- If the instrument is reset (zeroed) TH will be 0.0 so if you make a REM measurement with TH=0 the answer will be underestimated by the actual TH.
- A quick check can be made by using REM on the mirror target – the answer should be the TH.

v. Calibrating the Instrument

- Calibration must be completed before coordinates can be obtained
- 3 possible calibrations:
- Backsight by angle: must know instrument coordinates and have a landmark/target at a known azimuth:
  - Backsight by coordinate: must know instrument coordinates and have mirror target set on a position of known coordinates.
  - Resection (triangulation): must have 3 or more mirror targets established at known 3D coordinates.

vi. 3D Coordinates

- Coordinates may be absolute or relative depending on survey requirements.
- Surveying the area of a mining site would require relative coordinates, therefore, the initial instrument X, Y, Z coordinates may be 5000, 5000, 100

- Surveys that have to match a downloaded aerial photo from the USGS would have to match UTM NAD83 coordinates so the starting point would have to be determined by an accurate GPS receiver.

vii. Calibrate by Backsight by Angle (Figure 4.17)

- Remember that when the instrument is powered on it has a random X,Y coordinate. system: you must align the instrument with your working coordinate system.
- Level the instrument on the desired starting survey marker.
- Make sure that on the last levelling step the optical plummet is cantered on the survey point.
- Measure the target height and instrument height.
- Select [COORD] from the MEAS menu.
- Select “Stn. Orientation” and then “Stn. Coordinate”.
- Edit the “N0”, “E0”, and “Z0” fields to appropriate values (i.e. northing, easting, elevation of instrument).
- Enter the instrument and target height if necessary.
- Select [OK] when done   Backsight by A
- Select “Backsight” and then “Angle” from the menu
- Sight the landmark/target of known azimuth relative to instrument with telescope
- Select “Angle” from menu. Note that the menu displays the zenith angle (ZA) and current horizontal angle (HAR) and is waiting for you to enter the known angle with [EDIT]

Note: if you enter an azimuth angle as “85.4514” this will be interpreted as 85 degrees, 45 minutes, 14 seconds

IMPORTANT! You must select [OK] to accept the angle. Never use <Esc> to leave this screen!

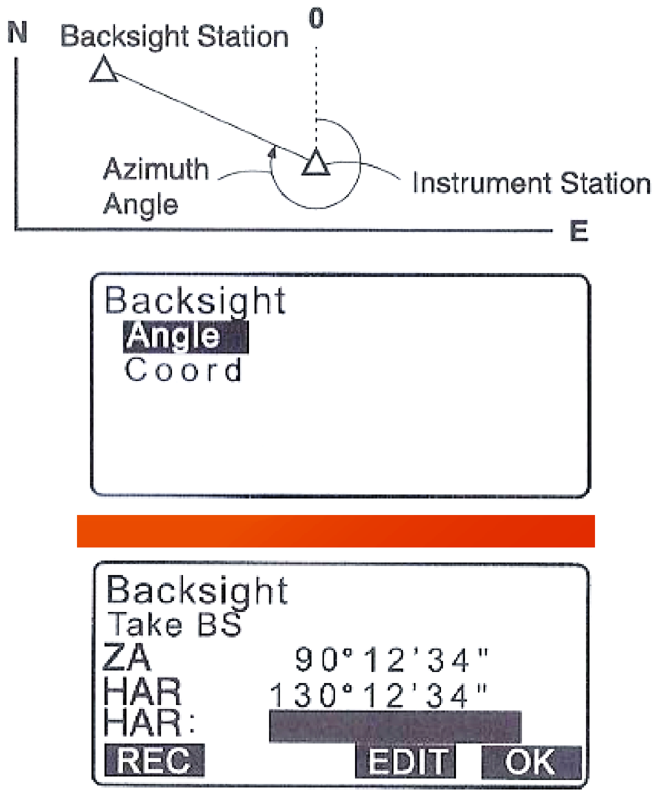


Figure 4.17 Back-sighting by Angle

- NOTE: because the backsight by angle simply sets the instrument horizontal angle encoder to match your desired coordinate system the mirror target is never "shot" by the beam. If you can accurately sight on an object or landmark such as a building corner the mirror target is not needed. Make sure the instrument is "locked" and accurately sighted with telescope before entering the backsight angle.

viii. Backsight

- Because there is no internal statistical measure of how well the backsight angle has been set it is imperative to check the backsight independently:
  - Known point; shoot the target at a position of known X,Y,Z such as a GPS point. The result should be within the resolution of the GPS.
  - Known angle; shoot to a landmark at a known azimuth from the instrument location- the angle should be within the resolution of the instrument.

ix. Backsight by Coordinate (Figure 4.18)

- Use this method when you have 2 known survey points with the instrument established on one and the mirror target on the other survey point
- From the "MEAS" menu select [COORD] and then "Stn. Orientation". Set the instrument coordinates with "Stn. Coordinate" and then select [OK] and return to "Backsight"

- Select "Coord" and then enter the backsight target coordinates (NBS, EBS, ZBS) and select [OK]
- Sight in the target and inspect the "Azimuth" (it should be reasonable for your coordinate system).
- Select [YES] to calibrate. If you don't select [YES] the coordinate system is still random.

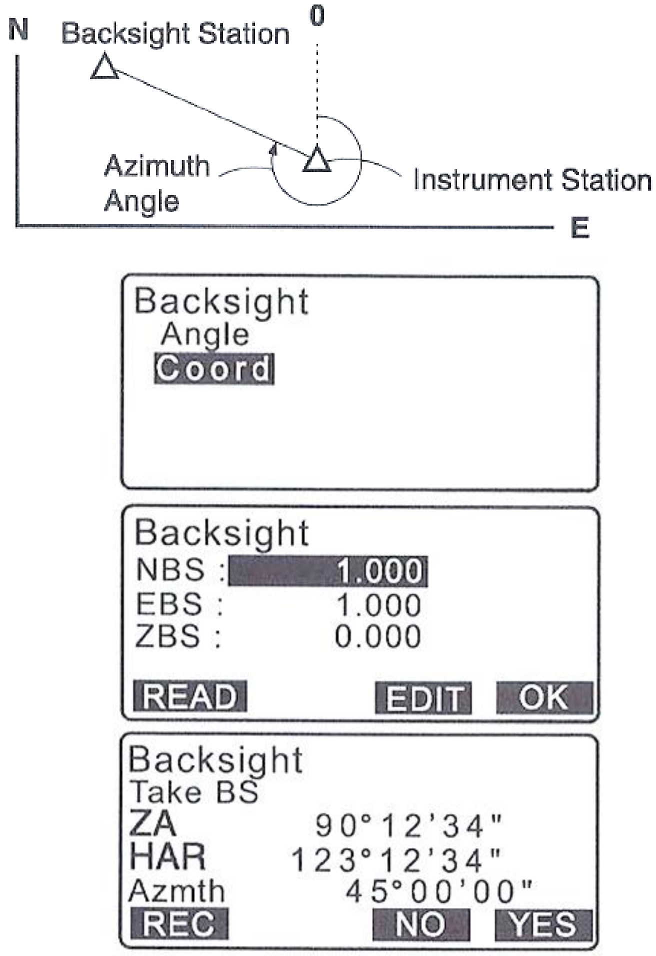


Figure 4.18 Back-sighting by Coordinate

- Always check the calibration of the instrument by shooting the target used for the backsight.
- The resulting X, Y, Z should be within the several cm resolution typical for a TS instrument.
- It is a very good idea to shoot other benchmarks within range to make sure accuracy is within acceptable limits.

x. Resection (Figure 4.19)

- Resection uses 3 or more known target survey points to automatically determine the X, Y, Z coordinates of the instrument.

- This has the significant advantage of not requiring the instrument to be levelled exactly on a survey point- any convenient location where you can sight the targets is OK.
- The ideal geometry is displayed to the right.

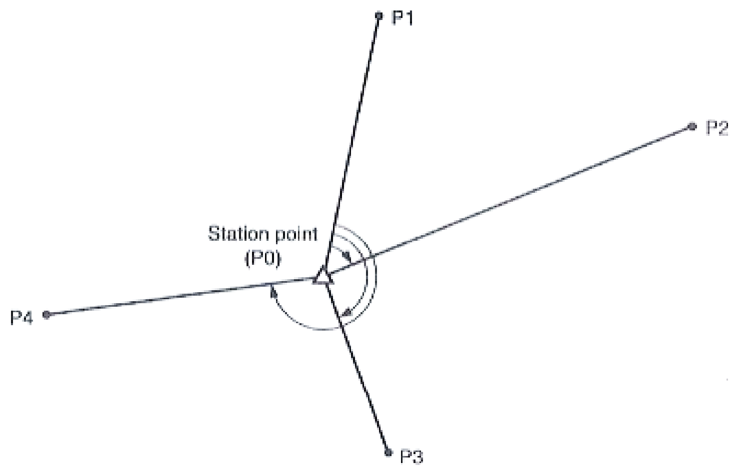


Figure 4.19 Resection Using Four Known Survey Points

- Prior to resection enter survey markers as known points through the “MEM” menu.
- From the “MEAS” menu select “[MENU]” > [RESEC].
- The resection procedure requires that the known coordinates be defined first, and in the order that they will be shot.
- In **Figure 4.20** , the 1st point has been defined and the 2nd point is being entered as shown in. You can use [READ] to read in previously entered or measured points.
- Press the “>” or “<” arrow to move to next or previous point.
- When all points are entered select [MEAS].

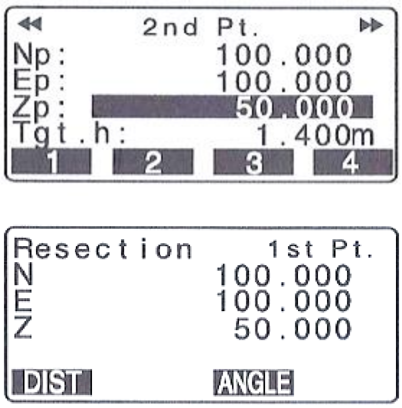


Figure 4.20 Coordinate entry for P1 and P2

- The [MEAS] screen in displays the point being shot – in this example (**Figure 4.21**) the 1st point.
- Choose [DIST] if you are shooting to a mirror target, [ANGLE] if not.

- Select [YES] to accept measurement, [NO] to re-shoot, [EDIT] to change target height.
- The [CALC] option will be displayed when the standard deviation of northing and easting can be displayed.

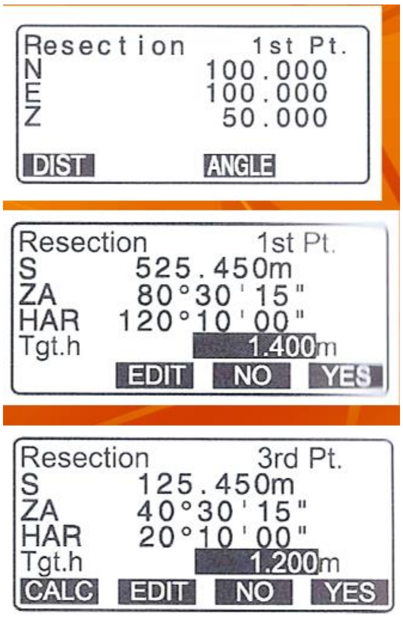


Figure 4.21 Display of Point 1 and 3 being Shoot (Targeted)

- Press [CALC] or [YES] on last point to display the calculated (**Figure 4.22**)instrument coordinates and the standard deviation of easting (σE) and northing (σN). ***Press [OK] to finish Resection, and then [YES] to set the backsight azimuth to the 1st shot point.***
- Press [RESULT] to display the residuals of each shot point- large deviations identify "bad" points.
- If there are no problems press {Esc} to return to main resection screen.
- The standard deviations are a measure of the accuracy. They should be in the range of several cm's for most surveys.

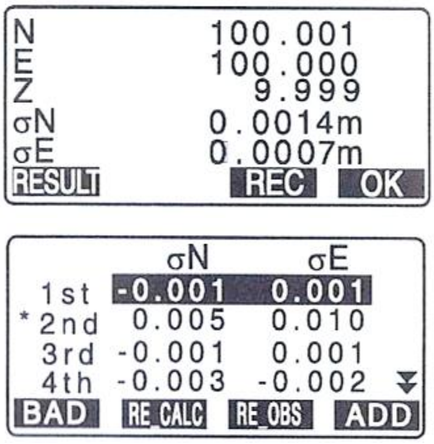


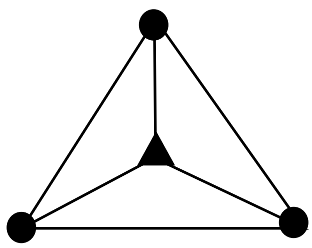
Figure 4.22 Display of Calculated Instrument Coordinates, Standard Deviation and Residuals

- Resection initializes the X, Y, Z coordinates of the instrument. Save this as a point since it represents a surveyed coordinate.
- Once the instrument is calibrated the mirror targets can be taken down and used elsewhere.
- The instrument height should be entered before resection is calculated.
- You can only begin shooting resection point 1 from the resection point #3 or higher coordinate entry.
- Certain Geometries should be avoided:
  - Targets and Instrument should not be arranged on a circle.

xi. Precaution When Performing Resection

In some cases, it is impossible to calculate the coordinates of an unknown point (instrument station) if the unknown point and three or more known points are arranged on the edge of a single circle.

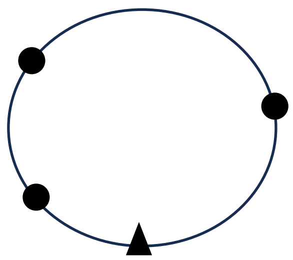
An arrangement such as that shown below is desirable.



▲△ : Unknown point

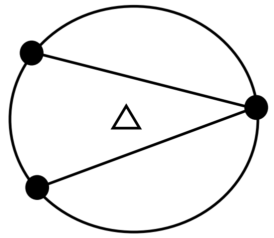
●O : Known point

It is sometimes impossible to perform a correct calculation in a case such as the one below.

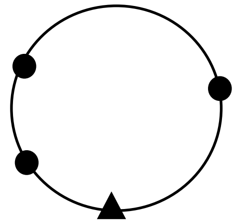


When they are on the edge of a single circle, take one of the following measures.

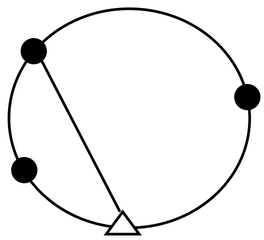
(1) Move the instrument station as close as possible to the centre of the triangle.



(2) Observe one more known point which is not on the circle.



(3) Perform a distance measurement on at least one of the three points.



xii. Coordinate Measurement (Figure 4.23)

- Used to determine XYZ coordinates of target point.
- Make sure the instrument height and target height are already set.
- Make sure backsight/resection have already "locked" the instrument into a mapping coordinate system.
- From MEAS select Menu > Coord > Observation

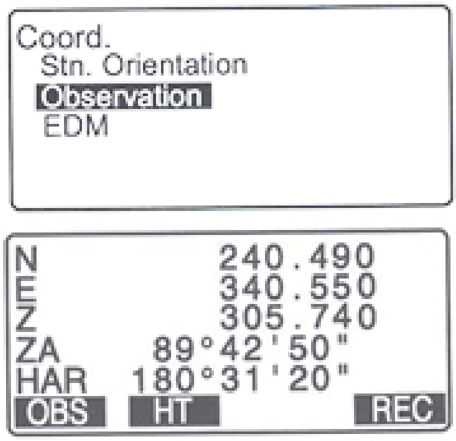


Figure 4.23 Display of Measured coordinates

xiii. Offset: Single Distance

- Single distance offset – used to measure points that cannot be "occupied" by the target.
- Examples: (1) centre of a large tree, (2) centre of a fountain, (3) centre of a building

xiv. Single Distance Offset measurement (Figure 4.24)

Finding it by entering the horizontal distance from target point to the offset point:

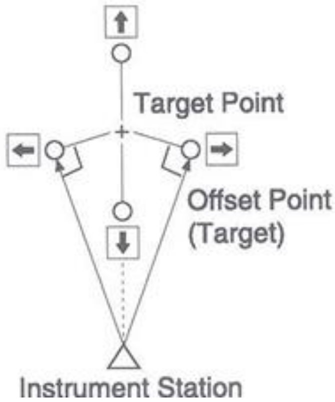


Figure 4.24 Single Distance Offset measurement

- Offset point can be right or left of the target but must be the same distance from the instrument.
- Offset point can be in front or behind target but must be on the same azimuth line.
- In any case the person/team holding the target must have a tape to measure the exact distance (to cm accuracy at least) of the offset.
- The instrument will request an observation to the target first, and then request the offset distance and where the target point is relative to the point of interest (left, right, front, back).

- Note the arrow that indicates that the target is to the left of the survey point by 2 meters (**Figure 4.25**).

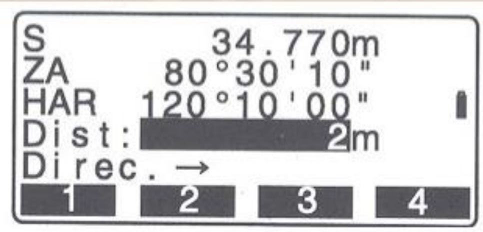


Figure 4.25 Display of Measured 2m offset

4.2.2.3 LEVELLING

The levelling work will start after benchmarks are established at appropriately as described in **Section 4.1.2**. Additionally, temporary benchmarks have to be established on either side of bridge crossings. Benchmarks will be located outside the road limit and where one is visible from the other along the road.

Following the establishment of benchmarks, differential levelling will be carried out. This is a closed loop levelling run between two benchmarks in the form of a run and a check back. This is done to determine the elevations of benchmarks from preceding benchmarks whose elevations are already determined. The misclosure on each previously established benchmark shall be within the prescribed tolerance. Where the difference is outside this limit the run must be repeated.

Profile levelling will be run between each pair of consecutive benchmarks, previously established, and the leveller must close on each successive benchmark as a turning point. For each succeeding length of profile any error, within the allowable, from the preceding length shall be discarded, the elevation of the intervening benchmark, previously established, being accepted, and used for the succeeding length of profile. Where it is not possible to close a day's work on a permanent benchmark in the case of failing light, a sudden storm, etc., a Temporary Benchmark (TBM) shall be established from which the work may be resumed the next time.

4.2.2.4 SURVEYING CREW

In order to handle the ground surveying work for detailed design of road properly, the following survey crews headed by the Geodetic/Geomatic Engineer should be formed and organized:

i. Location Crew
ii. GNSS Crew
iii. Total Station Crew
iv. Levelling Crew

The assignment of the survey crews is given in **Table 4.3**.

Table 4.3 Assignment of the survey crews

Name of Crew	Assignment
Location Crew	• Location crew will carry out the flagging, monumenting and setting out works. • First, the line will be flagged using the ranging poles or wooden pegs and a level in order to control the vertical gradients. The horizontal curvatures will also be controlled during the flagging work. Proper and most suited crossing sites for both major and minor drainages will be selected and controlled as well.
GNSS Crew	• This crew establishes monuments for the traverse control pillars, along the selected and flagged line. A control traverse should be established using GNSS equipment, coordinated and tied into the Ghana National Grid System. These control pillars shall be referenced in the field in permanent monuments, numbered, and shall be shown on the plan and profile drawings. The monuments should be established at certain offset away from the centreline since they should stay undisturbed at least until the construction time to be used as references. They should be established either from concrete as shown in **Figure 4.1**, or on rock outcrops or simply buried large boulders clearly marked or on headwalls of existing structures, over which GNSS equipment could stand. The primary control pillars will be established at approximately every 1km maximum. • Maximum of two inter-visible secondary control pillars should be established for each primary control pillar for cross-checking and as reserve points in case any one of them is lost or damaged. • The primary control pillar should be coordinated in X, Y, Z and should be tied to the Ghana National Grid System. • This crew will handle the profile levelling, the cross section levelling, and all other topographical survey works including details of bridge crossings, culverts, town sections and other areas which need detail survey. • The GNSS equipment is set on the primary control pillar and cross-check on the secondary control pillars and will continue its work on topography and detail(s) and close on the next primary control pillar, thereby checking the error of closure. If it is within the allowable, the routine work will continue, if not the former activity will be repeated. • The error from the proceeding successive control pillars, if it is within the allowable, shall be discarded and will not be carried over to the next section. • The output from the GNSS data collection is a computer file which contains horizontal coordinate points, vertical elevations, and a description of all points needed to develop a full topographic map of the area. • The computer file must be capable of being downloaded directly into a computerized design and drafting program. These programs should then be able to generate, if so desired, a three- dimensional digital terrain model. The data plot generated can be checked and verified by the surveyors shortly after the fieldwork.

Name of Crew	Assignment
Total Station Crew	• This crew establishes monuments for the traverse control pillars, along the selected and flagged line. A control traverse should be established using Total Station, coordinated and tied into the Ghana National Grid System. These control pillars shall be referenced in the field in permanent monuments, numbered, and shall be shown on the plan and profile drawings. The monuments should be established at certain offset away from the centreline since they should stay undisturbed at least until the construction time to be used as references. They should be established either from concrete as shown in **Figure 4.1**, or on rock outcrops or simply buried large boulders clearly marked or on headwalls of existing structures, over which Total Station equipment could stand. The primary control pillars will be established at approximately every 1km maximum. • Maximum of two inter-visible secondary control pillars should be established for each primary control pillar, for back sighting and as reserve points in case any one of them is lost or damaged. • The primary control pillars should be coordinated in X, Y, Z and should be tied to the Ghana National Grid System. • This crew will handle the profile levelling, the cross-section levelling, and all other topographical survey works including details of bridge crossings, culverts, town sections and other areas which need detail survey. • The Total Station is set on the primary control pillar and takes orientation or check on the inter-visible secondary control pillars and will continue its work on topography and detail and close on the next primary control pillar, thereby checking the error of closure. If it is within the allowable, the routine work will continue, if not the former activity will be repeated. • The error from the proceeding successive control pillars, if it is within the allowable, shall be discarded and will not be carried over to the next section. • The output from the Total Station data collection is a computer file which contains horizontal coordinate points, vertical elevations, and a description of all points needed to develop a full topographic map of the area. • The computer file must be capable of being downloaded directly into a computerized design and drafting program. These programs should then be able to generate, if so desired, a three- dimensional digital terrain model. The data plot generated can be checked and verified by the surveyors shortly after the fieldwork.
Levelling Crew	• The levelling crew runs the differential levelling between consecutive benchmarks/control pillars in a closed loop system that is by running the forward run, and by running the check back between the same two consecutive benchmarks/control pillars. The differential levelling is separately done because in the 'Z' co-ordinate the GNSS equipment is not reliable. It is only the level that is the most accurate equipment as far as the vertical control is concerned. And if available the digital level is recommended otherwise an optical level.

4.2.2.5 SURVEYING OF WATER CROSSING SITES

The collection of data shall be conducted in such a way as to acquire a complete and comprehensive field data on particular water crossing sites along the proposed road corridor.

For bridge sites, detailed topographic survey shall be done at 10 to 20 metres interval for a length of 100 – 400 metres or as specified by the Drainage Engineer on both upstream and downstream. This activity shall also be done on both approaches of the bridge site for a length of 100 to 200 metres depending on the terrain.

On existing structures, the data shall be collected at the top deck surfaces, and corresponding invert (bed levels on both sides) and also at existing and high-water marks including existing water levels, with the dates of data collection properly recorded, including all particular features of the bridges, flow channels and the surroundings. Additionally, data shall be taken at several points top and bottom on the wing walls and all feature points of approach roads and also high-water marks and existing water levels with its dates of data collection. For new crossings data shall be collected on top of banks, bottom, centre, high water marks, existing water levels and other natural features of the flow channel. **Figure 4.26** is a sketch for guidance to surveyors in topographic survey of water crossing or drainage structures sites.

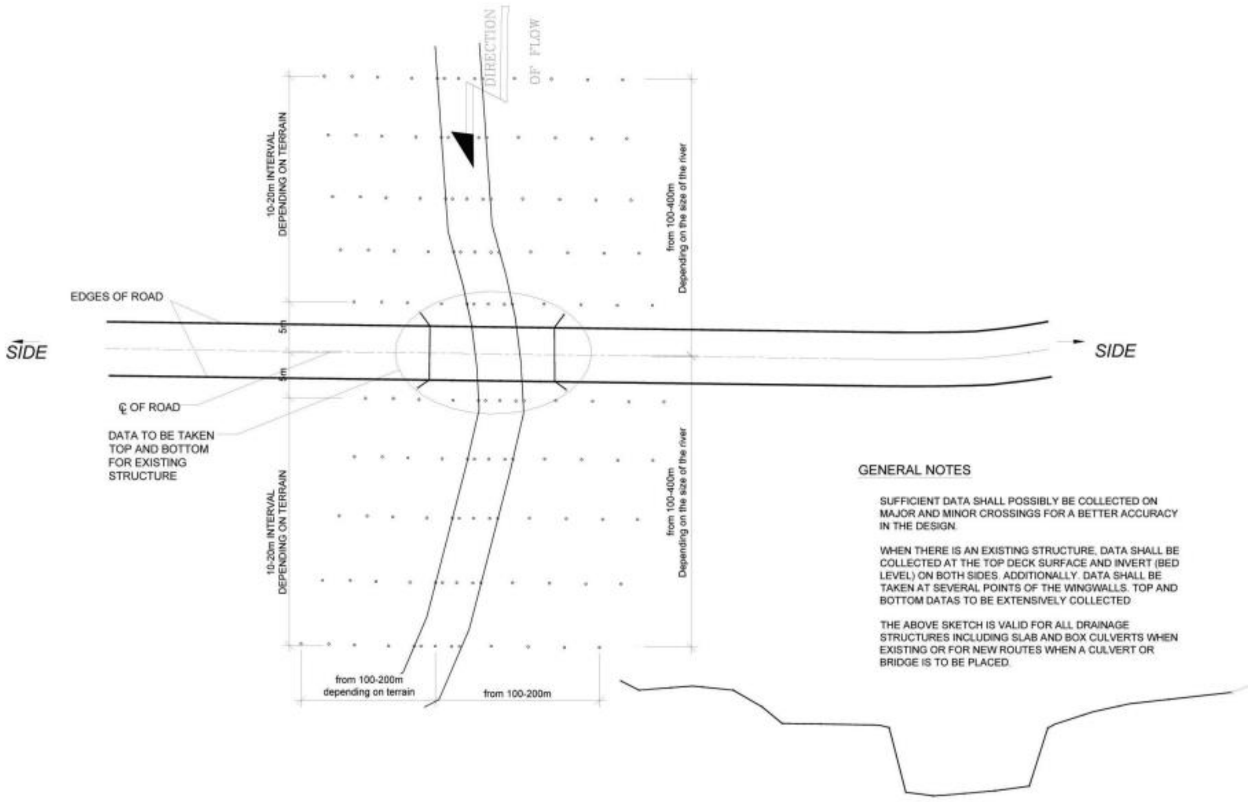


Figure 4.26 Guide for topographic survey of water crossing or drainage structures site

4.3 SURVEY CODES

Table 4.4 provides information on how points/strings should be coded.

Table 4.4 Survey Codes and Their Full Descriptions

CODE	TYPE (STRING/PONT)	FULL DESCRIPTION
BH	POINT	BOREHOLE
EP	POINT	ELECTRIC POLE
GL	POINT	GROUND LEVEL
GLF	POINT	GROUND LEVEL FARM
SH	POINT	SPOT HEIGHT
GLMSHY	POINT	GROUND LEVEL MARSHY
HTL	POINT	HIGH TENSION LINE
HYD	POINT	HYDRANT
MH	POINT	MANHOLE
ROCK	POINT	ROCK
STN	POINT	SURVEY CONTROL POINT
SL	POINT	STREETLIGHT
STP	POINT	STANDPIPE
TP	POINT	TELEPHONE POLE
TREE	POINT	TREE
TRSIGN	POINT	TRAFFIC SIGN
WVALVE	POINT	WATER VALVE
INV	POINT	INVERT
PYLON	POINT	PYLON
GCP	POINT	GROUND CONTROL POINT
MNUMT	POINT	MONUMENT
BD/BLD	STRING	BUILDING
TS	STRING	TEMPORARY STRUCTURE
BR	STRING	BRIDGE
CEM	STRING	CEMETERY
CHM	STRING	CHAMBER
CL	STRING	EXISTING CENTERLINE
CUL	STRING	CULVERT
DRN	STRING	DRAIN
RDL	STRING	LEFT EDGE OF EXISTING PAVEMENT
RDR	STRING	RIGHT EDGE OF EXISTING PAVEMENT
WALL	STRING	WALL

CODE	TYPE (STRING/PONT)	FULL DESCRIPTION
RV/STRM	STRING	RIVER/STREAM/POND
PIPELINE	STRING	PIPELINE
STAY	POINT	STAY
KBT	STRING	TOP OF KERB
KBB	STRING	BOTTOM OF KERB
RWALL	STRING	RETAINING WALL
FBR	STRING	FIBRE OPTIC LINE
TRFM	POINT	TRANSFORMER
SG	POINT	SURVEY CONTROL PILLAR
GHA	POINT	SURVEY CONTROL PILLAR (TRUNK ROAD)
DFR	POINT	SURVEY CONTROL PILLAR (FEEDER ROAD)
DUR	POINT	SURVEY CONTROL PILLAR (URBAN ROAD)

NOTE

1. A string code may be suffixed with an alphanumeric character to differentiate existing features with the same code. E.g., BD1 and BD2 represent two distinct buildings, BDCH1 and BDBANK1 stands for Church building No. 1 and Bank building No.2 respectively.
2. A prefix to “PIPELINE” should indicate the contents. E.g., W for water, G for gas, PT for petroleum etc.
3. Full descriptions should be provided for survey codes not included in **Table 4.4**.

4.4 SAFETY IN SURVEYING

4.4.1 INTRODUCTION

Road surveying occurs in many different hazardous environments. Rugged terrain, high speed traffic, tools used in surveying, and construction equipment are some of the elements that typify survey hazards. Most people have one thing in common with many who have experienced an accident: they believe it could not happen to them. A meaningful safety program requires that each team acknowledge that “it can happen to me.” Each must also ask, “What is my responsibility?” and “What can I do to keep it from happening?”

4.4.2 RESPONSIBILITIES

4.4.2.1 SURVEY ENTITY

The Entity intending to carry out route surveys will conduct an active and effective safety program designed to keep it in compliance with the safety and health rules and regulations of the Ministry of Road and Highways (MRH). This program may be delegated to Agency Unit Heads. They will have to ensure that all equipment and materials are available to carry out this program.

The objective of the program is to prevent work-related injuries and deaths, improve employee health and well-being, and control worker's compensation costs.

4.4.2.2 SAFETY OFFICER

The Safety Officer is responsible for conditions and performance regarding safety, instructing subordinates about safety policies and practices affecting them and other team members, and carrying out the more detailed safety procedures stated in this "Guide". He/she shall not knowingly permit an employee to work when his/her ability or alertness is impaired by fatigue or other factors, so that the employee or others might be exposed to injury. He/she is also responsible for:

i. New employee orientation.
ii. Personnel and routine job assignments.
iii. Ensuring that proper safety equipment is available and used.
iv. Safety training.
v. Formal corrective action.
vi. Ensuring that regular safety meetings are held and documented.

4.4.2.3 THE SURVEY TEAM LEADER

The Survey Team Leader is responsible for seeing that all safety rules and procedures are followed and that all work is performed safely. Do not attempt to delegate this responsibility. Survey Team Leader personally must ensure the use of the one best safe method for each operation.

As Survey Team Leader, you must see that a copy of this section of the "Guide" is always available to members of your team. You must enforce all elements of this section. Job planning is one of your major responsibilities. Discuss the safety aspects of each job with the Officer in charge of safety or your supervisor before beginning a job.

i. Incorporate safety in planning each survey.
ii. Develop additional safety practices as required for each job.
iii. Request enough personnel for safe surveying, spotters, flaggers, etc.
iv. Always plan around hazards, especially life-threatening hazards such as traffic.
v. Avoid assigning team members to solo-type tasks that isolate them from other personnel. Try to have each member working in pairs. (This is especially important in high hazard areas, such as along roads and mountainous areas.)
vi. If it is necessary to work alone, implement a communication plan.

You are responsible for ensuring that each team member possesses or has available the required personal safety equipment. You must see that employees use this equipment as required. If an employee refuses to use the required equipment, do not allow him/her to work. Return the team member to the office if practical and contact your supervisor.

It is your responsibility to make sure any new team member has been through the employee orientation as outlined in **Section 4.4.3.1**. Make sure they have been briefed and trained in the use and location of team equipment, safety material, survey safety procedures, first aid supplies, and all other needed equipment.

It is your responsibility to immediately correct unsafe practices by any member of your team. If an employee refuses to work safely or is inconsiderate of the safety of other team members, return the employee to the office and contact your supervisor. If an employee seems to be under the influence of drugs or alcohol, do not let him/her work. If an employee is obviously under the influence, return him/her to the office and report the incident to your supervisor.

Continually monitor employee safety performance and attitudes. Advise your supervisor about each employee's safety practices and attitudes. Your supervisor will need this information when doing the employee's performance appraisal.

You are responsible for conducting safety meetings with team members when situations occur where safety needs to be emphasized. Keep safety foremost in your team member's mind.

See that at least one member of your survey team has current first aid training and is designated to offer first aid. Employees should promptly give or obtain aid for an injured person. If first aid training is needed, advise your supervisor. You should thoroughly investigate all accidents and injuries and take corrective action as appropriate. See that all required reports are promptly prepared and submitted.

4.4.2.4 INDIVIDUAL EMPLOYEE RESPONSIBILITIES

Each employee is responsible for his/her own safety and the safety of co-workers.

A Safe Practices.

Each employee shall learn the personal and group accident preventions and injury treatments that are described herein and abide by them. Safety procedures and rules are not optional which should come with alternatives. Deviations are not allowed without prior formal approval. Each employee shall have a practical working knowledge of the safety section. In addition, each employee shall be alert to possible violations of safety policies. If violations are seen and you cannot correct them, you shall report them to your supervisor.

B Operational Practices

Each employee must routinely:

i. Report unsafe conditions or practices. (See the paragraph "Reporting Unsafe Working Conditions" below in this topic.)
ii. Promptly report all accidents and personal injuries to the supervisor.
iii. Render or obtain aid, as needed, for injured persons.
iv. Be alert for hidden hazards.
v. Be alert for hazards created by changing conditions, either natural or man-made.
vi. Avoid horseplay and practical joking.
vii. Store and secure all equipment and supplies when not in use. These must not be hazardous to persons or to vehicular operation.
viii. Help keep vehicle and working environment clean, sanitary, and litter free.
ix. Set aside defective and unsafe tools and supplies for repair or replacement. Report such problems to your supervisor.
x. Carry gear such as plumb bobs, hand levels, and hatchets in sheaths.
xi. Do not carry such tools in pants pockets.
xii. Heed all specific practices listed in Operational Safety, Chapter 4 of this manual.
xiii. Not have or use on the job, or transport in vehicles, any hazardous or potentially hazard-causing items such as: fireworks, firearms and ammunition, intoxicating beverages and drugs, and pets.
xiv. Before acting, mentally check the safety of each action.
xv. Face oncoming traffic when working on foot and near or on a travel way. If unable to face traffic, have a co-worker be a "spotter" and watch for you. (See spotter guidelines.) It is advisable to face traffic from any work site within the right-of-way, especially when near, at, or below roadbed level.

C Safety Meetings

Attend and participate in safety meetings.

D Proper Attire

Each employee must provide and wear clothing and footwear that provide adequate protection. Survey employees shall wear clothing that completely covers the body, except the head, neck and arms below the point of the shoulders. Work clothing should provide protection from the sun's rays, heat, cold, and vegetation. Wear clothing that will help keep you from being injured or diverted from safely performing the job at hand.

Foot protection shall be adequately catered for.

E Physical Condition

Each employee must report for work prepared to perform an alert, accident- free, full shift.

Employees will be free from the influence of drugs or alcohol. When a physician gives you a prescription, inquire if the drug might impair your safe functioning. If any impairment might result, ask the doctor what you can and cannot do while taking the medication. Notify your supervisor.

Do not report for work if you are under the influence of intoxicants. Do not report for work if any lingering effects from drinking or medication would diminish your alertness, keep you from reacting quickly, or impair your judgment.

F Vehicular Operations

Transportation surveying requires vehicular operations. To operate state vehicles, an employee must:

i. Possess a valid driver’s license.
ii. Must be trained in defensive driving.

G Accident Reporting

Vehicle accident reports and investigations must be completed promptly. Employees must participate or cooperate fully in determining the causes and prevention of accidents and injuries.

4.4.3 EMPLOYEE SAFETY

4.4.3.1 NEW EMPLOYEE ORIENTATION

Each new employee should be given a copy of the Safety section of this Guide to read and study. Supervisors should make certain the employee understands the basic requirements, and that each employee knows he/she is “responsible for his/her own safety and the safety of others.” Supervisors should be sure each employee knows how to access emergency aid for the areas where he/she works. If the survey team does not have a radio with which to summon help, the location of the closest medical services should be known. At least one employee on a survey team must have current first aid certification, and all field personnel should be trained in Defensive Driving. The employee should also be briefed on:

i. Medical care available through his/her employment.
ii. Worker’s compensation benefits.
iii. The accident investigation process and its purpose.
iv. Accident and injury reporting and their purposes.
v. His/her right to refuse to perform tasks that are dangerous or hazardous to his/her well-being.
vi. His/her responsibilities in case of personal or motor vehicle accidents.

4.4.3.2 PERSONNEL AND ROUTINE JOB ASSIGNMENTS

Supervisors must consider several things when assigning work. Trained personnel shall be assigned to teams working on jobs that require hazardous tasks, such as using a chain saw, climbing/descending precipitous or slippery slopes, or driving on rough terrain or unimproved roads. Highly allergic personnel must be kept away from jobs where toxic vegetation or substances cannot be avoided.

Before assigning a team to a new job, determine the hazards that are present and the preventive measures to be taken. Brief the Team Leader accordingly. In high hazard areas, plan for and brief the Team Leader on such things as narrow shoulders, escape routes, hospital locations, ambulance service, and rescue agencies. Surveys that are extremely hazardous because of immediate or short-term conditions should be postponed. Before assigning work on or alongside travel ways, determine which traffic controls are required to protect the team and the public. If extra personnel are needed for flaggers or spotters, assign accordingly.

4.4.3.3 SAFETY TRAINING

All personnel need to know about new safety equipment available and new techniques developed to aid in safe surveying. "Safety" as a topic should be on the agenda of each Team Leader meeting. Safety monitoring needs to be continuous. Immediately correct safety deficiencies that are seen on the job. Periodically rate the safety performance of Team Leaders and include it in their performance appraisal.

See that adequate safety equipment is in stock and that only safe supplies and equipment are issued. Whenever an employee refuses to work safely or when an employee's performance is affected by the use of drugs or alcohol, proper corrective action should be taken.

4.4.3.4 REPORTING UNSAFE WORKING CONDITIONS

All employees have a moral obligation to protect themselves, their co-workers, and the public by immediately reporting safety problems. Employees can report unsafe conditions by:

i. Promptly telling his/her immediate supervisor. He/she is often in the best position to take corrective action. If employee's supervisor is not available, use the chain of command.
ii. Refusing to work under unsafe conditions. If this occurs, the employee should give specific reasons to the supervisor in writing. The supervisor should take steps to provide the relevant safety equipment to execute the works.

Employees are not required to perform hazardous work or to operate hazardous equipment without at least one other person in the area, although the other person may be performing other related duties.

4.4.3.5 SAFETY EQUIPMENT AND HEALTH

Each employee shall be furnished with personal safety items which must be prudently and consistently used. Regular issue items shall include a hard hat, soft cap, safety boots, wellington boots and safety vest.

An approved hard hat must be worn anytime there is a danger from falling or flying objects, or electrical hazards. Inspect the hard hat shell and cradle at least twice a year. If either becomes defective or deformed, replace it. Hard hats should be replaced as needed, based on regular inspection.

Approved soft caps may be worn instead of hard hats in work areas not involving possible falling or flying objects, or electrical hazards.

A hard hat or approved soft cap must be worn by employees while on the roadway or right of way.

While working on the highway or right of way, outside upper body garments must be strong red, orange, strong yellow-green, or fluorescent versions of these colours.

In addition to the regular issue safety equipment, employees may use such specialty items as:

i. Safety Glasses – Wear these when exposed to flying particles, hazardous substances, injurious light rays, or while performing work where the expected hazard is from frontal impact only.
ii. Goggles – Wear mono-type ventilated goggles when exposed to blowing dust, swirling sand, or other windblown materials.
iii. Dust Masks – Use these when there is a likelihood you will breathe excessive dust.
iv. Gloves – Use them when working in hazardous, brushing line in thorny vegetation, or to protect against puncture, laceration, and splinter wounds.
v. Hearing Protection – Whenever operating gasoline chain saws, jackhammers, or other very noisy tools, use hearing protection. When an employee is exposed to noise levels above 90 decibels, hearing protection is required. Ear plugs and/or muffs shall be available.
vi. Safety Harnesses and Lifelines – Employees shall be secured by safety harnesses and lifelines whenever they work from unguarded surfaces above open pits or tanks; more than six-feet above water, ground, or floor; on a scaffold more than six feet above the surface; and in any areas where they would otherwise be exposed to dangerous falls.

Employees shall be protected by safety nets when the above procedures are impractical. When required, employees shall use lifelines which are at least 7/8”, wire-core manila rope.

4.4.3.6 FIRST AID AND SUPPLIES

One first aid kit should be provided for each employee who is isolated from the survey vehicle and other employees. The 16-unit kit should contain the following:

- 1 Package – Adhesive Tape, 1/2” x 90” Roll
- 2 Each – Triangular Bandage
- 2 Packages – 3” Bandage Compress
- 1 Package – 1” x 3” Adhesive Bandage (16)
- 1 Package – 2” Gauze Roller Bandage
- 1 Package – Bee Sting Relief Swabs
- 2 Each – 18” x 36” Gauze Compress
- 1 Package – Antiseptic Wipes
- 1 Pair – Scissors
- 1 Each – Rescue Blanket
- 1 Each – Micro-shield Breather 2 Pair – Gloves, Sterile

The kit should be stored in the primary survey vehicle. One kit should also be provided for each office or for each office unit that works isolated from other units. The kit should be stored where everyone can have easy access to it. All kits must be inspected periodically to ensure the supplies are usable. In addition to the first aid kits, each office and vehicle should have a readily accessible copy of a current First Aid Pocket Reference Manual.

Drinking Water. Use only clean containers which have been designed and used only for drinking water. Communicable diseases have been traced to dirty and improperly maintained water containers. Do not use for cooling or storing canned beverages, juices, etc. Use disposable drinking cups. If ice is added to the water, it should be carried to the water in a sanitary container.

4.4.4 OPERATIONAL SAFETY

4.4.4.1 CONSTRUCTION OPERATIONS

Before starting work, employees need to determine potential hazards from the natural environment, the public, and their operations. The Team Leader shall meet with his Team Members to discuss safety conditions in the work area and plan accordingly.

During work, employees should be extremely cautious around heavy and fast-moving equipment, especially on active roads and around equipment with limited driver visibility. Do not rely on the operator’s visibility, judgment, or ability. Establish communication with the operator before walking in front of or behind any piece of equipment. Use spotters as conditions dictate. It may be necessary to suspend survey operations when uncontrollable hazards develop and to resume work only when safe working conditions have been restored.

Display and use safety devices and gear as required and as needed for maximum safety. Do not walk on girders or along edges of raised platforms without guardrails unless safety nets are in place or safety lines are used. Do not work on or traverse any walkway, ramp, or other elevated structure over six-feet tall without using a safety harness and lifeline unless guard rails or safety nets are provided. Appropriate training is also required.

4.4.4.2 CUTTING TOOLS

Make sure you use the right type and size of tool for each operation. Keep all cutting tools sharp. When you sharpen a tool, use a file that has a handle. Turn dull saws in for replacement. When not in use, sheathe or store tools so the cutting edge is not exposed. Store and carry machetes and axes in the leather sheaths that shall be provided. Do not use tools with splintered or loose handles.

It is important that you properly use each tool.

Machetes – Don’t sharpen machete blades without a hand guard within 6 inches of the handle. Use gloves to protect your hands, especially in thorny bushes. While chopping, lean forward if possible and always chop away from the body. Swing with a full swing, but do not over swing or swing too hard. Before cutting larger vegetation, clear away small vines, etc. Do not use machetes for heavy cutting. If practical use long-handled lopping shears instead when cutting thorny bushes or briars.

Machetes are very dangerous if not used with extreme caution. Only use the machete when you have a firm grip on the handle and secure, balanced footing. Be careful to not over-swing or swing toward legs or feet. Take care to avoid glancing blows which can ricochet back toward the tool user. Do not allow co-workers to stand nearby.

Axes and Brush Hooks – Clear away any impeding light growth with a machete or hatchet before chopping. Make sure you allow ample space between adjacent choppers and keep others outside the area. Always carry an axe or brush hook with the handle gripped behind the head and the cutting edge facing outward. Do not use double-bit axes. For extended heavy brushing, use a small chain saw instead.

4.4.4.3 DIGGING TOOLS

These include tools such as pickaxes, mattocks, shovels, and ‘soso’. While using a pickaxe, do not use a pickaxe head that is either sharply pointed or badly blunted.

Make certain the head is “bound” tightly to a good handle before swinging. Allow ample space for swinging and do not over swing on the back swing. You should wear eye protection when digging in very hard material. As you swing, squat by flexing the knees so the pick handle will be horizontal when the point strikes the earth. This will also keep the point away from your feet.

Use a round-pointed shovel for digging in hard earth. Do not use the shovel as a pry bar. Also, do not use the shovel as you would a digging bar. Place the blade of the shovel on the earth and force it into the ground with your foot. Always keep one foot on the ground.

When using a 'soso', work with the feet widespread. Hold the bar close to the body and lift and drop it vertically. Keep the point sharp enough to do the job without having to lift the bar excessively high.

4.4.4.4 HAMMERING TOOLS

Always use the correct type and size tool for each driving operation. Check for defects before using. Do not use hatchets, axes, or other wood cutting or for driving or hammering metal. Never strike brittle or mushroomed metal with a hammer because bits of steel might chip off and cause serious injury. Use safety glasses when driving or cutting metal. Do not use tools with splintered or loose handles or with mushroomed or cracked heads. Allow ample space for swinging and swing so that the handle is horizontal when the face of the driving head contacts the object being driven. With long-handled sledges, this requires flexing the knees to lower the body during the swing. When squatting, use either a short-handled tool, or keep the long handle from between your legs (to avoid groin injuries). Never hold an object for someone to drive by full-swinging. When driving masonry nails, spikes, and stakes into asphalt pavement or very hard earth, use extra care. Be sure the object being driven is well started before releasing it and driving it with full swings of the hammer.

4.4.4.5 ELECTRICAL EQUIPMENT

Use only portable electric hand tools that are double insulated or that have a grounding wire. Do not remove grounding wires or prongs. Do not use any equipment that has a cord with broken insulation, or a damaged plug or socket. Do not use electrical equipment when you or the equipment is standing in water or on saturated soil.

4.4.4.6 FENCE CROSSINGS

It is best to use gates whenever possible and avoid fence crossings. Do not attempt to carry anything when climbing on or over obstacles. Cross barbed wire fences at the centre of a span and have a co-worker hold the wire(s) for you. When stepping over a barbed wire fence, lay a piece of heavy canvas, such as an empty materials bag, over the top strand.

4.4.4.7 ANIMAL HAZARDS

You must assume that all animals are potentially dangerous. Have owner's secure hostile-acting animals before entering enclosures containing such animals. Do not enter an enclosure with high fences if a hazardous animal is within. Carry a pointed stick or something similar to ward off an attacking animal. Retreat is usually advisable but do not turn your back and run unless you can reach a haven safely. Do not approach, attempt to capture, kill, or pet either domesticated or wild animals. This includes snakes and other reptiles. Be especially wary of animals that appear sick, animals with young, stallions, bulls, and guard dogs. Do not approach dead or seemingly dead animals, fowl, or reptiles.

4.4.4.8 HEAT STRESS AND SUN EXPOSURE

Heat stress and damage to skin can result from excessive sunlight. Employees should follow the preventive measures listed below.

i. Wear head coverings (a hard hat when required) that allow free air circulation and provide shade from the sun.
ii. Wear light-coloured, loose-fitting clothing that minimizes skin exposure.
iii. Drink enough fluids. Begin drinking before you feel thirsty. Water and

“sports” drinks like Gatorade and Lucozade are recommended. Caffeinated drinks (coffee, tea and sodas) are not as effective at keeping the body properly hydrated.

Team Leaders needs to provide a constant, readily available supply of potable water and see that employees wear proper attire. When the heat is extreme and the survey requires considerable exertion, the Team Leader should schedule work for cooler times of the day.

A typical symptom of heat stress is cramping of the muscles, especially in the legs, arms, and abdomen. It usually occurs when someone is doing strenuous activity in a warm environment, where large amounts of sweat are lost. Treatment includes direct pressure on the muscle, gentle steady stretching, and rest. Drink water to help balance sodium. If the problem persists, seek additional medical attention.

The signs and symptoms of heat exhaustion include sweating, weakness, dizziness, and headaches. The affected employee may also have moist and clammy skin, and rapid, shallow breathing. The best treatment is rest and removal to a cooler temperature. Elevate the legs and give water if the employee is conscious. If the employee does not respond to rest and fluids, seek additional medical aid.

Heat stroke is the most serious. Symptoms are red colour to the skin and the skin very hot and dry. Temperature can be 41° C and rising. Usually sweating stops but occasionally sweating continues. The affected employee may be disoriented and confused. There can be loss of consciousness or seizures. The employee needs to be cooled as quickly as possible. Use cold water, a cold bath, or blow cold air on the patient using ice and a fan. Do not, however, chill him/her. You need to get the temperature down to 39° C and maintain it at least that low. Seek medical attention immediately.

4.4.4.9 INSECT BITES AND STINGS

Some persons are highly allergic to the stings and bites of insects. More people die from bee stings than from snake bites. If an employee is stung or bitten, apply a bee sting relief swab from the first aid kit. If the employee is allergic to bee stings, he/she may have an emergency kit of his/her own. Assist with the medication as requested and seek medical attention. Treat spider bites of either the black widow (hourglass) or the aggressive house (hobo) spider the same as snake bites. Seek medical attention immediately.

4.4.4.10 LASERS

i. All employees operating a laser EDM must be aware of the following precautions:
ii. Do not look directly into a laser beam at close range.
iii. Do not look directly into a laser beam at any working range with binoculars or telescopes. The intensity of the beam is magnified by the square of the power of the optical instrument used.
iv. Do not expose the eyes to the laser for any prolonged time at any working range.
v. Check operating instructions for eye protection.

4.4.4.11 LIFTING

Lift only what you, or you and others, can safely handle. Do not be misled by bulk or lack of it. When you have any doubt, seek help or use a mechanical lifter. Check for splinters, sharp protrusions, spiders, snakes, stinging insects, and other hazards before lifting (particularly rocks). Before you begin, plan how the lifting, moving, and setting down of the object is to be done. Be sure you have a safe, obstacle-free path of travel. If stooping is required, crouch as close to the load as possible. Firmly grasp the object, keeping the spine straight, and then lift by pulling it into the pelvic area. Reverse this process when setting things down. Always lift or lower objects with the leg muscles, not the back muscles, and do not twist your body while carrying the object. Move your feet to turn your body.

4.4.4.12 MOUNTAINOUS TERRAIN

Use approved safety lines and harnesses whenever injury could result from work on precipitous slopes or slippery rocks. Lifelines used on rock scaling operations, or in areas where the lifeline may be subjected to cutting or abrasion, should be a minimum of 7/8”, wire-core manila rope.

For all other lifeline applications, a minimum of 3/4”, manila or equivalent, with a minimum breaking strength of 5400 pounds, shall be used. Use knots that will not slip and be sure the line is securely anchored. Always wear a hard hat when safety lines are required. Obtain required training in use of this equipment.

When traversing hazardous areas, test your footing and determine a safe route before proceeding. Avoid risky shortcuts and do not run downhill. Use the pair system in isolated areas. Never drink stream water or water from any untested source. Take drinking water with you.

4.4.4.13 NIGHT OPERATIONS

Hazards can become more dangerous at night. Therefore, surveying will not be done at night unless reasonable daylight alternatives have been considered. At night, make safety the number one priority. Allow extra time for all operations. Make certain there are enough personnel, equipment, and supplies. All team members should be properly briefed and issued adequate equipment.

In mountainous areas, always use the pair system. Use reflective material to flag safe roads and trails into work areas and to specific points. Radio communication for each work area is a necessity.

If traffic promises to be particularly hazardous, the team should seek assistance from the Police. Include public safety in your survey planning. All personnel shall wear reflective vests when working anywhere where vehicles are likely to be moving.

Night surveys can disrupt traffic and arouse the curiosity of the local residents. If this seems likely, the team should notify law enforcement agencies and the Police. Consider giving advance public notice through local news media.

4.4.4.14 POWER LINES

Regard all power lines as dangerous. Contact the utility company if lines are down or power poles are damaged.

4.4.4.15 POWER TOOLS

Power tool usage requires maximum alertness and adequate training. Employees must be given proper instruction before being allowed to operate powered equipment. Do not allow an employee to operate a power tool unless he/she has been trained in its use. Eye and hearing protection shall be used where chain saws, jackhammers, and ramsets are operated. Such protection must also be used by helpers. Nearby co-workers must use ear protection if the noise levels specified exceed 90 decibels.

4.4.4.16 PRESSURIZED SPRAY CANS

Serious injuries and costly cleanup have resulted from improper handling of pressurized spray cans. Do not puncture or incinerate them. Store them at temperatures less than 48° C. Check with and dispose of through local refuse disposal systems. Do not discard any spray can in a receptacle that is normally accessible to children. Store cans in a secure place during transport.

4.4.4.17 RADIO TRANSMITTERS

Mobile radio transmissions can interfere with critical communications networks frequencies or explosive charges. Check radio frequencies against existing communications channels and also with blasting supervisor before transmitting any messages.

4.4.4.18 WORKING NEAR RAILROADS

Guidelines used when working within an operating right-of-way are for the safety of the surveyor and the railroad. These general guidelines are:

i. Notify the railroad before entering any railroad right of way.
ii. Apply for required permits before entering the railroad right of way.
iii. Always be alert around railroads. Railroad equipment is not always heard, especially if there is other noise. If a railroad car is coasting or if a train is moving slowly, hearing alone might not provide adequate protection. When necessary, use a spotter.
iv. Never crawl under stopped cars and do not cross tracks between closely spaced cars. They could be bumped at any time as the engineer and the brakeman work only one side of the train.
v. Avoid use of the colour red. To a trainman, red means immediate danger and “Stop”, without exception. Surveyors must not wear red vests or red clothing when working near rails. Red markers, flagging, or lights will not be used for any reason.
vi. Normally, do not use flares on highways at railroad crossings. Only use flares if unmovable, injured persons or disabled vehicles are on the tracks, or if you have found a condition that could derail the train.
vii. Do not leave protruding stakes or any holes within three metres of the centreline of the tracks.
viii. Do not park vehicles within 3 metres of the tracks. Train teams need this area for their operations.
ix. When taping across railroad tracks, support steel tapes above the rails at all times. The contacting of both rails at once by a steel tape can activate signals. In switching areas, steel tapes can activate signals even when laid parallel to the track. Therefore, only let non-metallic tapes be grounded.
x. Do not leave instruments or other equipment unattended on or near tracks.

4.4.4.19 SURVEYING SIGHTS AND TARGETS

Do not leave cones, concrete-filled sight cans, or other similar sights where they might damage vehicles or be hazardous to pedestrians. All sights or points of any kind, both permanent and temporary, shall be guarded in a manner which protects the public as well as the survey point.

Do not use red flagging or red targets for signalling when working in or near traffic. Such signalling might confuse motorists. Be wary of using the standard surveying hand signals if they might confuse motorists.

4.4.4.20 SUB-SURFACE AND CONFINED WORKSPACES

Do not enter permit required confined spaces unless properly trained and equipped. In general, use ladders for places and situations that are difficult to enter or reach. On sloping concrete slabs and hard earth slopes, be cautious of slipping on loose sand and grit. Be just as cautious on wet and slimy concrete channel bottoms. Always be aware of snakes and spiders in manholes, trenches, sewers, and drains. Have an outside observer, a partner, in constant touch while subsurface work is in progress. Even though all other precautions are taken, use a lifeline whenever cave-ins or asphyxiation are at all possible.

On open excavations, if a trench is deeper than 1.5 metres, do not stand near, enter, or work in it unless a competent person has assessed the hazards and it is adequately shored or properly sloped. If a trench is less than 1.5 metres deep, do not enter it if ground movement appears possible. Do not park vehicles near the edges of excavations.

Confined Spaces and Enclosed Facilities. A confined space is defined as a space having the following conditions:

i. It is possible to enter.
ii. It has limited access or egress.
iii. It is not designed for continuous employee occupancy.

Permit required confined spaces are confined spaces that have known or suspected hazardous conditions that are potentially threatening to life and health. For example, manholes and underground utility vaults may have oxygen deficient atmospheres, contain toxic gases or have physical hazards (falls, moving machinery, water etc.)

Test for oxygen deficiency and for the presence of combustible gases or vapours. Do not enter these spaces without required training and equipment.

4.4.4.21 CULVERTS

Employees shall be allowed entry into culverts if all the following conditions are met:

i. Inspection is the only task
ii. Not deformed, bulging, crushed, or water running underneath
iii. Less than 0.6 metres of standing water.
iv. No rapid moving water
v. Not planning to change air quality (painting, welding, etc.)
vi. Detectable air flow (should be able to feel)
vii. No sheen or hazmat present on water surface
viii. Culvert is 0.9 metres or greater
ix. No other conditions that may be immediately dangerous to life or health.

4.4.4.22 WORKING IN TRAFFIC

When working in or near traffic, all personnel must be alert and watch out for each other. Additional help may be required to work safely. You may need to close lanes or the shoulder when a fairly lengthy operation is anticipated, narrow shoulders with no escape route are encountered, or you are working in a heavy-use area. All team members should maintain an awareness of the location of moving traffic, how to avoid it and, if needed, possible escape routes. This includes work on shoulders as well as on the travel way. You should face oncoming traffic at all times or be guarded by a co-worker acting as a spotter. When working in a zone between two-way traffic, stand parallel to the travelled way and use a spotter. Move deliberately: Do not make sudden movements that might confuse a motorist and cause him/her to take evasive action, panic, or stop and cause an accident. Also, be careful and deliberate when using surveying hand signals. You don't want motorists to mistake them for flagging signals and get confused. Use radios if available.

Use off-set lines as much as possible to avoid interrupting traffic. Minimize the crossing of traffic lanes. Drive around by way of ramps or surface streets to assure a safe crossing. When possible, work one side of the road at a time. For example, when cross sectioning, keep a Rodman on each side of the road if this will eliminate lane crossing. Whenever possible place a barrier vehicle or shadow vehicle between traffic and the workers. When carrying equipment, walk parallel to traffic and be careful to keep level rods, range poles, etc., from extending into a traffic lane. Avoid working near moving vehicles, e.g. rollers, graders, or restricted view equipment. Be especially careful when there are "competing" activities, such as vehicular accidents, maintenance activities, construction operations, hawkers, or distracting objects alongside the road. Be constantly aware of possible quick escape routes to avoid dangerously veering traffic.

4.4.4.23 WATER OPERATIONS

i. Employees working over or near water where danger of drowning exists shall wear Ghana Maritime Authority approved life jackets or buoyant work vests.
ii. Prior to and after each use, the buoyant work vests or life jackets shall be inspected for defects which would alter their strength or buoyancy. Never use any defective unit.
iii. Ring buoys with at least 30 metres of line shall be provided and readily available for emergency rescue operations.
iv. At least one lifesaving boat shall be immediately available at locations where employees are working over or adjacent to water.
v. Employees shall wear approved buoyant protective equipment at all times while working on or over water, such as:
    a. On floating pontoons, rafts, and floating stages.
    b. On open decks of floating plants (such as dredges, pile drivers, cranes, pond saws and similar types of equipment) which are not equipped with bulwarks, guardrails, or lifelines.

c. During the construction, alteration, or repair of structures extending over or adjacent to water except when guardrails, safety nets, or safety harnesses and lifelines are used.
d. Working where there are potential drowning hazards, regardless of other safeguards provided.
e. On floating logs, broomsticks, and unguarded walkways.
f. On boom boats and other work boats.

If in a boat, wear a life jacket. Work with a partner and do not overload the boat. Use only boat operators who have been trained in boating safety. Always follow the common rules of boating safety.

Do not wade barefoot in the water. Wear your life jacket if the water is over knee deep. It is good practice to always work with a partner. In still waters, wear chest waders and probe ahead with a pole for holes before proceeding. Limit wading to waist deep water. When working in moving water, do not wade in if the water is more than mid-thigh deep.

Only schedule work at the ocean shore for low tides. Never work in heavy surf. Always wear a life jacket. When working on inland shores, do not walk on floating debris and be cautious of recently puddled trenches and dredging fills.

4.4.4.24 VEHICLES

Defensive driving is the key to safe driving. The use of seat belts is mandatory for drivers and passengers in state vehicles and in private vehicles used on state business. The majority of vehicular accidents are of three types. They are:

i. Backing – Whenever possible, park so that backing up is not necessary. Never back without first checking to the rear. When visibility is limited, use a second person to provide guidance. Never back into a traffic lane unless adequate visual checks are made.
ii. Colliding with the vehicle ahead – Follow at a safe distance. In inclement weather, increase this distance (at least two seconds) to allow for poor conditions. Observe conditions as far ahead as possible. Pay attention to traffic, stay alert, and check for clear areas in case evasive action is needed.
iii. Rear end collisions – If you are about to be rear ended when you are not moving and cannot take evasive action, firmly apply the brakes and press yourself against the seat back and headrest.

Adjust your speed to the weather. Posted speed limits are for ideal conditions only. Reduce speed in rain or patchy fog. In heavy fog park off the road and turn off all exterior lights except flashers; otherwise, another driver might think you are still on the road and hit you from the rear. When roads are slippery, start braking earlier than you would under ideal conditions.

Do not drive unless you are physically and emotionally able to drive safely. Alcohol, fatigue, and illness slow reflexes. Some medicines impair driving performance. So called "stay awake" drugs are not effective.

Unless vehicles are being used as protective barriers, park them completely away from highway traffic. If a vehicle must be parked within 3 metres of a traffic lane for more than fifteen minutes, and if the consequences of its being hit include possible injury to personnel, close the shoulder.

4.4.4.25 BASIC SAFETY RULES OF OPERATION

In addition to the guidelines above, the following are some basic rules to be used while operating or riding in a vehicle. Never let job urgency transcend safety. Use defensive driving techniques at all times. Allow for limited visibility, acceleration, braking, and the large size of survey vans and other heavily loaded survey vehicles. Check on the safety of the vehicles before operating them and do not knowingly operate an unsafe vehicle. Use seat belts and require all passengers to have their seatbelts fastened before the vehicle is under way. If you have been drinking alcohol or taking medications, drugs, or any substance that might impair your physical or mental faculties, do not drive. Do not stand in any part of the vehicle while it is in motion. Passengers must be seated with their seat belts fastened. Always Park vehicles in a safe manner and in a safe place. Routinely double check to see that the hand brake is firmly set and the transmission is in low, reverse, or park if the vehicle has an automatic transmission. Turn the wheels when the parking site presents a possibility of a roll away. Chock blocks may also be used to avoid roll away.

When in doubt, check overhead clearances. Keep all tools and equipment securely fastened in their designated places. Obey all traffic laws, signs, speed limits, and signals. Keep all survey truck cabinets closed when not in use. Do not overload vehicles and never exceed the intended capabilities of a vehicle. Always obey the Highway Code. The only time the Highway Code does not apply to employees is when they are working on the surface of a highway and have sought approval. Verify the safety of each vehicle with a pre-drive check before operating it. This includes but is not limited to checking:

i. Tires for inflation and adequate tread.
ii. All illuminating directional and warning lights, as well as gauge lights.
iii. Windshield wipers and condition of blades.
iv. Brakes and steering.
v. Mirrors.
vi. Horns.
vii. door and hood latches, windows, seat belts, etc.
viii. Trunk or other storage for jack, lug wrench, reflectors or flares, safety flares, spare tire,

first aid kit, and fire extinguisher.

ix. Motor oil and coolant levels.

Do not have more than manufacturer's recommended number of people in the vehicle. Except in emergencies, do not push a vehicle with another vehicle. Check the lug nuts on the survey vans weekly.

4.4.4.26 FLAMMABLES

Carry flammables in approved safety cans. Manufacturer's, as well as general safe charging, discharging and storage guidelines for batteries shall be strictly followed. If wet cell storage batteries must be used, carry them in tilt proof and splash proof boxes. Secure the boxes in the vehicle so they cannot shift or slide. Provide adequate ventilation.

4.4.4.27 UNDERGROUND UTILITIES

There are regulations and guidelines governing excavation where buried pipes, wires and cables are located. Caution should be used when any excavation or setting up of survey points is done in an area that could contain an underground utility. As well as being an inconvenience to the public if service is disrupted, damaging an underground utility can be financially costly and a serious threat to health and safety. Engage the relevant utility institutions for their network maps before beginning any excavation prevents damage to underground facilities, service interruptions, and bodily injury.

4.4.5 TEMPORARY TRAFFIC CONTROL

[For specific guidance regarding temporary traffic control, refer to the current edition of the Ghana Highway Authority 'Guidelines for SIGNING AT ROAD WORKS.

4.4.5.1 INTRODUCTION

The primary function of temporary traffic control is to provide safe and efficient movement of road users through or around survey work areas while protecting workers.

Surveying work may involve multiple, short-duration activities using lightweight, portable equipment and often a single support vehicle. Therefore, quick deployment and portability are important in minimizing worker exposure and risk of injury.

Several techniques can be employed to adequately protect workers involved with this type of work that may appear to deviate from typical applications.

i. Portable Changeable Message Signs (PCMS), additional lighting and other more dominant devices such as high intensity rotating, flashing, oscillating or strobe lights on work vehicles may replace more typical advance warning sign sequences.

ii. Advance warning signs should be used to indicate the presence of workers in the road, as appropriate.

iii. Work vehicles may be placed to provide additional protection to the workers.

iv. Use of a spotter is recommended for work within the road where the worker is unable to monitor or respond to traffic themselves.

If it is necessary to work alone, a job hazard assessment must be performed. If the assessment indicates special risks for the task being performed when working alone then a communication plan must be developed.

4.4.5.2 BICYCLE AND PEDESTRIAN CONSIDERATIONS

Accommodate all road users (motorists, bicyclists, and pedestrians, including those with disabilities or visual impairments) at all times within a temporary traffic control work zone.

The placement of additional temporary signing and Traffic Control Devices (TCDs) for the control of non-motorized vehicles and pedestrians should be considered where a reasonable volume of users are expected and where work is expected to last longer than one hour.

Make every practical effort to satisfy the following:

i. Match the level of accommodation to the existing facilities available prior to the work.

ii. Use appropriate TCD to keep bicycles and pedestrians outside active workspaces and away from work equipment.

iii. Avoid placing bicycles and pedestrians in conflict with traffic, work site vehicles, materials or operations.

iv. If using an alternate route, provide sufficient and appropriate advance warning and detour signing for bicycles and pedestrians.

v. Unless an alternate route is provided, maintain a 4-foot minimum width for bicycles.

4.4.5.3 NIGHT OPERATIONS

Working at night when there is less traffic on the road can be the only practical way to accomplish some work tasks. Any time drivers must use their headlights for visibility should be considered the same as night conditions. Use the following basic principles for adjusting your traffic control for night conditions:

i. Use enough lighting to provide a safe work environment without creating glare in the path of road users.

ii. All devices, including flagger STOP/SLOW paddles, shall be retro-reflective.

iii. Signs, cones and worker safety apparel used at night should be kept in like new condition.

4.4.5.4 FLAGGING

When one direction of the road is closed and road users must alternately share the remaining open portion for both travel directions, flagging, pilot car operation or portable signals shall be used for the safety of workers and road users, including bicyclists and pedestrians.

4.4.5.5 SETTING UP THE WORK ZONE

This section provides guidelines and procedures for setting up the work zone.

4.4.5.6 TEMPORARY TRAFFIC CONTROL ZONE COMPONENTS

The temporary traffic control zone as shown in **Figure 4.27** has four parts and extends from the initial advance warning signs through the last temporary traffic control device.

i. **Advance Warning Area**: An advance warning area is necessary for all traffic control zones.
ii. **Transition Area**: In a transition area, traffic is channelized from normal public road lanes to the path required to move traffic around the workspace.
iii. **Activity Area**:
   a. Buffer Space is a short section of clear road between the cone taper and the work space which can provide an extra margin of safety for both traffic and workers. Buffer spaces should be provided when space is available but are optional.
   b. Work Space is that portion of the road which contains the work activity and that is set aside exclusively for surveyors and equipment.
iv. **Termination Area**: The termination area provides a short distance for traffic to clear the workspace and return to normal operation.

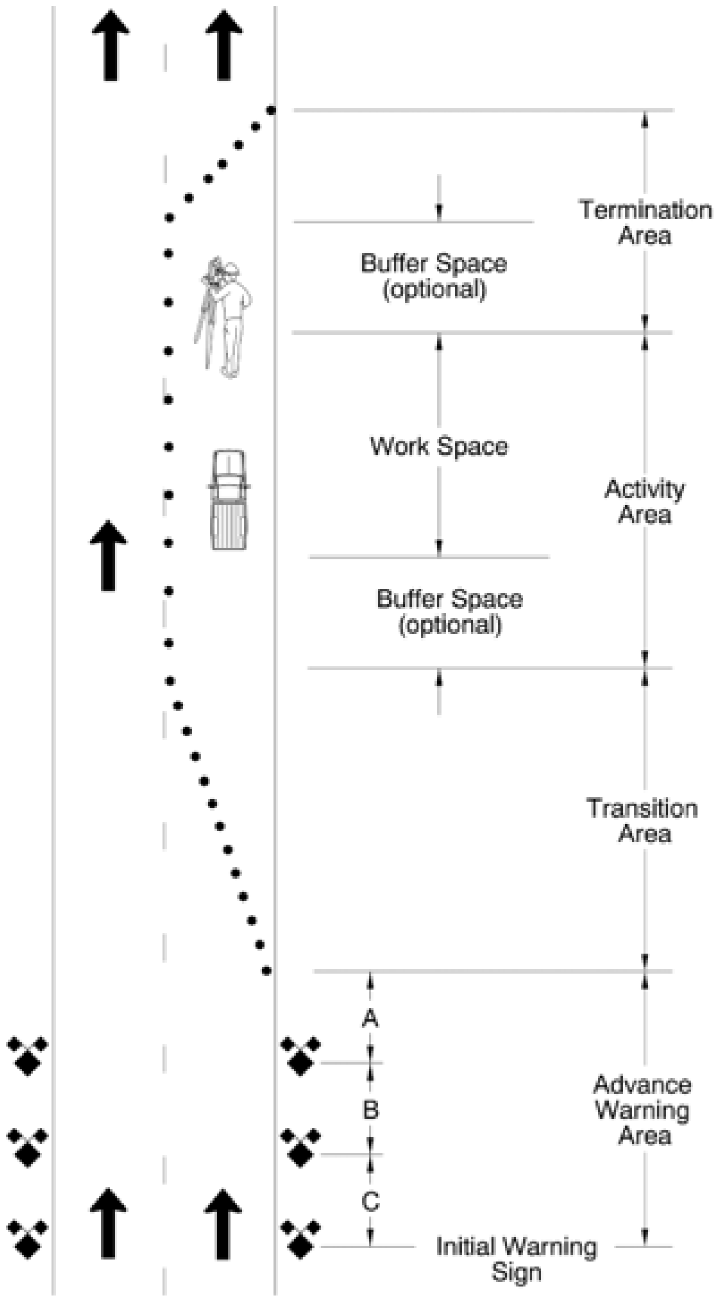


Figure 4.27 General Details — Work Zone Components

4.5 REFERENCES

1. https://surveyorinsider.com/control-points-surveying/#google_vignette
2. https://www.takeoffpros.com/2019/07/31/gps-surveying-explained/
3. https://www.southalabama.edu/geography/allison/GY301/Total%20Station%20Setup%20and%20Operation.pdf
4. Road Geometric Design Manual, Ministry of works, Tanzania 2011 Edition
5. The Ethiopian Road Authority Geometric Design Manual, 2013.
6. Survey Manual, Colorado Department of Transportation, 2021.

7. Highway Design Guidance Manual, Commonwealth of Kentucky Transportation Cabinet, March 2017.
8. Geometric Design Manual of Uganda, (2005).

CHAPTER 5

ROAD SAFETY, DESIGN CONTROL AND CRITERIA

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CHAPTER 5 ROAD SAFETY, DESIGN CONTROL AND CRITERIA

5.1 ROAD SAFETY

5.1.1 INCORPORATING SAFETY INTO ROAD DESIGN

Globally, road crashes account for an estimated 1.35 million deaths and 50 million injuries each year, with over 90 percent of the reported deaths occurring in developing countries [1]. In Ghana, it is estimated that 1,770 deaths and 12,500 injuries occur annually [2]. There are known, cost-effective solutions that can be implemented to address this global crisis. The 2030 Agenda for Sustainable Development recognizes that road safety is a prerequisite to ensuring healthy lives, promoting well-being, and making cities inclusive, safe, resilient, and sustainable. The Decade of Action for Road Safety 2011–2020, officially proclaimed by the United Nations (UN) General Assembly in March 2010, had a goal to stabilize and reduce the forecasted level of road traffic deaths around the world. To continue this global focus on improving road safety, the UN General Assembly has adopted a new resolution on global road safety, proclaiming the period 2021–2030 as the Second Decade of Action for Road Safety with the goal to reduce road traffic deaths and injuries by at least 50 percent by 2030.

A substantial reduction in road deaths will only be feasible if concerted efforts are made, following the “Safe System” approach involving all elements of road safety, management, and delivery. This includes all pillars of the Safe System - starting from road safety management, safe roads and roadsides, safe speed, safe vehicles, safe road users, and post-crash care as depicted in **Figure 5.1**.

Safe Road Users	Safe Vehicles	Safe Speeds	Safe Roads	Post Crash Care
The safe System approach addresses the safety of all road users, including those who walk, bike, ride transit and travel by other modes.	Vehicles are designed and regulated to minimise the occurrence and severity of collisions using safety measures that incorporate the latest technology.	Humans are unlikely to survive high speed crashes. Reducing speeds can accommodate human injury tolerances in three ways: reducing impact forces, providing additional time for drivers to stop, and improving visibility.	Designing to accommodate human mistakes and injury tolerances can greatly reduce the severity of crashes that do occur. Examples include physically separating people traveling at different speeds, providing dedicated times for different users to move through a space and alerting users to hazards and other road users.	When a person is injured in a collision, they rely on emergency first responders to quickly locate them, stabilize their injury, and transport them to medical facilities. Post-crash care also includes forensic analysis at the crash site, traffic incident management, and other activities.

Figure 5.1 The five pillars of safe system approach for Road Safety

This guide focuses on elements of safe road and roadside designs for road networks that can provide safe mobility to all road users, as well as complementary changes to improve speeds, vehicle safety, road user behaviours, and post-crash care. A balanced road design must take into account these complementary system elements to maximize safety benefits. The energy carried by a moving object is proportional to the square of its speed. A well-designed "forgiving roadside" ensures that this energy is dispersed in a crash, and as a result, less energy is transferred to the occupants.

Road infrastructure design plays a vital role in road safety outcomes. Safe infrastructure supports other road safety pillars by encouraging appropriate road user behaviour (such as appropriate speed and correct lane position) and by providing a forgiving road environment if things go wrong. Poorly designed road infrastructure can give rise to dangerous road user behaviour. One of the key realizations of the Safe System approach is that drivers make mistakes and will continue to do so, even if we can reduce how often these occur. This road user error has long been recognized as a significant contributor to poor road safety outcomes. However, roads of any given speed can be designed to reduce the likelihood of crashes occurring, and there is very clear evidence that the severity of outcomes when crashes do occur is significantly influenced by the road design. [3]

The Safe System approach highlights that a shared response is required to address road safety. This means that road users will continue to take responsibility for their actions, for instance by being alert and compliant with road rules. However, it is also recognized that road managers and designers have a significant responsibility to provide a road system that protects all road users. This can be achieved through appropriate designs of roads.

As an example, if a driver runs off the road and sideswipes a tree at high speed, there is a very high probability of a fatal or serious crash outcome. In this same situation, if road users were protected from the tree by a well designed and installed roadside barrier, the risks to the occupants would be significantly reduced to the extent that it is likely that only minor damage would occur to the vehicle, but that there would be no significant injuries (assuming a reasonably safe and well-maintained vehicle). This is regardless of the cause of the crash: impairment, misjudgement of speed, fatigue, distraction, drugs, alcohol etc. The same protection occurs when pedestrians and cyclists are adequately separated from motorized traffic, or when speeds are managed through traffic calming to appropriate levels considering the presence of road users. Similarly, when vehicles travelling in opposing directions at high speeds are separated by barriers, the risk of a head-on collision occurring is greatly reduced.

The provision of this safe road infrastructure relies on good decision-making by recognizing key risk factors while planning road infrastructure and incorporating appropriate design elements to address these risks. This also requires an understanding of the key crash types that result in death and serious injury. These crash types include collisions with vulnerable road users (including pedestrians and cyclists); run-off road, head-on, high-angle collisions including right-angle crashes at intersections; and rear-end crashes. A long-term vision is required to provide improved design to support safe road design and use following safe system principles.

5.1.2 SAFE SYSTEM GUIDING PRINCIPLES TO SAFER DESIGN

The following Safe System principles are recommended to ensure safety in sustainable road transport system design:

1. **Inclusiveness**

   Road design needs to be for all road users - not only for motorized vehicles.

   The implication of this is that designers need to cater for the most vulnerable road users present. In doing so, safety will typically be improved for all road users.

2. **Road functionality**

   Roads serve two functions: “access and mobility” or “movement and place.” Roads serve two primary functions or “roles”: to facilitate the movement (mobility) of people and goods and to act as places (access) for people. For safe design the “actual function,” not the “intended function” should be identified. In cases where mono-functionality cannot be realized in the short term, efforts should be made to provide adequate safety through safe speeds, starting with provision for the most vulnerable road users.

3. **Clarity**

   Design should meet road users’ expectations and be free from any surprise to road users. In case of practical limitations, clear delineation (e.g., markings and signs), adequate sight distance (e.g., decision sight distance), and/or speed management should be used to provide safety for all road users. In addition, variations in key design parameters along the road have an impact on traffic flow and safety. Such transitions should be supported by safe speed reductions, for example, traffic calming. This is applicable in case of variation in cross-section design near bridges/culverts, for roads passing through villages and towns, at-grade crossing facilities for vulnerable road users, and so forth.

4. **Homogeneity**

   Design should limit differences in vehicle speed, direction of travel, mass, and size. The design should ensure that vehicles (road users) travelling at different speeds are not able to interact (e.g., fast moving vehicles and vulnerable road users); that those travelling in different directions are not able to collide, especially at higher speeds, (for example in head-on conflicts), and that road users of different mass or size do not mix (for instance, trucks and vulnerable road users). Where it is not possible to provide designs that ensure separation, speeds need to be low. The implication of this principle includes that:

   i. Design should ensure the safe segregation of vulnerable road users from motorized traffic where operating speeds need to be above 30km/h, i.e., conforming to Safe System speed.

ii. Designs should ensure, whenever possible, physical separation between bi-directional traffic in situations where speeds are above human tolerance levels (e.g., 70km/h for motorized vehicles that have modern safety features) and more so when visibility is restricted.

5. **Safe Speed**

   Design should support Safe System speeds. The determinant of "safe design" is the safety of the most vulnerable or least protected road user and their tolerance to impact forces during a collision. This survivability is largely dictated by the impact speed for different road users. Hence, similar to "design vehicle," the concept of "design road user" should be adopted to ensure safety, especially when considering the speed environment.

6. **6. Forgiving roads and roadsides**

   Roads and roadsides should be forgiving, i.e., free from hazards. In higher speed environments roads and roadsides should be free from permanent as well as temporarily fixed objects, such as rigid structures, trees, stopped/parked vehicles, etc., and should be protected if vehicle departure is non-recoverable.

7. **Minimized exposure**

   Design needs to minimize exposure to risk for all users. This can be achieved at the planning stage by providing good quality, safe infrastructure that encourages modal shifts (e.g., from motorcycles to mass transit systems in cities). Exposure to risk can also be managed through the provision of safe infrastructure elements. As an example, intersections can be designed to remove or eliminate exposure by banning turning movements across multiple lanes of traffic.

8. **System design**

   Road design should be done in a way to support other elements of the Safe System. For example, it may be possible to build post-crash response into the design (e.g., providing shoulders to park disabled vehicles or access of emergency vehicles, providing for safe enforcement activity).

5.1.3 KEY ROAD DESIGN PRINCIPLES IN THE CONTEXT OF SAFE PLANNING

This section discusses the key design principles in the context of safe planning. They are:

i. General Road Design Principles
ii. Road Function and Land Use
iii. Vehicle and Road User Type
iv. Context Sensitive Design
v. Community Engagement
vi. Innovation

5.1.3.1 GENERAL ROAD DESIGN PRINCIPLES

Road infrastructure design plays a significant role in road safety outcomes, but typically safety is just one consideration among many during the road design process and is often not prioritized. Road design and construction involves the geometric and structural design of a roadway. A key objective of this is to optimize operational safety and transport efficiency within constraints (including budgets, environmental concerns, and other social outcomes). Design needs to consider both the traffic volume and type that would be expected to use the road. This covers all user groups - motorized and non-motorized.

Road engineers design road geometry to ensure stability of all vehicles when negotiating curves and grades and to provide adequate sight distances for undertaking passing and stopping manoeuvres. The design choices related to geometric road design will depend upon the environment through which the road passes - principally habitation and topography - and the interactions between these design features and the environment have a fundamental impact on safety. Each design situation is unique and there are no ‘off the shelf’ solutions that will fully address all situations encountered. The rigid application of charts, tables and figures is unlikely to lead to a successful and safe design outcome. Good design requires creative input based on experience, knowledge about the local environment (including road user considerations), and a sound understanding of design, allowing evidence-based principles and solutions to be effectively applied with refinements to the exact local circumstances. Processes and tools, such as road safety audit, to ensure safety is embedded in a proactive way in design are also required.

Any design **must**:

i. Address the needs of all road users.
ii. Be undertaken by a qualified road designer under the supervision of a professional engineer/senior design engineer, both with appropriate road design experience in line with the scope of the project.
iii. Be safe, ensuring that any recommended safety provisions are not reduced in favour of saving costs during the design and construction process.
iv. Be context sensitive, including being suitable for the land use.
v. Demonstrate cost-effectiveness through value engineering processes, cost benefit analyses, and consideration of life cycle costs (which include safety benefits).
vi. Be fit-for-purpose, i.e., the function it is supposed to serve, while trying to achieve the highest possible standard of design, safety, and operational efficiency within the context of the site, the project scope, and budget.
vii. Be subjected to an audit process by independent and qualified road safety auditors.

It **should** also**:**

i. Be considerate of environmental, cultural heritage, and social requirements.
ii. Recognize the increasing impact of climate change on the resilience of road infrastructure.
iii. Maintain or improve the performance of an existing road.
iv. Fully document the rationale behind design decisions.
v. Meet the objectives of the project while being mindful of the objectives for the road link and network.
vi. Be able to demonstrate it appropriately balances all of the above principles within the limits of the project scope and constraints and is complementary to the network.
vii. Consider and cater for the interaction between all road users and the roadway.
viii. Meet current needs while also providing for future needs.
ix. Be developed in accordance with sound design guidance. Innovative designs may be developed using the foundations provided in accepted design guidance; however, all other road design principles should be maintained.

In the context of designing and providing a safer road environment, the Safe System approach aims to ensure that potential collisions are avoided and, if they occur, that the crash impact forces do not exceed human tolerance. Findings from Sweden identified that, while there was a strong interaction between the three system components of vehicles, road infrastructure, and road user, road-based factors, including speed, were most strongly linked to fatal crash outcomes [6]

Roads should therefore be designed to reduce the likelihood of crashes occurring and minimize injury to the road users even when a crash occurs, and there is very clear evidence that suggests that the severity of outcomes when crashes do occur is most heavily influenced by the road design. In particular, this includes the features that indicate to drivers the speed at which the corridor is designed to operate and the features which force lower speeds. The elements that are typically thought to impact on efficiency and safety include intersections, horizontal and vertical curves, camber (superelevation), gradients, cross sections (lane and shoulder width, medians and roadside provision), and merge and diverge areas.

It is noteworthy that roads should be designed to cater for a defined function and use. By adopting a consistent and clearly differentiated design for each function group, the road can create a better appreciation of risk in (most) drivers. This in turn encourages road user behaviour consistent with the safety standard of the road. The same general functional management principles should be applied in both urban and rural networks.

Appropriate design choices are needed for roads serving different functions to minimize the number of crashes likely to occur and to mitigate injury severity, particularly on higher-speed

roads. Further, it is also important to state that a consistent selection of minimum design criteria is not a good practice and that such choices often lead to unsafe and inconsistent design.
While road engineers concentrate on the geometric parameters, road users are more concerned with the context of the road and rely on visual cues and roadside details to determine safe and appropriate speed and risk. These elements need to be provided in such a way as to give all road users sufficient time to make appropriate decisions to avoid conflict and injury collisions. A balance is needed between too much and too little information, but whatever is provided needs to enable road users to assess an appropriate and safe behaviour.

Road infrastructure should be designed to proactively take account of the same injury tolerance criteria as those developed for vehicle occupant protection and pedestrian impacts, so that roads and vehicles together provide an effective safety system.
Safety is fundamental to the design and operational life cycle of a road. Safety should not be left to reliance on road users behaving safely - the millions of crashes and injuries globally each year show that this does not work. The process should start with a safety impact assessment of a proposal even before a decision is made where to site a new road.

A proactive approach is required to improve road safety. Safety audits are then undertaken at specific points during the design, construction, and post-opening stages to ensure all aspects of detailed design that might affect safety are addressed. A safety audit during the construction phase also helps to ensure that construction workers and road users are not at risk during developing and changing road conditions.

Once the road is built and accepted by the road agencies, they have a responsibility to ensure its safe operation. This is best done through a combination of a crash investigation and on-road inspection to enable cost-effective remedial programs to be developed; many tools exist to support these activities.

5.1.3.2 ROAD FUNCTION AND LAND USE

Different road classifications offer different levels of mobility and accessibility depending on their overall usage which require different traffic speeds, segregation of users and other driving actions e.g., readiness to deal with cyclists and pedestrians (including young children). Networks should reflect the development of a hierarchy of motorized use, with expressways/motorways at the highest level of motorized use and local/access roads at the lowest. In practice, a basic hierarchy will occur naturally through the more heavily trafficked routes being engineered to higher standards. But it is important that the hierarchy is established to clear guidelines linking design to actual function to provide the desired levels of mobility and accessibility. Additionally, road networks interlace and connect residential, commercial, urban, and suburban areas of cities, towns, and villages. They fulfil many functions along their routes catering for many types of activity, not just journeys by different modes. Thus, roads need to be designed for their actual function, and it should be recognized this may differ along their length.

It is not safe to assume that the intended function of a road will be its function along its whole length, or for its whole lifetime. By failing to take account of the changing context along the route this classification system limits understanding of how improvements, maintenance, or safety should reflect the wider functions that routes serve. Local roads within an urban area are often referred to as “streets” and are typically lined with buildings and non-travel activities including trade, play, and other forms of engagement. While movement is still an important requirement on streets, the ability to undertake other functions safely becomes increasingly dominant. These hierarchies often face a major challenge due to the typical distribution of mode shares consisting of a significant portion of non-motorized road users.

All roads can be designed to create different expectations about how road users should act on them. The aim is that different classes of roads should be distinctive, and within each class, features such as width of carriageway, road markings, signing, and use of street lighting would be consistent throughout the route and matched to their functional use. Drivers would thus perceive the type of road and “instinctively” know how to behave. The environment effectively provides a “label” for the particular type of road, and there would be less need for separate traffic control devices such as additional traffic signs to regulate traffic behaviour. They become “self-explaining,” i.e., more intuitive, to all users.

However, simply spending precious resources to achieve consistency on an otherwise safe and efficient corridor might not be acceptable. Therefore, a less onerous philosophy to achieve an acceptable level of consistency of facility along a corridor may be applied.

That philosophy is one of predictability. That is for successive sections along a road within a consistent environment (rural vs. urban), there should be little or no variation in the level of cross section, horizontal or vertical geometric standard, or sight distance provided. A “no-surprises” approach has a consistency of context that provides the users with appropriate and relevant information in a timely fashion to facilitate their decision-making. Any rapid or isolated changes, e.g., sharp curves or shoulder narrowing, would be considered “out of context” and would ideally be eliminated, but if they are unavoidable, then more specific, local treatment should be considered to give advanced warning of their presence to drivers.

Such approaches use simplicity and consistency of design to reduce driver stress and driver error and help guide driver behaviour and their speed selection. It is already used for the highest road classes (expressway/motorway), but on low-class roads, consistency in design is often compromised by other objectives such as high access levels, variable alignment, mixed use, and variable roadside development, which result in a lack of consistency and a lack of differentiation between road classes.

Developments affecting parts of the road system that have customarily been used for social or commercial purposes should therefore be handled with particular care. If it is possible to retain the social or commercial function, then care should be taken to separate through traffic movements from the local traffic in mixed activity areas and ensure that a high-speed environment is not imposed on it. If it is not possible to retain the social and commercial functions, then a suitable alternative site for these activities should be found, and the new road facility which replaces the former mixed activity area should be clearly identifiable as primarily a traffic facility. In a situation where high-speed through traffic cannot be separated from the local traffic and activities, downgrading of the functional class is needed to maintain safe travel speeds through such areas with the help of suitable infrastructure design and speed enforcement.

5.1.3.3 VEHICLE AND ROAD USER TYPE

The type, quality, and volume of motorized and non-motorised traffic and experience of road users are unique in each road link, primarily due to the socioeconomics, affordability of vehicles with modern technologies, and above all, the policies of the Ministry of Roads and Highways, Ministry of Transport and Ministry of Local Government.

As a result, there are several variants of vehicles under both light and heavy vehicle categories, with a broader range of acceleration-deceleration capability and the top speed that could be maintained. Furthermore, there is also a very high share of pedestrians, cyclists, makeshift vehicles, overloaded vehicles, vehicles for agriculture and farming, and animal-drawn vehicles. Such a broad heterogeneous traffic mix, with a high variation in travel speed creates an environment of increased accident potential.

While the vehicle dynamic characteristics vary widely in mixed traffic, there are few separate transport facilities; thereby, all vehicles use the same carriageways, often with poor or zero lane discipline. In addition to the motorized vehicles, the share of non-motorized traffic and pedestrians is very high on Ghana’s road network. However, most often there are no dedicated facilities for non-motorized transport (NMT) road users, resulting in higher conflict with motorized and NMTs, and a high share of crashes and injuries involving these vulnerable road users, including physically challenged persons. This often means that the standard of design and infrastructure provision needs to be higher for NMT (e.g., provision of adequate walkway and footbridges); yet the amount invested per kilometre on road projects is significantly lower.

In addition, where significant numbers of animals and animal-drawn carts use a path, consideration for collision risks and slow-moving vehicles should be given to the design to accommodate them safely (e.g., additional width, special signage, fencing, road furniture such as noise barriers or guardrails, segregation at intersections and crossings). Nonetheless, there is evidence that segregating a diverse traffic mix (especially where speed is involved), such as high-speed through traffic and low-speed local traffic with the help of service roads, and separating vulnerable road users such as motorcyclists, cyclists, and pedestrians through the introduction of motorcycle lanes, cycle lanes, and walkways, respectively, is likely to produce significant safety benefits.

Finally, integrating public transport facilities [e.g., bus rapid transit (BRT)] with well-designed crossing facilities is found to be effective in enhancing the safety of public transport users who are mostly pedestrians before and after using public transport facilities.

5.1.3.4 CONTEXT SENSITIVE DESIGN

A road design cannot be considered safe, fit for purpose, or conforming if it simply adopts design minima, particularly in combination, for elements of the design. Most design criteria (range, desirable, absolute) have been researched or developed in isolation from each other (although there may be some implicit relationships) and when used in combination with other elements, while conforming to the published guidelines, may result in a solution that compromises safety or operational efficiency.

Any road also has to operate appropriately within the natural and built environment to meet a range of expectations of the users and the broader community. Consequently, the design cannot be carried out in isolation, but must be sensitive to the context in which the road will operate and, as a result, competing or conflicting criteria often need to be compromised to achieve a balanced, safe and cost-effective solution.

Context-sensitive design (CSD) is an approach that provides the flexibility to encourage independent designs tailored to particular situations [7] while giving due consideration to all factors.

A “design domain” can be thought of as a range of values that a particular parameter might take. This applies to a range of design parameters that, when used in context, provide acceptable safe, efficient, and effective outcomes. They are justified in an engineering sense using a consistent set of principles, based on test data and sound reasoning, for example, and therefore can have a reasonable level of defence if challenged [8]. The design domain approach places emphasis on developing appropriate and cost-effective designs rather than providing a design that simply meets standards.

CSD seeks to produce a design that combines good engineering practice in harmony with the natural and built environment, and meets the required constraints and parameters for the project. It refers to roadway standards and development practices that are flexible and sensitive to community values. It also makes allowance for the use of narrower lanes, lower design speeds, sharper turns, and special features not included in generic road design guidelines to help create a more balanced and efficient transportation system and meet community land use objectives. However derived, a design should demonstrate value engineering and acceptable lifecycle costs to cater for all road engineering disciplines including safety, geometric design, traffic, drainage, pavements, asset management, and stakeholders (e.g., road users, vulnerable road users, freight, public transport, emergency services, environmental), while taking into account current and future needs.

At the beginning of the project lifecycle, the project needs to determine what road users are present and how they will be catered for. The suitability of a design should also consider the effects the design may have on adjoining road sections and the surrounding network. Designs require decisions to be made on the value of improving the standard of a road and the impact this might have on the ability to fund improvements elsewhere on the road system. Depending on the road agency's funding priorities, for example, this may be focused on safety, environment, or efficiency, which may drive different outcomes.

5.1.3.4.1 DESIGN EXCEPTIONS

Design exceptions are situations where the design does not conform to the minimum or limiting criteria set forth in the standards, policies, and standard specifications. They are most likely to occur due to challenging terrain, constrictions due to existing infrastructure, services, property boundaries, environmental conditions, cultural heritage and community expectations.

Design exceptions have the potential to negatively affect highway safety and traffic operations. For this reason, consideration of a design exception should be deliberative and thorough, and a clear understanding of the potential negative impacts should be developed through a risk assessment that is unbiased and supported by crash analysis. Sometimes the reasons for adopting design exceptions such as these may be for social, environmental, or economic reasons. However, the risk assessment must show that the decisions associated with adopting such a low standard outweigh the potentially higher cost of fatal and serious injury crashes. If the decision is made to go forward with a design exception, it must be formally approved by the relevant road agency and supported by a well-documented justification. It is also especially important that measures to reduce or eliminate the potential negative impacts be evaluated and, where appropriate, implemented.

Documentation for design exceptions should describe all of the following:

i. Specific design criteria that will not be met.
ii. Existing roadway characteristics.
iii. Alternatives considered.
iv. Comparison of the safety and operational performance of the roadway and other impacts such as right-of-way, community, environmental, cost, and access for all modes of

transportation.

v. Proposed mitigation measures.
vi. Compatibility with adjacent sections of roadway.

Design exceptions **should** not be used where any one of the following applies:

i. There is a crash history linked to the use of the design exception e.g. police crash reports indicate that limited visibility was a contributing factor to the crash. This is even more important in the following cases:
    a. If more than one such crash is reported
    b. Mitigating devices are already in place.
ii. Use of the same, or similar, design exception has been known to cause safety problems elsewhere on the network.
iii. The value of the design exception is well outside the range of values of the design domain.
iv. The design exception is an isolated case, for example, if a roadway contains generous horizontal curvature except for one (or a few) very substandard horizontal curves. In this case, drivers become used to the general standard of horizontal curvature and are less likely to adequately perceive and negotiate the substandard element(s). This is different from a roadway comprising tighter, but more consistent horizontal alignment which would cause drivers to be more alert and have a greater expectancy of tight geometric elements.
v. A design exception is combined with other geometric minima, especially other design exceptions. The greater the number of minima combined, the lower the likelihood that a design exception can be tolerated as one of these minima.
vi. On road restoration projects comprising higher function and/or higher traffic volume roads.
vii. The parameter being considered is intersection sight distance. In this case, the EDD values are the lowest that should be provided.
viii. Where little effort and expense is required to avoid using the design exception.
ix. On road restoration or low volume road projects where the pavement is being replaced, especially if minimal earthworks are required.

5.1.3.4.2 DESIGN FOR ROAD USER CHARACTERISTICS AND COMPLIANCE

Conventional roadway design standards and guidance define features such as lane and shoulder widths, design speeds, and minimum parking supply. They often reflect the assumption that bigger and faster is better, leading to a design that effectively exceeds the standard required for its intended purpose. This can result in higher traffic speeds, increased project costs, and roadways that contradict other planning objectives. For example, wider and straighter roads tend to increase traffic speeds and disperse destinations, which can reduce accessibility, safety, and liveability.

At the beginning of the project life cycle, the project needs to determine what road users are present and how they will be catered for. This requires that data are collected about who uses the road and how they use the road e.g., where do pedestrians walk, what percentage of vehicles are motorcyclists and what is the actual speed vehicles travel at. These data are important in understanding the true design environment, as opposed to a design for how people "should" behave. It is also important that at the start of each project phase the road-user and stakeholder requirements are clearly documented so that the designer can clearly understand how to develop a design that addresses the needs and requirements of all road users and balances these within the overall design solution.

5.1.3.4.3 COMPLETE STREETS

The complete streets approach is a modern approach to urban design aiming to address the safety and amenity challenges of all road users and redressing the old school focus on motorized vehicles. Complete streets are streets designed and operated to enable safe use and support mobility for all users. This includes people of all ages and abilities, regardless of whether they are travelling as drivers, pedestrians, cyclists, or public transportation riders. The concept of complete streets encompasses many approaches to planning, designing, and operating roadways and rights-of-way with all users in mind to make the transportation network safer and more efficient.

Complete street policies are set at the national, regional, district and local levels and are frequently supported by roadway design guidelines. Complete streets approaches vary based on community context. A complete street in a rural area will look quite different from a complete street in a highly urban area, but both are designed to ensure safety and convenience for everyone using the road, including pedestrians with disabilities.

In context, community roads are generally used by all modes of transport, with a high share of non-motorized users. However, with increased motorization, the streets and roads are taken over by motorized vehicles. The safety threats to the non-motorized users are on the rise due to a lack of planning and design in general, and the lack of speed management in particular. In this regard, a complete street design, especially for the mixed use is very relevant.

Complete streets may address a wide range of elements, such as walkways, cycle lanes, bus lanes, public transportation stops, crossing opportunities, median islands, accessible pedestrian signals, kerb extensions, modified vehicle travel lanes, streetscape, and landscape. They also reduce motor vehicle–related crashes and pedestrian crashes, as well as cyclist risk when well-designed cycle-specific infrastructure is included. [9] They can promote walking and cycling by providing safer places to achieve physical activity through transportation, which may in turn have positive impacts on health, including reduced obesity.

The design of a street can always be improved. Successful streets cannot be imposed but need a collaborative effort between the road agency or municipality and the local community which they serve.

5.1.3.5 COMMUNITY ENGAGEMENT

In road projects, community engagement is an inclusive process conducted throughout the project life cycle: during conceptualization, design, construction, maintenance, and operation. It is important that at the start of each project phase the road user and stakeholder requirements are clearly documented so that the designer can understand how to develop a design that addresses these needs and requirements. The process should be well thought through and planned, with clear programs for facilitators and experts. If done well, it will enhance local ownership and create an interface between the road implementing organizations and the community. The benefits of community engagement include:

i. Providing an opportunity to inform the community about why there is a need for the project, including the safety and broader benefits. That way the community can understand the options and make informed decisions.
ii. Good decision-making resulting from accessing good/additional information from the community.
iii. Establishing new networks and relationships (and further developing of existing networks).
iv. More local ownership of solutions to current problems and a higher level of responsibility for creating that future.
v. Increased local support for change, or even the power of community in demanding change.
vi. Strengthening communities by keeping them informed about local issues.
vii. Building trust and confidence among stakeholders and the community.
viii. Contributing to the identification and development of leadership in community road safety.
ix. Providing a say to those who tend to be less involved in or have barriers to participating in decision-making processes.
x. Extending democratic processes to stakeholders and the community in regard to community road safety.
xi. Fostering a sense of belonging and empowerment from working together.

Engagement with a community can be especially important in situations where difficult decisions need to be made. As examples, decisions on land acquisition, provision of bypasses, and changes in speed limits are areas where a community has a big role in improving safety. It is important to work closely with communities on these and similar topics to ensure all stakeholders have inputs to decisions and understand the broad implications of these decisions.

The time it takes to get community partnerships established is very worthwhile, as the people can provide valuable insights in relation to problem identification and the design of actions, and can act as "key informants," providing qualitative data that can help prioritize the problems identified by a data analysis.

Where crash statistics are inadequate, it is even more important that road users are consulted so that local knowledge helps ensure the correct problems and appropriate, acceptable solutions are identified. For example, the community can provide information on hazardous locations (where crashes often occur) and participate in offering solutions and developing measures aimed at addressing safety issues. These may include the addition of walkways, median barriers, bridge upgrades to accommodate pedestrians, improved lighting, signage, and fencing, as well as alignments around crossings with the purpose of reducing speed.

An important outcome of this approach is the information gathered from the community, which would not have been available through the normal processes of visual assessments and data collection and analyses. At the same time, the community takes ownership of the solutions implemented to address the problem.

5.1.3.6 INNOVATION

Generally, it takes several years from when new solutions or approaches are identified, introduced and evaluated, and then adopted into formal design guidance [10]. In other cases, there may be a need to identify new solutions, because no solution currently exists, or existing solutions are not fit for purpose (e.g., not producing the required safety benefits; costing too much; or changing demands, including from road users). For these reasons, there is often a need to go beyond what is currently included in design guidance in order to achieve objectives. Innovation is often needed to deliver safety and other project outcomes. However, this innovation must be done in a considered, evidence-based manner. Risks (whether safety related, financial, or other) need to be minimized, and so a robust process is required. The Permanent International Association of Road Congresses (PIARC) Road Safety Manual [11] suggests the following reasons in regard to road safety interventions:

i. Lack of knowledge regarding the treatment and its effectiveness.
ii. Lack of experience of how to install and maintain a treatment.
iii. Issues regarding transferability and differences in local conditions.
iv. Concern about legal liability if something goes wrong.
v. Concern about public understanding or acceptability.

There is a need to take care when trying innovative approaches, and new designs should be tested (pilot test) and shown to have positive benefits with no unacceptable negative impacts before they are implemented more widely. This may involve identification of positive case studies from other jurisdictions and further analyses (including reviews of literature on effectiveness and broader impacts; communication with those who have tried an innovative approach), small-scale trials (on-road, or off-road if risks are high), larger-scale implementation (including as part of demonstration projects), and then eventually full adoption. Each stage requires careful monitoring and documentation; based on the learnings from each stage, refinements might need to be made.

5.1.4 KEY ROAD DESIGN ASPECTS IN THE CONTEXT OF SAFE ENGINEERING

Speed is an important aspect closely linked with road design. In road design, “design speed” is used as a design control and is used to determine the various geometric features of the roadway. The assumed design speed should be logical for the topography, anticipated operating speed, adjacent land use, and functional classification of the road. On the other hand, travel speed or “operating speed” is the speed at which vehicles generally operate on a road. Excessive speed is the most significant contributor to fatal and serious crash outcomes. When a pedestrian is struck by a car at 30 km/h, they have a reasonable chance of survival, but above this, the chances reduce dramatically. The critical threshold for cars colliding at an intersection is 50 km/h, above which chances of survival decrease rapidly. Providing effective speed management can have profound benefits in terms of safety and other positive outcomes for urban, interurban, and rural roads.

5.1.4.1 DESIGN SPEED AND OPERATING SPEED

There are concepts of desirable and absolute minimum design speed for a particular type of facility. While the idea is to use the desirable design speed for the design of geometric elements, in no case should it go below the absolute minimum design speed for that facility. Absolute minimum design speed is specifically crucial to avoid substandard design due to restrictions in land availability and so forth.

Unfortunately, a designer has few variables that may be used to convey the design speed to a driver, especially outside built-up (urban) areas. The relationship between the desirable design speed, curve radii, and their superelevation, that is, side friction demand, should be consistent and so should the forward sight distance along the route or at intersections. Therefore, the level of demand on the driver is very important. There are significant safety factors built into the parameters that are dependent on the selected design speed.

As far as practicable, the road should be designed to operate at a speed equal or slightly higher (5 km/h) than the posted speed limit. This can be assessed by “sensitivity testing” the design for drivers travelling at higher speeds. Two examples of how this could be achieved are assessing the superelevation on the curves and the sight distance requirements. However, it should be acknowledged that geometry is not an appropriate mechanism by which to control speed, primarily because it relies too heavily on driver interpretation and feel. This is particularly relevant, for example, when horizontal straights and straight gradients are used in generally flat terrain.

In current practice, the term “operating speed” is defined as the speed at which drivers are observed operating their vehicles during free-flow conditions. This may not be at a safe speed and should not be used to define the appropriate speed limit. The 85th percentile of the distribution of observed speeds is the most frequently used measure of the operating speed associated with a particular location or geometric.

The posted speed or speed limit is the speed displayed with a regulatory sign and is used to set the legal maximum or minimum speed at which road vehicles may travel on a given stretch of road. Speed limits are often close to the 85th percentile operating speed of the facility, but as highlighted above, this measure should not be used to set speed limits for existing roads.

However, in some instances, speed limits are set at levels that are too high given the prevailing road corridor conditions (geometry and roadside) and the mix and volume of road users, particularly near built-up and market areas where there are many pedestrians and cyclists. It thus becomes difficult to achieve safe travel conditions under these circumstances, and several infrastructural, and enforcement-related interventions become essential. For further details on design speed and operating speed refer to **Section 5.7**.

5.1.4.2 SAFETY IMPLICATIONS

While the relationship between the operating speed and posted speed limit can be defined, the association between design speed and either the operating speed or posted speed limit cannot be defined with the same level of confidence. Further, below are common challenges that may arise while working with design speed.

i. First of all, it is possible that due to higher design standards and prevailing traffic conditions, the operating speed of a particular facility ends up being higher than the design speed. Such high operating speed would result in unsafe conditions for the existing land use and endanger road users of the facility.
ii. On the other hand, it is also possible that due to restrictions of site conditions, the minimum design speed could not be followed, which raises the issue of consistency in design.
iii. Additionally, design elements following minimum design speed as a criterion may lead to value design that may not always lead to safer performance.

5.1.4.3 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

i. Setting a target maximum operating speed is often very important, where speed enforcement is mostly absent.

ii. It is also essential to use infrastructure-based as well as enforcement-based road safety interventions to help restrict the maximum operating speed in a facility.

iii. The importance of such interventions is increased when the difference between the operating speed and posted speed limit is high, and the consequence of higher operating speed may lead to fatal and severe crashes.

iv. Infrastructure-based management of speed should ideally limit speeds to safe levels, which certainly means at the design speed. Often even this is not enough for safe operation.

5.1.4.4 SPEED MANAGEMENT AND TRAFFIC CALMING

Effective speed management involves identifying the actual functional road use for different parts of the network (reflective of all road user groups), selecting a safe speed limit to match that use, and providing appropriate infrastructure to support these speed limits where required. This can include developing treatments to reinforce the change in the road environment and appropriate speed requirement. It may also require support from police in enforcing the required speeds, particularly where matching the safe speed, design speed, and speed limit has not been adequately considered in the design process. Increasingly, in-vehicle technologies are assisting in ensuring appropriate speeds are maintained. In regard to road design, speed management needs the strong support of road infrastructure to ensure road users can clearly understand their required speeds. Particularly in lower speed environments, well-designed roads also contribute significantly to a road user's choice of speed. Refer to **Chapter 10** for further information.

5.1.4.4.1 SAFETY IMPLICATION

i. Effective speed management can reduce vehicular travel speeds, with subsequent safety benefits.

ii. Where safe speeds are provided (matching required road and roadside activity), there can be a significantly reduced frequency and severity of collisions (up to and even exceeding 60 percent reductions in death and serious injury). [12]

iii. Even with minor changes in speed, there can be significant safety benefits.

iv. Appropriate speed management can reduce the need for police speed enforcement, freeing up resources for other enforcement activity.

v. There are also numerous benefits beyond those for road safety, including potential greater incentives for using active modes (particularly walking and cycling, which produce broader health benefits; reduced emissions, noise and fuel consumption; and more "liveable" space for residents and visitors.

5.1.4.5 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

The current implementation factors in traffic calming include:

i. For maximum effect, combinations of traffic calming measures should be used, preferably as part of an integrated transport strategy.
ii. Community engagement on safety benefits may be required to avoid negative public feedback due to perceived inconvenience and a misconception of additional injury. This should be factored into timelines for project delivery.
iii. Where relevant, schemes should be designed to cater for cyclists and essential emergency services and other heavy vehicles so that these are not hindered.
iv. Narrowing the vehicle travel lanes is effective at reducing speed and providing space for sustainable modes.
v. Cost-effective traffic calming design solutions should be used.
vi. In many cases cheaper options (such as line markings to narrow lanes rather than fully constructed islands) can be as effective.
vii. Monitoring the effects of the treatments is also important, potentially starting with the lower cost, to fully understand how each one contributes and therefore where the highest value is achieved.
viii. Clear signing may be required, especially at isolated traffic calming devices, to alert road users and prevent traffic calming measures from becoming traffic hazards. Some treatment types can act as a road or roadside hazard.
ix. Speed limits should be consistent and aligned to the function, standard, and use of the road.
x. Speed humps and other devices need to be well designed to provide maximum safety benefits. Nonstandard designs that are not well understood by road users may create a hazard. (Refer to Chapter 10 on traffic calming measures)
xi. Some treatment types (humps, rumble strips etc.) can act as roadside hazards if not properly designed, signed, and maintained.
xii. Speed limits should seem realistic and credible so that drivers will adhere to them.
xiii. Maintenance of speed calming infrastructure should be prioritized after implementation to ensure continuous safety.

5.1.4.6 SIGHT DISTANCE

Sight distance is needed to provide drivers with enough reaction and manoeuvring (including braking) time to adapt to the road conditions. The faster people drive, the further they need to look ahead and vice versa, in order to read, understand, and react in time to a hazard. Warning and information signs may sometimes be so sited that they have poor conspicuity, and the detailing of the road may not provide sufficient additional clues as to the hazard or decision ahead.

Stopping sight distance is the minimum sight distance that must always be provided at any point on a roadway. It ensures a driver travelling at an appropriate speed can safely and effectively bring the vehicle to rest, including being able to see any objects along the vehicle path.

Intersection sight distance enhance visibility and awareness for all road users at an intersection. It is typically defined as the distance a motorist can see approaching vehicles before their line of sight is blocked by an obstruction near the intersection. The driver of a vehicle approaching or departing from a stopped position at an intersection should have an unobstructed view of the intersection, including any traffic control devices and sufficient lengths along the intersecting roadway to provide the driver with enough time to anticipate and avoid potential collisions. Pedestrians also need to see and be seen, and crossing movements are often concentrated at or near intersections. Refer to **Section 7.2.6** for further information.

Insufficient sight distance can be a contributing factor in crashes. Examples of obstructions include herds of animals, plants, parked vehicles, utility poles, buildings, and the horizontal and vertical alignment of the roadway.

5.1.4.6.1 SAFETY IMPLICATION

Insufficient sight distance, and the corresponding reduced time to react, increases the risk of rear-end crashes on the approaches and high angle crashes within the intersection. This is because motorists may be unable to see and react to traffic control devices (i.e., signals and stop signs) or approaching vehicles from both major and minor roads.

5.1.4.7 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

Adequate sight distance is essential to provide drivers with enough reaction and manoeuvring time to adapt to the road features and to other road users. This involves improving the triangle sight distance (visibility splay) at intersections, enhancing visibility for all road users at the intersection, and, in some cases, reducing excess sight distance that could encourage early decision-making, bearing in mind that it is always necessary to maintain the minimum sight distance required.

Below is a summary of strategies to improve sight distance. Depending on the crash risks and crash types, a combination of countermeasures should be considered. The measures taken should aim to achieve a situation in which the available sight distance is made sufficient through reduced operating speeds (not just speed limits) or other measures.

i. **Signs and markings:** For a conventional unsignalized intersection, an enhancement to the typical signs and pavement markings should be considered, although the effect may be limited.
ii. **Traffic calming devices:** Sight triangles required for stopping and approach distances are typically based on ensuring safety at intersections with no controls at any approach. This situation rarely occurs in urban environments and occurs only at very low speed,

low volume junctions. At uncontrolled locations where volume or speed presents safety concerns, consideration should be given to the addition of traffic controls or traffic calming devices on the intersection approaches. (Refer to **Chapter 10** on traffic calming measures).

iii. **Relocating obstacles**: If the most frequent crash types are angle crashes due to insufficient sight distance with an overgrowth of foliage, the most effective countermeasure would be to clear the intersection's sight triangles to improve sight distance. Similarly, signals, signs, buildings, and so forth also should be relocated when they obscure sight distance.

iv. **Physical barriers and medians:** As only placing signs is proven to be unreliable to control movements, physical barriers and medians should be installed to reinforce to drivers what is expected.

5.1.4.8 LINEAR SETTLEMENTS

Linear settlements (which are a group of buildings, small villages, or other developments) along major routes, inevitably lead to a mismatch between road design and use of the road. Traffic problems occur due to poor road network planning, poor enforcement of planning rules (where these do exist), and pressure from local businesses who see these locations as providing useful commercial access to passing motorists. These problems are accentuated by a lack of understanding of the safety risks that are present.

5.1.4.9 SAFETY IMPLICATIONS

i. These settlements lead to a mixing of high speed through traffic and local slow-moving traffic and vulnerable road users. This mixed function can lead to very high risks, particularly for vulnerable road users who may be attempting to cross and walk alongside the road.

ii. Other risks include poorly designed pickup and set down points for public transport (whether formal or informal), which also pose risks for pedestrians attempting to cross or walk along the road.

iii. There may also be slow-moving local traffic which may be manoeuvring, including turning movements into and out of local access points or side roads, and making U-turns. Despite these road user movements, the design of these roads often remains unchanged, with wide roads, poor facilities for vulnerable road users and local traffic, and high speeds.

iv. In essence, what were previously highways have been converted over time to local streets in regard to road use, but the road design may be unchanged. This creates confusion for road users and high levels of risk. This issue can occur at very discrete points on the road (one or two vendors selling goods to passing road users) through to sections that may be several kilometres in length.

5.1.4.10 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

Various solutions can be applied to addressing this problem of linear development. These solutions are of two main types: regulatory and infrastructure.

i. Regulatory approaches include development and enforcement of strict road and land use planning to prevent the development of houses and businesses at the side of the road. This may also require appropriate legal and enforcement powers and adequate resources to apply these. These approaches may also require education of the local community regarding the road safety risks and possible penalties for breaking planning laws.

   Roadside markets that are dotted along the carriageways/walkways and whose activities spills on the carriageway/walkways pose a major hazard in linear settlements. This situation puts pedestrians at high risk of death/injuries as they are compelled to share portions of the carriageway with motorised traffic. These must be addressed through the provision of safe off-road market facilities with parking spaces.

ii. A variety of infrastructure solutions are also available:

   a) The highest cost and most substantial response are to provide a bypass road around the affected area. It is important to ensure the new route has strict planning controls, and that new residential and commercial development are not allowed on this bypass route. This approach also requires infrastructure improvements for the linear settlement (the existing road) to provide better, lower speed facilities to cater for the road users that are present. This often involves road narrowing, widening of walkways, and the provision of safe pedestrian crossing facilities. With significant reductions in traffic, what may have been a four-lane road (two lanes in each direction) can now be narrowed to just two lanes, with adequate provision for pedestrians and other slower road user groups.

   b) Provision of a service road which provides lower speed access for local traffic and vulnerable road users. These may be used as a location for permanent businesses, public transport stops, or for temporary markets and sellers. For smaller areas of roadside activity, a well-designed lay-by may be adequate. Further measures are likely to be required on the main through road, as there will typically be a need for local road users to cross the road. There also needs to be good provision for entry and exit points between the through road and service road.

   c) Reduction in speeds for all road users through the application of traffic calming measures. Particular care is required to provide low speed, safe crossing points for pedestrian. Refer to **Chapter 10** on traffic calming measures.

5.1.4.11 ACCESS CONTROL

Access management/control is one of the critical elements of geometric design and is related to the management of interference with through traffic. A high-speed road with unlimited access will not serve the purpose of mobility, and at the same time, will pose a high risk to its road users.

The aims of access management are to limit the number of conflict points, separate the conflict points, removing turning volumes and queues from through movements. The benefits include not only reducing crashes but also increasing capacity and reducing travel time. For further information on access control refer to **Section 5.8**.

5.1.4.11.1 SAFETY IMPLICATIONS

i. Imbalance of access and mobility (movement and place) leading to high-speed environments where non-motorized and vulnerable road users are not separated from high-speed traffic.

ii. Inadequate consideration of the travel needs of non-motorized road users in the planning and design process as shown in **Plate 5.1**.

Plate 5.1 Lack of pedestrian walkway

iii. Improper and unsafe crossing opportunities for non-motorized road users as shown in **Plate 5.2**.

Plate 5.2 Improper and unsafe crossing points

iv. The unsafe crossing of pedestrians in a high-speed environment, with large numbers of uncontrolled access from local streets onto the main highway.

5.1.4.12 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

For better safety outcomes, it is helpful to have separate corridors that have designated restricted usage or priorities, that is, not all corridors are provided for all users. Some may be designated to the movement of freight/car priority with limited access to vulnerable road users, while others prioritize public transport and cycling with high accessibility. In case such separation is not possible, to tackle the issue of unsafe access management, the following treatments and design practices need to be followed whenever a highway enters built-up areas and settlements.

i. At-grade crossing facilities with marked uncontrolled crossings at two-lane and controlled and/or grade-separated crossings for wider roads such as four, six, or higher lane highways.
ii. Provision of walkway and cycle lanes to separate pedestrian and cyclist traffic from through traffic.
iii. Provision of pedestrian guardrails to channelize pedestrians only at the marked crosswalk such that random crossing of roads at undesignated locations could be prevented.
iv. Safe and marked public transport stops with bay facilities for boarding and alighting.
v. Where major roads are bordered by commercial or residential development, multiple minor accesses may be connected to a service road that connects into the main highway via a properly designed junction.

5.1.4.13 CONSTRUCTION, OPERATION, AND MAINTENANCE

As part of the construction, maintenance, and operation of a road network, there will be a requirement to review safety features and implement measures to ensure safe use of the network by all users. This will often require road works, temporary closures, or incident management while allowing traffic to flow as freely as possible. In addition, additional reviews of safety features will be needed throughout the lifetime of the road to ensure that safe operation of the roadway is maintained. **Plate 5.3** through **Plate 5.5** illustrate some unsafe practices in work zones.

Plate 5.3 Poor barricade and sub-standard stop/"road closed" sign

Plate 5.4 Inappropriate sign

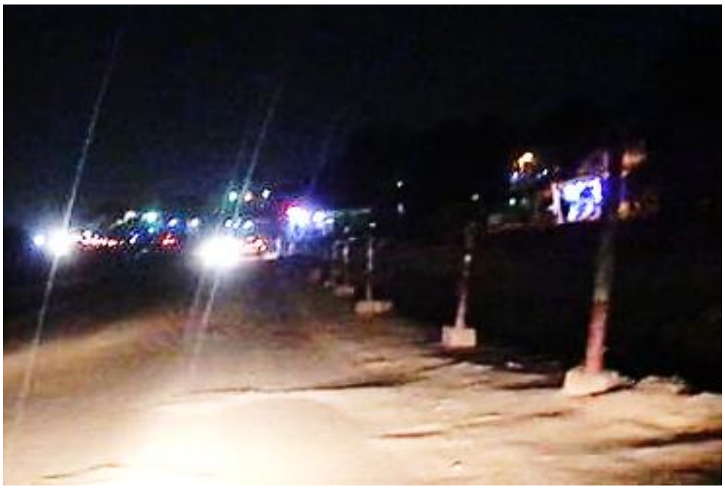


Plate 5.5 Low reflectivity of delineators

5.1.4.13.1 SAFETY IMPLICATIONS

i. Even the best design will produce poor outcomes if construction is poor (including not following design, use of different materials or design solutions during construction, and not adequately adapting to local factors (such as utilities and traffic mix).
ii. Poorly defined work zones can increase road safety risk for all users as shown in Plate 5.6.
iii. Even where adequate and comprehensive work zone traffic management arrangements are provided, they do not change with each phase of operation and materials and objects are often not protected or are left behind when construction is completed in that area.
iv. Construction materials/objects are often not removed even after the road is open to the public.

Plate 5.6 Absence of lateral buffer along active work zones

5.1.4.14 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

All work activities should be planned to optimize road safety, road space, and work efficiency while minimizing congestion, delays, and inconvenience for all road users. Refer to "Guidelines for the Signing at Road Works, 2007".

A. CONSTRUCTION AND MAINTENANCE

i. All reasonable steps should be taken to ensure that disruption due to the work is reduced to a minimum.

ii. Work zones must be clearly defined and protected to allow both road workers and the general public to adapt safely to the change in space and alignment.

iii. Traffic and road worker safety in work zones should be integral and high priority elements of every road construction project or road maintenance activity, from the planning process until project construction or maintenance work is complete.

iv. Work zone traffic management must not be associated with substandard traffic safety and if anything, the unusual and/or restrictive conditions found in work zones can require even higher standards of safety.

v. Subject to achieving an acceptable level of road user and worker safety, traffic amenity in a work zone should be as close as possible to that provided for in the normal operation of the road, including speed, permitted movements, access to abutting property, and provisions for non-vehicular traffic. However, in many cases restrictions on some or all of these aspects are necessary. These restrictions require clear advance warning, signage, and direction to operate safely.

vi. The same geometric and safety design principles which apply to the design of permanent roadways also govern the design of work zone traffic management treatments. For example, lane drops, lane narrowing, sharp curves, or other abrupt or frequent geometric changes must be appropriately designed and implemented in terms of design speed, advance warning, signage, and delineation to provide road users with effective clear and positive guidance.

vii. This may also require the introduction of geometric changes in individual steps or stages. for example, the closure of two lanes on a multilane highway should be done in two individual stages to allow traffic to change lanes smoothly and safely, and a lane closure should not end, and a sharp horizontal curve begin at the same point but should be separated.

viii. Road construction materials (whether in use or surplus) should be contained within a demarcated construction zone. If materials need to be placed along the roadway, delineation, demarcation, and signage should be given to warn and guide drivers.

ix. All construction materials/stored materials on the Right of Way (ROW) which can potentially harm road users or cause them to behave in such a way that can potentially lead them to an unsafe situation should be removed. **Plate 5.7** depicts materials placed along a project road which can potentially lead to traffic crashes.

Plate 5.7 The material on the road is inappropriate for traffic safety

x. All construction phases (i.e., different site layouts and access/routing arrangements) need to be subjected to an independent road safety audit.

xi. The whole of the construction process should be subject to a thorough safety assessment that considers the risk to both road worker and road users during the implementation of any works, including road safety audits during construction. This compares options for design, construction, operation, and decommissioning of the asset and assesses which has the lowest risk to the workforce and the travelling public during each phase. This does not necessarily lead to a change in preference for options; however, the risks should be identified so that they are taken into account during subsequent phases of the project. A specific Traffic Management Plan (TMP) needs to be developed that demonstrates safe routing of motorized and non-motorized traffic during construction, together with appropriate protection of construction site workers. The TMP should be subjected to road safety audit. A typical TMP is shown in **Figure 5.2**.

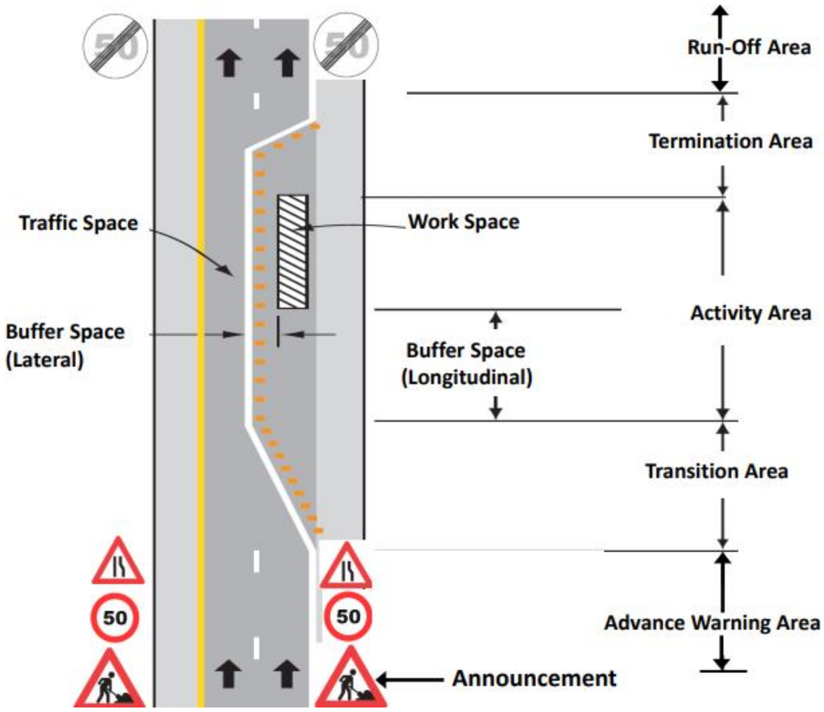


Figure 5.2 Typical Traffic Management Plan

B. OPERATION

i. When a scheme is implemented and open to use, it is still important to monitor and review the safety performance of the design to ensure that the predicted safety is achieved.

ii. Road Agencies therefore need to introduce a system for monitoring and reviewing the performance of any implemented road safety inspection or road safety assessment recommendations. This can then be used to identify the most appropriate safety improvements to incorporate into revised design standards.

5.1.5 VULNERABLE ROAD USER INFRASTRUCTURE DESIGN

Vulnerable road users generally refer to those modes of travel that do not include cars, public transport, or licensed commercial vehicles - those where the road users are protected from injury by an enclosed vehicle. It includes pedestrians, cyclists and motorcyclists.

5.1.5.1 SAFETY IMPLICATIONS

i. Roadway design generally caters for the needs of four-wheeled motorized traffic, neglecting the needs of pedestrians, cyclists, or motorcyclists.

ii. Facilities for a “typical” pedestrian may not accommodate a significant portion of users, including older adults, physically challenged persons, and children.

iii. Increased vehicle speeds are associated with increased injury severity and death for vulnerable road users. The provision of arterial roadways, intersections, and fast traffic lanes without adequate attention to facilities for other modes results in an increased likelihood that they will be killed or injured when using the road.

iv. Motorcycles, bicycles, and pedestrians are less easy to see, especially by faster moving vehicles.

v. High speed and volumes of motorized vehicles require the separation and protection of both pedestrians and cyclists as depicted in **Plate 5.8**. The risk of pedestrian injury is high when pedestrians share the road with vehicles travelling at fast speeds (greater than 30 km/h). Vehicle–pedestrian collisions are 1.5 to 2.0 times more likely to occur on roadways without walkways. [13]

Plate 5.8 Separation and protection of both pedestrians and cyclists from high-speed road

vi. Roadway designs in which facilities such as defined walking routes and signalized crossings are missing, inadequate, or in poor condition increase the risk of injury for pedestrians.

vii. Pedestrians falling into roads occurs where there is too little friction or traction between the footwear and the walking surface due to wet surfaces, weather hazards, and flooring or other walking surfaces that do not have same degree of traction in all areas. In addition, obstructed visibility of walkways (e.g., improperly placed signs or trees, poor lighting) also increases the risk. The quality of walkways is important for the safety of its users, including physically challenged persons.

viii. Intersections are associated with high rates of collisions and injuries because they include many conflict points. Uncontrolled intersections exacerbate such conflicts, as vulnerable users may encounter oncoming vehicles that are not required to stop or yield travelling at elevated speeds.

ix. Vertical separation (overpasses and underpasses) is expensive and require large amounts of space. They may also be inaccessible to some users, or even be unsafe from a personal security perspective.

Specific design requirements for pedestrians, cyclists, and motorcycles are considered in the following sections.

5.1.5.2 PEDESTRIAN FACILITIES

Pedestrian facility is a continuous, unobstructed, reasonably direct route between two points that is intended and suitable for pedestrian use and it include but are not limited to walkways, access ways, stairways and pedestrian bridges. It serves as the very foundation of the multimodal transportation system and they reduce traffic congestion and pollution by providing alternate means to vehicular travel. Pedestrian facilities are provided in areas where pedestrians need to cross or walk along the road. In practice, this would include residential areas, villages, markets, school zones, healthcare and hospital precincts, around places of worship, university hubs, public transport hubs and major train station zones, city centres, and central business districts (CBD).

Regardless of the primary means of transportation chosen (auto, bus, rail), nearly everyone is a pedestrian at some point in nearly every trip and pedestrian facilities serve nearly everyone. Because of this, it is important to provide dependable pedestrian facilities that are usable year-round by people of all abilities.

Pedestrians are often not provided with a safe and frequent crossing point or given the ROW. This means that they are left to signal their intent to cross by standing in the roadway, thereby increasing their exposure to a potential collision, which may result in death or injury. Refer to **Sections 8.10.1** and **11.10** for detailed information on pedestrian crossing facilities.

5.1.5.2.1 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

5.1.5.2.1.1 WALKWAYS

i. Mixed use of the road space is common in both urban and rural areas. A key consideration in providing safe routes and facilities for vulnerable users is the speed, size, and volume of all vehicle types.
ii. To promote a safe environment for walking, pedestrians must be provided with a complete network with sufficient space to walk along the public right-of-way.
iii. In urban and suburban areas where pedestrian volumes may be high, the most common form of provision is the inclusion of a paved or sealed walkway immediately adjacent to, and raised above, the vehicular carriageway as shown in **Plate 5.9** and **Plate 5.10**.
iv. 1.5m is considered the absolute minimum clear width to allow pedestrians to pass each other without having to move into the vehicular path. Increased width may be needed as pedestrian flows increase to prevent overspill into other use areas (i.e., cycle lanes or traffic lanes).

Plate 5.9 Urban walkway without protection

Plate 5.10 Urban walkway with protection

v. A zoning concept that divides the corridor into three main zones—the frontage zone, the pedestrian zone, and the furniture zone - can allow for the safe and convenient use of pedestrian space. Each of these zones plays an important role in a well-functioning pedestrian corridor.

vi. Walkways should be raised above the vehicular carriageway by at least 75 mm, with a defined boundary on both sides.

vii. In rural areas, where pedestrian traffic might be less frequent, walkable shoulders may be sufficient where vehicle flows are high. Care will also be needed to ensure that these shoulders do not become running or stopping lanes that might endanger pedestrian use.

viii. Pedestrians and vehicles are able to share the same space safely where speeds are less than 20 km/h. In these shared zones, pedestrian movements have equal priority with vehicles and vehicle speeds are low. Often this is a result of the high number of pedestrian movements compared to vehicles. Crucially, these are not major transport corridors, and alternative through routes for vehicles must be available.

ix. At speeds of 30 km/h, separate provision needs to be made where frequent pedestrian use is expected.

5.1.5.2.1.2 CROSSING FACILITIES

A crucial aspect of designing a safe and accessible pedestrian route is adequately dealing with crossing requirements of the motorized corridor. This can be done in several ways that are dependent on the concentration and volume of pedestrian and vehicle movements. Often pedestrians need to be guided to appropriate crossing points or deterred from crossing in unsafe locations. This is often achieved by using pedestrian fencing or guardrails close to the kerb edge. Unless alternative safe crossing points are available that are perceived as being convenient to use, any barriers may soon become damaged or stolen to recreate the more direct (even though dangerous) crossing point.

When considering pedestrian crossings at intersections, the ability to cross the minor road safely is as important as the crossing of the main road in order to provide consistent route continuity for pedestrians. The level of provision on the minor road need not be the same as on the major road, but it is usually safer to maintain the same level of control on each arm.

Additional consideration may need to be given at school crossing locations given the extra vulnerability of children. This may include lower speed zones, additional signage, enhanced crossing facilities, or even crossing supervisors/traffic wardens. Equal consideration needs to be given to pedestrians' crossing of minor roads and accesses away from formal junctions.

A. GRADE-SEPARATED CROSSING

i. Grade-separated crossings, whether underpass or overpass, are expensive pieces of infrastructure to install and need to be justified by demand and provide convenient crossing, otherwise they will be ignored.
ii. Where high volumes of pedestrians are concentrated in infrequent and specific locations, grade-separated crossings can be appropriate, either as a pedestrian overpass or underpass. They involve separating pedestrians from vehicular traffic by placing them at different levels and are often used where pedestrian crossing signals would cause delays and queuing or crashes (due to high traffic speeds). Pedestrian overpasses and underpasses require users to deviate from their preferred desired line. Pedestrian route selection is typically determined by the shortest, fastest, or most convenient route.
iii. Any deviation from the desired line, either vertically or horizontally, reduces the attractiveness of that route and increases the likelihood that it will not be used. Closure or obstruction of the direct route is needed to encourage use of the safer alternative.
iv. Ideally these facilities should have ramps rather than steps to accommodate the mobility impaired, but this often increases the length of any diversion.
v. Clear sight lines on approach and through the crossing and sufficient lighting must be provided with no places for people to hide, as they can be seen as a security hazard with the opportunity for personal attacks, especially at night.
vi. The risk of personal attack reduces their attractiveness and increases the likelihood that

crossings will not be used.

vii. To be effective they need very careful design and location to ensure ease of access. They also require sufficient lighting, adequate drainage, and proper maintenance to keep them in clean and tidy conditions.

viii. Often the provision of planned retail or vendors is good for increased security. Such design should be encouraged.

B. AT-GRADE CONTROLLED CROSSINGS

It is much easier to provide crossings at the same level as the rest of the route, but then this requires segregation in time, i.e., specific times for pedestrians and vehicles to use the same space.

i. Signalized pedestrian crossings at intersections aim to reduce vehicle/pedestrian conflicts.

ii. They provide right-of-way access to pedestrians during a green pedestrian phase when conflicting or all traffic is stopped.

iii. Pedestrian green time should be timed to give pedestrians long enough time to complete their crossing before the signals change to allow vehicle traffic to start passing through the crossing again. (Assume pedestrian walking speed of 1.2 m/s).

iv. Long waiting times for pedestrians can increase the likelihood of violations.

v. There can be compliance issues with vehicles failing to obey signals, or failing to give way when turning at signals is a common issue. A lead phase can be included at signals to give pedestrians an early start at signals before other road users are allowed to start. This is useful to reduce the incidence of turning vehicles hitting pedestrians at intersections, as this gives greater visibility to crossing pedestrians.

vi. Tactile paving should be provided to guide the visually impaired pedestrians through the crossing, and parking should be removed from the immediate vicinity of the crossing to provide adequate sight lines.

vii. To maintain the safety and segregation of uses, it is important that filter lanes are omitted where pedestrian crossings are in place.

viii. Countdown timers at signals can also provide phase duration information to pedestrians. The timers display the time remaining until the end or start of a pedestrian green phase and remove some of the doubt for all users.

ix. In addition to signalized crossings, other crossings that give priority to pedestrians typically consist of signs and painted road markings (“zebra crossings”).

x. These formalize the crossing location giving pedestrians the right-of-way over vehicles. They also increase the awareness for other road users that pedestrians may be present, improving expectations about the need to stop.

xi. They also cater for the mobility impaired with walkways ramped down to carriageway level or the carriageway lifted to walkway level.

xii. Audible and tactile warning of the pedestrian crossing phase can also be provided on the traffic signal pole.

xiii. Especially where vehicle approach speeds are high, at-grade raised pedestrian crossings can improve safety, but need to be clearly signed and have sufficient advance warning for drivers to react to their presence.

xiv. Extra care is required when designing signalized pedestrian crossings either at intersections, or away from intersections, in higher-speed, multilane environments. Drivers may fail to stop either because they fail to see the signals or do not comply, and this results in high severity outcomes.

xv. Raised pedestrian crossings have a similar profile and speed reduction effect as flat top speed humps, but they differ in that they give priority to pedestrians rather than motorists.

xvi. Narrowing of the roadway can also provide safety benefits; as pedestrians have less distance to cross, facilities can be included to make pedestrians more visible, and speeds may be reduced. Alternatively, the crossing movement can be split into two with provision of a protected median or refuge for pedestrians.

C. AT–GRADE UNCONTROLLED CROSSINGS

i. Wide crossings (of more than two lanes) can be narrowed by providing central refuge islands to enable pedestrians to cross the road in two stages, thereby limiting the amount of time pedestrians are exposed to traffic.

ii. Pedestrians and drivers need to maintain alertness where pedestrians are crossing multilane roads, as they are often hidden from drivers' view, and vice versa, by vehicles in adjacent lanes.

iii. Pedestrian refuges are raised median islands in the middle of the road that provide an area for pedestrians to safely wait until an appropriate gap allows them to cross.

iv. Islands need to be wide enough (minimum of 2.0m) to protect pedestrians with strollers (and cyclists) from passing traffic.

v. Refuges are particularly useful for those who are wheelchair-bound, elderly, or otherwise unable to completely cross the road in one movement.

vi. Islands can also have additional benefits, including acting to separate traffic moving in opposite directions, controlling vehicle speeds by narrowing the roadway, and providing motorists with an indication of where pedestrians might cross a roadway.

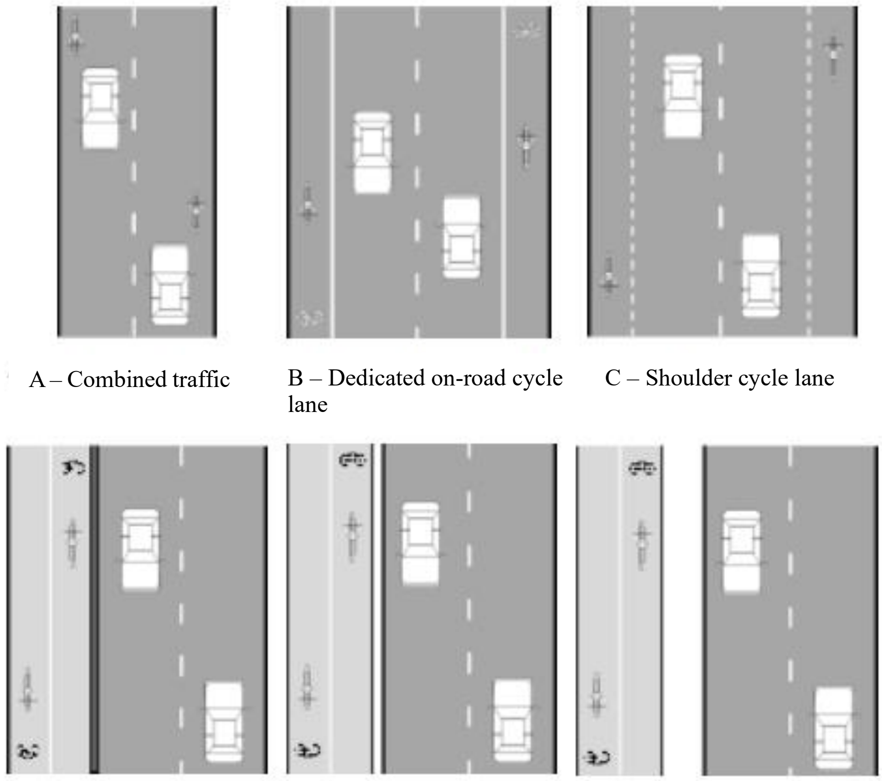
vii. Walkway ramps with tactile paving need to be included to make them appropriate for all mobility conditions.

viii. Refuges alone do not give any priority for pedestrians to cross.

5.1.5.3 CYCLIST FACILITIES

Safe cycle provision can be achieved in a number of different ways, from separate cycle networks to on-road painted cycle lanes as shown in **Figure 5.3**.

Cycle highways are separate paths for cyclists (and pedestrians) away from motorized traffic. They can facilitate daily, long distance cycle journeys. This may be as a regional connection, a commuter route into a business district, or between residential areas.



A – Combined traffic

B – Dedicated on-road cycle lane

C – Shoulder cycle lane

D – Separated kerbed cycle path

E – Separate cycle lane with narrow separation

F – Separate path for pedestrians and cyclists

Figure 5.3 Examples of Cycle paths

Cycle highways provide direct, flat, and continuous tracks that often link popular origins and destinations. Cycle roads (also known as “boulevards”) are a form of mixed-traffic road where the needs of cyclists (and possibly pedestrians) are prioritized over motor vehicles. Cycle lanes provide a physically separated space in which people who cycle can travel without mixing with motor vehicles - through either a physical barrier or raising the track to a higher level (or both), incorporating appropriate side clearance as shown in **Figure 5.3E** or **Figure 5.3F**. Cycle lanes can be relatively quick and inexpensive to implement, making them one of the most common forms of cycle paths implemented in cities. They can be either on-road (as indicated in **Figure 5.3B** or **Figure 5.3C**) or off-road or shared footways (as shown in **Figure 5.3D**) and allow people who cycle to take advantage of the accessibility that the existing road network provides.

5.1.5.3.1 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

Cyclists consist of a wide range of abilities and uses, from occasional recreational use to regular commuters and sports cyclists. The needs of each group are different and need to be accommodated in any specific provision. Basic quality design principles aim to increase actual and perceived safety, and include:

i. Limiting conflict between cyclists and other cyclists, pedestrians, or motorists.
ii. Ensuring low-stress environments where mixing with other users is limited and controlled.
iii. Separating main routes for cyclists from pedestrian routes.
iv. Reducing motor vehicle traffic volumes and speeds around cyclists, especially when road users mix.
v. Separating cyclists from fast/heavy motorized traffic, reducing the number of dangerous encounters - including separation on routes and/or at intersections and on-street parking.
vi. Ensuring conflict points at intersections and crossings are clearly presented so that users are aware of the risks and can adapt behaviour appropriately.
vii. Visibility of cyclists to motorists should be maximized at the approach to intersections.
viii. Ensure cycling infrastructure is well maintained - especially quality of pavement and continuity through intersections. Wide shoulders may be provided to allow for cyclists' use, along with protection from vehicular traffic using shoulder rumble strips or physical barriers.

A. CYCLE TRACKS

i. For cyclists, the use of segregated cycle tracks as shown in **Plate 5.11** and **Plate 5.12** is the ideal solution.
ii. To be effective, they require parking enforcement to avoid vehicles blocking them, and careful treatment at junctions.

Plate 5.11 Urban Cycle Track

Plate 5.12 Rural Cycle Track

iii. Along straight sections of the carriageway, cycle tracks provide greater protection for people who cycle compared with cycle lanes, as they are physically separated from the traffic lanes.

iv. Buffer zones between cycle tracks and parked vehicles or moving vehicular traffic are strongly recommended.

v. At intersections, designs must ensure that the visibility of people cycling to motorists is maximized.

vi. Where possible, priority should be given to people who cycle at intersections on cycle tracks (especially where it is given to traffic on the adjacent carriageway).

vii. Clear markings and accompanying signage should be in place to increase the visibility of the cycle tracks as shown in **Plate 5.13**.

Plate 5.13 Clear makings on cycle tracks

viii. Cycle tracks should be wide enough for people who cycle to feel comfortable and safe (minimum 3m) and allow overtaking between cyclists moving in the same or opposite directions. (Overall width will depend on the volume of cyclists).

ix. Where they allow two-way cycling, centreline marking should be provided to regulate safe and expeditious flow of cyclist traffic.

x. The surface of cycle tracks should be smooth (closed surface paving) and level and well maintained.
xi. Preferably the surface should be coloured and cycling symbols used to improve awareness and understanding.
xii. Roadside objects can present a hazard to cyclists, especially at higher speeds, and so should be removed or protected where possible.

B. CYCLE LANES

i. When the design of the cycle lane follows best practice and implementation is part of a coherent network, cycle lanes offer a safe and convenient route for people who cycle to travel around a city.
ii. In rural areas, cycle lanes can also be provided on the paved shoulders.
iii. They should only be applied on roads with medium or low motor vehicle volumes and speeds.
iv. Where vehicle speed and/or volume are high, then separate cycle lanes should be used.
v. Cycle lanes should be wide enough for people who cycle to feel comfortable and safe, allowing for comfortable clearance of other users, with surfaces smooth and level.
vi. Minimum recommended width is 2m for a single direction.
vii. Clear markings and accompanying signage should be in place to increase the visibility of the cycle lanes.
viii. Buffer zones may be considered between the cycle lane and motorized traffic where safety is of concern, particularly where there is heavy freight traffic.
ix. Buffer zones between the cycle lane and parked vehicles are strongly recommended.
x. Parked vehicles on cycle lanes put cyclists at high risk of injury/death as they are compelled to share the traffic lane with moving traffic as shown in **Plate 5.14**.

Plate 5.14 Parked vehicles on cycle lanes

C. CONTRAFLOW CYCLE LANES

i. Contraflow cycling can contribute to improving conditions for cycling more generally within a city, improving the convenience to travel. This can be implemented through the following:

a. Unsegregated two-way cycling on an unmarked road (quieter roads), which can be implemented through the use of signage.

b. The use of designated contraflow lanes on one-way roads with a high traffic volume.

ii. Since almost all conflicts take place at road crossings, it is often considered sufficient to mark contraflow lanes at the crossings only (10m length).

D. INTERSECTIONS

The traffic intensity, speed and number of traffic lanes should guide the choice of the most appropriate intersection design. At any intersection, there will be conflict points between transport modes, but effective intersection design can reduce possible conflicts and increase safety and comfort for cyclists. Knowledge is increasing about types of infrastructure that can be provided at intersections to improve safety for cyclists. Good design will generally include the following principles:

i. Avoid mixing motor traffic with cyclists where the traffic flow and/or speed is typically high.

ii. On carriageways with low traffic volumes and low traffic speeds (typically 30km/h or less), cyclists usually mix with other road traffic, and cycling specific infrastructure is typically not necessary at intersections.

iii. Maximize separation of cyclists from dangerous traffic movements.

iv. Separate traffic light phases for people cycling and people motoring or separate routes by over/ underpasses.

v. Maximize the visibility of cyclists.

vi. Make drivers aware of cyclists on the approach to an intersection.

vii. Intersections should be easy to identify, understand, and safe to use by all transport users. This requires specific designs to underline the priority status of cyclists.

viii. For any type of intersection, the primary consideration for safety is visibility of cyclists.

ix. In situations where cyclists and motor traffic are approaching the intersection in close vicinity (i.e., cycle lanes or mixed traffic), it is assumed that drivers are aware of cyclists.

x. In situations where cyclists are separated from the carriageway, it is advised that the cycle path should be designed alongside the carriageway on the approach to the intersection to increase drivers' awareness of cyclists.

xi. Advanced cycle stop line gives cyclists advantage away from signal stop lines.

xii. Turning provisions may be needed at intersections for motorized vehicles cutting through cycle lanes to ensure cyclists are highly visible. This includes coloured road surfacing for the cycle lane and additional signage.

xiii. Single circulatory lane roundabouts are considered the safest intersection design for all users on moderately busy roads if designed correctly. They reduce the speed of approaching traffic and allow the smooth flow of traffic through the intersection. Two-lane roundabouts can be particularly dangerous for cyclists due to the movement of motor traffic between lanes. **Plate 5.15** depicts a single circulatory lane roundabout for cyclists.

xiv. When a busy cycle route crosses a main road with high traffic volumes, a grade-separated crossing is preferred.

Plate 5.15 Single Circulatory Lane Roundabout for Cyclists

5.1.5.4 MOTORCYCLIST FACILITIES

Although few physical engineering facilities to improve motorcycle safety exist, some measures have been identified and are considered important. Furthermore, motorcyclists will benefit from speed reduction measures where there is mixed traffic, as they are less visible to drivers (having a smaller profile) and often appear where least expected. Particular care needs to be given to the design of road and traffic engineering facilities where a large number of motorcyclists can be expected in the traffic stream. Although such measures will not completely eliminate motorcycle crashes, they will minimize their occurrence and reduce their severity when they do occur.

5.1.5.4.1 SAFETY IMPLICATIONS

i. Unlike other forms of motorized transport, there is very little protection for motorcycle riders and passengers due to their size, lack of stability, and manoeuvrability.
ii. When crashes do occur, they often have very severe consequences, especially at higher speeds or in situations where larger vehicles are involved.
iii. The chance of a motorcycle rider or passenger surviving a collision with a car is greatly reduced at speeds over 30km/h.

While many motorcycle, crashes involve collisions with other vehicles, a significant number are single vehicle crashes. These crashes include a rider:

i. Losing control and running off the road
ii. Overtaking or crossing the centreline (usually on curves)

iii. Hitting another vehicle (or other obstruction) from behind
iv. Being thrown from the motorcycle and hitting the road surface

The road environment has a significant influence on the risk of crashes involving motorcyclists. Contributing factors include:

i. Interaction with larger vehicles (cars, trucks).
ii. Road surface issues (such as roughness, potholes or debris on the road) and poor skid resistance.
iii. Water, oil, or moisture on the road.
iv. Excessive line marking or use of raised pavement markers.
v. Poor road horizontal and vertical alignment.
vi. Presence of roadside hazards.
vii. Number of vehicles and other motorcyclists using the route.
viii. Motorcycles also have very different road performance characteristics than other types of vehicles. They:
    - are less stable
    - can accelerate much more rapidly than other vehicles
    - may appear in positions where other road users do not expect them
    - may also suddenly change their lane position to avoid a surface hazard or irregularity
    - are much more manoeuvrable than cars or heavier vehicles
    - can negotiate constraining alignments much more easily.
ix. This latter characteristic poses major challenges for road designers and is a significant influence on the risk of crashes involving motorcyclists, as is the quality of the road surfacing and maintenance of potholes and utility covers.
x. Where drivers emerge from side roads, or come to the end of segregated lanes, their view can be obscured, making it more likely they will fail to see motorcyclists.
xi. Wide entries to priority intersections can encourage drivers to pull up on the offside of the rider, especially if the latter is on a low-powered machine. This increases the potential for injury when moving off and competing for the same forward lane space.
xii. Excessive entry width of the entry can also encourage two cars to pull up side by side, obscuring the adjacent driver’s view of oncoming traffic on the main road and increasing risk for motorcyclists.
xiii. The positioning of road furniture and vegetation affects clear visibility, which is critical for safety at intersections.

5.1.5.4.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

Increased safety can be achieved by the separation of motorcycles from other motor vehicles. This segregation can take one of two forms. Either exclusive motorcycle lanes or inclusive lanes can be provided. These joint lanes provide routes that pedal cyclists and other non-motorized vehicles can also use. Motorcycles can also share bus priority lanes where necessary.

A. EXCLUSIVE MOTORCYCLE LANES

Exclusive motorcycle lanes require a carriageway separate from that used by other vehicles.

i. They can minimize crashes at intersections by providing segregated routes or control.
ii. Their width and appropriateness will depend on specific usage - the higher the use, the greater the width and junction control.

B. INCLUSIVE MOTORCYCLE LANES

Inclusive motorcycle lanes are installed on the existing road and are usually located on the driver nearside of the main carriageway (next to walkways or shoulders) for each direction of traffic flow.

i. Motorcycle lanes may be separated from the rest of the road by painted lines or physical barriers.
ii. Some motorcycle and motor vehicle separation can be achieved by allowing the shared use of bus lanes. However, full consideration of the traffic flows of both types of vehicles is important - shared use at specified times of the day could be a possible acceptable measure.
iii. Alternative measures may be needed on shared links to prevent four-wheeled vehicle access, i.e., by using posts at the entry/exit points.
iv. Care is needed to not encourage the sharing of all facilities such as pedal cycle measures at intersections or even on footbridges, due to the differences in respective vehicle speeds.

Despite the provision of separate small moving vehicle lanes, the shared use by non-motorized vehicles and motorcycles is generally not allowed, and motorcycles must usually use the main carriageway.

C. ALIGNMENT

To cater fully for the needs of motorcyclists, road design needs to consider:

i. Consistent horizontal alignment such as avoiding bends that tighten after entry.
ii. Smooth transitions in vertical alignment to minimize loss of tire adhesion and to prevent water collection. This has a greater effect on motorcycles than on twin-track vehicles (i.e., traffic calming ramps at junctions).
iii. Cross-sectional designs consistent with the speed of the road and the radius of the bends where adverse camber or inadequate superelevation can have graver consequences for motorcyclists than other vehicles.

iv. Specification and positioning of road furniture, including impact characteristics when struck by a fallen or sliding body, are crucial to minimize the number of obstacles, especially on higher speed bends, and to use supports that do not shear off leaving sharp remains or protrusions that could snag a fallen rider.

v. On higher-speed roads consideration must also be given to the "swept path" of the rider leaning into bends to avoid roadside features and oncoming traffic.

D. INTERSECTION

i. A significant proportion of collisions between motorcycles and cars in urban areas are caused by drivers failing to see the approaching or adjacent motorcycle. This can be helped by advanced stop lines for motorcyclists similar to those common for pedal cyclists as shown in **Plate 5.16** and **Plate 5.17**.

Plate 5.16 Motorcyclists at intersection

Plate 5.17 Advance motorcycle stop line

ii. It is important to optimize sight lines and to provide good braking surfaces for all users.

iii. Motorcyclists should be able to brake and stop while upright, travelling in a straight line, and on a surface which offers consistent grip. High friction surfacing at intersections can maximize the rider's chances of braking safely.

iv. Ensure consistent and appropriate skid resistance including that of extra surface features such as coloured patches and thermoplastic markings. Clear advance warning and direction signs should minimize the need for such surface signing. The requirement to lean when cornering increases the likelihood of loss of control when there is a substantial variation in the skidding resistance between two different types of material. The following should be kept in mind:
   a. Avoidance of different surface materials, for example granite blocks, to emphasize a change in circumstances at turning points.
   b. Thermoplastic road markings, some types of block paving, and metal utility covers can be particular problems for motorcyclists in these situations.
   c. Careful thought should be given before using large areas of hatching.
   d. The use of a high quality, cold-applied, coloured antiskid material provides the required visual effect without presenting a hazard for motorcyclists.
   e. Roundabouts also need to be designed with the correct entry path curvature and width to help reduce the speed of vehicles and ensure that approaching vehicles are not positioned at an excessively oblique angle.
   f. Concentric overrun areas feature on roundabouts to increase the deflection, reduce speeds, and be more conspicuous to approaching vehicles.

v. Care needs to be taken with this kind of treatment to ensure that it does not introduce an additional hazard for circulating motorcyclists. For example, where overrun areas have a slight kerb up-stand (10–20 mm) between the extended area and the remaining carriageway, as a motorcycle must lean over to negotiate a roundabout, crossing the up-stand can cause a rider to lose control.

vi. Single lane roundabouts are considered the safest intersection design for all users on moderately busy roads. They reduce the speed of approaching traffic and allow the smooth flow of traffic through the intersection. Two-lane roundabouts are particularly dangerous for motorcyclists due to the movement of motor traffic between lanes.

E. ROADSIDE BARRIERS

i. Roadside crash barriers are designed to contain an impacting twin-track vehicle and prevent it from crossing the path of oncoming traffic or leaving the running lane and colliding with a severe hazard.

ii. The majority of the roadside safety barrier systems in use today are designed to bring passenger cars and/or heavy vehicles to a controlled and safe stop. However, when struck by errant motorcyclists these systems may fail to provide this same level of protection.

Refer to **Section 12.5** for detailed information.

5.1.5.5 PUBLIC TRANSPORT, BUS RAPID TRANSPORT AND OTHER MODES

Public transport is generally thought of as referring to buses, coaches, and possibly trams that run regular and advertised schedules both in rural and urban areas wholly within the confines of the public right-of-way. In urban areas, public transport provides an efficient form of transport for large numbers of people and reduces congestion in busy cities.

Well-regulated public transport systems run along fixed routes with set embarkation and disembarkation points to a prearranged timetable.

Control and regulation of setting down and picking up points for paratransit service (such as taxis or small cars) are difficult and can lead to the use of inappropriate and unsafe locations.

5.1.5.5.1 SAFETY IMPLICATIONS

i. A common issue is that there is danger, not only when moving around the road network, but also when picking up or dropping off passengers, and extra care needs to be taken at such locations.
ii. Buses may also block the view of pedestrians attempting to cross at the signals. There is therefore an increased risk of crashes associated with unintentional non-compliance with the signals.
iii. Siting of bus stops that obscure intersections or signs, or obstruct traffic movements present particular safety problems for all users.
iv. Bus lanes appear to lead to an increased number of crashes, at least injury crashes. In order to move in or out of such bus lanes, several lane changes may be necessary (especially on three or more traffic lanes in the same direction) [14].
v. There may be major differences in speed between a bus lane and the other traffic lanes. This high-speed variability appears to increase the accident potential.
vi. When turning at an intersection, it may be necessary to cross the bus lane. In dense traffic, the differences in speed between a bus lane and the other traffic lanes may be relatively large.

5.1.5.5.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

A. BUS RAPID TRANSIT

i. Bus Rapid Transit (BRT) as shown in **Plate 5.18** is a high-quality, efficient mass transport mode providing capacity and speed comparable with the broad mix of traffic on urban roads.
ii. In cities of the developing world, the implementation of median-running BRT systems has generally proven to have a positive impact on safety. Research from Australia indicates that bus priority systems (with dedicated lanes) also had a positive safety

impact [15]

iii. On average, BRTs in the Latin American context have contributed to a reduction in fatalities and injuries of over 40 percent, and a reduction in Property-Damage Only (PDO) crashes of 33 percent on the streets where they were implemented. The mean effect is quite consistent across different regions of the world, as evidenced by the similar impacts of the Janmarg BRT in Ahmedabad, India. [16]

iv. The main reason that BRT systems have had positive safety impacts in Latin America is because in order to accommodate the BRT infrastructure, the city removed lanes, introduced central medians, shortened and provided improved crosswalks, and prohibited crossing turns by general traffic at most intersections.


Plate 5.18 Dedicated bus lanes for bus rapid transit system

B. BUS LANES

i. These are dedicated lanes within the main carriageway to allow buses to bypass traffic congestion. **Plate 5.19** shows dedicated bus lanes within the main carriageway. They are usually located at the nearside of the carriageway to allow easy access for passengers from an adjacent walkway. They are often separated from main traffic by a single solid white line, although in some instances they can be separated by a median.

Plate 5.19 These are dedicated bus lanes within the main carriageway

ii. Provision of dedicated bus lanes prevents use by general traffic and restricts parking and loading for adjacent properties. Obstruction of the bus lane by other vehicles negates the advantages of a dedicated lane and requires a dangerous manoeuvre for both vehicles to enter and leave the main traffic stream.

iii. Particular care is needed at intersections where the bus lane ends to allow all traffic to queue or buses to make turns across the main traffic flow.

iv. Additional benefit can be given to buses at signal-controlled intersections with specific stop lines and call stages.

C. LAY-BY (BUS BAY OR BUS STOP)

i. Pedestrians must be able to access bus stops safely. If pedestrians have to cross busy roads where complex manoeuvring occurs in order to access or leave buses, pedestrians will be at risk of crashes.

ii. In rural areas where services are less frequent, clear identification of formal stopping places is needed to prevent unsafe manoeuvres and deterioration of the road shoulder as shown in **Plate 5.20**.

Plate 5.20 Rural bus bay provides safe off-road operation

iii. Bus stops need to be clearly identified and safely accessed whatever form of vehicle uses them and wherever they are located.

iv. Bus stops must not be located such that stopped buses will obstruct the sightline to the traffic signal.

v. Several bus stops may be grouped together to facilitate easy transfer between routes. These may be arranged in a simple row along the street, or in parallel or diagonal rows of multiple stops.

vi. Whichever level of provision is made, the key elements are to ensure that:

   a. Vehicles should be able to enter, stop, and leave the location safely and smoothly.

   b. Lay-bys should be positioned on straight, level sections of road and should be visible from a good distance in both directions.

c. Access to a lay-by should be convenient and safe for vehicles and, also for pedestrians in the case of bus stops.
d. Advance warning signs should be erected to alert drivers of the approach to bus stops, and to the possible presence of pedestrians ahead.
e. Passengers are provided with sufficient advance warning (either within the vehicle or by external signage) to allow them to stand safely and comfortably.
f. Adequate queuing areas should be available so that waiting passengers do not use the road or a dedicated bus lay-by.
g. Pedestrian crossing facilities should be placed before the bus stop to aid visibility of crossing pedestrians and ease bus egress from the stop, whether at the kerb or within a lay-by.
h. Adequate and safe routes are provided to and from the stops to the surrounding pedestrian network.
i. Locations for stopping and waiting are clearly identified and protected.
j. Informal stopping on the roadway or shoulder should be prevented.

vii. Improvements to walkways and well-maintained pedestrian routes and short distances between bus stops can reduce walking distances and thus the number of injuries.

5.1.6 CROSS SECTION AND ALIGNMENT

Road design can be broken into three main parts: horizontal alignment, profile/vertical alignment, and cross section. Combined, they provide a three-dimensional layout for a roadway. Each of these parts are comprised of geometric design elements, including horizontal curves and straights, vertical curves and gradients, lane widths, shoulder widths, median widths, superelevation, and crossfalls, among others. The design of these elements influences safety and very restrictive designs, such as sharp horizontal curves or very narrow lanes, relative to the travel speeds, and often results in considerably higher crash rates. Certain combinations of these elements may also result in severe crash consequences. It is important to keep in mind the principles to good geometric design as discussed in **Sections 7.2** and **7.3**. The design should result in a road environment that is consistent with the road users' expectations or "non-surprising," as well as "forgiving" in the sense that road users' mistakes can, as far as practicable, be corrected, if not avoided. The selected design speed on which road alignment and cross-section characteristics are determined needs to be realistic and compatible to the expected operational speed as discussed in **Section 5.7**. It should also be in accordance with the type and functional requirements of the road and compatible to the roadway environment.

5.1.6.1 ROAD WIDTH

Road width is the total width of the portion of the roadway that allows for the movement of through vehicular traffic, including shoulders but excluding facilities such as kerbs, separated cycle facilities, walkways, or parking lanes. It is the full width of the carriageway, including all the travel lanes and adjacent shoulders (if present).

5.1.6.1.1 SAFETY IMPLICATIONS

It is important that the assignment/function of available space on the road is consistent and well considered to achieve a high level of readability and predictability. This means that all modes sharing the road corridor understand where they each should be and their position relative to each other, whether in adjacent lanes and shoulders or opposing lanes. Wider roads/lanes generally encourage, and are associated with, higher operating speeds than narrower roads/lanes. As such, the use of wide roads/lanes can pose significant safety risks, especially within the urban traffic environment where pedestrians, cyclists, and crossing vehicles are embedded in the traffic mix. Higher operating speeds and associated greater stopping sight distances can make it more difficult for motorists to bring their vehicles to a quick stop to avoid crashes. This is because the following dimensions at higher speeds may appear excessive, leading to vehicles cutting across on multilane roads and a tendency for drivers to drive closer to the car in front of them. The severity of crashes may also be increased.

i. Wider road/lane widths in urban areas increase exposure and crossing distance for pedestrians at intersections and midblock crossings.
ii. Lanes that are too narrow (typically less than 2.8~3.0 m) have increased risks of crashes under poor lane discipline at high speeds, such as single vehicle runoff-the-road crashes and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe collisions. This may be due to encroachments onto adjacent lanes, insufficient space for overtaking wide vehicles, or reduced sightlines to other vehicles in congested conditions. Within urban areas, narrowing of lanes can be used to control speed.
iii. Lanes that are excessively wide have increased crash frequencies. Studies have shown that the safety benefit of widening lanes stop once lanes reach a width of roughly 3.4 m, with crash frequencies increasing as lanes approach or exceed 3.7 m. [17,18]. The use of lanes greater than 3.6 m may in fact be used as two lanes, which can lead to increases in sideswipe crashes. Higher speeds may also be expected, which increases the likelihood and severity of crashes.
iv. Narrow lanes at curves may not provide an adequate tracking width or swept path for wide vehicles or room for driver error and may result in head-on crashes, sideswipes (particularly with vulnerable users in shoulders), or run-off-the-road crashes. Superelevation around curves can be applied to help maintain good lane tracking.
v. Narrow turning lanes at intersections may not accommodate the swept path of larger vehicles such as trucks and buses, which may lead to encroachments onto adjacent lanes increasing the risk of sideswipes (particularly with vulnerable users), and head-on and run-off-the-road crashes.

5.1.6.1.2 BEST DESIGN PRACTICE/TREATMENTS/ SOLUTIONS

i. The selection of the lane width and the number of lanes will depend on various factors including:
   a. Target vehicle speed (design speed, average speed, and posted limits) and lateral displacement.
   b. Context (existing or future function of roads and land uses).
   c. Level of pedestrian and cycle activities and facilities.
   d. Vehicle volumes and capacity.
   e. Vehicle type (large vehicles, transit vehicles, trucks) and the degree of truck proportion in total traffic.
   f. Provisions for other users.
   g. Nature, direction, and number of lane uses (turning lanes, through lanes, kerbside lanes).
   h. Situation adjacent to the lane (delivery, on-street parking, boulevards).
   i. Emergency vehicle operations. Travel lanes should not be too narrow (less than 2.8~3.0 m) for vehicles pulling out from emergency vehicles' paths, and long uninterrupted medians should be avoided. Multiple lanes leave sufficient space for drivers to pull out of the way of emergency vehicles.
   j. Topography and geometry (continuous median, horizontal alignment, crossfall, or slope of the road).
   k. Other considerations (maintenance, bridges and crossing points, planned changes of roads).

ii. Narrowing lanes is an effective tool for speed management since narrower lanes generally bring down operating speeds closer to safer speed limits, while maintaining consistent speed and minimum impact on corridor travel time.

iii. In urban areas, the use of narrower lanes has numerous benefits when considered within the assemblage of a given road, and urban roads can be redesigned to accommodate the needs of all road users through a road diet as shown in **Plate 5.21**. A road diet is generally described as reducing the number of travel lanes and/or narrowing travel lanes in a roadway and utilizing the space for other uses and travel modes. The benefits include:
   a. Reclaimed space to serve other modes, including cycle lanes and walkways, which improves mobility and access for all road users.
   b. Reclaimed space for geometric features that enhance safety, such as medians, pedestrian refuge islands, and turn lanes.
   c. Allow greater and more attractive space for pedestrians to relax and linger.
   d. Shorter pedestrian crossing times because of reduced crossing distances.
   e. Reduced interference with surrounding development.
   f. More economical to construct.

g. Less storm water runoff as more space can be left as vegetation.

Current

Proposed

Plate 5.21 Willow Glen Diet Trial

iv. Wider lanes may be necessary at turning locations, including curves, turning lanes, and roundabouts, especially when designed to accommodate larger vehicles. This allows more space for drivers to get around a curve/turn without encroaching onto the adjacent lane, shoulder, or even footpath. The amount of widening per lane will depend on the radius of the curve, the type/size of vehicle operating on the road, and some allowance for steering variations by different drivers.

5.1.6.2 SHOULDER WIDTH AND TYPE

A shoulder is the portion of the roadway contiguous with the travelled way that, depending on the width, design, and maintenance, performs several functions. The benefits include:

i. Accommodation of stopped vehicles for emergency use

ii. The provision of a controlled recovery area for drivers who inadvertently stray from their lane, thus reducing the risk of run-off-the-road crashes (especially in high-speed locations)

iii. Provision of space for evasive manoeuvres to avoid potential crashes or reduce their severity

iv. Provision of a defined space for cyclists or pedestrians were designed safely, in the absence of separated facilities

v. The provision of structural support to the pavement

vi. Reduction of pavement breakup by allowing storm water to be discharged farther from the travelled way, and therefore have both safety and asset management benefits

vii. Provision of lateral clearance to roadside objects, e.g., curbs, signs, and guardrails

viii. Improvement of sight distance in cut sections and enhancement of highway capacity and efficiency by encouraging uniform speeds.

Shoulders can be paved, unpaved (i.e., granular or earth shoulders), or partially paved (i.e., shoulders comprising of a paved section and unpaved section). These are also known as composite shoulders

5.1.6.2.1 SAFETY IMPLICATIONS

i. Shoulders that are too narrow do not provide an adequate recovery width for stray vehicles and enough through traffic clearance to vehicles stopped on the shoulder. These vehicles actually create roadside hazards and result in increased risks for run-off-the-road crashes, head-on and rear-end collisions, and sideswipes.
ii. Shoulders with inadequate skid resistance may cause a vehicle that leaves the travelled way, especially one travelling at high speeds, to lose traction and control resulting in run-off-the-road crashes, with severe consequences upon impact with roadside objects or other vehicles.
iii. In the absence of other facilities with more separation, a shoulder that is too narrow or in poor condition inevitably displaces non-motorized traffic onto the carriageway where they face increased safety risks due to exposure to high-speed traffic.
iv. Unpaved shoulders, especially on roads with high heavy-vehicle volumes in areas with heavy rainfall or abundant water runoff such as sag curves, may be eroded with time, resulting in pavement edge drop-offs as shown in **Plate 5.22**, where there is a difference between the height of the road surface and the height of the shoulder. Edge drops may cause a driver who drifts out of the travelled way to lose control of the vehicle and either veer off the road or overcorrect and veer into the oncoming traffic.
v. Roadways with no clear distinction between the carriageway and the shoulder (due to lack of signs, pavement markings, or low kerbs) may encourage the use of shoulders by motorized traffic, even when the shoulder is of a different surface material and intended to serve a separate purpose to the carriageway.

Plate 5.22 Pavement edge drop along the shoulder

vi. Excessively wide shoulders, especially when paved, can pose some safety risks including:
   a. Wide, paved shoulders result in higher operating speeds which, in turn, may impact the severity of crashes.
   b. Paved shoulders greater than 2.5 m may be interpreted as a travel lane by drivers, or even as temporary places for commercial activity (such as selling items to passing motorists).
   c. Wide shoulders may generate voluntary stopping on the shoulders.
   d. Wide roadway widths, resulting from wide shoulders and wide lanes, combined with limited right-of-way, may result in steeper side slopes or backslopes.
   e. Objects such as electricity posts, cable ducts, and raised drainage covers that are located along the shoulders are hazardous to all road users.

5.1.6.2.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. It is recommended that the shoulder be constructed of the same materials as the carriageway in order to facilitate construction, improve pavement performance, and reduce maintenance costs.
ii. The ideal width of shoulder depends on the road design class. A wide enough shoulder is recommended to provide an adequate refuge for vehicles to pull over and a recovery area in case of a roadway departure but not too wide to encourage the use of the shoulder as an additional lane as shown in **Plate 5.23**.
iii. The shoulder surface should provide sufficient skid resistance to prevent the loss of traction and control for stray vehicles. A sealed surface will provide the best grip for tires.

Plate 5.23 Wide sealed shoulder

iv. Shoulders should be continuous regardless of the width to avoid intermittent stopping on the travelled way. This also provides a continuous path for cyclists and pedestrians when shoulders are used as cycle lanes or walkways.

v. The shoulder surface should connect to the pavement at approximately the same level to prevent loss of control for vehicles that erroneously leave the travelled way.

vi. Sealing shoulders (full width or partially) on otherwise gravel shoulders can reduce the amount of erosion on the gravel shoulder and provide a safe recovery zone for shoulder encroachments.

vii. The quality of shoulders, on low radius curves in particular, requires special attention given the higher probability of encroachment at these locations. This may be due to intentional driver behaviour or inadvertent “off-tracking” of an articulated trailer. Wide, paved shoulders improve the safety of curves as shown in **Plate 5.24**, particularly on the inside edge (Refer to **Section 5.1.6.3**).

Plate 5.24 Wide, paved shoulder on curve

viii. In the absence of other facilities with greater separation, wide, paved shoulders provide space for pedestrians and cyclists, thereby potentially improving the safety of vulnerable users. Noting that pedestrians and cyclists may be at risk when drivers inadvertently drift off the road, shoulder rumble strips or edge-line rumble strips could be installed to mitigate against this risk as shown in **Plate 5.25**.

Plate 5.25 Paved shoulder with rumble strips

ix. Rumble strips or textured edge markings, which can be produced either by cutting grooves or adding ribs to the road, may be placed on the shoulders (near the edge of the travel lane) to alert drivers when they are leaving the roadway. These are highly effective and significantly reduce runoff-the-road crashes due to inattention, distraction, and fatigue.

x. Signs, pavement markings, and textured edge markings provide the necessary distinction between the shoulders and the carriageway and should be used to deter the use of shoulders by motorized traffic except in emergencies. In urban areas, kerbs along the edge of the carriageway may be used.

xi. Objects that are located along the shoulders should be moved and/or buried beyond the shoulders, and where possible, beyond the clear zone (Refer to **Section 5.1.6.7**). It is essential that shoulders remain traversable to serve their function.

xii. The management and maintenance of the road and shoulder should be routine and simple.

5.1.6.3 HORIZONTAL CURVATURE

Horizontal curves are associated with higher safety risks compared to tangent sections. This difference becomes particularly apparent at radii less than 1,000m and becomes increasingly significant as curve radii reduce further (< 200m) [19]. This is often the result of a mismatch between the radius, superelevation, and negotiation speed chosen by the driver. This creates an imbalance in the forces exerted on the vehicle and does not match driver expectations, among other factors. It should be noted, however, that while short horizontal curves of low radius may increase the crash risk in high-speed environments, the same curve length and radius may be appropriate in low-speed residential environments to help facilitate slower speeds, or as part of a series of curves. In addition, high radius curves may be introduced into rural designs to manage vehicle speeds and reduce monotony.

5.1.6.3.1 SAFETY IMPLICATIONS

i. Unexpectedly sharp curves often result in loss of control, skidding, or crashes onto roadside objects or oncoming vehicles when drivers mismatch their speed to the geometry and are forced to perform sudden corrective actions. This is made worse when the sharp curve is "out of context" or does not match with the alignment conditions adjacent to the sharp curve section that mislead or encourage high speeds, for example, a sharp curve after a series of gentler curves or after a long straight section.

ii. Obstructions located too close to the carriageway on the inside of the curve without the necessary horizontal sightline offset, limit the sight distance and the driver's ability to see and anticipate road features ahead of the curve as shown in **Plate 5.26**.

Plate 5.26 Tree located too close to the carriageway on inside of curve

iii. Sharp curves increase the width of a vehicle's swept path, which may cause a vehicle to cross into the path of an approaching vehicle on narrow carriageways, or onto the shoulders and pedestrian areas, increasing the crash likelihood for other road users. This is worse for wide or long vehicles/trucks.

iv. Drivers may overtake on curves when it is unsafe despite the "no-overtaking" provision.

v. Other factors that influence the safety on curves include the roadside profile (whether level or drop-off if a vehicle leaves the road), the presence of unshielded roadside hazards, poor visibility, poor delineation, poor drainage, inadequate or reverse superelevation, inadequate lane widths or lack of extra widening on curves.

vi. Poor coordination of horizontal and vertical curvature can result in visual effects that may mislead drivers thus contributing to crashes as indicated in **Plate 5.27**. This usually occurs when horizontal and vertical curves of different lengths occur at the same location.

Plate 5.27 Poor alignment showing optical breaks caused by steep sag curves and horizontal tangent

5.1.6.3.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

i. It is important that a road is designed for a speed that exceeds, as a minimum, the speed often referred to as the operating speed or at which it is anticipated (or intended) drivers will travel.

ii. At the design stage, consistency and predictability of the driver experience are very important, and unexpectedly tight curves should be avoided. This can be done by either increasing the radius of the curve or ensuring the transition to sharper curves is carried out through gradual and progressive reduction of the radii along sequential curves.

iii. For tight horizontal curves, which are out of context compared to the rest of the design and cannot be re-aligned for financial or environmental reasons, special treatments at these curves should be specified and carried forward to the design and construction phases. These special treatments can be in the form of specific signs or markings that alert the driver to the change in conditions.

iv. Forward visibility and sight distances are important to help the driver assess the road ahead and adjust their speed in anticipation of the road condition. Sight obstructions on the inside of curves or the inside of the median lane in divided highways need to be removed to provide appropriate sight distance. In situations where it is impractical to remove such obstructions (such as retaining walls, cut slopes, concrete barriers, buildings, and longitudinal barriers), the sight distance should still be optimized, but the design needs to evaluate the risk associated with the deficiency and assess the options for mitigating that risk. Because of the many variables in the design of curves, i.e., the alignment and cross section, and the number, type, and location of potential obstructions, it is necessary to conduct a specific study for each individual curve. Using sight distance for the design speed as a control, the designer can then check the actual conditions on each curve and make the necessary adjustments to provide adequate sight distance. These adjustments should take into account the extent or duration of the obstruction. For example, a retaining wall may represent a significant length of deficiency, thereby requiring some adjustment of design or speed, whereas a single building or clump of trees represents only a momentary reduction and therefore the risk is much lower.

v. Curves should be superelevated in proportion to their radius and speed; superelevation should be changed gradually and equally between curves of a different radius.

vi. Transition curves may be provided between a tangent and a circular curve or between two circular curves, allowing the gradual introduction of superelevation. The length of the transition curves should be equal to the distance over which the superelevation changes. The full nature of approaching curves should be evident to the driver. Long transition curves that mask a sharp final radius should be avoided.

vii. While simple curves are preferred, compound curves can be used to satisfy topographical constraints that cannot be as effectively balanced with simple curves.

viii. Higher skid resistance materials can be used on critical bends, particularly in wet environments.

ix. Large radius horizontal curves may be introduced on straight alignments to break driver monotony and enable drivers to make better judgments of the speeds of approaching vehicles.

x. Lane (or curve) widening is normally applied to the inside edge of curves and is often necessary on lower radius curves to provide room for "off tracking" of articulated vehicles. Especially relevant where radii are <500m, this allows for the difference between the path of the rear axles of the trailer compared to the truck (tractor) unit.

xi. Where it is not possible to entirely separate horizontal and vertical curves, they should be combined with common changes for intersection points (where the ends and centre of the vertical curve are coincident with the corresponding ends and centre of the horizontal curve) to avoid the presentation of misleading information to drivers. Where possible, they should be of the same or similar length, and where this is not achievable, the preference is for the extent of vertical curves to lie wholly within a single horizontal curve. This arrangement should produce the most pleasing, flowing, three-dimensional result, which is more likely to be in harmony with the natural landform as shown in **Plate 5.28**.

Plate 5.28 Example of good combination of horizontal and vertical curve

xii. Sharp horizontal curves in combination with a pronounced crest vertical curve should be avoided since drivers may not perceive the horizontal change in alignment, especially at night.

xiii. Sharp horizontal curves at or near the low point of a pronounced sag vertical curve should be avoided since the view of the road ahead would be foreshortened, and curves in these locations tend to be sharper than they appear.

xiv. On two-lane roads where combinations of curves are likely, tangent sections may be provided with good passing sight distance to provide opportunities for safe overtaking.

5.1.6.3.3 SUMMARY OF TREATMENTS FOR HORIZONTAL CURVES

A. MARKINGS AND SIGNS

Pavement markings are important in providing continuous information to help drivers navigate roadways successfully. These include:

i. Longitudinal pavement markings (centrelines and edge lines). Wider centreline markings can be used where space allows to increase separation between vehicles travelling in opposite directions.
ii. Guideposts/delineators along the side of the road. They can be used for both unpaved and paved roads.
iii. Retroreflective pavement markings and chevron alignment signs as shown in **Plate 5.29** enhance visual guidance at nighttime.

Plate 5.29 Retroreflective pavement markings and chevron alignment signs at night time

B. PAVEMENT COUNTERMEASURES

The following countermeasures may be applied to improve the pavement's skid resistance:

i. High-Friction Surface Treatments (HFSTs)
ii. Widening at curves to allow wide centreline treatment.

The following treatments are applicable to shoulders:

i. Shoulder widening to provide a recovery area for drivers to regain control in case of a roadway departure, especially on the inside of the curve.
ii. Shoulder paving to replace unstable or narrow shoulders.
iii. Rumble strips may also be installed to warn fatigued, distracted, and inattentive drivers when leaving their travel lane by either milling/cutting grooves or placing ribs/bumps on the road. These can be placed on the shoulders near the edge of the travel lane or at the edge of the travel lane in line with the edge line marking or at/near the centreline of an undivided highway as shown in **Plate 5.30**. They can be created by milling grooves in the road surface or the addition of intermittent raised edge line markings.

(a) shoulder rumble strips **(b) Edge line rumble strips** **(c) centreline rumble strips**

Plate 5.30 Pavement countermeasures

5.1.6.4 SUPERELEVATION AND CROSS SLOPE

On horizontal curves, superelevation is the transverse slope provided perpendicular to the direction of travel to counteract the centrifugal force generated by the speed in a circular motion. It is applied by raising the outer edge of the pavement with respect to the inner edge throughout the length of the horizontal curve **(Figure 5.4)**. It is usually applied over the length of a circular curve to reduce the sideways frictional demand between the tires and road surface and to increase comfort. The superelevation value is usually selected by road designers to be consistent with the combination of design speed, curve radius, and the road authority's policy for maximum superelevation. A transition curve, inserted between the tangent and circular curve, may be used to remove the adverse cross-slope (adverse camber) created by the road crown and introduce superelevation.

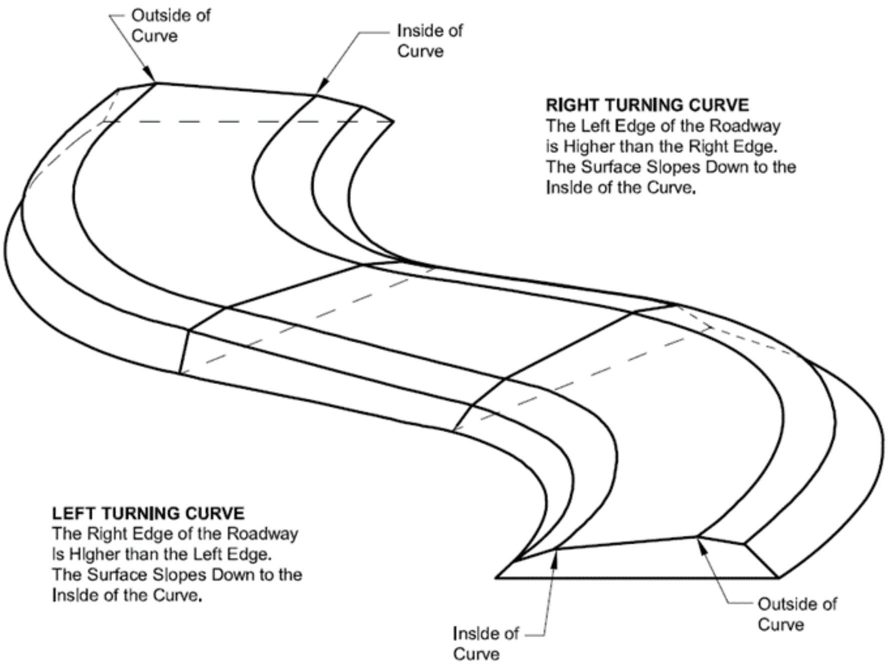


Figure 5.4 Superelevation for Left and Right Turning Curves

5.1.6.4.1 SAFETY IMPLICATION

i. If a road is not superelevated, the centrifugal force tends to push the vehicle toward the outside of the curve. At high speeds, the driving task will be uncomfortable and more demanding. As a result, vehicles may become less stable, lose traction, and skid due to insufficient frictional force between the tire and the road to counterbalance the centrifugal force, or topple sideways if the centre of gravity is high. Vehicles on the outside of curves are more likely to experience run-off-the-road crashes resulting in collisions with road users or objects on the outside shoulders or rolling over.

ii. The lack of superelevation also encourages drivers to use the centre of the road or the inside lane irrespective of the direction of travel, which increases the probability of head-on collisions, especially on two-way, two-lane roadways.

iii. Since superelevation also assists with the drainage of water, too low superelevation is more susceptible to surface defects that may result in standing water on the carriageway, which increases the risk of loss of pavement friction. The film of water developed during and after rainfall increases the risk of hydroplaning/aquaplaning. Standing water may also result in pavement damage and loss of shape in the long term, which presents an additional safety hazard. A curve with a worn or polished pavement surface that provides little skidding resistance presents increased crash likelihood, particularly in wet conditions.

iv. Change in superelevation, also known as roll over, from one side of the road to the other or between straights and curves, inevitably creates areas of road surface without any camber or crossfall. It is important that these areas do not coincide with very shallow longitudinal gradients, or the water will simply collect, and ponds will form.

v. Inadequate superelevation poses greater safety risks to motorcyclists given their lower vehicle stability, i.e., only two points of contact with the road. In the absence of superelevation, motorcyclists rely completely on the tire grip to remain on the road.

vi. Too high superelevation will result in the possibility of slow-moving vehicles sliding sideways or, in extreme cases, overturning. It has been observed that a 12 percent superelevation can cause trucks with high loads to lose their loads or tip over completely when trying to negotiate a curve at a low speed. [20]

5.1.6.4.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

i. Where the radius of a curve is less than the specified minimum for each design speed, the introduction of superelevation and curve widening will minimize the intrusion of vehicles onto the adjacent lanes and encourage uniformity of speed, thus increasing vehicle safety at the curves. This consistency is achieved by using minimum acceptable side friction factors between the tires of a vehicle at the design speed and the road surface. Acceptable friction factors vary from 0.12 for 100km/h to 0.33 for 50km/h.

ii. By designing to a side friction demand of 50 – 60 % of the maximum, a safety factor is built into the process that allows for a margin of error in a driver's choice of negotiation speed. This means that a driver travelling faster than intended around a curve will feel increasingly uncomfortable well before the side friction demand exceeds that available and they lose traction. This discomfort triggers the driver's natural response to ease off the accelerator.

iii. On major roads with a superelevation deficiency, it is desirable to reconstruct the outer lane around the curves to provide superelevation that suits the operating speeds. It has been reported that improving superelevation may reduce the number of crashes by 5 to 10 percent [21]. It is also important to ensure that there is smooth transition between the crowned and superelevated cross sections on each end of the curve.

iv. Drainage conditions should be checked to ensure that combinations of slopes along and across the road are adequate to remove water from potential flat areas that can lead to standing water and potential risk of aquaplaning. Technical solutions, including transverse road gutters, diagonal slopes, and surface grooving may be applied to prevent aquaplaning for consecutive opposed curves where the vertical alignment is not helping with the drainage of pluvial waters from the carriageway [22]. The design of curves should be checked for consistency, with the selected value of maximum superelevation applied consistently based on traffic function. This will ensure that there are no variations in the rates of superelevation for curves of equal radius. It is widely accepted that drivers select their approach speeds to curves on the basis of the radius that they see and not on the degree of superelevation provided. For this reason, a lack of consistency with regard to superelevation would almost certainly lead to differences in side friction demand with possible critical consequences.

v. By applying superelevation relative to the maximum value, the driver will experience a consistent level of comfort when travelling round a superelevated curve.

vi. If a superelevation deficiency cannot be reasonably or readily addressed, other safety measures may be considered, including:

   a. Advance warning signs to warn drivers of a tight curve ahead and an indication of the reduced speed required to safely negotiate the curve.
   b. Road markings, signs, and posts to draw the driver's attention to the curve.
   c. Provision of shoulder and hazard free areas to provide a safe recovery area.
   d. Improving the surface friction of the outside lane.
   e. Erecting safety barriers or designing clear zones around the outside of the curve (Refer to **Sections 12.5** and **6.16** respectively).

vii. Curves on residential streets and built-up areas are often constructed without superelevation due to the assumption of lower operation speeds. In such areas, speed management and traffic calming rather than road surface shape, is usually a more appropriate solution to reduce the additional risks created by vehicles travelling at high speeds.

viii. Rainfall after a long dry spell reduces side friction, especially in areas where the surface is polluted by oil spills, rubber, and other debris. Where any of these circumstances are likely to occur, a lower value of maximum superelevation is recommended in design.

5.1.6.5 VERTICAL ALIGNMENT

Vertical alignment involves the road grade (the rate of change of vertical elevation) and vertical curves (i.e., crests and sags). Its design is a derivative of the interaction between sight distance criteria, the topography of the roadway, and the designer's need to meet ancillary goals (e.g., balancing excavation and fill quantities) as important safety factors of roads.

Most passenger cars can readily negotiate grades as steep as 4 to 5 percent without an appreciable loss in speed below that normally maintained on level roads. On steeper upgrades, speeds decrease progressively with increases in the grade. Specifically, speed differentials between passenger cars and heavy vehicles should be considered when conducting a safety analysis. On downgrades, passenger car speeds generally are slightly higher than on level sections, and there are increases in braking distances (refer to **Table 7.11**). The severity or sharpness of vertical curves is usually referred to in terms of a radius (circular arc) or "K-value" (parabola). **Section 7.3.4** provides comprehensive information on the design and calculation of vertical profiles (alignments) using parabolic curves. For the combination of horizontal and vertical alignment, see **Section 7.7**.

5.1.6.5.1 SAFETY IMPLICATIONS

i. Vertical alignment influences a driver's sight distance. Crest vertical curves may limit sight distance by restricting a driver's line of sight. The crash frequency on crest curves with reduced sight distance is 52 percent higher than on curves with no reduction in sight distance [23].
ii. Overtaking will be of higher risk at this location without auxiliary lanes (i.e., climbing or passing lane), especially on rural roads.
iii. Steep gradients may increase the vehicle's speed by up to 5 percent; therefore, large vehicle drivers may choose to descend grades at slower speeds to maintain better control of their vehicles.
iv. Long, steep downgrades may result in loss of control of vehicles, especially if present before horizontal curvature to perform sudden corrective actions. The higher the grade rate is, the higher the crash risk is. The crash risk rises more rapidly for grades over 6 percent as vehicle speeds become more difficult to manage. [24]
v. Increases in braking distances and the possibility of heavy vehicle brake overheating should be concerning because this can lead to brake failure.
vi. Crash frequency and severity are higher on downhill grades than on uphill grades, with a high involvement of heavy vehicles.

5.1.6.5.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

i. On grades, the designer must avoid a combination of features that increase the probability of having to carry out difficult manoeuvres, including intersections or other crossings (railway, crosswalk, bike path, etc.), sharp horizontal curves, narrow structures, and no provision of protection.
ii. Overtaking must be prohibited to avoid head-on collisions, preferably by physical separation between the opposite directions of travel.
iii. Steep side slopes should be avoided. The maximum gradient that can be travelled by errant vehicles is in the order of 1:3 to 1:4. The angle between shoulder/slope and slope/adjacent land should also be smoothed in surface level as shown in **Figure 5.5**.

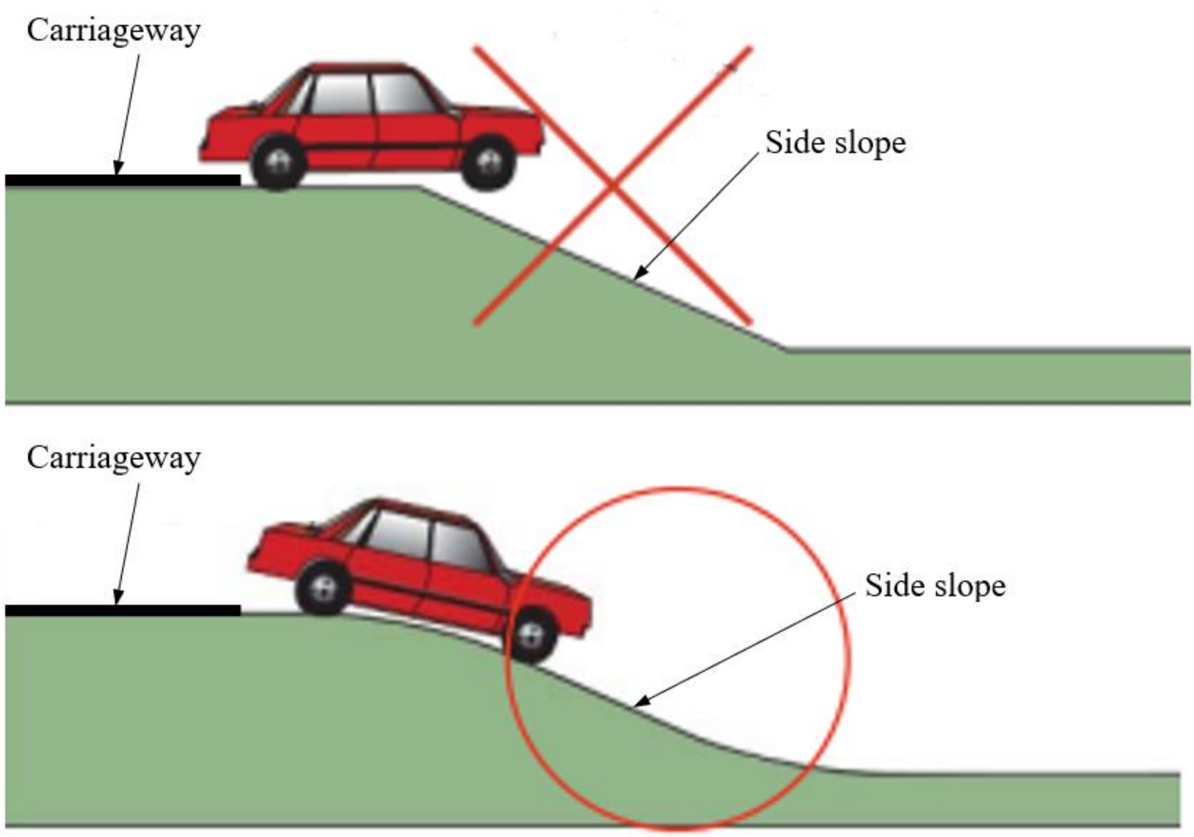


Figure 5.5 Smoothing of side slopes

vii. At descending steep slopes, a short passing lane, auxiliary, or "slow vehicle turn-out" can be provided. If not provided, operations may be degraded for faster-moving vehicles from behind, creating an increased risk of rear-end crashes and risky passing manoeuvres.

viii. Increasing the superelevation on horizontal curves that coincide with steep down gradients improves heavy vehicle stability when braking [25].

ix. Increasing radii at the bottom of steep downgrades is advisable, as these curves are often misread by drivers, and the visual distortion leads to "overdriving." [26]

x. At descending steep slopes, the provision of emergency escape or runaway ramps for brake failure should be provided **(Refer to Section 11.8)**.

xi. Sag vertical curves at underpasses should be designed to provide vertical clearance for the largest legal vehicle that could use the undercrossing without a permit. A tractor trailer will need a longer sag vertical curve than a single-unit truck to avoid the trailer striking the overhead structure as indicated in **Figure 5.6**.

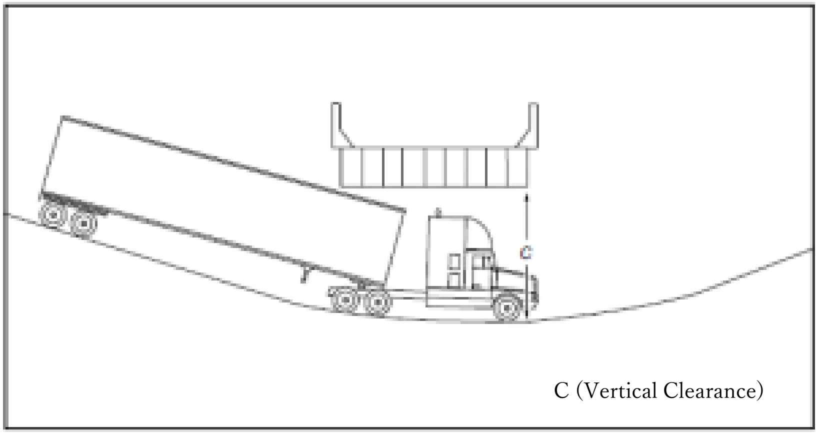


Figure 5.6 Vertical clearance at undercrossing

xii. Grade rate of change is critical for sag curves where gravitational and vertical forces act in opposite directions. The hidden dip (roller coaster) type of profile should be avoided. Such profiles may occur on relatively straight, horizontal alignment where the roadway profile closely follows a rolling natural ground level. Hidden dips may create difficulties for drivers, because the passing driver may be deceived if the view of the road or street beyond the dip is free of opposing vehicles, even with shallow dips as shown in **Figure 5.7**.

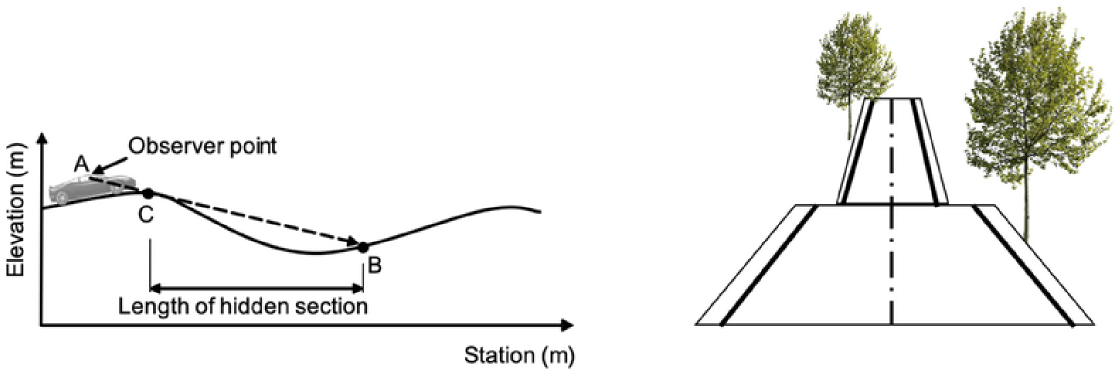


Figure 5.7 Hidden side dip: Left – road vertical profile; Right – road frontal view

xiii. A "broken-back" gradeline (two vertical curves in the same direction separated by a short section of tangent grade) generally should be avoided. This effect is particularly noticeable on divided roadways with open median sections.

xiv. A minimum grade of 0.4 percent is needed for proper drainage on sag vertical curves to avoid mishandling related to ponding. However, it may be necessary to use flatter grades in some instances (refer to **Section 7.3.3**).

5.1.6.5.3 OTHER TREATMENTS FOR VERTICAL ALIGNMENTS.

A. Marking and signs

i. Signs can be used to provide drivers approaching a steep grade with advance warning. Signs help drivers to adjust their behaviour to deal with approaching hazards or decision points. Use of advance warning signs as a stand-alone measure is unlikely to sufficiently mitigate a design exception for grade, but it can be an effective component of a more comprehensive approach.
ii. Poorly designed/maintained/located signs must be re-installed. The retro-reflectivity of signs is an important consideration for road use at night and when wet. Maintenance of signs can be problematic; signs may be stolen and broken in some areas.
iii. For increasing braking distance at downgrade, signs can indicate a lower speed and that a lower gear is required.

B. Roadside improvements - barriers

i. When locations have poor visibility, steep slopes, level difference, and a slower vehicle speed is likely, overtaking should be prohibited.
ii. Flexible posts (delineators) also contribute to improve visibility of the median of the road because of level differences. Treatment for visibility must be made, even though other vertical treatments will be costly (Refer to **Section 12.2**).
iii. Furthermore, flexible posts can also be applied where there is a requirement for lane discipline along with traffic calming.
iv. Flexible posts are a quick and easy solution but can result in a high maintenance cost if repeatedly struck and require replacement. In these circumstances more robust and substantial treatment may be more appropriate (barrier or concrete median).
v. On a downgrade where there is a significant drop in speed limit, speed control treatments can be used. Speed control treatments include a combination of prominent signs, road markings, and traffic calming measures.

C. Terrain modifications

i. Modifying a vertical alignment is often too costly and can have significant impacts to adjacent land uses. It is much better to design the road well before it is built than to rebuild it. The reconstruction of a crest vertical curve should be implemented when the hill crest hides major hazards from view, such as intersections, sharp horizontal curves, or narrow bridges as shown in **Plate 5.31**.
ii. "Roller coaster" or "hidden dip" type of profiles should be avoided by use of horizontal curves or by more gradual grades.
iii. Road widening (either as wider shoulders or passing lane) over a crest with less than adequate sight distance can be an effective countermeasure rather than flattening the

crest. Long and steep downgrades can result in heavy vehicles travelling at crawl speeds to avoid loss of control on the grade. Slow-moving vehicles of this type may impede other vehicles. Auxiliary lanes can be provided to address this risk. They can be constructed on uphill and downhill grades to enable safe passing manoeuvres by slower vehicles. Since auxiliary lanes encourage passing manoeuvres at relatively high speeds that are incompatible with the slower speeds of vehicles accessing and exiting the road, they should not be located in conjunction with intersections or other access points.

iv. Since safety problems in grades primarily involve heavy vehicles, solutions aimed at limiting their presence at high-risk locations may also be considered, when permitted by the configuration of the road network (dedicated heavy vehicle roads).

Plate 5.31 Alignment modification to eliminate a sharp curve at the bottom of a steep grade.

5.1.6.6 PASSING LANES

Constructing passing lanes results in safer operational conditions, perceived safety by motorists, and historical crash reductions [27]. Studies indicate that injury crashes after a passing lane have been constructed are likely to be in the range of 20–40 percent less than if it had not been constructed. The extent to which the reduction in crashes applies, however, varies from being specific to the passing lane itself, the passing lane and its immediately adjacent road, or for an entire route. However, if not well designed, constructed, and maintained, passing lanes may pose some safety risks. Refer to **Section 7.5** for further details.

5.1.6.6.1 SAFETY IMPLICATIONS

i. Limited sight distance at the start and end points of passing lanes. This is particularly hazardous on merging sections implemented along or near horizontal or vertical curves with limited sight distance to drivers along the passing lane. This sight distance should be provided to the road markings so that drivers can see precisely where the lane and shoulder tapers, associated with the end of the passing lane, start and finish.

ii. Passing lanes located near towns, major intersections, or high-volume access roads may result in collisions from the high interaction of passing vehicles with turning movements and vulnerable road users. It should be noted that vehicle speeds are influenced for a significant distance after a passing lane, as overtaking drivers desire to distance themselves from the overtaken vehicle. As a result, the need for minor improvements to the downstream road corridor should be assessed.

iii. Passing lanes that have intersections within their length should be avoided if at all practicable.

iv. Narrow shoulder widths on passing sections. In many cases, the sealed shoulder width is reduced as part of the passing lane construction design, which can lower the safety benefit of three-lane passing sections. Narrower sealed shoulders are a safety risk as they may not provide an adequate space for vehicles to perform evasive manoeuvres should it be necessary to avoid another vehicle.

v. Inadequate signage and pavement markings. This adversely affects both the effectiveness and safety of the passing lane, as drivers are not sufficiently guided on the most appropriate action to take on the passing section, including the approaches and merges.

5.1.6.6.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

i. The design of passing lanes should be considered carefully with regard to the road designs and conditions at both ends. The passing lane location should provide adequate sight distance to the road markings at the lane addition and lane drop tapers. The length of the tapers should also be adequate in relation to the operating speeds.

ii. The location of passing lanes will depend on the particular local needs and constraints. However, there are sites where passing lanes should not be constructed, including sites close to towns and/ or high-volume access roads and sites with major intersections. This is because collisions may result from the interaction of passing vehicles with high turning movements and vulnerable users in these areas. Locations with other physical constraints, such as bridges and culverts, should also be avoided if they restrict the provision of a continuous shoulder. Highway sections with low-speed curves are not appropriate for passing lanes since passing, which requires drivers to speed up, may be unsafe.

iii. Proper signing and pavement marking (Refer to **Section 12.4** and **12.3** respectively) are required to enhance the driver's understanding of the intended use of the passing section and inform them of upcoming opportunities to overtake, which results in increased efficiency and safety of the passing lane.

For optimum signage, signs should be provided in the following six areas:

a. In advance of the passing lane.
b. The transition area of the lane addition of the passing lane.
c. In advance of the termination of the passing lane.
d. The transition of the lane reduction of the passing lane.
e. The downstream area adjacent to the passing lane.
f. In the opposing direction of the passing lane.

iv. A strategy for advance signing of passing lanes is desirable to alert road users of upcoming passing opportunities. This reduces unsafe overtaking prior to the passing lane, as motorists know that safer passing opportunities will be available shortly.

v. On three-lane sections, vehicles travelling in the opposite direction to a passing lane should be discouraged from overtaking due to the high risk of a head-on collision. This information can be provided through longitudinal pavement markings or a combination of both signs and pavement markings. A site-by-site review is desirable to determine which passing lane sites are critical in the prohibition of passing by opposing traffic on the basis of limited sight distance, unusual geometrics, roadside development, and high-traffic volumes. The use of flexible posts along the centreline on these critical locations, in addition to the signs and pavement markings, may enhance the prohibition of passing by opposing traffic.

vi. "2 + 1" Roadways are a safety countermeasure for two-lane highways where a continuous three-lane cross section on which the central lane serves as a passing lane in alternate directions is provided throughout the length of the facility as shown in **Figure 5.8**.

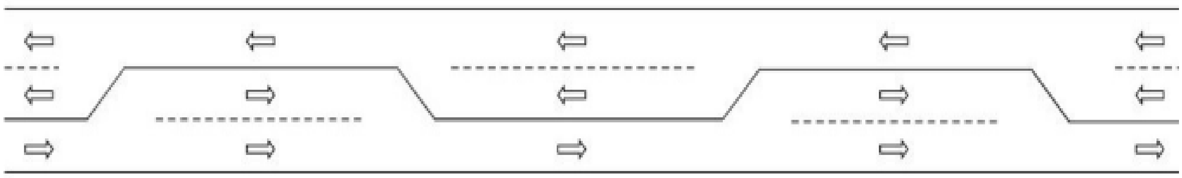


Figure 5.8 Schematic view of "2+1" Roadway

Travel directions are separated by a median which could be physical or painted as shown in **Plate 5.32** and **Plate 5.33**. They are a cost-effective solution where a two-lane road is not providing enough safety and/ or traffic efficiency and the expansion to a four-lane roadway seems unjustified due to cost, demand, or environmental issues. Before implementing a "2+1" Roadway, unique design aspects linked to this configuration should be considered, including traffic volume; passing lane length; transition areas; cross section, intersection, and access design; and signing and markings.

Plate 5.32 Highway with flexible barrier

Plate 5.33 Highway with painted median

5.1.6.7 ROADSIDES—FORGIVING ROADSIDES AND CLEAR ZONES

The fundamental principle for designing safe roadsides is based on the knowledge that drivers (or riders) will make mistakes; occasionally they will lose control of their vehicles and leave the road. When this happens, a collision with unyielding objects such as trees or poles, or non-traversable features such as drains, steep side slopes, or rough surfaces, may result in the vehicle vaulting, rolling over, or coming to a sudden stop. This can lead to severe injuries or death for the occupants.

Providing a forgiving roadside is intended to minimize the consequences of a vehicle leaving the roadway by providing a safe and forgiving area that is free of rigid objects, has flattened, smooth-sloped embankments and no other hazards, in which an errant vehicle can safely recover and stop. All aspects of the roadside should be designed to minimize the possibility of an occupant of an errant vehicle being seriously injured or killed.

5.1.6.7.1 SAFETY IMPLICATIONS

i. Several studies have revealed that run-off-the-road crashes are not only frequent, but are especially serious, resulting in more severe injuries and deaths than most other crash types. The main factors that influence run-off-the-road crash outcomes are the existence of vehicle recovery areas, roadside barriers, and the presence of infrangible objects. If a vehicle leaves the roadway but recovers on the shoulders or grassed verge, the outcome is likely to be no damage or minor damage. However, if an errant vehicle hits a rigid lighting column or a substantial tree at speed and comes to a sudden stop, the outcome is likely to be a severe injury or a fatality.

ii. The closer a roadside hazard is to the traffic lane and the higher the traffic speed, the higher the likelihood that the hazard will be struck by an errant vehicle. The presence of road curves adds to the overall likelihood of a run-off-the-road crash, as the driver needs to take more action to maintain the vehicle on the road. On-going traffic exposure to roadside hazards will increase the likelihood of crashes, i.e., higher traffic volumes increase the risk of a collision with the hazard over time. Rural roads have been shown to be more likely to produce severe consequences of run-off-the-road crashes due to generally high operating speeds and typically low levels of roadside modification, e.g., retaining original trees along the roadsides.

iii. Steep roadside slopes increase the risk of a rollover in case of a run-off-the-road crash, which generally has high severity. High speeds will add to the risk of high severity crashes. Slopes steeper than 1:4 are deemed as non-recoverable, i.e., a typical errant vehicle will travel to the base of the slope before being able to recover. The surface condition of the embankment also influences the recovery of an errant vehicle, with smooth firm slopes offering a better chance of recovery than soft, uneven slopes. High-volume roads with unshielded steep slopes tend to have a higher record of casualty crashes than roads with relatively flatter slopes or road safety barriers.

iv. Unprotected end posts of bridges are hazards due to their solid (rigid and infrangible) construction and proximity to the traffic. The narrower the bridge is, the higher the risk of a collision with an end post, as the hazard is closer to the traffic lane. Single lane bridges that can be approached at high speeds and have no active traffic control have a high risk of head-on crashes, crashes into the bridge end posts, and pedestrian/cyclist crashes. Bridges with narrow lane widths, especially two-lane bridges and bridges that lack pedestrian/cyclist separation, can lead to increased risk of sideswipes, head-on crashes, or large vehicles becoming wedged.

v. Overgrown or poorly planned vegetation can be a serious hazard depending on its location with respect to the road. When located close to the carriageway, they can obscure signs, hazard markers, and roadside hazards like ditches. Roadside vegetation may also interfere with sight distances at intersections and on road curves, which increases the risk of intersection, run-off-the-road, and head-on crashes.

vi. Overhanging tree branches can also interfere with the driving task, especially for buses and trucks, causing drivers to swerve into adjacent lanes to avoid damage to the vehicle or load. In urban areas, low decorative shrubs can block the visibility of pedestrians (especially children) at road crossing points, while overhanging tree branches can block sightlines to traffic signal displays. Trees, however, provide benefits, including shade for pedestrians and reduced soil erosion on site, and those less than 100mm diameter are less likely to contribute to the severity of a crash.

vii. On high-speed facilities, kerbs may be a safety hazard as they could cause an errant vehicle at high speed to jump or roll over.

5.1.6.7.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. The safety of a roadside (or median) may be gauged by the width of the clear zone, which depends on operating speeds, traffic volumes, roadside slopes, and the road geometry. Wider clear zones are recommended near intersections or bends, where the complexity of the driving task and interaction with other vehicles adds to the likelihood of run-off-the-road crashes. It should be noted that the clear zone widths are not a guarantee of safety but a compromise, a way of managing roadside risks. Nonetheless, generous forgiving roadside widths should be provided where feasible.

ii. In addition, the longer an errant vehicle traverses the roadside area, even if this is an extended clear zone, the greater the likelihood that the vehicle will roll over [28].

iii. Clear zones need to be of good quality and well maintained to maximize their safety benefit. Uneven surfaces or exposed tree roots can snag vehicles causing them to roll, and this often results in severe crash outcomes.

iv. Avoid locating any new hazardous objects within the clear zone when designing a new road. This can be achieved through the development of policies that restrict the placement of new potentially hazardous objects on the roadside.

v. All existing fixed roadside objects that are 100mm in diameter or larger should also be removed from the clear zone. In circumstances where it is not possible to completely remove a hazard from the clear zone, consideration should be made to relocate the hazard, preferably beyond the clear zone. Rigid poles, rigid lighting columns, and drains can be relocated to reduce the risk or replaced with frangible/passively safe columns.

vi. The removal of trees, on the other hand, needs to be undertaken with consideration to the environment and community values. Large trees (more than 100mm in diameter) that are close to the carriageway may be replaced with more appropriate plants to avoid soil erosion and regrowth affecting the site. Care should be taken not to leave large stumps and deep holes upon the removal of a tree, as these are also hazards. It is also important to trim and regularly maintain vegetation along the roadside.

vii. In locations where removing or relocating a hazard that is within the clear zone is not feasible or practicable due to economic or environmental constraints, altering or modifying the hazard can reduce the severity of a crash and the potential for serious injury. Common modifications of hazards include:

a. Modifying open longitudinal drains by piping them or covering them with a drivable cover.
b. Modifying end walls of driveway culverts to make them drivable.
c. Redesigning rigid signposts and lighting columns to provide more forgiving frangible (breakaway) posts and lighting columns, i.e., impact absorbent or slip-base types (**Refer to Section 12.10**)
d. Flattening a steep fill slope to make it drivable.
e. Replacing bridge rails with safety barriers with appropriate end treatments.
f. Shielding bridge piers with rigid barriers.

viii. In situations where the road reserve may be limited, it may not be practical to create a clear zone. Reducing the operating speeds instead may be a more appropriate solution. A safety barrier system may also be considered that in itself presents a (reduced) collision risk but needs the terminals appropriately treated to minimize risk.

ix. Good geometric design and the prudent use of road features can help to keep vehicles on the road and reduce the risk of a run-off-the-road crash. The geometric standard should be based on a realistic assessment of the likely operating speed of a road section considering the road function, the terrain through which the road exists, and the road environment. Some of the road design features that assist in keeping vehicles on the road include appropriate lane widths and shoulder widths, predictable horizontal and vertical alignment, sufficient sight distance, and a sound road surface with proper drainage.

x. There are various low-cost treatments that reduce the risk of run-off-the-road crashes, including proper delineation, chevron alignment markers (CAMs), warning signs, provision of hazard markers before any roadside obstruction such as bridge parapet wall, provision of sealed shoulders and tactile edge lines, and wide centreline treatments. All these can be applied to help vehicles stay on the road.

xi. Delineation and signage are essential safety aspects of preventing run-off-the-road crashes, as they serve as visual guidance to drivers along a highway. Such information and guidance become particularly important at night, requiring that the devices are fitted with retro-reflective material. Good design and installation of signs and guideposts, as well as regular maintenance of the devices, are important to ensure that the devices perform as needed for the road conditions.

xii. Concrete guideposts should not be used as they are a hazard to errant vehicles. Narrow flexible guideposts made of timber, sheet metal, or plastic should be used as they present a lower risk to errant vehicles' occupants, particularly motorcyclists, if hit.

xiii. As a last resort, it may be necessary to ensure that each hazard (particularly trees) is delineated so it can be more easily seen by drivers as shown in **Plate 5.34**.This should be considered as a last option when treating hazards, as delineating a hazard will likely reduce incidental collisions or "innocent hits" but will not assist the occupants of an errant vehicle that is out of control. Delineating a hazard that is too close to the carriageway could be accompanied by other treatments, including a reduction in the speed of the highway (for instance to no more than 50 km/h) or protection by safety barriers. The object hazard marking provided should be retro-reflective to ensure visibility at night.

Plate 5.34 Delineation of trees with reflectors

5.1.6.8 BARRIERS

Barriers are used to shield hazards from errant vehicles. They can be used along the median (sometimes referred to as non-traversable medians) to prohibit movement of traffic across the median or on the roadsides to shield roadside hazards. They are designed to redirect an impacting vehicle and dissipate crash forces in a controlled manner, thus reducing the severity of crashes involving out-of-control vehicles. Barriers broadly fall under three categories: flexible barriers (e.g., wire-rope safety barriers), semi-rigid barriers (e.g., steel beam), and rigid barriers (e.g., concrete). Each type of barrier has various benefits and constraints that make them suitable for some locations, but unsuitable for others. To avoid installing unsafe barriers or wasting resources, engineers need to understand the benefits and the limitations of each barrier type. Detailed description of each barrier type is provided in **Section 12.5**.

5.1.6.8.1 SAFETY IMPLICATIONS

i. Barriers that are not well-designed fail to perform satisfactorily and can be a safety hazard as indicated in **Plate 5.35** and **Plate 5.36**.

Plate 5.35 The use of Bollards as safety fence is inappropriate for traffic safety

Plate 5.36 Rail units overlap in the wrong way

ii. A barrier that is too low can lead to an impacting vehicle to vault over it. A barrier that is too high (for flexible and semi-rigid barriers) can cause an errant vehicle to pass beneath the cables or railing leading to severe consequences.

iii. A barrier that is too close to the road leads to increased incidental impacts with the barrier, while a barrier that is too far from the road (and closer to the hazard it is shielding) means less opportunity for the deflection of a flexible and semi-rigid barrier and may result in the impacting vehicle hitting the hazard. The farther away the barrier is from the road also means the greater the chance of a high angle impact which could result in severe injuries to the vehicle occupants especially when impacting a rigid barrier.

iv. A critical aspect of a barrier's design is the "length of need" required in order to adequately protect a hazard. A barrier that is too short in its length may cause the errant vehicle to pass behind the barrier and strike the hazardous object or collide with oncoming traffic.

v. The end points (terminals or end treatments) of barriers can be dangerous if not properly designed, constructed, and maintained. The end of a guardrail, for example, can spear through an errant vehicle that strikes it (unless a correctly installed safe terminal is used) as shown in **Plate 5.37**.

Plate 5.37 Guardrail spearing through an errant vehicle

vi. Poorly designed transitions between different barrier types and insufficient offsets to hazards may lead to pocketing, that is, where an errant vehicle strikes a barrier but is directed by that barrier into a fixed object. This can occur, for example, where a guardrail is poorly connected to a concrete bridge parapet causing a vehicle that strikes the guardrail to be directed to hit the blunt end of the concrete parapet upon the deflection of the guardrail.

vii. A kerb in front of and close to a barrier can cause an errant vehicle hitting the kerb at high speed to jump and either vault over the barrier or hit the barrier at a greater height than provided for in the design and testing. Injuries can be more severe in such crashes.

5.1.6.8.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. Traditionally it has been thought preferable to remove, relocate, or modify roadside hazards, but in some situations, shielding a hazard with barriers may be the only practical option where it is not feasible or economically viable to treat the hazard in other ways. It is important to first assess the need for a barrier before installing one to determine if there are other ways to treat the hazard. This is because the barrier itself can represent a hazard to errant vehicles. A collision with the barrier should be less severe than collision with the hazard that the barrier is shielding.

ii. Safety barriers should only be installed if the manufacturer of the product has subjected it to an internationally accepted crash test to confirm it performs satisfactorily. The barrier should then be installed fully to the supplier's instructions, following the applicable standards on which the crash test was performed.

iii. Roadside barriers should be sufficiently offset from the travel way to allow space for vehicles to pull off the traffic lane.

iv. Since rigid barriers can cause serious injuries if struck at a high impact angle, they are located close to the traffic lane (usually within 4m of the edge of the nearest traffic lane) to minimize the risk that vehicles will impact the concrete barrier at a high angle. On the other hand, it is desirable to install roadside flexible and semi-rigid barriers further from the traffic lane to maximize the chance of a driver regaining control of the vehicle before impacting the barrier.

v. When located on horizontal curves, safety barriers may need to be offset further from the edge of the traffic lane so that they do not impede horizontal sight distance. Sight distance is a factor that also needs to be considered near intersections, median breaks, pedestrian crossings, and driveways.

vi. It is preferable that the slope in front of a barrier is installed as designed. This essentially means vertical for semi-rigid and flexible systems or to the required, tested slope for rigid systems. This is irrespective of the barrier manufacturer used. This is because safety barriers perform best when they are impacted by vehicles with their centre of gravity at or near the normal position.

vii. The terminals of barriers should be well designed to provide controlled deceleration of errant vehicles below recommended values that cause injury to vehicle occupants. They should also ensure that the vehicle is not speared, vaulted, snagged, or rolled on impact.

viii. Semi-rigid barriers are often used to shield concrete bridge parapets that could result in a serious crash. The transition from the approach barrier to the bridge parapet should provide a continuous face along which an errant vehicle can be controlled. To prevent pocketing of the vehicle upon impact, it is important to enhance the strength and stiffness of the barrier gradually as it approaches the parapet, e.g., through reductions in the post spacing's and to affix/embed the barrier firmly to the parapet as shown in **Plate 5.38.**

Plate 5.38 Safe connection between guardrail and rigid barrier on bridge

ix. Minor damage to flexible and semi-rigid barriers needs to be repaired in a timely manner to maintain the integrity of the barrier. If incidents are not reported, manual inspections of the barrier systems may be required.

5.1.6.9 MEDIANS

A road median is an area of separation between opposing flows of traffic. In effect, the median converts a “two-way” movement into two “one-way” movements. It can be constructed (often referred to as “raised”) using kerbing or median barriers; provided via paint (sometimes called a “flush” median, “ghost” island or wide centreline); or provided using an unpaved or grassed area. Vehicles are physically prevented from crossing the median with constructed medians, while they are only discouraged when using other types of medians. Various types of medians are shown in **Plate 5.39**.

(a) “Ghost” Island

(b) Concrete barrier

(c) Kerbed median

(d) Grassed median

Plate 5.39 Types of road median

Medians provide a degree of separation between opposing directions of traffic, meaning that when vehicles stray from their lane, they have time to recover and return safely to their lane, or are physically directed back into their lane (in the case of median barriers, and to some extent with kerbed medians). They can be used in urban areas as well as on high-speed roads. They may be supplemented with rumble strips, particularly on higher-speed roads, to alert inattentive or distracted road users that they are leaving their lane. In some settings, a median can provide a holding point for pedestrians trying to cross multiple lanes of traffic, especially when the median is accompanied by a pedestrian refuge island.

Median openings are typically provided for cross traffic movement at intersections and sometimes at access points allowing left, right (or both), and U-turns on the roadways where a physical median is present. Although median openings facilitate traffic movements, they can also introduce risks, especially when no turning bay is provided, or if the median is not of adequate width. It is essential that adequate turning provision be provided, especially in higher speed locations.

5.1.6.9.1 SAFETY IMPLICATIONS

i. Medians are used to improve overall safety and efficiency for vehicles, and if designed correctly may also provide benefits for vulnerable road users. By providing a central refuge, pedestrians are required to cross traffic from only one direction at a time.
ii. Road crashes can result from the presence of unnecessary or less predictable (readable) median openings (both for pedestrian crossings and vehicular U-turn movement) and smaller medians. Narrow medians and lack of adequate turning provisions, especially in higher-speed environments, can significantly increase crash risk.
iii. Proper planning and designing of median openings are critical for safety, access control, and maintaining traffic flow. This includes allowance for large vehicles, particularly buses or articulated ones to turn without their envelope encroaching into a through lane. Median openings should also not encroach on the functional area of another median opening or intersection.
iv. Specific benefits of adequately sized medians, especially non-traversable medians, include:
    a. Reduced chance of vehicles travelling in opposing directions colliding (reduced head-on crashes).
    b. Reduced lane width can lead to reduced vehicle speeds on the roadway.
    c. Better access control.
    d. Providing refuge area for pedestrians crossing the road.
    e. Managing the location of intersection traffic conflict points.
    f. Provide space to install improved lighting at pedestrian crossing locations thereby reducing night-time pedestrian fatalities at pedestrian crossing points.
v. For painted medians or where rumble strips are used, the risk for two-wheeled vehicles can be increased due to reduced or variable skid resistance.
vi. Where kerbing or raised pavement devices are used, there may be an increase in risk for two wheeled vehicles and pedestrians (trip hazard).
vii. There are also some efficiency benefits in addition to those for road safety, including decreased delays for motorists and increased capacity of roadways.

5.1.6.9.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. Reduction in head-on crashes can be achieved through selection of a suitable width of the median.

ii. Adequate widening to provide an optimum turning radius should be provided to ensure vehicles do not block the roadway while turning. Dedicated turning lanes should also be provided to ease congestion or conflicts between turning and non-turning vehicles. This includes allowance for large vehicles, particularly buses or articulated ones, to turn without their envelope encroaching into a through lane.

iii. Unnecessary median openings (both for pedestrian crossings and vehicular U-turn movement) should be removed, and smaller narrow medians should be avoided where possible.

iv. Restriction of turning movements may be an issue for raised medians, and community input and acceptance should be sought. Regular provision of gaps may be needed to address this issue (ensuring that such gaps are well designed with appropriate turning facilities).

v. Appropriate median widths should be determined according to the road classification and function of the median. This will include whether turning movements are required (into and out of side streets), U-turn requirements, and pedestrian use.

vi. There should be adequate sight visibility for drivers turning in/out of accesses and at intersections.

vii. Median should be highly visible both night and day and should contrast with the travelled way.

viii. Clear advance warning and visibility should be provided for raised medians.

ix. Appropriate drainage facilities should be included when installing raised medians.

x. Placement of rumble strips, kerbs, and raised pavements should be carefully considered so as to avoid being a hazard for two-wheeled vehicles. Similarly, painted medians with poor skid resistance should not be used where they might become a hazard for these road users.

xi. Raised medians can also be used to provide additional plastic shields to prevent glare from opposing traffic lanes.

5.1.6.10 ROAD SURFACING

Road surface characteristics affect road safety in several ways. One way is the surface friction which affects the resistance to sliding or skidding of tires across the road surface. This friction force, known as skid resistance, provides the grip that a tire needs to maintain vehicle control and for emergency stopping. Skid resistance is particularly important during wet weather conditions, as water on the pavement acts as a lubricant, reducing the direct contact between the tire and the pavement.

In addition to climate and water on the pavement, the potential for a skidding crash depends mainly on the speed of the vehicle, the cornering path, the magnitude of acceleration or braking, the condition of the vehicle tires, and the characteristics of the road surface. The road surface characteristics that influence surface friction include microtexture, macrotexture, megatexture/unevenness, chemistry of materials, temperature, thermal conductivity, and specific heat.

5.1.6.10.1 SAFETY IMPLICATIONS

The relationship between skid resistance and crash risk is well understood, with low skid resistance being directly related to increased crash risks [29], especially on wet roads. Low skid resistance is likely to result in longer stopping distances and may cause longitudinal or sideways skidding and loss of vehicle control. Loss of control of the vehicle may lead to run-off-the road crashes, head-on crashes, sideswipes, and rear-end crashes. Research performed in some European Union (EU) countries demonstrated that the use of road pavements with sufficiently high skid resistance could improve road safety by not only reducing the vehicle's sliding risk but also the crash risk and severity [30]. This is because drivers who lose their ability to brake effectively are more likely to encounter higher impact speeds compared to vehicles that decelerate prior to impact. The risk of crashes is also much higher at high traffic volume intersections than at low traffic volume locations owing to increased exposure to the pavement deficiency. Pavement defects that indicate poor skid resistance include:

a. A polished surface (rounded or worn-out aggregates) in the wheel path.
b. "Bleeding" of the pavement (upward movement of bitumen/asphalt as evidenced by a shiny black surface film).
c. Accumulation of oil or localized spill of slippery substance (especially on curves and intersection approaches).
d. Loss of top layer of aggregate (bitumen pavement).
e. A significant difference in friction between wheel paths.

5.1.6.10.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

i. Skid resistance is most important at locations where enhanced braking performance may be required including curves, approaches to intersections, areas near pedestrian crossings, etc.
ii. Crash rates can significantly be reduced by implementing proper measures to increase skid resistance at potentially dangerous locations such as curves, intersections, and bridges. There are two main options for the treatment of pavements with low skid resistance:
    a. Retexturing: This treatment type involves mechanical reworking of the existing road surface to improve its frictional characteristics. The methods include

diamond grooving, shot blasting, bush-hammering, and high velocity water blasting.

b. Resealing: These include relatively low-cost thin surfacing treatments that not only improve the surface texture and resistance to wet road skidding but can also seal the surface against water penetration and arrest disintegration of the existing road surface. They include surface dressing applications and high friction surfacing.

iii. Special attention on the pavement's skid resistance should be given on road sections where the effect of aggregate polishing caused by traffic is known to be most frequent. These include curves, roundabouts with small radii, sections where vehicles accelerate or decelerate, and in areas close to crossings.

iv. The choice of aggregates and bituminous mixes that retain skid resistance (rather than polish with wear) may be considered.

v. There are a number of safety strategies that can be implemented to mitigate increases in crash risks resulting from increased speeds due to resurfacing. When included at initial design, many of these interventions can be included at low cost or even no additional cost. These include:

   a. Traffic calming at key locations
   b. Gateway treatments on entering a village or other built-up area
   c. Speed limits
   d. Provision of wide sealed shoulders
   e. Visual narrowing of roads
   f. Segregated walkways
   g. Widening of curves
   h. Centreline and edge-line marking
   i. Advanced warning signs
   j. Advanced warning signs with advisory speeds.
   k. Chevrons
   l. Barriers (median and roadside)
   m. Improved sight distance
   n. Increasing visibility of intersections

5.1.6.11 DRAINAGE

The primary purpose of road drainage facilities is to prevent surface runoff from reaching the roadway and to remove rainfall or surface water efficiently from the roadway. Water pooling has a number of accident potentials, but the most serious one is the threat of hydroplaning. Tyres are designed to push water out of the way so they can stay in contact with the road at all times. Unfortunately, when there are large pools of water, tyres cannot funnel this water. As a result,

tyres can lose traction, compromising the driver's ability to steer, brake and accelerate.
Drainage facilities, including channels, shoulders, and surfaces, capture sheet flow from the road pavement and backslope and convey that runoff to larger channels or culverts within the drainage system. The gradient of drainage typically parallels the grade of the roadway. A stable conveyance design is a critical component in roadside channels. Refer to **Chapter 13** for further information on road drainage.

5.1.6.11.1 SAFETY IMPLICATIONS

i. The lack of good drainage can lead to the ingress of water into the road structure leading to structural damage and costly repairs, while surface water can form a road safety hazard. Water on the pavement will contribute to crashes from hydroplaning and loss of visibility from splash and spray. The aim of drainage facilities is to prevent surface runoff from reaching the roadway and to remove rainfall or surface water efficiently from the roadway.
ii. Water may accumulate on shoulder areas like ponds, also creating a risk.
iii. Poor drainage causes early pavement distresses and damage of shoulders, leading to driving problems and structural failures of the road.
iv. One study showed that 22 percent of all run-off the-road (ROR)/rollover crashes involved hitting a ditch or embankment and another study determined that 55 percent of ROR rollover crashes result in injury. A very high proportion of these are on rural roads.
v. Culverts that have unprotected headwalls close to the carriageway are a hazard to vehicles using the sealed shoulder.
vi. Drainage is usually more difficult and costly for urban than for rural roads. This is because of more rapid rates and larger volumes of runoff, costlier potential flood damage to adjacent property, higher overall costs from more inlets and underground systems, greater restrictions from urban development, lack of natural water body areas to receive flood water, and higher volumes of vehicular and pedestrian traffic.

5.1.6.11.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. An important part of road design is consistency, which prevents discontinuities in the road environment and considers the interrelationship of all road elements. The interrelationship between the drainage channel and side slopes is important because good roadside design can reduce the potential severity of crashes that may occur when a vehicle leaves the roadway.
ii. Discontinuous sections of kerbing, as at the gore of ramps, and variable kerb offsets should not be used as expedients to handle pavement drainage where these features could contribute to loss of control by vehicles that run off the road.
iii. Deep and open drainage structures near the roadway must be avoided, as they constitute

rigid obstacles that may aggravate crash severity.

iv. Adequate sight distance for drivers must be provided to ensure vehicles can stop before entering any floodwaters. The floodway longitudinal profile should be horizontal so that the same depth of water exists over the entire floodway length. The floodway length should be limited and on a straight stretch of road where possible. Adequate permanent and temporary signage and delineation must be installed.

v. Hydraulic capacities and locations should be designed to take into consideration damage to upstream and downstream property and to reduce the likelihood of traffic interruption by flooding.

vi. Inadequate drainage can lead to high maintenance costs and adverse operational conditions.

vii. Median areas should preferably not drain across travelled lanes, and often the inside lanes and shoulder of multi-lane roads will drain to the median area where a centre swale collects the runoff. Medians may be drained by drop (grate) type inlets.

viii. Drainage channel design in rural areas should incorporate traversable roadsides, good visibility, control of pollutants, and economical maintenance. This may be accomplished with flat side slopes, broad drainage channels, rain gardens, and liberal warping and rounding. In urban areas, runoff is often captured in enclosed storm drains, and rain gardens may be used to reduce the amount of runoff.

A. FOR ROADSIDE USERS

i. Drainage facilities should be designed to minimize their impacts on motor vehicles. Culvert end treatments should not be an obstruction, either through relocation of the feature outside of the 5 m clear zone from the edge of the running lane, or where this is not possible, an assessment should be undertaken to establish whether the end treatments can be made traversable. If neither remedial treatment is possible then safety barriers should be considered. All shoulder slopes into ditches should be at a maximum of 1:3 and desirably 1:6.

ii. In areas where roadway surfaces are warped, such as at cross streets or ramps, surface water should be intercepted just before the change in cross slope. Flumes are used to carry the water collected by intercepting channels down cut slopes and to discharge the water collected by shoulder kerbs. Flumes can either be open channels or pipes as shown in **Plate 5.40**, but closed flumes or pipes are preferred to avoid failure due to settlement and erosion.

Plate 5.40 Piped flume

iii. When the capacity of the kerb/gutter/pavement section has been exceeded (e.g., near low points of sag vertical curves, pedestrian crossings, etc.), drainage inlets that are connected to a storm drain-pipe can be installed to divert runoff from the roadway surface. However, grate inlets alone are not recommended in sag locations because of potential clogging.

iv. Pit lids of inlets and channels should be designed to ensure the safety of motor traffic, maintenance vehicles and plants, and pedestrians and cyclists. Pit lids should be designed to carry the appropriate motorist's, cyclist's and pedestrian's passing and loading if necessary.

v. For high-speed roads, pit lids should not be located within the traffic lanes. If necessary, they should be located outside the clear zone. On low speed roads, pit lids should also be located outside of traffic lanes, as the lids can cause impacts, cause noise, come loose, and cause safety problems.

B. CROSS SLOPES

i. Drainage of kerbed roadways on sag vertical curves needs careful profile design. At a level point on a crest vertical curve, there is no difficulty with drainage if the curve is sharp enough or the road surface has sufficient crossfall or superelevation.

ii. Flat grades can typically provide proper surface drainage on unkerbed roads where the cross slope is adequate to drain the pavement surface laterally. With kerbed roads, longitudinal grades should be provided to facilitate surface drainage.

iii. In the superelevation transition section, the combination of an inadequate crossfall and a longitudinal design gradient may result in the edge of the pavement having negligible longitudinal fall. This can lead to poor pavement surface drainage, especially on kerbed cross sections. This length of the transition section should be closely contoured to understand the wider pavement shape, including the tangent runout section and an equal length of the runoff section on the curve (Refer to **Section 7.2.8** on superelevation).

iv. For these problems, providing a minimum profile grade or a minimum edge-of-pavement grade in the transition section can be considered to maintain a certain profile grade and edge-of-pavement grade.

5.1.6.12 KERBS

Kerbs are raised or vertical elements located very near the edge of the travelled way that usually extend 75 to 200 mm above the road surface. They serve the following purposes: drainage control, delineation of the pavement edge, delineation of pedestrian walkways, right-of-way reduction, reduction of maintenance operations, aesthetic purposes, and assistance in orderly roadside development.

Kerbs are commonly used in urban areas, with a major benefit in containing drainage within the pavement area, separating pedestrians from traffic flow, and in channelizing or controlling traffic into and out of adjacent properties. They can be placed on medians or edges of the travelled way. Kerbs may be constructed using various materials, including cement concrete, granite, and asphalt/bituminous concrete and are often combined with gutter sections.

There are two basic kerb design types: vertical and sloping kerbs. Vertical kerbs, also referred to as barrier or non-mountable kerbs, have a vertical or nearly vertical face and deter vehicles from leaving the roadway. Sloping kerbs, also referred to as mountable kerbs, have a sloped face to permit vehicles to encroach on them readily when needed. They are usually used in situations where it is desirable to provide access to the roadside in emergency situations and to adjacent properties. From these basic kerb design types, there are further types, including semi-barrier and semi-mountable kerbs with a variety of designs.

5.1.6.12.1 SAFETY IMPLICATIONS

i. Kerbs are primarily used on low-speed facilities, and caution should be applied when installing kerbs on high-speed facilities. According to AASHTO [31] installing kerbs instead of narrow flush shoulders on urban four-lane undivided roads appears to increase off-the-road and on-the-road crashes of all severities. Installing kerbs on suburban multilane highways instead of narrow flush shoulders appears to increase crashes of all types and severities.

ii. Vertical kerbs have an ability to redirect an errant vehicle in a direction parallel to the travelled way provided the impact velocity and angle are modest; a situation applicable to low-speed facilities. The re-directional capabilities occur at speeds of approximately 40 km/h or lower.

iii. Vertical kerbs or steeply sloped kerbs can be a hazard to cyclists and motorcyclists.

iv. On high-speed facilities, vertical kerbs are a safety hazard as depicted by **Plate 5.41**. A high-speed impact with the kerb will introduce a roll moment since the vehicle's centre of gravity is much higher than that of the top of the kerb. This in turn introduces instability into the vehicle's trajectory that may limit a driver's ability to control the vehicle. Since kerbs are primarily used for drainage purposes, they are often found in conjunction with steep side-slopes where a rollover would be even more likely.

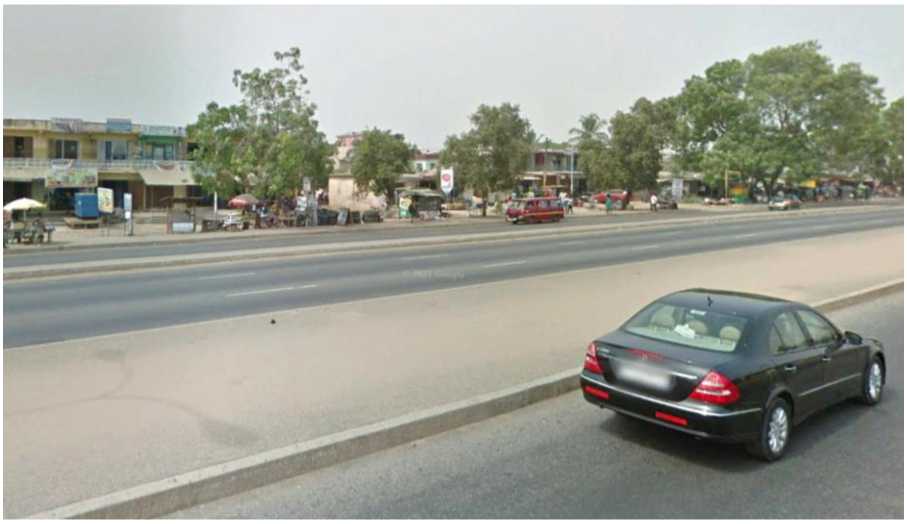


Plate 5.41 Hazardous vertical kerbs on high-speed road

5.1.6.12.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. Vertical kerbs are recommended for built-up areas adjacent to walkways with considerable pedestrian traffic, shared use paths, and at lay-by (**Plate 5.42**). This is because they reduce the risk to pedestrians, not only as a physical barrier, but also as a psychological barrier as drivers generally tend to shy away from the kerb line.

ii. While kerbs are to be designed to discourage motorists from encroaching onto the pedestrian realm, it is desirable that pedestrians can still step up and down from the pedestrian realm to the travelled way. The typical preferred kerb height is 150 mm.

iii. At pedestrian crossing locations, dropped kerbs are ideal, as they allow pedestrians, particularly the physically disabled, elderly, and those with prams/ strollers, to cross the road with ease

iv. As an alternative, particularly for unsignalized pedestrian priority crossings, the carriageway can be raised to footway level and act as a speed control measure.

v. Where drop kerbs are used, they need to be matched at both ends of the crossing location, with a tactile paving surface to facilitate the movement of visually impaired persons. Drop kerbs are also used to allow access by vehicles to properties.

vi. Where encroachment by motorists onto the pedestrian realm is an issue, protective techniques such as bollards and planters may be employed, rather than higher kerbs.

vii. The use of kerbs is generally discouraged in higher-speed roadways (greater than 60 km/h) because of their effect on a vehicle's trajectory upon impact. However, they may be required because of restricted right-of-way, drainage considerations, access control, delineation, and other kerb functions. It is recommended that sloping kerbs be used where such a need exists and should be located at the outer edge of shoulders rather than the edge of the travelled way. Sloping kerbs would also enable access to the roadside in case of emergency situations, and motorists can park clear from the travelled way in case the width of the sealed shoulders is not wide enough.

viii. Since the appearance of cement concrete and bituminous concrete kerbs offers little visibility in contrast to normal pavements, particularly during foggy conditions or at night when surfaces are wet, marking of kerbs with reflectorized materials such as paints and thermoplastics or attaching reflectorized markers to the top of the kerb enhances their visibility. Periodic cleaning or repainting is required to maintain this visibility.

ix. For kerb-barrier combinations, it is important to note that a kerb can have an effect on a vehicle's trajectory, which often involves the transformation of longitudinal kinetic energy to vertical and rotational kinetic energy that is hard to control. For this reason, one approach to the design is to place the kerb behind the face of the barrier or flush with the barrier and limiting the deflection of the barrier by stiffening. It is recommended that the kerb to be used should be of the sloping type and not more than 100 mm in height. Careful consideration and safety risk assessment are needed for locations where the above solution cannot be achieved to determine whether a modified outcome is safer than providing no barrier at all.

Plate 5.42 Non-mountable kerb adjacent to walkway and lay-by

5.1.6.13 ROAD TRAFFIC SIGNS

Traffic signs aim at providing information, warning and/or regulations for road users, in order to help them make sound decisions regarding their speed, lane choice and other parameters of their behaviour. The correct positioning, size and condition of signs will ensure that they will be observed and recognized, thereby providing the driver with adequate time to react and take action to prevent a crash from occurring.

Apart from signs warning of approaching features, there are others for use at the site itself, such as direction chevrons at bends or intersections and regulatory signs at the point of enforcement. The three main functions of traffic signs are to regulate, warn, and inform. In addition, there are increasing amounts of commercial or advertising signage on the highway. These are not strictly traffic signs but do impact on road user safety.

5.1.6.13.1 SAFETY IMPLICATIONS

i. A common safety hazard is inadequate traffic signs on the roadway such as the provision of non-standard and poorly located/maintained signs. In some instances, the provision of consistency of sign appearance and use are essential for road safety, as is the selection of sizes appropriate for the prevailing traffic speed.
ii. Signs need to be visible in enough time to understand the message and take appropriate action.
iii. Signs may not be visible at night because of poor illumination, lack of regular maintenance, or continuous power supply.
iv. Reflective signs not regularly cleaned may not maintain their design properties.
v. Maintenance is vital as poor quality, bent, or missing signs are not able to convey messages clearly.
vi. A recurring problem with signs is their obscuration, either by permanent features such as road furniture, road alignment, and vegetation or by parked vehicles and, on dual carriageways, by moving vehicles in the nearside lane.
vii. Signs can themselves obscure other features and may be visually intrusive from an environmental point of view.
viii. A major issue is the theft and vandalism of signs.
ix. Overuse of signs is distracting to the road user.
x. Too many signs can detract from their objective by overloading the driver with information leading to confusion, or to a situation where the driver ignores some signs. This unsafe situation is shown in **Plate 5.43**.

Plate 5.43 Overuse of signs is distracting

xi. Warning signs sited at different distances from the associated hazards in different localities, for instance, could mislead road users who venture outside their local area.

xii. Inconsistency in route guidance can result in drivers making unsafe and inappropriate lane and turn decisions.

xiii. Advertising signs are designed to attract the user's attention and pose a major distraction. Their prominent use at junction and complex locations is dangerous as shown in **Plate 5.44**.

Plate 5.44 Road signs overshadowed by advertising signs

5.1.6.13.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. In order to achieve safe and efficient operation of a road network, it is essential that all signing provided are necessary, clear and unambiguous.

ii. Generally, road traffic signs aim at providing information, warning and/or regulations for road users, in order to help them make sound decisions regarding their speed, lane choice and other parameters of their behaviour. The correct positioning, size and condition of signs will ensure that they will be observed and recognized, thereby providing the driver with adequate time to react and take action to prevent a crash from occurring.

iii. Road Signs should not provide an unnecessary distraction.

iv. To obtain the fullest benefits of uniformity, therefore, there should not only be uniformity of signs but also uniformity in their use, siting, and illumination.

v. The siting of signs is critical - they need to be far enough in advance of a feature to give sufficient time for the message to be understood and obeyed, but not so far in advance for the message to be forgotten by the time the feature is reached.

vi. Signs must be visible in darkness. This can be achieved with the use of retro-reflective signs.

vii. Regular maintenance is important to maintain visibility, function, and presence of signs.

viii. Making signs with material that have little value or other uses can make them less susceptible to theft. Secure fixing to supports also helps. The support should be frangible, and signs should not constitute a hazard in themselves to vehicles leaving the road or obstruct visibility or movement.

ix. Signs should be sited far enough away from the running lanes as to not present a hazard should a vehicle leave the roadway.

5.1.6.14 ROAD MARKING

Road marking is any kind of device or material that is used on a road surface to provide guidance and information to all road users.

The essential purpose of road markings is to guide and control traffic on a road. They supplement the function of traffic signs, serve as a psychological barrier, and signify the delineation of a traffic path and its lateral clearance from traffic hazards for the safe movement of traffic. Hence, they are very important to ensure the safe, smooth and harmonious flow of traffic. This is likely to become more important with autonomous vehicles that rely on good quality road markings for lane guidance.

They can be used to delineate traffic lanes, inform motorists and pedestrians, serve as noise generators (when installed with an audio tactile raised profile) when run across a road, or attempt to wake a sleeping driver when installed in the shoulders of a road. Road surface markings can also indicate regulations for parking and stopping. They can be either longitudinal (along the roadway); transverse (across the roadway) or provide written words or symbols. Refer to **Section 12.3** for further information.

5.1.6.14.1 SAFETY IMPLICATIONS

i. Road markings have their limitations:
   a. They provide less skid resistance than the surrounding road surface.
   b. Removal and repositioning of road markings can leave a ghost marking that can confuse users.
   c. Their conspicuity is impaired when wet or dirty.
   d. Their effective life is reduced if they are subjected to heavy trafficking.

ii. They make a vital contribution to safety, e.g., by clearly defining the path to be followed through hazards, by separating conflicting movements, and by delineating the road edge on unlit roads at night.

iii. They can also help to improve junction capacity and make best use of available road space. In particular, widespread use of lane markings is desirable as they encourage lane discipline and improve the safety and efficiency of traffic flow.

iv. The guidance function is less critical (although still important) in daylight or on lit roads because there are many visual cues available to enable the driver to judge course and position. On unlit roads at night, conditions are very different; the visual stimuli in the distance and to the sides of the road are largely absent. Road markings then become the most important aid in enabling the driver to follow the road.

v. Collaborative European research has shown that drivers need to be able to detect guidance markings at a distance equivalent to a minimum of two seconds of travel time. If the visibility is less than this, drivers tend to adjust too late when the road changes direction. They run too close to the centreline on nearside bends, or too close to the road edge on offside bends. The higher the prevailing traffic speed, the greater the visibility distance required to maintain this two second "preview time." If it is not provided, drivers tend to miss the curve, or proceed in a series of staggers. Almost all the recent crash research has been geared toward adding edge lines to roads. Recent crash studies as well as those more than a half century old have conclusively shown that adding edge lines to rural two-lane roads can reduce crashes and fatalities. In a recent study, driver workload was reduced after edge lines were added to narrow two-lane roads. [32]

vi. Visibility distance is adversely affected by glare from oncoming vehicles, dirty headlamps, or windscreen, and especially by rain; the glass beads which produce the nighttime luminance are drowned by excess water, greatly reducing the brightness of the line.

vii. Older drivers also see a marking less well than younger drivers; someone seventy years old might suffer a reduction in visibility distance of more than 20 percent compared with drivers in their twenties.

5.1.6.14.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

i. Line marking layout should always be considered in detail at the design stage of any scheme.

ii. A variety of factors influence the visibility distance of a road marking. It is increased when a line is wider, has a higher mark-to-gap ratio, or has a higher coefficient of retro-reflected luminance at night-time and higher contrast with the road surface during day light.

iii. Longitudinal lines should be designed to ensure a flowing alignment, avoiding sudden changes of direction or sharp tapers of inadequate length as shown in **Plate 5.45**.

Plate 5.45 Unexpected deviation of line marking

iv. For line markings to be effective, they need to be clearly visible both by day and by night.

v. Most line markings that have a guidance function are required to be illuminated by retroreflecting material). Retro reflectivity is achieved through the addition of glass beads applied directly to the surface of the line marking during the application process and, in the case of thermoplastic, through the presence of glass beads incorporated within the material itself. This makes the marking much brighter at night than non-reflectorized materials.

5.1.6.15 ROADWAY LIGHTING

Road light is a raised source of light, usually situated on top of a light pole (column), lamppost, or lamp standard on the side of a road or path, or in the median of a divided carriageway. It may also be suspended on wires over the carriageway.

Lighting is most appropriate in urban streets, intersections, bridge approaches, accident prone section and key places where pedestrians cross.

The introduction of adequate street lighting can help reduce night-time accidents and is an established accident prevention measure in urban areas. It is particularly important where there are high proportions of pedestrians, cyclists or other poorly lit road users including animals. Refer to **Section 12.8** for further information on roadway lighting.

5.1.6.15.1 SAFETY IMPLICATIONS

i. Major advantages of street lighting include prevention of crashes and increase in safety [33].
ii. It is estimated that 40% of all annual roadway fatalities in Ghana occur at nighttime and 80% of such fatalities at night time occur on dark roads or roads without lighting [2].
iii. There are also physical dangers to the posts of road lamps. Road lamp post pose a collision risk to motorists and pedestrians.
iv. Most of the information drivers utilize in traffic is visual. Visual conditions can therefore be very significant for safe travel.
v. In the dark, the eye picks up contrast, detail, and movement to a far lesser extent than in daylight. This is one of the reasons why the risk of a crash is higher during darkness than during daylight for all road users.
vi. The loss of night vision because of the accommodation reflex of drivers' eyes is the greatest danger for drivers in terms of optical safety risk.
vii. As drivers emerge from an unlit area into a pool of light from a streetlight their pupils quickly constrict to adjust to the brighter light, but as they leave the pool of light, the dilation of their pupils to adjust to the dimmer light is much slower, so they are driving with impaired vision.
viii. As a person gets older the eye's recovery speed gets slower, so driving time and distance under impaired vision increases.

ix. Oncoming headlights are more visible against a black background than a gray one. The contrast creates greater awareness of the oncoming vehicle. Lighting therefore needs to highlight the silhouette of an approaching vehicle or pedestrian effectively.

x. High winds or accumulated metal fatigue also occasionally topple road lights if not maintained.

xi. Similarly, road lights are only effective when working. Poor maintenance or lack of consistent power supply can render them ineffective and a collision hazard.

5.1.6.15.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. Lighting is most appropriate in urban roads, and key locations include intersections and places where pedestrians cross.

ii. The level of illumination needs to be consistent, and maintenance is most important.

iii. Lighting should provide a uniformly lit road surface against which vehicles, pedestrians, or other objects are seen in silhouette as shown in **Plate 5.46**.

iv. The design of the lighting system should relate to the road surface reflection characteristics in order to provide the optimum quality and quantity of illumination.

Plate 5.46 Lighting at night-time

v. Light coloured surfaces give better silhouette vision than do dark ones.

vi. Lighting systems can be expensive to install and maintain. Frequent interruptions to power supplies can also reduce effectiveness. Recent technological advances in solar power generation are making lighting more appropriate for remote communities and hazardous roadway sections as indicated in **Plate 5.47**.

Plate 5.47 Solar powered streetlights at Okyereko near Winneba

vii. Spacing between light poles is typically 2.5 times the height of the light source. A single row of lights might be sufficient for a narrow street, but multiple sources are needed for wider streets.

viii. Light poles that are too far apart result in areas of darkness and can leave users feeling unsafe, as well as affecting the driver's perception of shadow and silhouette.

ix. Rigid lighting columns may be redesigned to provide more forgiving frangible (breakaway) posts and lighting columns, i.e., impact absorbent or slip base types.

x. Collision risk can be reduced by locating columns away from runoff areas or designing them to break away when hit (frangible or collapsible supports), protecting them by guardrails, or marking the lower portions to increase their visibility, particularly for pedestrians.

5.1.7 INTERSECTIONS

An intersection is a location on the road network where two or more roads or streets meet or cross. They may be classified by:

i. Number of roads that meet (approach arms)

ii. Level (grade-separated or at-grade),

iii. Form of traffic control (uncontrolled, signalized, or unsignalized)

iv. Layout ('T','Y', roundabout, raised)

Grade-separated intersections are sometimes referred to as interchanges. It is often difficult to determine the best intersection type for any particular location, taking into account all relevant factors and several options that may be possible.

The selection of an intersection involves considerations of safety and operational performance,

including capacity, compatibility with adjacent intersection treatments, topography at the site, among other factors. Refer to **Chapter 8** and **9** for further information on at-grade and grade separated intersections respectively.

A. SAFETY IMPLICATIONS

i. The safety needs of all road users, including pedestrians, cyclists, motorcyclists, and people with mobility difficulties, must be considered, as their needs may be a significant factor in the choice of treatment and the type of traffic control adopted.

Vehicle speeds through an intersection must be managed safely. Low relative impact speeds provide a safer environment for conflicting manoeuvres. When collisions do occur at lower speeds, the severity outcome tends to be lower. Speeds above 50km/h for motorized vehicles, and above 30km/h for non-motorized road users lead to increasingly severe crash outcomes. Lower speeds enable drivers to break and stop more quickly when there are hazards; to make easier judgements regarding speeds of other vehicles and therefore decisions about appropriate gaps in traffic; and to accept smaller gaps thus reducing delays and increasing capacity.

B. BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. The basic principles of good intersection design are that they should allow transition from one route to another or through movement on the main route with minimum delay and maximum safety. To do this, the layout and operation of the intersection should be obvious and unambiguous, with good visibility between conflicting movements. These objectives need to be achieved at reasonable cost, so the provision of unnecessarily high standards as well as inadequate ones needs to be avoided. Different intersection types will be appropriate under different circumstances depending on traffic flows, speeds, and site limitations.

ii. Intersections should be as simple as practicable and designed to guide users safely through conflict points.

iii. Intersections introduce an elevated level of risk due to the number of conflict points. One strategy for reducing risk is to remove unnecessary intersections, although this requires the existence of alternative and safe options for road users.

iv. The various types of intersection layout can provide a hierarchy of alternative layouts catering for increasing levels of traffic flow. These are:
   a. Intersections without any designated priority - uncontrolled intersections.
   b. Simple priority intersection - Stop or Yield control.
   c. Priority intersections with channelization.
   d. Roundabouts or signal-controlled intersections.
   e. Grade separated intersections.

v. Road network planning must be well considered to avoid creating multi-arm and skewed intersections. Inappropriate approach angles will obscure a driver’s sight triangle in the intersection area.

vi. The potential for severe injury within an intersection can also be minimized through reductions in speed, reduction in the number of conflict points, separation of road users, and/or reductions in the angle of vehicle impact.

vii. Large intersections with little channelization or deflection can create large open unregulated spaces with multiple conflict points and high vehicle speeds. While solutions would be site specific, the general principle of reducing speed and managing conflict points should be applied to all intersection designs.

viii. Conflict points can be reduced through geometric design, including channelization and provision of roundabouts, the addition of deceleration lanes, realignment of the intersection, turn bans, and a reduction in traffic lanes. In general, the number of conflict points at four-leg intersections is much greater than for T-intersections. However, the number of lanes also greatly affects the number of conflict points. Roundabouts result in the fewest conflict points for a four-arm intersection as shown in **Figure 5.9**

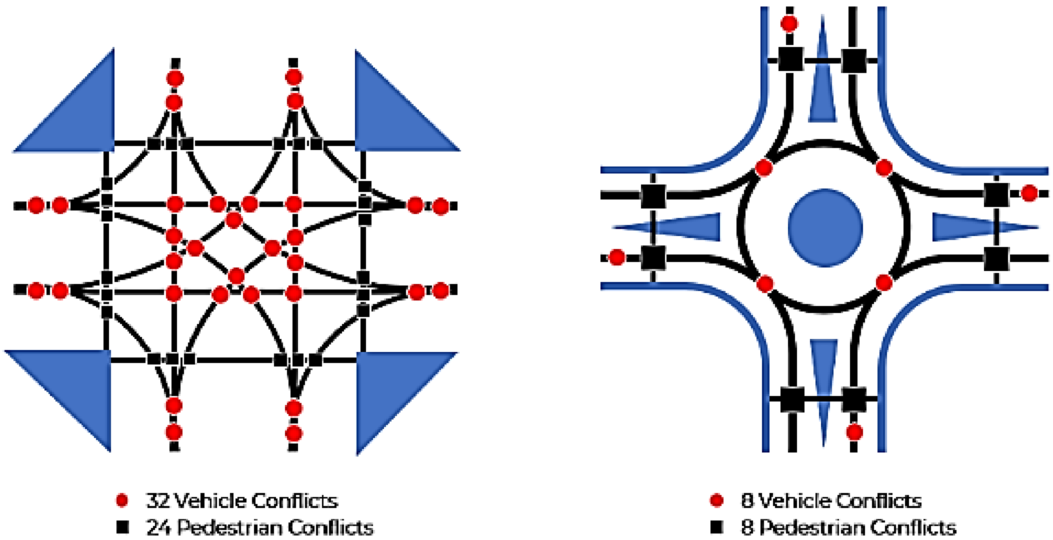


Figure 5.9 Conflict points at four-arm intersection and a roundabout

ix. Separation of traffic at intersections is another effective means to improve safety and can also produce benefits in traffic capacity. Grade separation (underpasses and overpasses) are the most substantial form of separation. These substantially reduce the change of conflict between vehicles, especially when well designed.

x. Other strategies to address intersection risk include the application of traffic control devices such as signs, markings, and traffic signals. These have benefits in reducing crash risk but do not always reduce the severity of crashes. It is often beneficial from a safety perspective to combine these devices with other measures (such as reductions in speed) to achieve significant safety benefits.

xi. Cost and necessary activities for maintenance at an intersection should be considered.

5.1.7.1 UNCONTROLLED AND UNSIGNALIZED (YIELD) INTERSECTIONS

An uncontrolled intersection is an intersection controlled by only general road rules (i.e., traffic laws), with no traffic control devices such as signs, road markings additional lanes, or channelization in place. They are the simplest form of intersection provided on the road network. Uncontrolled intersections are usually limited to very low-volume roads in rural or residential areas.

If traffic control devices are in place, then the intersection can be called an unsignalized or priority intersection. Unsignalized intersections can also be subdivided into those where the minor approach is required to yield to traffic on the main road and those where circulatory movement controls the entry of approaching traffic. This section only considers those intersections where no circulatory control is provided.

All control of potential conflicts at yield intersections, including those achieved by regulatory signs or road markings, are supported by relevant road rules. At uncontrolled intersections, only general road rules control traffic.

However, yield intersections still often account for a high proportion of network delays, conflicts between vehicles, and conflicts between vehicles and other road users (e.g., pedestrians). Yield intersections are suitable for situations where there are no operational problems, such as excessive delays/queues or safety problems (i.e., low traffic volume and low-speed roads, etc.).

5.1.7.1.1 SAFETY IMPLICATIONS

i. Straight four-arm intersections often have a poor safety record because of minor road traffic failing to stop for main road traffic, either because of driver indiscipline or because the driver is not aware that there is a major road ahead.
ii. The major crash types at both uncontrolled and yield intersections are where vehicles fail to stop, implying inadequate visibility or awareness of the intersection.
iii. Crashes with emerging vehicles suggest inadequate sight lines along either the major road or minor road.
iv. In most of unsignalized intersections, the minor roads lack adequate sight distance, mainly due to encroachments.
v. Wrong turns and chaotic traffic movements are commonly observed at these locations. Such untreated minor intersection and access roads may lead to unsafe movement of pedestrians and vehicles whenever present.
vi. Where intersections are uncontrolled, the lack of awareness by main road drivers for turning vehicles can result in rear-end collisions.
vii. If the yield line is in the dip at the edge of the major road camber, it can be invisible from a distance on the minor road.

viii. Speeds of approaching vehicles are also a major cause of collisions.

ix. For all types of uncontrolled or yield intersections, the problem of delay exists for minor road traffic. If the delays are excessive, emerging drivers may take undue risks in order to enter or cross the mainstream.

x. Multiple lane approaches place greater demands on the emerging driver and tend to be more hazardous locations.

xi. Slow-moving or stationary vehicles turning into a side road across a main road stream of traffic are often the cause of serious crashes, particularly at night.

xii. Problems can also be caused in urban areas by inadequate kerbs that give an unclear layout and make little or no provision for pedestrians.

5.1.7.1.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. In cases where there are no control devices (i.e., traffic signals and roundabouts), designating or clarifying priority rules (e.g., stop or yield signs/markings) must be provided to give clear indication of expectation to drivers as shown in **Plate 5.48**. This will also aid separation of conflicting movements in addition to the general intersection rules. These devices prevent or discourage inappropriate traffic movement at the intersection.

Plate 5.48 Yield sign being used as intersection control

ii. Traffic islands (e.g., triangular left-turn islands) and medians would help to provide delineation and direct traffic into the appropriate path through intersections.

iii. Although controlling traffic by police officers (or authorized persons) is often used in exceptional circumstances (e.g., peak traffic hours, road work, incidents), this might result in extra delays at the intersection.

iv. In case any safety treatments cannot be implemented at an uncontrolled intersection, redirecting traffic to a higher quality intersection should be considered.

v. Improving intersection conspicuity and driver's sight distance at intersections must be prioritized to increase awareness and readability.

vi. All obstacles within intersection areas must be removed as shown in **Plate 5.49** and all unnecessary conflict points must be eliminated. For example, placing a waiting space at the centre of an intersection is dangerous because passengers have to enter the intersection to reach the space. Furthermore, the waiting space will be an obstacle for other road user's sights.

Below is a summary of treatments for uncontrolled/ unsignalized intersections:

A. Approach and minor road treatment

i. Advanced warning signs and road markings would help to indicate the existence of an intersection to drivers.

Plate 5.49 Sight triangle obstacle from minor road at cross intersection

ii. Placing stop signs on the minor road approach to an intersection can be effective where the sight distance from the minor leg of the intersection is insufficient and it would be unsafe to proceed without stopping. But reassignment of a priority might not perform safely if placed contrary to driver expectation and it does not work as a stand-alone treatment.

iii. A decision as to whether a stop sign rather than a yield sign is required is based on sight distance available for drivers on the minor road approach, i.e., whether the sight distance from the minor leg of the intersection is inadequate and it would be unsafe to proceed without stopping. It has been found that the use of stop signs in locations with adequate sight distance does not provide additional safety benefits and can lead to a loss of credibility, and their effectiveness will be compromised.

iv. Speed management, also known as "traffic calming" features (e.g., speed humps, raised intersections, etc.) are used in conjunction with stop/yield signs on approaches of intersections to help control speed (see **Chapter 10** on traffic calming measures)

v. Channelization, adequate sight distance, or supplemental visibility enhancement, including lighting, should be made available at all the minor junctions.
vi. Provide flexible poles on both major and minor roads to separate traffic from the opposite direction. This can reduce certain types of crashes.

B. Movement prohibition measures

i. Prohibition of selected movements (e.g., left-in left-out, no left or right turn, full-time or part-time, etc.) can reduce certain types of crashes related to limited sight distance and pedestrians that involve left or right turning vehicles. This strategy can also reduce the frequency and severity of crashes.
ii. The prohibitions can be implemented by channelization, markings, and/or signs. Signs and/or markings alone will require other physical interventions.
iii. The prohibitions may be appropriate where a turning movement is considered to be high risk and other strategies are impractical or not possible to implement. This strategy may be difficult to justify at a major intersection unless the left-turn volumes are very low. It is generally preferred to more safely accommodate the turning movement at the point where the driver desires to turn than to displace the turn activity to an alternative location.
iv. An auxiliary lane provides separation for the manoeuvring of a vehicle and is typically used in rural areas where high-speed, low-volume traffic occurs and the volume and slow manoeuvring of turning traffic is sufficient to create a conflict with following traffic.

The following are the summary of treatments for Y- (skewed) intersection:

i. The speed of approaching vehicles at the intersection is affected by approach angles. Approach angles also affect the crossing distance (footprint) of vehicles at the intersection. Furthermore, appropriate approach angles may improve driver's sight triangle in the intersection area. Approach angles must be determined to achieve the following principles:
   a. Limit turning speed around obtuse angle. Acute angled intersections reduce visibility for motorists, while obtuse intersections allow for high-speed turns. A right-angle treatment can work as speed enforcement and can improve a driver's sight triangle as shown in **Figure 5.10**.
   b. Shorten the crossing distance (footprint) of vehicles. Compact intersections reduce pedestrian exposure, slow traffic near conflict points, and increase visibility for all users. Both acute- and obtuse-angled intersections create unnecessarily long pedestrian crossings.
   c. Separate vehicle flows to reduce conflicts as shown in **Figure 5.11**.

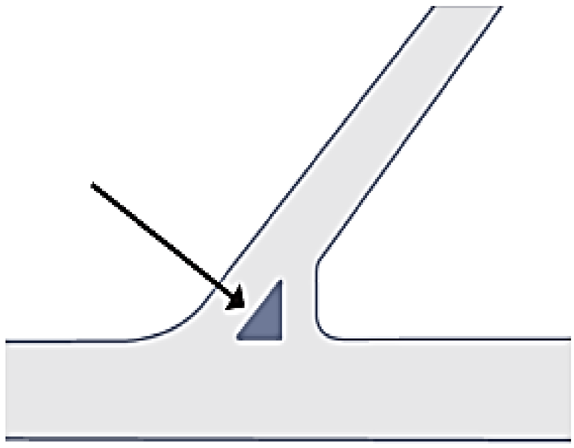


Figure 5.10 Island separating traffic at centre of minor road as indicated by the arrow

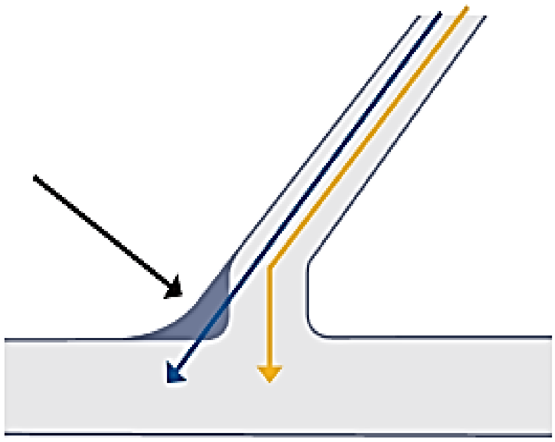


Figure 5.11 Kerb changing angle of entering intersection from minor road as indicated by the arrow

ii. Realignment of an intersection may impact sight distance and/or the impact angle for vehicles involved in collisions at the intersection. Realignment of an intersection is often too costly. It is much better to design the intersection well before it is built than to rebuild it. The reconstruction of an intersection should be implemented when adequate sight distance and countermeasures are not available.

5.1.7.2 SIGNALIZED INTERSECTIONS

A traffic signal-controlled intersection restricts conflicting traffic movements in time or space by only allowing non-conflicting movements to proceed through the intersection at the same time. It controls vehicular and pedestrian traffic and assigns the right-of-way to the various traffic movements for a given duration, thereby profoundly affecting free traffic flow.

Traffic signals operate on the basis of phases and stages. A signal phase is a single movement stream that is assigned a green signal to move or a red signal to stop. Several phases can be combined to create a single signal stage. Once all phases have been allowed to proceed, a full signal cycle of movement through the intersection is completed.

The amber signal is used to warn drivers of the approaching change in status between stop and go. The amber period is required to allow for driver reaction time and clearance of conflicting movements through the intersection. The potential conflict points vary by approach and size of the intersection. Refer to **Section 11.3** for further information on signals.

5.1.7.2.1 SAFETY IMPLICATIONS

i. Appropriate phase control sequences can reduce the frequency and severity of certain types of crashes, especially right-angle collisions, by separating these from other conflicting movements, including pedestrians.

ii. The common practice of allowing nearside turns through a signal-controlled intersection can still result in substantial collision risk for crossing pedestrians.

iii. Traffic control signals are sometimes installed at locations where they are not needed, adversely affecting the safety and efficiency of vehicular, bicycle, and pedestrian traffic. The judgment of implementation of traffic control signals at an intersection must be done after consideration of alternatives (e.g., installing pedestrian beacon/pelican, roundabout, and so forth). (Refer to **Section 8.2.4** on Intersection selection.)

iv. The improper or unjustified use of traffic signal control can result in:
   a. Excessive delay.
   b. Disobedience of the signal indications.
   c. Increased use of inadequate routes to avoid the traffic signals.
   d. Increases in the frequency of collisions (e.g., rear-end collisions).

v. It is important to understand that installation of traffic control signals is not a "cure all," and there may still be several risks (e.g., from noncompliance, lack of maintenance, remaining crashes, etc.).

vi. Visual obstructions of traffic signals and other traffic control devices should be removed. Traffic signals often are hidden by branches of a tree or other obstructions. This makes urban travel particularly difficult and potentially hazardous.

vii. Reduced conflict points for both vehicle-to-vehicle and vehicle-to-pedestrian can reduce certain types of crashes. For example, there are 32 vehicle-to-vehicle conflict points and 24 vehicle-to-pedestrian conflict points in a typical four-leg intersection as depicted in **Figure 5.12**. During a green light phase for pedestrians and vehicles approaching from the same direction, the number of vehicle-to-pedestrian conflict points can be reduced to only one if nearside turn on red is permitted as shown in **Figure 5.13**.

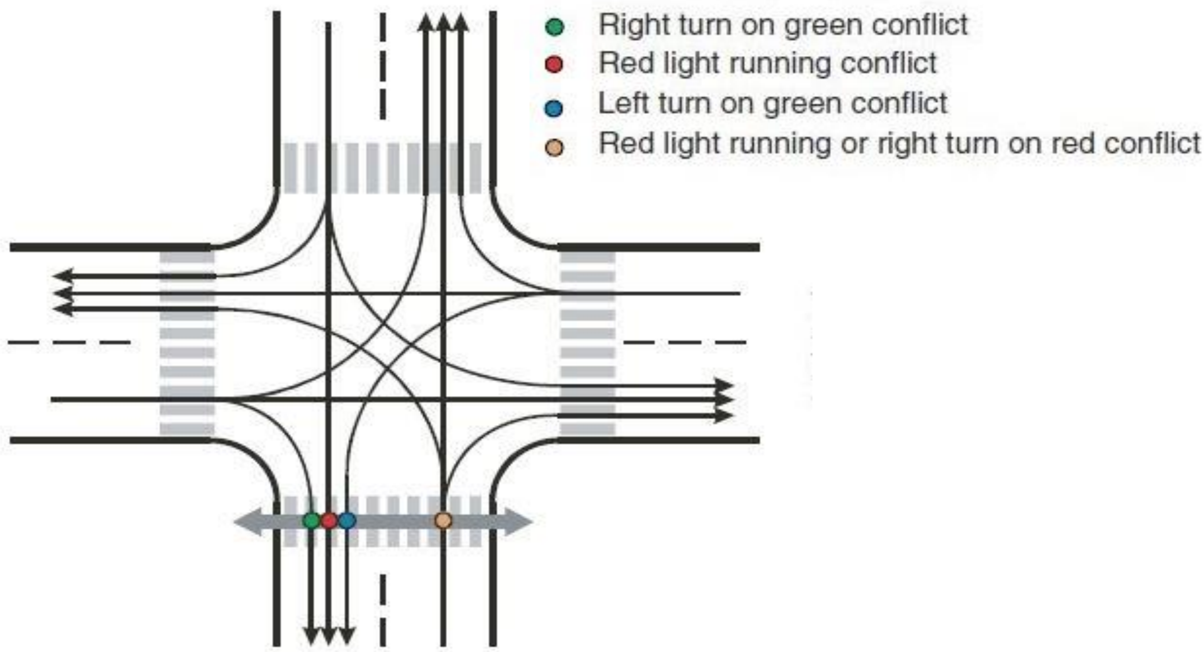
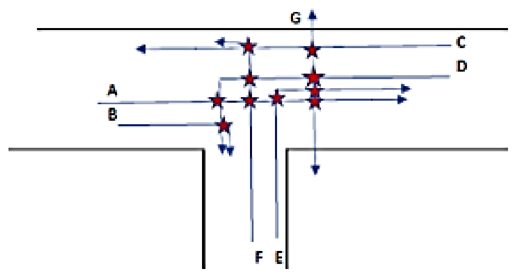


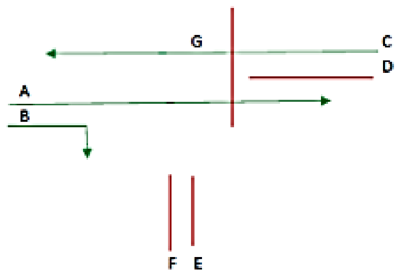
Figure 5.12 Example of Pedestrian vehicular conflict at four-leg intersection

5.1.7.2.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. Signal intersections simplify drivers' decision-making by preventing conflicting movements as illustrated in **Figure 5.13**. The possibility of misjudging whether it is safe to enter or cross an intersection by both the vehicles on a minor street and pedestrians crossing the street can be reduced.
ii. Layout of traffic signals must be considered with the visibility of signals for road users. Driver's sight triangle and the height of signals must also be considered.
iii. The signal head must also be visible at a point in the crosswalk which allows the pedestrian clear sight before and while crossing. Consideration should be given to the provision of LED signal head which has longer detection distance.
iv. Pedestrians must have sufficient time to travel to the centre of the farthest travelled lane before crossing vehicles receive a green [34].
v. Special attention for road users should be given if signals become dysfunctional or hidden.
vi. Alternative staging of signals can reduce all potential conflicts, but care is needed to maintain cycle times that do not result in users becoming impatient for change. Cycle times between 90 to 120 seconds are preferred.

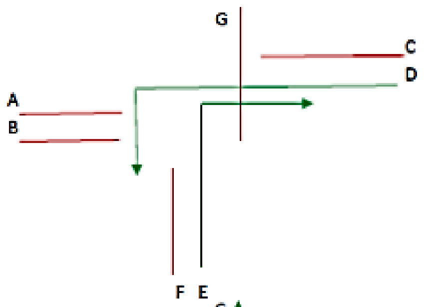


- A simple three-leg uncontrolled intersection with a pedestrian crossing on arm CD has 10 potential crossing points.
- All these can be removed under signal controls by preventing the conflicting movements operating together.
- By identifying each movement stream separately, alternative staging can be considered, depending on traffic volumes



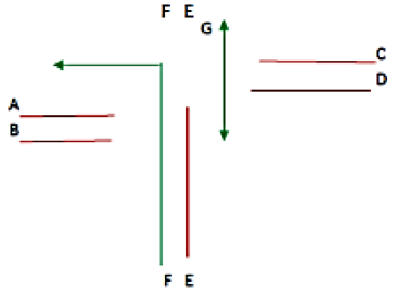
Stage 1

- Green indicates those movement streams moving, and red indicates those movement streams stationary.
- The amber period between Stage 1 and 2 is required to allow phases A-D to clear the conflict point as being the longest.



Stage 2

- Green indicates those movement streams moving, and red indicates those movement streams that are stationary.
- The amber period between Stages 2 and 3 is required to allow phases D-F to clear the conflict point.



Stage 3

- Green indicates those movement streams moving, and red indicates those movement streams that are stationary.
- The amber period between Stages 2 and 3 is required to allow phases A-G to clear the conflict point.

Figure 5.13 Typical Signal Cycle for above stages

vii. Supplemental pole mounted traffic signals may be placed on the nearside of intersections particularly where sight distance is an issue such as on approaches to intersections on curves as shown in **Plate 5.50**.

viii. Advanced stop lines at traffic signals are helpful in improving the visibility of pedestrians to motorists. Motorists may ignore the line if placed too far in advance of the pedestrian crossings as shown in **Plate 5.51**.

Plate 5.50 Supplemental signal for intersection around a curve

Plate 5.51 Unsafe manner at stop line (overcrossing stop line)

ix. At signalized intersections, advance stop lines back from the crosswalk at traffic signals must be placed away from the crosswalk to allow pedestrians and drivers to have a clear view of each other and more time in which to assess each other's intentions [35].

x. At large signalized intersections with multiple turn lanes, continuation of the lane markings through the intersection can provide additional guidance for motorists and reduce the occurrence of side impact collisions.

5.1.7.3 ROUNDABOUTS

A roundabout is a form of intersection channelization in which traffic circulates in an anticlockwise direction around a circular central island, and all entering traffic is required to give way to traffic circulating in the roundabout.

Benefits include reduced conflict points and therefore driver workload associated with perpendicular junctions and, depending on traffic flows, reduced queuing associated with traffic lights. They provide facility for U-turns within the normal flow of traffic, which often are not possible at other forms of intersections.

When entering, vehicles only need to give way at relatively low speeds, and do not always perform a full stop. As a result, by keeping a part of their momentum, the intersection performs more efficiently from a traffic flow perspective. In addition, engines will require less effort to regain the initial speed, resulting in lower emissions. Research has also shown that slow-moving traffic in roundabouts makes less noise than traffic that must stop and start, speed up and brake.

Because low speeds are required for traffic entering roundabouts, they are physically designed to manage the speeds of traffic approaching and entering the junction to improve safety. Approaches are designed so that vehicles enter the circulating carriageway with limited vehicle path radius naturally slowdown (141).

Roundabouts can be used satisfactorily at a wide range of intersection sites, including:

i. Urban local and collector roads
ii. Arterial roads in urban areas
iii. Rural roads
iv. Expressway/motorway ramp terminals
v. As a grade-separated treatment at an interchange.

Refer to **Section 8.11** for further information.

5.1.7.3.1 SAFETY IMPLICATIONS

i. Roundabouts provide a highly readable and consistent physical intersection layout that predictably and consistently limits the potential for higher speed and high impact angle conflicts.
ii. Transforming the control method from a two-way stop or traffic control signal to a roundabout with single/two lanes is effective in reducing the percentage of fatalities and injuries at intersections.
iii. For well-designed single lane roundabouts in particular, the rate of crashes between pedestrians and vehicles can be significantly reduced.
iv. By limiting the entry path curve and thereby introducing horizontal deflection to the approaches, vehicular entry speeds can be reduced, which provides drivers more time to react to potential conflicts and reduces crash severities.
v. There are fewer vehicular conflict points and less potential for high severity conflicts, such as right-angle, left-turn, and head-on crashes because of the roundabout’s design and because all drivers are going in the same direction.
vi. Generally, there is a reduced speed differential between vehicles travelling through the intersection, which reduces crash severity.
vii. They are effective during power outages. Unlike traditional signalized intersections, which must be treated as all-way stop or require police (or traffic warden) to direct traffic, roundabouts continue to work as normal.
viii. As remaining safety risks, the following factors can be considered; however, because of the reliably low-speed environment, the severity of injuries from roundabout crashes,

even for vulnerable users, tend to be low:

a. Misunderstandings of rules and not every driver knows roundabout rules.
b. Poor judgement of gaps by drivers entering a high-speed flow of circulating traffic, especially when there are multiple lanes.
c. Rear-end collisions between vehicles waiting to join the roundabout may increase (although these are far preferable than the high-speed impacts seen at other intersection types).
d. Sideswipe collision during changing lanes or entering/exiting the centre circle.
e. Pedestrian/cycle collision by not yielding to pedestrians and cyclists.
f. Painted or low height islands become less visible and negligible for drivers. Drivers may not make sense of what looks like painted circles on intersections that are meant to act as roundabouts.

5.1.7.3.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. Properly designed roundabouts control the angle at which traffic enters the intersection and the speeds of vehicles entering and going through the intersection by creating geometric curvature with centre and splitter islands. This feature results in safer intersections than other at-grade intersections where vehicles can enter the intersection without slowing their speeds.
ii. Newer designs may also include raised platforms or humps on the approach that have been successfully used to slow the approach speed of vehicles, reducing the need for geometric curvature, and sometimes significantly reducing construction costs.
iii. Circulating space within the roundabout is often restricted to a single lane; however, multiple lanes can be used provided there is sufficient size to allow the inner flow of traffic to manoeuvre to the outer lane to exit. However, it should be noted that, as circulating widths increase, the ability to control speed into and through the roundabout becomes less predictable.
iv. A key element of safe roundabout operation is to ensure that the central island or splitter islands provide sufficient deflection from the straight-ahead movement to ensure slow vehicle speeds through the intersection (an example of a poorly designed roundabout is shown in **Plate 5.52**). Where sufficient deflection is not possible (for instance due to restrictions in road space), raised platforms have been used successfully instead.

Plate 5.52 Unsafe roundabout with no deflection on the approach splitter island

v. With splitter islands, pedestrians are required to cross only one direction of traffic at a time at a roundabout and contend with slower-moving vehicles because of the splitter islands.

vi. Flat/low height islands (such as markings) are inappropriate for traffic safety. Centre and splitter islands should be physically raised to provide readability.

vii. Decoration and vegetation at centre and splitter islands must not obstruct driver's sight distance of approaching or circulating traffic. However, it should be sufficiently high to obstruct the straight through view of the road ahead and concentrate drivers' awareness on the roundabout.

viii. The centre island and the splitter islands must be large enough to force approaching vehicles to reduce their speed in order to enter the intersection. Too small centre islands and splitter islands may not work to reduce the speed of approaching vehicles during passing through the intersection because turning along the centre island is not required. This defeats the purpose of a roundabout.

ix. A key factor in determining the size of a roundabout, both the central island and the width of the circulating carriageway, is the safe negotiation of the design vehicle for all movements. For example, when designing for the safe passage of a semitrailer unit, as the central island radius decreases, so the circulating width must increase to allow the vehicle to get around the island. This effect depends on the vehicle dimensions and hinge point. Because this can result in less deflection and therefore higher negotiation speeds, it may be preferable to provide a slightly raised apron or ever-run area on the central island. With a low vertical lip (50 mm), this feature allows the large vehicles to negotiate safely while still providing a narrower "target" for small vehicles and maintains predictability.

x. A traversable truck apron can be provided at roundabouts to accommodate large vehicles while minimizing other roundabout dimensions (Refer to **Section 8.11.2**). A truck apron provides an additional paved area to allow the over-tracking of large semitrailer vehicles on the central island without compromising the deflection for smaller vehicles. At roundabouts which do not have truck aprons, the circulatory lanes become too wide to accommodate larger vehicles. This can cause an inappropriate usage of lanes. These roundabouts have higher vehicle speed through the intersection.

xi. Humps and platforms can be used to reduce speeds, especially where there is not enough deflection on approach.

xii. Pedestrian and cyclist facilities can be included in the intersection design.

xiii. Lane lines must be provided with appropriate widths. In the absence of lane lines, approaching vehicles will miss the courses they should drive in the intersection, and consequently crashes between vehicles will occur. The lane boundaries must be provided as per the approaching roads. This can also guide vehicles to turn along the centre island appropriately and reduce their speed.

xiv. In low speed constrained urban environments, mini-roundabouts (those with no physical island and only a painted circular road marking) can be effective if flows are low and speed is well controlled. Deflection though the intersection is provided through traffic rules and approach alignments that require the central marking to be passed to the offside.

xv. Mini-roundabouts may be an optimal solution for a safety or operational issue at an existing stop-controlled or signalized intersection where there is insufficient right-of-way for a standard roundabout installation. Mini roundabouts are characterized by a small diameter and mostly traversable (painted circle or low dome) islands (central islands and splitter islands) and offer most of the benefits of regular roundabouts with the added benefit of a smaller footprint.

xvi. Signage for indication of a roundabout ahead in a clear and consistent way throughout the network is very important. The variation in the use of signs and markings reflects either the lack of knowledge, the lack of attention to detail, or the lack of clear guidance for the implementation of road signs and road markings. Similarly, the variation of road markings also causes driver’s misbehaviours.

xvii. The performance of some congested roundabouts can be improved with traffic signal control by balancing entry flows and/or a continual flow of traffic on the circulating carriageway to prevent long queues causing long delays and blocking back into preceding junctions. Signals are able to keep the circulatory traffic flow fluid and hence balance and improve the roundabout capacity.

xviii. The number of pedestrians (and cyclists) can increase crash risks and delays because traffic is governed by yield-control entry at a roundabout, especially at intersections with a low volume of pedestrians. Providing specific crossing points and routes around the intersection separate from motorized traffic can improve pedestrian and cycle safety at roundabout intersections (Refer to **Section 5.1.5** on vulnerable users).

xix. Traffic rules and design of roundabouts must coordinate with other transportation modes to avoid increasing crash risks in arterial roads with cycle lanes and public transportation lanes (Refer to **Section 5.1.5.5** on Public Transport).

5.1.7.4 CHANNELIZATION (INCLUDING TURN/SLIP LANES)

Channelization is the provision of dedicated traffic lanes for different movements at intersections. It aims at improving the performance and safety of intersections by separating traffic flows (either through road marking or physical islands) and making driving patterns and right-of-way rules transparent. Such channelization can reduce the area of conflict as well as improve intersection angles. It may also be added to increase capacity, improve the visibility of traffic control devices, and reduce crashes. It can be included in all types of intersections, irrespective of layout or control.

Channelization can be included on both/or side roads and main roads. Separation of movements can be with traffic islands, medians, or road markings, together with auxiliary lanes or designating lanes for specific movements such as left-turn, right-turn, or U-turn.

5.1.7.4.1 SAFETY IMPLICATION

i. A primary goal of intersection design is to limit and/or reduce the severity of potential road user conflicts.

ii. The basic principles of intersection channelization that can reduce conflicts are:

   a. Separate points of conflict. Separation of conflict points can ease the driving task while improving both the capacity and safety at an intersection. The use of exclusive turn lanes, channelized right turns (for those driving on the right), and raised medians as part of an access control strategy are all effective ways to separate vehicle conflicts. (Refer to **Section 5.1.7.5** on right-in right-out).

   b. Define desirable paths for vehicles. The approach alignment to an intersection as well as the intersection itself should present the roadway user with a clear definition of the proper vehicle path at risky locations with complex geometry or traffic patterns, such as highly skewed intersections, multi-leg intersections, offset T-intersections, and intersections with very high turn volumes. Clear definition of vehicle paths can minimize lane changing and avoid "trapping" vehicles in the incorrect lane.

   c. Discourage undesirable movements. Designers can utilize corner radii, raised medians, or traffic islands to prevent undesirable or wrong-way movements, including restriction of turns and designing approach alignment to facilitate intuitive movements.

   d. Encourage safe speeds. On low-speed roads with pedestrians, turning speeds should be lower by smaller turning radii, narrower lanes, and/or channelization features.

On high-speed roads with no pedestrians, speeds for turning vehicles should be comparable with straight through speeds to remove turning vehicles from the through traffic stream as quickly and safely as possible. This can be accomplished with longer, smooth tapers and with associated deceleration length to corner at a slower speed.

e. Facilitate the movement of high-priority traffic flows. Accommodating high-priority movements at intersections addresses both drivers' expectations and intersection capacity. The highest movement volumes at an intersection define the highest priority movements, although sometimes route designations and functional classification of intersecting roads should be considered. In low density suburban and rural areas, giving priority to motor vehicle movements may be appropriate; however, in some urban locations, pedestrians and cyclists at times may be the highest priority users of the road system. Separating movements by channelization can reduce crossing widths for pedestrians and increase their opportunity to cross busy roadways.
f. Facilitate the desired traffic control scheme. Visibility of signs and markings at intersections can be maintained by channelization. Other equipment at the intersection should not block sight distance and should facilitate preventive maintenance by field personnel. Intersection layout should be designed for simultaneous left-turning movements and potential U-turning movements. Operational impacts and the design of pedestrian facilities should be taken into account during the intersection's design.
g. Accommodate decelerating, slow, or stopped vehicles outside higher-speed through traffic lanes. Speed differentials between vehicles in the traffic stream are a primary cause of crashes. Speed differentials at intersections are inherent as vehicles decelerate to facilitate turning. The provision of exclusive left- and right-turn lanes can improve safety by removing slower-moving turning vehicles from the higher-speed through traffic stream and reducing potential rear-end conflicts. In addition, through movements may experience lower delays and fewer queues. However, care is needed not to induce higher speeds for through and turning traffic and obscure the view for side road traffic.
h. Provide safe refuge and way finding for cyclists and pedestrians. Intersection channelization can provide refuge and/or reduce the exposure distance for pedestrians and cyclists within an intersection without limiting vehicle movement.

iii. Channelization separating through and turning lanes may constitute a hazard because of its placement when a raised treatment is applied, especially on high-speed roads [36].

iv. Several studies from high-income countries confirm that the provision of turn lanes has been found to reduce crash rates. [37], [38] & [39]

v. The provision of median islands on the approach to an intersection can assist drivers to identify the location of the intersection and raise their alertness to select their travel path through the intersection. Median islands provide some protection for turning vehicles when a turning lane is provided to take the turning vehicle out of the through lane. This treatment can achieve a reduction in head-on, rear-end, and right-turn type crashes by 20 percent. If the median island is placed through the intersection, thereby removing the cross-movement, head-on, right-turn and right-angle type crashes can be eliminated.

vi. For wider medians (generally more than 5.4 m) offsetting the turn lane provides the following safety benefits:
   a. Better visibility of opposing through traffic.
   b. Decreased possibility of conflict between opposing left-turn movements within the intersection.
   c. More left-turn vehicles served in a given period of time, particularly at a signalized intersection [39].

vii. The provision of indented turn lanes with painted islands can achieve a 20 percent reduction in opposing turn and rear-end crashes; and with a median island, reductions of 40 percent in rear-end, 30 percent opposing turn, and 20 percent loss-of-control crashes can be achieved [40].

5.1.7.4.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. Raised channelization with sloping kerbs is recommended over channelization accomplished through the use of pavement markings alone (flush) for left- and right-turn lane treatments at intersections on all roadways with operating speeds of less than 20 km/h. (AASHTO 2009. Highway Safety Manual).

ii. Raised islands should be semi-mountable kerbs. Barrier kerbs and other profiles are not favoured for use on islands.

iii. Island noses should be offset from the edge of the adjacent traffic lane to provide additional clearance to the kerb to enhance comfort for approaching drivers and prevent any tendency for them to shy away from the kerb.

iv. As a general guide, the island nose should be offset as specified in **Table 8.16**. On narrow islands where an offset to the approach nose is not practicable, a fully mountable nose may be provided, which requires a smaller offset and nose radius than a kerb. However, where this cannot be achieved because of limited visibility to intersections that are located on crests or relatively tight curves, raised median islands in the major road can be used to improve driver perception of the intersection. In such cases the island nose should be designed to a length that carries it over the crest or around the curve to a point where it can be easily seen (Refer to **Section 5.1.6.9** on Median).

v. Kerbed islands are sometimes difficult to see at night because of the glare from oncoming headlights or from distant luminaires or roadside businesses. Kerbed islands generally should not be used in rural areas and at isolated locations unless the intersection is lighted and kerbs are delineated, such as with kerb-top reflectors.

vi. Channelization at lower cost is the placement of painted islands/medians to narrow the lanes and reduce approach speeds. This is supplemented by rumble strips within this median and along the outside of the edge lines of the pavement (Refer to **Section 5.1.4.4** on traffic calming).

vii. An auxiliary lane should be of sufficient width (including that of shoulders adjacent to auxiliary lanes) and length to enable a driver to manoeuvre a vehicle into it properly, and once in it, to reduce speed for turning at the intersection.

viii. The storage length should be sufficient to avoid turning vehicles stopping in the through lanes waiting for a signal change or for a gap in the opposing traffic flow. A longer lane should be considered in situations where there is a high volume of trucks turning, a grade, or a high design speed. The inability of turning vehicles to access turn lanes can adversely affect the capacity of an intersection and result in vehicles encroaching onto medians and causing maintenance issues.

ix. However, the taper length should not be too long to ensure that the commencement of the auxiliary lane is well-defined, and drivers do not inadvertently enter the lane during inclement weather or on a horizontal curve.

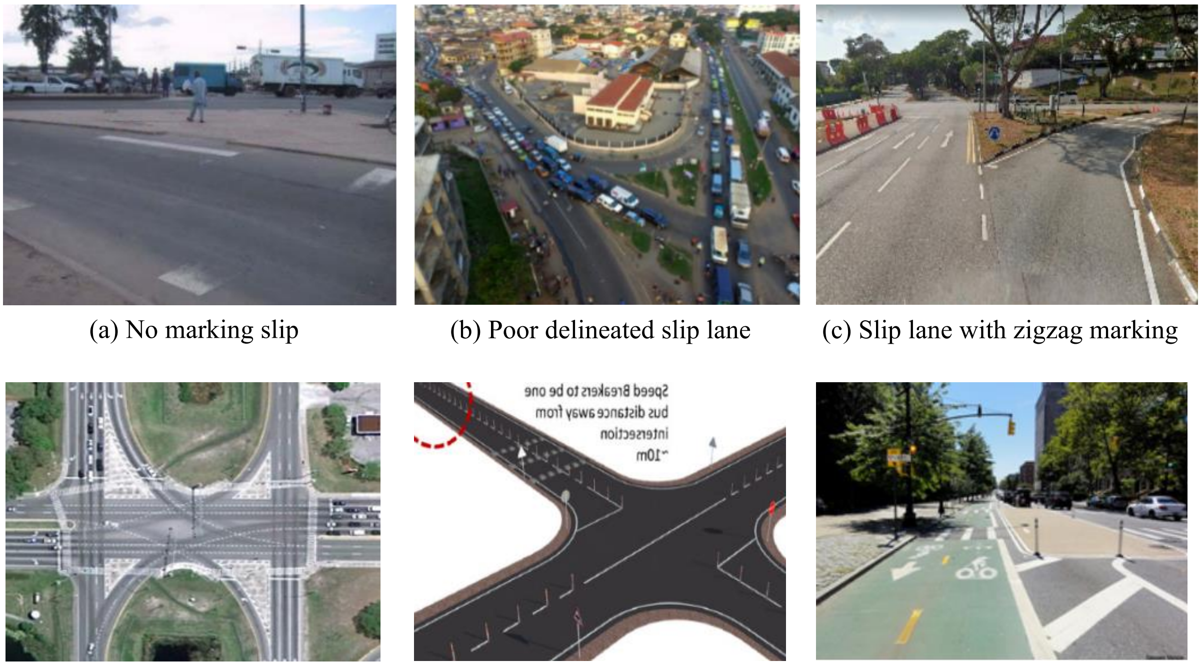
x. The design should allow for an occasional large truck to turn by swinging wide and encroaching on other traffic lanes without disrupting traffic significantly.

xi. Where kerbing is to be used adjacent to the auxiliary lane, an appropriate kerb offset should be provided to be able to accommodate vehicles.

xii. Parking should be restricted for a distance in advance of the right, nearside turning radius to avoid encroachment on adjacent spaces of the turning lanes.

xiii. For arterial road design, adequate radii for vehicle operation should be balanced against the needs of pedestrians and the difficulty of acquiring additional right-of-way or corner setbacks. Because the corner radius is often a compromise, its effect on both pedestrians and vehicular movements should be examined.

xiv. **Plate 5.53** show some good and bad examples of delineation for turning movements.



(a) No marking slip

(b) Poor delineated slip lane

(c) Slip lane with zigzag marking

(d) Large urban intersection with pavement marking delineation for turning movements

(e) Minor road treatment with flexible poles

(f) Pedestrian refuge and cyclist way finding

Plate 5.53 Good and bad examples of delineation for turning movements

For vulnerable road user safety:

i. Install a raised island of adequate size to provide refuge where pedestrian crossings are expected. Islands used for channelization should not interfere with or obstruct cycle lanes at intersections.

ii. Drivers should not be suddenly confronted with an unusable area in the normal vehicle path. Islands first approached by traffic should be indicated by a gradually widening and marking or a rumble strip on each side.

iii. Place the crosswalk in the centre of the turning roadway (further away from the intersection corner) perpendicular to the direction of travel (without making it an inconvenient detour for pedestrians), and use landscaping, etc., to prevent pedestrians from crossing elsewhere. In addition, the crosswalk and kerb ramp should be kept a distance equivalent to one or two car lengths (i.e., usually 6m or 12m) back from the holding line so that the crossing is coincident with a space between queued cars, which will allow drivers on the approach leg to look for and yield to pedestrians before reaching the intersecting roadway and scanning for gaps in traffic.

iv. Adequate stopping sight distance should be provided to pedestrians, particularly to crossings of slip lanes where speeds are higher than locations with smaller corner radii. Other situations where special consideration of cyclists and treatments is required to assist access and safety include on approaches where the skew of an intersection necessitates provision of a slip lane on the corner of a roundabout (e.g., marked cycle lanes). The driver may not see (cognitive and physical) cyclists crossing the road the driver wants to turn into (potentially due to driver distraction, or cyclists speed misjudged).

v. Whenever feasible, signal and other utility poles and signs should be placed outside of paved pedestrian walkways and landing areas. Care should be taken to avoid placing these objects in conflict with future pedestrian facilities.

vi. Providing a buffer space whenever walkways are constructed adds separation between pedestrians and the travelled way.

vii. Appropriate cycle treatments, including line marking and signs for drivers using the slip lane to watch for cyclists, may be required adjacent to the island forming slip lanes.

viii. Priority at crossings should be clear for all road users (i.e., whether motorists, pedestrians, or cyclists have priority).

ix. At intersections with channelization, lighting systems should be installed for illuminating islands, diverge and merge locations, turning roadways, and pedestrian crossings.

x. A refuge island for pedestrians at or near a crosswalk or cycle path that aids and protects pedestrians and cyclists who cross the roadway should be provided with slip and turn lanes.

xi. Raised kerb corner islands and centre channelizing or divisional islands can be used as refuge areas. Refuge islands (for pedestrians and cyclists crossing a wide street, for loading or unloading transit riders, or for wheelchair ramps) are used primarily in urban areas.

xii. Where pedestrians and cyclists are expected to cross a slip or turn lane, low vehicle speeds should be encouraged at the crossing point.

xiii. Using physical devices (e.g., road hump or special marked sections) crossing on slip and turn lanes can reduce vehicle speeds and improve visibility of crosswalks as shown in **Plate 5.54**.

Plate 5.54 Raised Crosswalk on Slip Lane

5.1.7.5 RIGHT-IN RIGHT-OUT

Right-in right-out (RIRO) refer to a type of three-way road intersection where turning movements of vehicles are restricted. RIRO is typical when vehicles drive on the right. This is because minor roads usually connect to the outsides of two-way roads. A RIRO permits only right turns. “Right-out” refer to turns from an intersection (or a driveway or parcel) to a main road. They are implemented to prevent the turning manoeuvre across opposing lanes of traffic.

5.1.7.5.1 SAFETY IMPLICATION

i. RIRO configurations generally improve road traffic safety and efficiency by reducing the number of conflict points between vehicles. In particular, they eliminate the high severity risks of turning traffic versus through traffic. Turning movement restrictions are a type of access management strategy used to improve the safety of stop-controlled intersections and driveways. Restricted and prohibited turn movements reduce the number of turning conflict points at intersections, which are generally known to reduce crash risk [41]
ii. According to the literature, 74 percent of driveway crashes involve offside turn manoeuvres where emerging vehicles have to cross opposing traffic lanes [42]
iii. RIRO are only effective where this turning manoeuvre is effectively prevented, usually by a physical barrier or raised island. Legal restrictions on turning makeovers (those without physical restrictions) are much less effective and open to abuse. Therefore, they are most common where there is a divided carriageway and no median crossing.
iv. There is some evidence that RIRO without physically preventing left turn movements can result in higher crash rates than those with a physical prohibition [43]
v. A RIRO configuration may improve safety and operations at one intersection while consequently worsening them at another intersection upstream or downstream.
vi. Crash migration is a potential issue related to restricted turning movements at a given access point. This occurs when crashes at a treated site shift to another site. While RIRO operations eliminate turns across opposing flows at the subject location, U-turn movements and related crashes potentially increase at the next intersection downstream that allows U-turns.
vii. They also introduce additional collision patterns as vehicles attempt to cross the main running lanes and merge with traffic in opposing directions (**Plate 5.55**). As such, at a full movement signalized intersection within a corridor, there could be an increase in U-turn movements from both directions along the main line if the stop-controlled intersections are converted to RIRO along the corridor.

Plate 5.55 RIRO junction with too close right turn in Ukraine

viii. RIRO junction with offset right turn too close and insufficient weaving length between movements (also incorporates pedestrian crossing and public transport stop). Turning and storage needs to accommodate all vehicles, including heavy goods vehicles (HGVs).

5.1.7.5.2 BEST DESIGN PRACTICE/TREATMENTS/SOLUTIONS

i. RIRO intersections should be designed with a physical median in the mouth of the junction that is effective in preventing an unauthorized turn.

ii. Single lane approaches are most effective in preventing this unauthorized turn.

iii. Where the main road is a single carriageway, a physical barrier is also needed on the main highway.

iv. Where the main road is a dual carriageway, the intersection should be designed as though the deceleration lane were an off-ramp and the acceleration lane an on-ramp, with physical separation between the decelerating/accelerating traffic emphasizing the intersection and converting the turn movement into a merge.

v. The ability to undertake the prohibited manoeuvre at the RIRO intersections needs to be possible at the next available intersections. As these are effective U-turns into the offside of the opposing traffic stream, they must be made under controlled conditions with higher quality turning facilities (either a signal control or at a roundabout as shown in **Figure 5.14**).

vi. The use of an offset median crossing merely transfers the merge problem to another location.

vii. Wherever this turn is allowed, it should be at a sufficient distance from the RIRO to allow emerging traffic to safely cross the main traffic lanes and allow approaching vehicles to anticipate vehicles slowing for the offside turn.

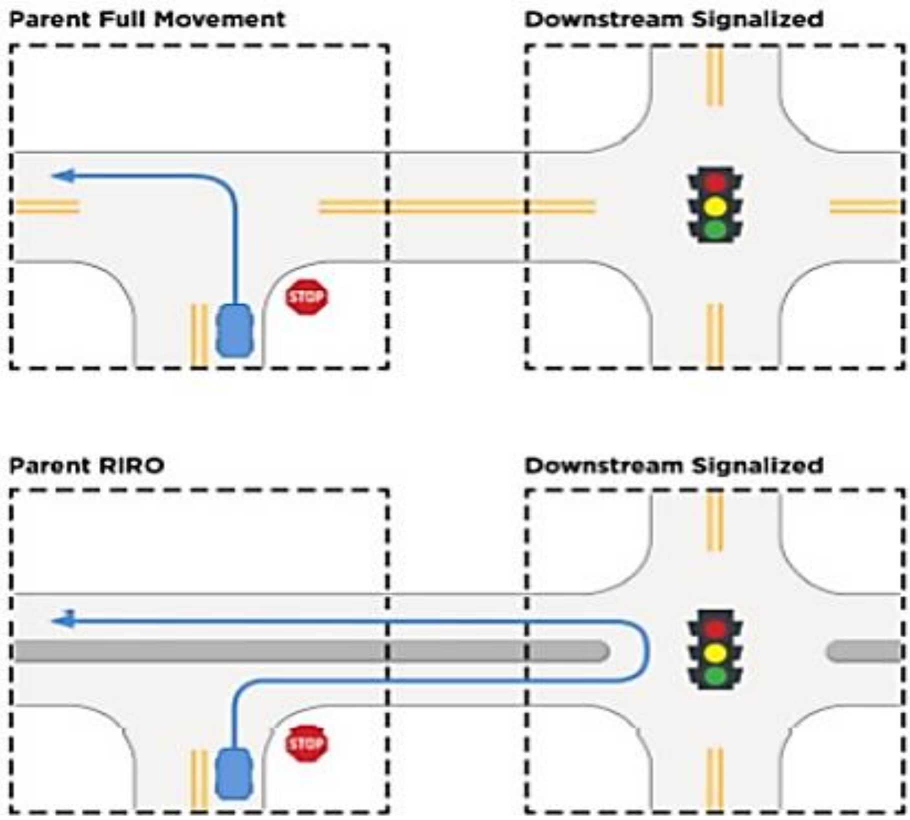


Figure 5.14 Illustration of replacing turning movement at downstream junction

5.1.7.6 ACCELERATION AND DECELERATION LANES

Acceleration/deceleration lanes (also known as speed change lanes) provide drivers with an opportunity to speed up or slow down in a space not used by high speed through traffic (**Figure 5.15**).

Merging can occur at on-ramps to expressways/motorways or multilane highways, or when two significant facilities join to form a single traffic stream. Merging vehicles often make lane changes to align themselves in lanes appropriate to their desired movement.

Diverging occurs when one traffic stream separates to form two separate traffic streams. This occurs at off-ramps from expressways/motorways and multilane highways, but can also occur when a major facility splits to form two separate facilities. Again, diverging vehicles must properly align themselves in appropriate lanes, thus indicating lane changing; non-diverging vehicles also make lane changes to avoid the turbulence created by diverge manoeuvres.

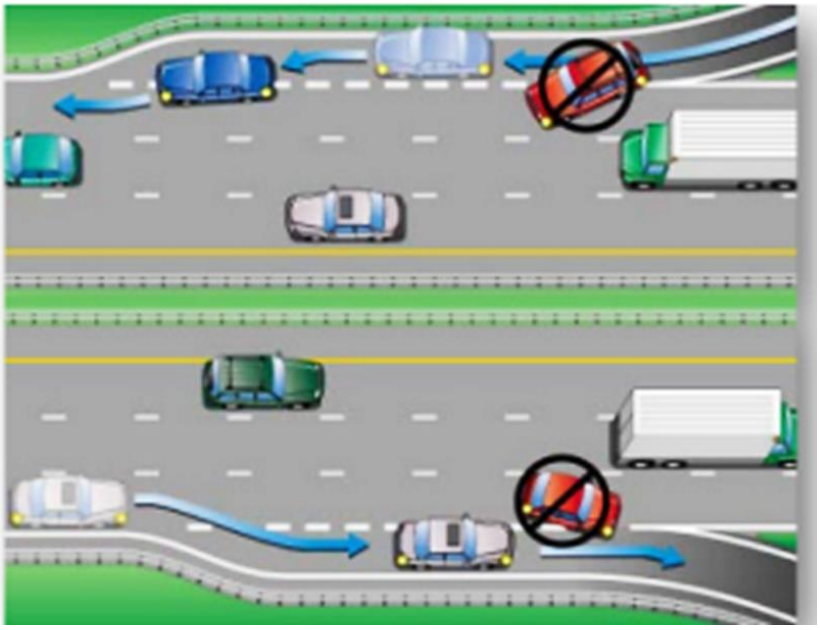


Figure 5.15 Illustration of acceleration and deceleration lanes

5.1.7.6.1 SAFETY IMPLICATIONS

i. Acceleration and deceleration lanes may be blocked by parked or stopped vehicles.
ii. Drivers using acceleration lanes have a narrow angle of vision with the main road flow.
iii. Drivers merging in a stream of vehicles may have difficulty in watching both the vehicles on the mainstream and those that are merging.
iv. Those wishing to leave the multilane highway into a speed change lane need ample warning to move safely into the nearside lane in sufficient time to enter at the start of the lane.
v. Congestion, if the number of vehicles goes beyond the capacity, can increase collisions as vehicles slow or stop unexpectedly.
vi. If lanes do not have sufficient capacity for all vehicles, then queues can back onto the main carriageway causing additional rear-end collisions.
vii. Where speed change lanes are included on multilane highways, the lane changing of vehicles within the main streams can reduce free flow capacity.
viii. Late entry and early exit from a speed change lane can increase the risk of collisions.

The spacing between merge and diverge speed change lanes can result in disruption to main line flow and result in excessive sideswipe and rear-end collisions.

5.1.7.6.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

i. Good visibility should be maintained for both emerging and approaching traffic.
ii. Clear signing and marking of lanes are crucial to safety.
iii. Visibility in the night can be enhanced by using reflective road studs of different colours.
iv. In the case of a perpendicular approach to merging lanes, the line of sight should be kept

free from road furniture, barriers, and road signs.

v. To avoid obstruction on the lanes, parking restrictions should be implemented and strictly followed.

vi. Speed change lanes should be kept free in case of congestion. Therefore, the capacity of the main road and volume of merging traffic need to be calculated to allow free flow conditions under all circumstances. When queues develop, the effective length of the lane is reduced.

vii. Similarly, the upstream capacity of the main road is a major consideration when large amounts of traffic need to use the deceleration lane off a multilane highway and relative speeds and lane changing will be an issue.

Length of the lanes should be long enough to accommodate all the traffic if traffic volume is very high on the mainstream. Where intersections with speed change lanes are close together, sufficient weaving length is needed to maintain stable flow conditions between intersections.

5.1.7.7 GRADE SEPARATION AND RAMPS

Most crashes happen at intersections. The best way of stopping conflicting intersection movements is placing the intersecting roads at different levels, or grade separating them. This can be done with overpasses or interchanges.

An overpass is a simple grade separation of two roads whereby there is no actual link between them and hence no exchange of traffic is possible (**Plate 5.56**). Overpasses are typically used when a minor road crosses a major road, and where a rail line crosses a road.

Interchanges are grade-separated intersections where traffic from one main road is connected to another main road via free flow connecting roads. An interchange allows traffic to move between two or more roads that are grade separated. Interchanges vary from simple arrangements with ramps and intersections at the minor road to complex layouts where two or more expressways/motorways (major roads) connect. Refer to **Chapter 9** for further information.

Plate 5.56 A sample overpass with no connection between two routes

5.1.7.7.1 SAFETY IMPLICATIONS

i. Studies have shown that the crash rate is lower at grade-separated intersections than at at-grade intersections [44]. The largest differences have been found in four-way intersections. At these, the reduction of the number of injury crashes is larger than the reduction of the number of property-damage-only crashes.

ii. The crashes around grade-separated intersections include crashes on ramps, but not crashes on comparable stretches of road immediately before and after at-grade intersections. If these crashes were included in the calculation of the effects on crashes, still larger reductions of the number of crashes on grade-separated intersections would probably have been found. However, ramps are a new road element when grade-separated intersections are constructed, and their effects on safety should be included in the effects of grade-separated intersections.

iii. Partly grade-separated intersections have been found to be less safe than grade-separated intersections, but safer than at-grade four-way intersections. When at-grade four-way intersections are equipped with speed cameras, these are safer than partly grade-separated intersections without speed cameras. No significant difference has been found between partly grade-separated and signalized intersections.

iv. Diamond interchanges (simple and comprehensive, with straight ramps, and with minor roads running above the main road) appear to be the safest form of grade-separated interchanges.

v. Diamond interchanges have lower crash rates than most other types of interchanges. Most differences are only small and not significant. Diamond interchanges are most favourable in comparison with trumpet interchanges and intersections with direct access ramps. There are several factors that make diamond interchanges relatively safe:

- The layout is relatively simple and thereby reduces confusion or errors among drivers.
- Ramps in diamond interchanges are straight, and crash rates are smaller on straight ramps than on curved ramps or loops.

vi. The studies have found that there are more crashes in curves with a smaller radius than in curves with a larger radius [45].

vii. It is possible that the higher speeds on expressway/motorways on the approach to loops may be a contributory factor to crashes, particularly on diverge loops.

viii. HGVs are particularly susceptible to rollover incidents on curved ramps or loops due to the tight radius and potential for high speed.

ix. Short or frequent spacing between intersections can result in short weaving lengths between associated merge/diverge/speed change lanes.

5.1.7.7.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

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xviii. Short or frequent spacing between intersections can result in short weaving lengths between associated merge/diverge/speed change lanes.

5.1.7.8 RAIL CROSSINGS

Rail networks are defined corridors where vehicles move on defined and immovable rails. They are commonly situated in dedicated corridors with only limited and controlled interaction with other forms of land transport (cars, vans, motorcycles, cycles, and pedestrians) on the road network.

Rail crossings are intersections where a road crosses a rail track at-grade and are the physical intersection of two very different vehicle-carrying surfaces and areas approaching the physical intersection. Within the crossing area, physical design characteristics of each structure, i.e., rail and road, may have to be specifically adjusted to accommodate the other transportation mode smoothly and safely.

Some international rules have helped to harmonize level crossings, for instance, the 1968 Vienna Convention which requires standard warning signs and lines, and potential barriers. Early crossings had a flagman in a nearby booth who would, on the approach of a train, wave a red flag or lantern to stop all traffic and clear the tracks.

5.1.7.8.1 SAFETY IMPLICATIONS

i. Level crossings constitute a significant safety concern internationally. On average, each year around 400 people in the European Union and over 300 in the United States are killed in level crossing crashes. According to the Ghana Railway Development Authority, three (3) train crashes with vehicles have occurred at level crossings in Ghana between 2018 and 2022.

ii. Collisions can occur with vehicles as well as pedestrians; pedestrian collisions are more likely to result in a fatality [46].

iii. Rail crossings can be dangerous if:

   a. There is poor sight distance to a signal display, or to approaching trains.
   b. Traffic control is inadequate.
   c. Vehicles queue across tracks due to congestion or nearby intersections.
   d. There is a lack of pedestrian facilities.

e. Either road or rail pavement is not maintained.
f. Signalling equipment is located too close to the road that can result in unnecessary damage by passing vehicles.
g. Vertical profile of road over rail crossing results in grounding of road vehicles.

iv. Signalized intersections at or near grade crossings possess added concerns over intersections that are stop controlled. If traffic signals are not properly coordinated with railroad operations, severe crashes can occur.
v. When a rail crossing is located near a signalized intersection, it is possible that queues from the intersection could extend over the grade crossing and potentially cause stopped vehicles to become trapped on the tracks.
vi. Similar situations can occur at uncontrolled intersections close to rail crossings where long vehicles can block the crossing.
vii. When a long-wheelbase or low-ground-clearance vehicle negotiates a roadway having a high vertical profile, such as a roadway-railway grade crossing, roadway crown, or driveway entrance, the vehicle may become lodged or stuck on the "hump." A somewhat common occurrence is one in which a railway is on an embankment and a low-ground-clearance vehicle on the crossing roadway becomes lodged on the track and is subsequently struck by a train.

5.1.7.8.2 BEST DESIGN PRACTICE/ TREATMENTS/SOLUTIONS

i. Trains have a much larger mass relative to their braking capability, and thus a far longer braking distance than road vehicles. With rare exceptions, trains do not stop at level crossings and rely on vehicles and pedestrians to clear the tracks in advance.
ii. Level crossings as shown in **Plate 5.57** are controlled through either passive or active systems. Passive control systems provide warnings through signs and line markings. They do not react to the presence of an approaching train. Active traffic control systems warn road users of approaching trains.

Plate 5.57 Level rail crossings

iii. Adequacy of sight distance is critical at passive crossings; however, even where active devices are present or will be provided, sight distance is beneficial to confirm the ability to cross the tracks.
iv. Warnings at active controlled crossings consist of flashing lights and sounds (combined with static controls such as signs and pavement markings) which are triggered by a train.
v. As with passive crossings, adequate visibility of these devices is necessary for approaching road users.
vi. Another level of active control is achieved by placing a barrier between vehicles or pedestrians and trains. This is done with electro-mechanical devices such as pedestrian gates and vehicle boom barriers used in combination with other active and passive controls.
vii. Intersections near roadway-railway grade crossings require special attention to coordinate the movements of vehicle, train, and pedestrian traffic.
viii. To avoid queues from an intersection blocking a crossing, traffic signals located near roadway-railway grade crossings need to be synchronized when trains approach in order to clear vehicles off the tracks before the train arrives. This synchronization is normally achieved through an electrical interconnection circuit between the railway grade crossing warning system and the roadway traffic signal controller assembly. The geometric design of any signalized intersection near a roadway-railway grade crossing should consider interconnection and synchronization.
ix. Sufficient space is needed to ensure that waiting vehicles can wait safely to clear a crossing.
x. Approach to rail crossings therefore needs to be as flat or straight as possible to allow clearance for long wheelbase vehicles.

5.1.8 DESIGN TOOLS FOR SAFE OUTCOMES

5.1.8.1 INTRODUCTION

Existing road design guides are generally technically sound and are essential for the design process, but they will not enable designers to achieve safety outcomes on their own. Even designing strictly to existing guides will result in designs that allow death and serious injury. It is therefore very important that additional tools and processes be used to ensure that road safety objectives are met through the life cycle of a road or network. In response to knowledge on this issue, various approaches have been developed over time to help ensure safety is adequately considered throughout the life cycle of a road.

The comprehensive approach to the safe design, operation, maintenance, and use of roads is generally referred to as Road Safety Infrastructure Management. The objective of road safety management is to integrate all road safety activities throughout the design and operation of an individual road or network such that a systematic approach is taken to reducing death and serious injury. Examples of safety techniques used at each stage of the road life cycle are provided in **Figure 5.16**.

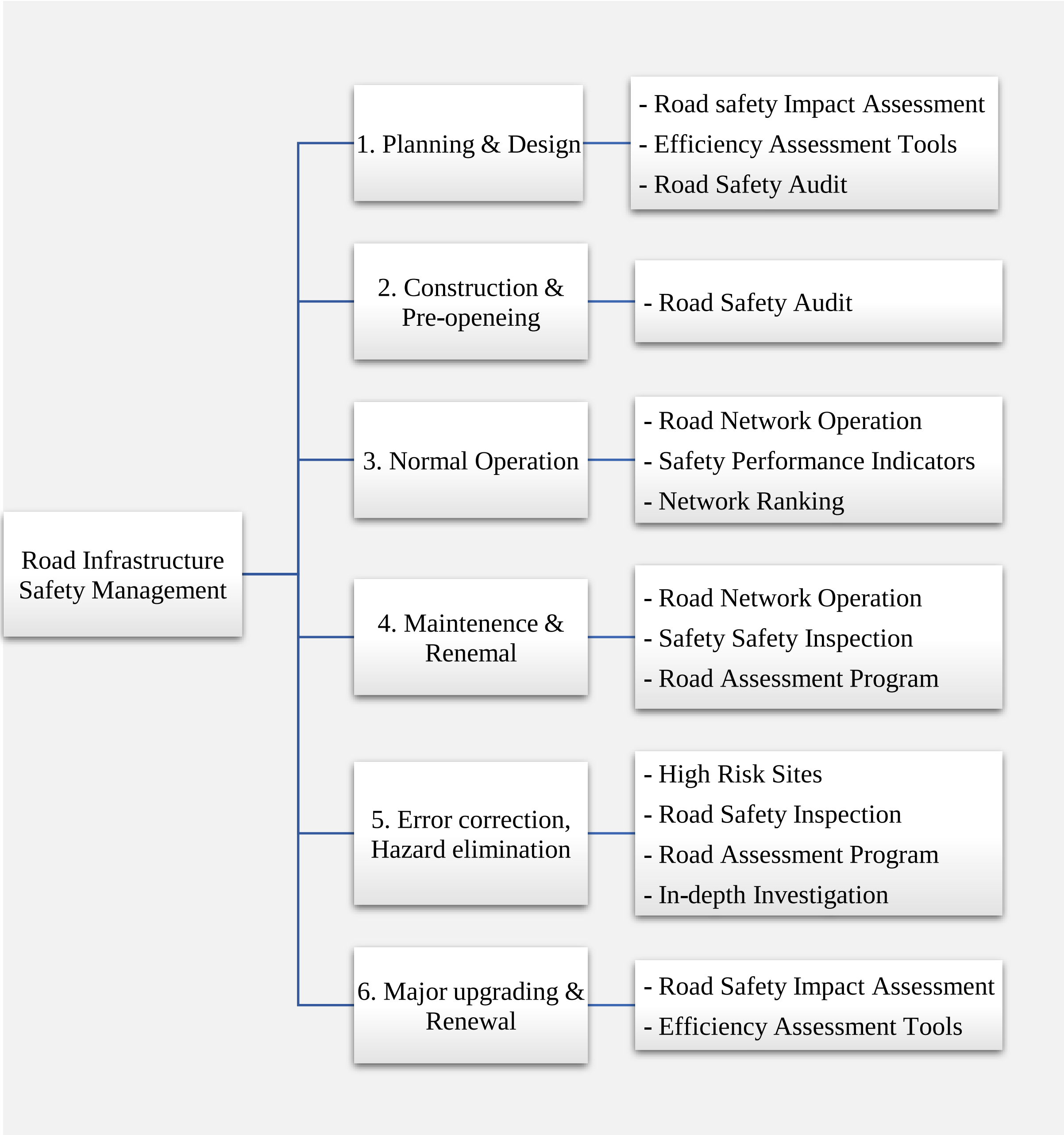


Figure 5.16 Road Safety Techniques for different stages of the road life cycle

The strategies implemented in road traffic safety management can include both reactive and proactive approaches.

i. A reactive approach to road safety is associated with the following:
   a. Identification of locations experiencing safety problems (screening).
   b. Problem definition (diagnosis).
   c. The identification and implementation of countermeasures (cure) from a detailed examination of crash data.

ii. Road safety improvements are proposed in response to identified safety problems brought to light by crashes that have occurred after the road has been designed, built, and opened to the travelling public.

iii. A proactive approach to road safety is associated with the prevention of safety problems before they manifest themselves in the form of a pattern of crashes focusing on what is known about the impact of different situations, road features, and treatments on road safety injury or crash outcomes. The proactive approach applies this knowledge to the roadway design elements or to improvement plans on existing roads to diminish the likelihood and severity of crashes.

iv. The reactive and proactive approaches are often used in combination with the emphasis shifting from one to the other, depending on the maturity of the overall safety management processes in the Road Agency, or even in different road environments. As an example, for a rural route with high numbers of run-off-the-road crashes, it is desirable that all potential high severity locations be treated, regardless of whether crashes have happened there yet or not. This is in contrast to a crash-based analysis that addresses just those points on the road where crashes have previously occurred. Equally risky locations (in terms of road and roadside features) should not be ignored.

v. Whichever approach is used, it is necessary to identify safety deficiencies that need to be investigated to diagnose safety problems, and then identify and implement countermeasures or design improvements to remedy the deficiencies before they create serious harm to road users.

vi. In order for these deficiencies to be addressed effectively through the design process, there must be some form of performance measure of the effectiveness of the design to achieve safety outcomes, in the same way as Key Performance Indicators (KPIs) are applied to other design aspects.

5.1.8.2 ROAD INFRASTRUCTURE SAFETY PERFORMANCE INDICATORS

Road Infrastructure Safety Performance Indicators (SPIs) are any measurement that is causally related to crashes or injuries and can be used in addition to the figures of crashes or injuries in order to indicate safety performance or understand the process that leads to crashes. Road Infrastructure SPIs aim to assess the safety hazards by infrastructure layout and design (e.g., percentage of road network not satisfying safety design standards).

The inclusion of performance indicators is common practice within major infrastructure projects. They are quantifiable measures of performance over time and provide targets for teams to aim for, milestones to gauge progress, and insights to help organizations make better decisions.

“What gets measured gets done.”

Managing with the use of performance indicators includes setting targets (the desired level of performance) and tracking progress against that target. Historically, specific performance indicators to reflect safety outcomes have seldom been applied in road design. At best, phrases such as “improved safety outcomes” are used when defining project objectives, but these are not measured in any tangible way. With the development of better assessment techniques for safety in design, and even the quantification of safety impacts from design decisions, there is now the ability to better specify safety outcomes in design.

5.1.8.3 INFRASTRUCTURE TOOLS AND TECHNIQUES

This section contains introductory information on some of the road safety infrastructure tools and techniques that can be used to assess risks and identify solutions. In general, the earlier such tools can be applied in the design process, the better. Early changes to design are likely to be more feasible and will generally be at lower cost. The tools included are those most likely to be used by road designers. Examples of the most common tools are included as well as information on some promising emerging tools. These are presented in the same order as they appear in **Figure 5.16**.

5.1.8.3.1 ROAD SAFETY IMPACT ASSESSMENT

Being able to explicitly estimate the impact on road safety that results from building new roads or making substantial modifications to the existing road infrastructure that alter the capacity of the road network in a certain geographic area is of crucial importance if road safety is not to unintentionally suffer from such changes. The same applies to other schemes and developments that have substantial effects on the pattern of road traffic. The procedure that has been designed for this purpose is known as Road Safety Impact Assessment (RSIA). This procedure is intended to be applied at the planning stage, often proceeding to a definite design for the scheme. A safety impact assessment thus precedes and complements the eventual safety audit of any specific design for the scheme.

A scenario method is used to carry out an RSIA. The starting point is the existing road network, the current pattern of traffic on that network, and the level of reported road injury collisions within the area. Current traffic patterns include usage for all users - motorized and non-motorized - although non-motorized travel data are notoriously difficult to access at the same level as motorized data. It is helpful, though not essential, to have all this information represented in a digital form within a Geographic Information System (GIS).

The information needed relates to a road network which is made up of roads of several types that have different road safety characteristics. Each road consists of intersections and stretches of road between the intersections, with associated traffic volumes for each user group, and the number of collisions and casualties.

Alternative scenarios to this current situation are the possible changes being studied in respect to the physical infrastructure and the associated traffic volumes in the road network in the future.

If, for example, a new road is to be added to the existing network, the traffic and transport models can be used to estimate what this will mean for the traffic volumes throughout the network in the future.

The central step is to interpret these changes in terms of the impacts they will have on the numbers of crashes and casualties. To accomplish this, what are needed are quantitative indicators of risk (such as casualty rates per million vehicle-km) for each type of road and user, supplemented, if possible, by corresponding indicators for each main type of intersections. One way of obtaining such indicators is to estimate them at a national level and adjust them if necessary, using data for the area in question. In addition, thought should be given to any expected changes over time in the level of risk for each type of road or junction. These kinds of information enable safety impacts to be estimated for the duration of the road's life cycle.

If the various data are accessible from a computer, calculations of safety impacts for a range of scenarios and comparisons between impacts of different scenarios can be made quite readily. The procedure can be adapted to help identify what changes are needed in a given scenario in order to bring the safety impact within some target range.

When implementing this scenario technique, it is important to bear in mind the quality of the information being used. It is also important for the information to be accessible in such a way that calculations for a range of scenarios can be elaborated at relatively modest costs within a short period of time. For this purpose, the traffic and transport models should be set up in such a way that an RSIA module to apply the relevant indicators of risk for future years can be readily linked with them.

Adopting this sort of methodology of risk assessment allows consideration to be made of future land use changes and the potential for the land use changes to encroach on the road corridor and change the consequent function and safety risk. Additionally, it allows for opportunities to influence road user behaviour by introducing cycle ways and walkways to encourage sustainable travel modes.

5.1.8.3.2 ROAD SAFETY AUDIT

Road safety audits have been applied to road design projects for several decades and are a well-established approach in developed countries as well as developing countries.

The Road Safety Audit (RSA) process involves independent teams of experts assessing designs at one or more stages of project development as shown in **Figure 5.17** through a formal process to identify safety-related risks.

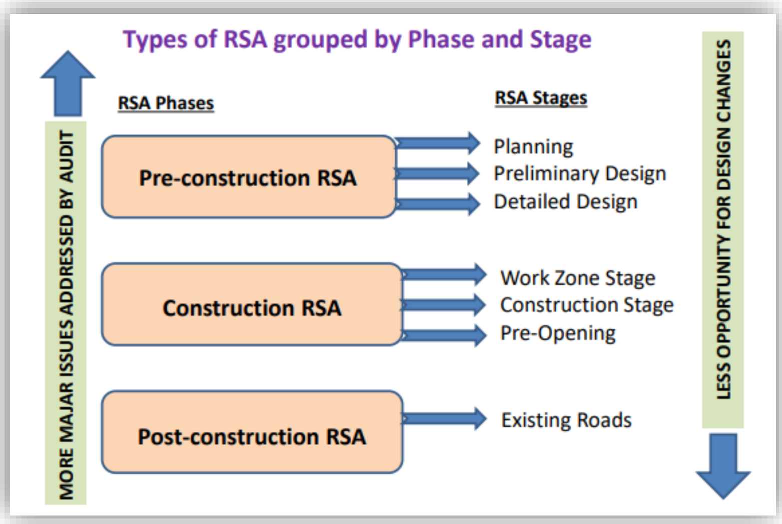


Figure 5.17 Road Safety Audit Phases and Stages

The sooner that an audit is undertaken in the design process, the greater and easier the safety benefits are to achieve. Road safety audits are not a check of compliance against design standards (as identified in **Section 1.2**, compliance with such standards does not guarantee a safe design). These teams of experts review designs and make assessments on safety impacts for all road users based on experience, road safety engineering, among others. Road safety issues are documented, and a priority for addressing these issues is typically given. It is expected that designers would address each of these issues wherever practicable.

Guidance documents on how to conduct audits include Ghana Highway Authority Manual of Road Safety Audit and Department of Urban Roads' Procedures for Road Safety Audit. The process outlined in each of these documents is broadly similar. Although road safety audits can lead to substantial road safety benefits, this will only occur if audits are conducted correctly by an experienced team.

5.1.8.3.3 OBJECTIVES OF ROAD SAFETY AUDIT

A. Stage 1: Feasibility/Preliminary Design Stage RSA

RSA Stage 1 identifies the potential safety problems that can influence the:

i. Scope of project
ii. Route choice
iii. Design standard
iv. Impact on existing road network
v. Route continuity
vi. Provision of interchanges/intersections

vii. Access control
viii. Number of lanes
ix. Route terminals
x. Stage development etc.

B. Stage 2: Draft Design Stage RSA

It identifies the potential safety problems that can influence the:

i. Horizontal and vertical alignment
ii. Sightlines distances
iii. Location of accesses
iv. Intersection layout
v. lane and shoulder width
vi. Pavement crossfall and super elevation
vii. Overtaking lanes
viii. Bus bays/lay bys
ix. Provision for pedestrians and cyclists etc.

C. Stage 3: Detail Design Stage RSA

RSA Stage 3 identifies the potential safety problems that can influence the:

i. Correct type and placement of signs and line markings
ii. Delineation
iii. Lighting
iv. Intersection details
v. Vulnerable Road Users (Pedestrian crossing facilities)
vi. Drainage
vii. Poles and other roadside objects
viii. Landscaping
ix. Guard fencing
x. Signals
xi. Landscaping etc.

D. Construction work zone (CWZ) traffic management audit

The following are the reasons for conducting a CWZ traffic management audit:

i. Road works sites typically involve a change in speed environment, additional conflicts and confined road space, which can increase the potential for crashes

ii. Traffic arrangements during road works can change several times and can bear very little resemblance to permanent arrangements. Audits at design stage can give little indication of the safety of the temporary works
iii. Construction contractors may not appreciate the finer points of traffic management, roadside safety and the operation of safety devices (especially from the viewpoint of the road user, rather than from that of the construction team)
iv. To evaluate that standard arrangements are applied, for consistency and for adequacy under those particular conditions
v. To avoid conflicting messages between permanent and temporary devices and between traffic signs, markings and other devices
vi. To review the appropriate use of signs and guidance to the road user
vii. To evaluate the safety of the access to the construction site from the “public” road network and locations where construction traffic is in conflict with the general travelling public.

E. Stage 4: Pre-Opening RSA

The following are the reasons for conducting a Stage 4 RSA:

i. To check for issues that may have been missed through the design process
ii. To check for issues resulting from poor or incorrect construction
iii. To check for issues resulting from genuine mistakes
iv. To ensure that safety needs of all road users are adequate
v. To determine if hazardous conditions exist which were not evident in the previous audits
vi. It is the last chance to rectify any problems before exposing them to the travelling public

F. Stage 5: Pre-Opening RSA

The following are the reasons for conducting a Stage 5 RSA:

i. To check that issues raised in the pre-opening audit have been satisfactorily addressed
ii. To ensure compatibility between the safety features of a road and the functional classification of the road
iii. To identify any feature that can, with time, create a safety problem – for example vegetation blocking a sign
iv. To identify all features in the road environment that pose a safety hazard to any of the road users.

v. Often not all road features are complete at the time of the pre-opening audit, hence it is also an opportunity to check the completed road

vi. To check that traffic is coping adequately with the new road conditions

5.1.8.3.4 ROAD SAFETY INSPECTION

A Road Safety Inspection (RSI) is a systematic review of the safety provision on an existing road, in particular in relation to the provision and hazards associated with traffic signs, roadside features, environmental risk factors, and road surface condition. RSIs are based on similar approaches as Road Safety Audits (RSAs). The key difference between RSIs and RSAs is that RSAs are carried out on new or rehabilitation schemes where design teams are in place and RSIs are undertaken on existing roads where no proposals are yet in place for improvement. An RSI is a proactive approach that involves a systematic review of an existing road by driving and walking to identify hazardous conditions, faults, and deficiencies in the road environment that may lead to road user injury and to develop countermeasures for treating potential accident sites. Guidance documents on how to conduct RSI (Road Safety Audit – Stage 5) include Ghana Highway Authority Manual of Road Safety Audit and Department of Urban Roads' Procedures for Road Safety Audit.

5.1.8.3.5 HIGH-RISK SITES (BLACK SPOT MANAGEMENT)

Road accidents happen in many forms and in many locations and it is not feasible nor useful to analyse all accidents in detail. The key is to identify locations where accidents are most frequent, as these are potentially worthwhile sites for investigation and treatment. Depending on the quality and details recorded in crash data, several different types of analyses may be undertaken, each with a differing level of granularity.

i. Specific site analysis is undertaken to identify locations across the network where a concentration of crashes has occurred. These are then investigated in detail to understand the nature of the crashes, and a site visit is undertaken. A remedial treatment program is then designed and implemented. Standard Process for Identifying and Treating Accident Site is shown in **Figure 5.18**.

ii. Corridor/route analysis is undertaken to identify stretches of roads that perform badly. These can then be investigated, inspected, and a treatment program developed.

iii. Area analysis is undertaken to understand the types of crashes occurring in an area which may be more widespread than for a single site or route.

iv. Once identification has been undertaken, sites can be prioritized to maximize casualty reduction for the available budget.

v. Guidance documents on Black Spot Management include Ghana Highway Authority Manual on Identifying and Treating Accident Sites, (January, 2002 Edition), among other best practice manuals.

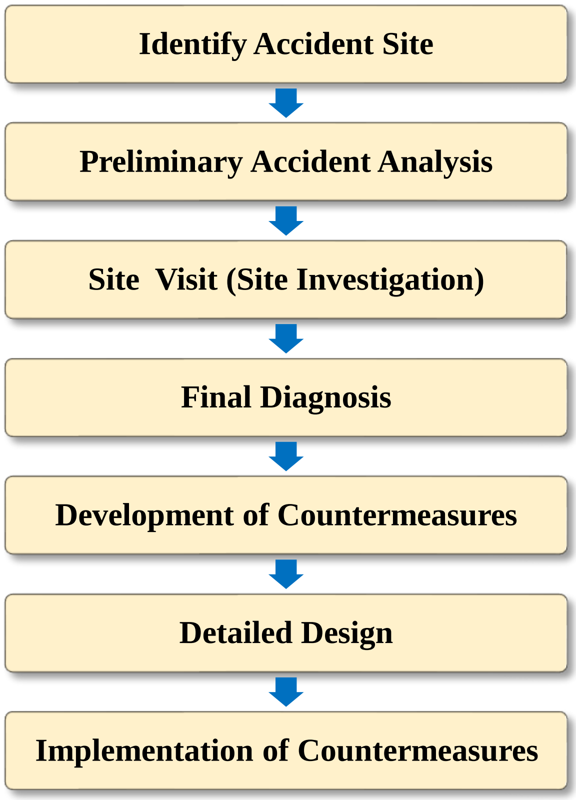


Figure 5.18 Standard Process for Identifying and Treating Accident Site

5.2 ESTIMATION OF DESIGN TRAFFIC VOLUME

The design traffic volume for road planning is determined from the estimated traffic demand in consideration of the area of development, industrial and economic activities, appropriate population spread etc. in the target year, and the daily vehicle traffic volume that is estimated as a result of the relationship with the road network at the target year.

The target year differs according to the character and importance of the planned road; however, it is generally given as 20 years in the future at the time of planning. The target year may also be given as 10 years in the future in the case of low-grade roads, however, when setting the target year, it is desirable to decide it in consideration of the road's character and the envisaged timing of construction.

5.2.1 PROCEDURE FOR ESTIMATION DESIGN TRAFFIC VOLUME

The design traffic volume is generally given as Annual Average Daily Traffic that is expected to pass along the planned road. The road geometry is determined upon considering the design traffic volume and also the heavy vehicle traffic volume, trip length and other traffic characteristics deemed to reflect the road's functions. **Figure 5.19** shows the procedure for estimating design traffic volume.

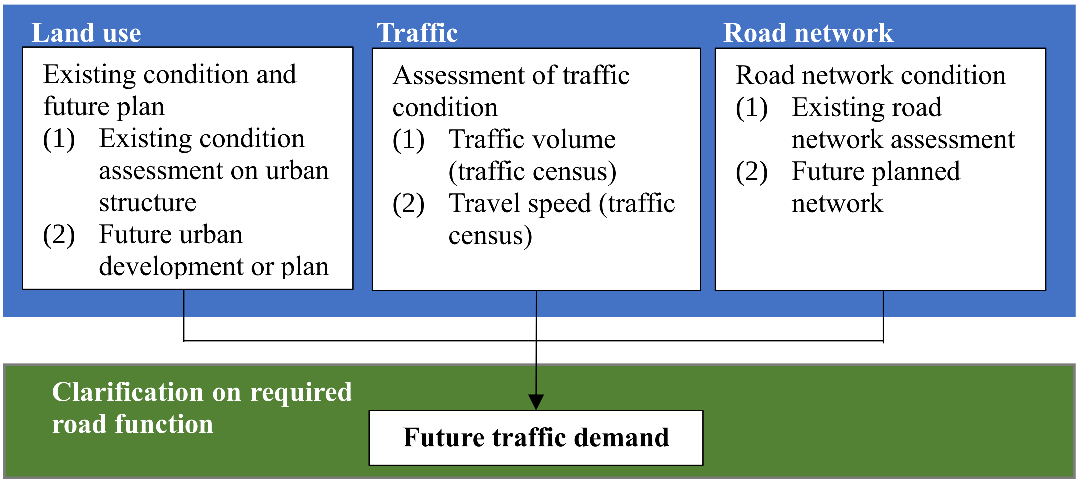


Figure 5.19 Procedure for estimation design traffic volume

5.2.2 ROAD TRAFFIC CHARACTERISTICS

The road geometry is determined according to the design class and design traffic volume. However, since roads have complex functions, there are cases where the design traffic volume alone cannot fully express the road's inherent functions. Accordingly, when deciding a road geometry, it is also necessary to consider the hourly distribution, the directional distribution, vehicle trip lengths, daytime-night time traffic volume ratio, weekly distribution rate, and other road traffic characteristics.

5.3 CALCULATION OF POSSIBLE TRAFFIC CAPACITY

When the road and traffic conditions meet the free flow condition, it is termed the base traffic capacity. This is the traffic capacity for calculating possible traffic capacity. **Figure 5.20** shows the flowchart for determining the possible traffic capacity.

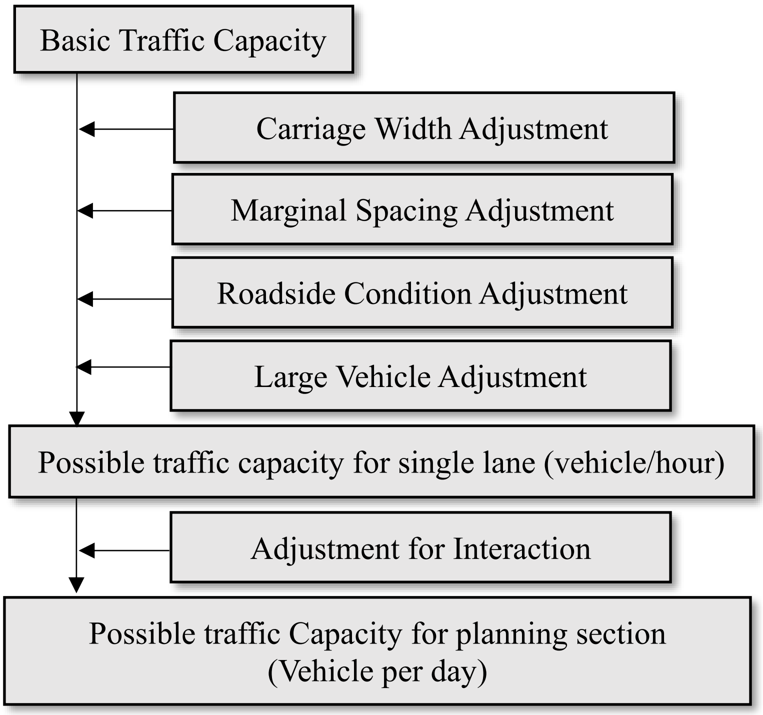


Figure 5.20 Determination of possible traffic capacity

5.3.1 BASE TRAFFIC CAPACITY

The base traffic capacity is the maximum number of passenger car unit (pcu) per hour under the ideal conditions for uninterrupted flow as follows:

i. Traffic flow, free from interference of vehicles and pedestrians from the side
ii. In-stream flow of passenger cars
iii. Traffic lanes of adequate width and shoulders with no lateral obstructions within 1.8m from the edge of the carriageway
iv. Horizontal and vertical alignment suitable for an average speed of 100km/h on multi-lane highways in rural areas.

These ideal conditions are rarely met with on actual roads; hence, the theoretical capacities are never realised. According to HCM US, the theoretical capacity under ideal conditions is about 2200 per lane with free speeds of 100km/h for multi-lane highways. The base traffic capacity is shown in **Table 5.1**.

Table 5.1 Base traffic capacity

Road conditions	Base traffic capacity (pcu/h)
Per lane on a multi-lane road	2,200
Per lane on a single direction lane road (one-way road)	2,200

5.3.2 POSSIBLE TRAFFIC CAPACITY

Possible traffic capacity refers to traffic capacity under actual road conditions and traffic conditions on the target road. This is calculated upon conducting corrections on lane width, marginal spacing, roadside conditions, large vehicles and intersections with traffic signals for the impact of road and traffic conditions on the base traffic capacity.

i. Lane width correction

The traffic capacity is greatly affected by lane width. For this, the correction coefficients in **Table 5.2** are used.

Table 5.2 Lane width correction coefficient

Lane width (m)	Correction coefficient
> 3.50	1.00
3.25	0.94
3.00	0.85
2.75	0.77

ii. Marginal spacing correction

The necessary and ample marginal spacing width in terms of traffic capacity is around 1.75m per side. If the marginal spacing is less than 1.75m, the correction coefficients in **Table 5.3** are used.

Table 5.3 Marginal spacing correction

2-lane road							
Marginal spacing width (m)	1.75	1.50	1.25	1.00	0.75	0.50	0
Insufficient on single side only	1.00	0.98	0.96	0.93	0.91	0.88	0.85
Insufficient on both sides	1.00	0.96	0.92	0.86	0.81	0.75	0.70
Multi-lane road							
Marginal spacing width (m)	1.75	1.50	1.25	1.00	0.75	0.50	0
Insufficient on single side only	1.00	1.00	0.99	0.98	0.97	0.95	0.90
Insufficient on both sides	1.00	0.99	0.98	0.97	0.94	0.90	0.81

iii. Holiday and bottle neck (tunnel sag) correction

Road traffic capacity is known to decline due to holidays and bottlenecks (tunnel sag). Correction coefficients in **Table 5.4** are used as standard values.

Table 5.4 Holiday and bottleneck correction coefficient

Design class	Number of lanes	With bottleneck (Tunnel sag)		Without bottleneck (Tunnel sag)	
		Heavy holiday traffic	Small difference between holidays and weekdays, or there are few holidays	Heavy holiday traffic	Small difference between holidays and weekdays, or there are few holidays
A	1 lane	0.70	0.90	0.90	1.00
	2 lanes	0.75	0.90	0.90	1.00
	≥3 lanes	0.85	0.90	0.90	1.00
B - E	≥1 lane(s)	0.90	1.00	0.90	1.00

The sag conditions that give rise to congestion are as follows:

- Algebraic difference of the vertical grades ≥6%
- Vertical curve radius: 15,000m or more, and vertical curve length: 1,500m or more

iv. Roadside characteristics correction

The roadside characteristics correction is intended to cater for the reduction in traffic capacity that arises from interference of the traffic flow due to entry and exit of vehicles from roadside facilities and intersections with no traffic signals and the parking of vehicles on the road. **Table 5.5** describes the standard roadside characteristics correction coefficients adopted depending on the class of road and conditions of the urban/rural area.

Table 5.5 Roadside correction coefficients

Design class	Urbanisation conditions		
	Non-urbanised	Semi-urbanised	Urbanised
A1/A2	1.0	1.0	1.0
B1, C1, D1, E (rural)	1.0	0.9	0.8
B2, C2, D2, E (urban)	0.9	0.8	0.7

v. Large vehicle correction

Large vehicles have a major impact on the flow of traffic because they occupy a large area of the road and have slower speed on gradient sections. Accordingly, large vehicle correction is conducted using a converted value (passenger car conversion coefficient) indicating how many passenger vehicles one large vehicle is equivalent to and the large vehicle mixing rate.

Basically, the pcu value is obtained by multiplying the vehicle type pcu conversion value with the results of vehicle type-separate traffic volume survey on similar roads. Values of pcu based on the vehicle type is shown in **Table 5.6**.

Table 5.6 Passenger car unit (pcu)

Vehicle	pcu value
Pedestrian	0.15
Bicycle	0.20
Motorcycle	0.25
Auto rickshaw	0.35
Bicycle with trailer	0.35
Motorcycle with trailer	0.45
Animal drawn cart	0.70
Light goods vehicle	1.00
Bullock cart	2.00
Bus	2.00
Medium goods vehicle	2.50
Heavy goods vehicle	3.50
All based on a passenger car = 1.00	

It is desirable to appropriately set traffic capacity correction rate based on traffic volume data on the target road and local area, however, if sufficient data cannot be obtained, the large vehicle correction coefficient and pcu value are sought by using **Equation 5.1.** Large vehicle correction coefficients are shown in **Table 5.7.**

Traffic capacity correction rate:

$$\gamma_T = \frac{100}{100 - [P_T + (E_T \times P_T)]} \tag{5.1}$$

Where,

γ_T: Traffic capacity correction rate (%) based on mixing of large vehicles

P_T: Large vehicle mixing rate (%)

E_T: Passenger car conversion coefficient of large vehicles

Table 5.7 Large vehicle correction coefficient

Design class	Large vehicles mixing rate	Large vehicle pcu conversion coefficient
A1	15%	1.8
A2	10%	1.8
B1, C1	15%	1.7
D1, E (Rural)	15%	1.7
B2, C2, D2, E (Urban)	10%	1.7

5.4 TRAFFIC ANALYSIS – CAPACITY AND LEVEL OF SERVICE

The concepts of capacity analysis and Level of Service (LOS) are fundamental to the planning, design and operation of traffic facilities.

Highway Capacity Manual (HCM) 2016 provides detailed information on capacity analysis and is the primary reference document on this topic.

5.4.1 TYPES OF TRAFFIC FACILITIES

Traffic facilities may be classified into two broad categories:

i. **Uninterrupted flow facilities**, on which traffic flow conditions are the result of interactions between vehicles in the traffic stream, and between vehicles and the geometric and environmental characteristics of the road; there are no fixed elements external to the traffic stream, such as traffic control signals, that cause interruptions to traffic flow; examples include two-lane rural roads, multi-lane rural roads, and motorways.

ii. **Interrupted flow facilities**, on which traffic flow conditions are subject to the influence of fixed elements such as traffic signals, stop signs, give-way signs, roundabouts or other controls which cause traffic to stop periodically, irrespective of the total amount of traffic; examples include urban streets, unsignalised and signalised intersections.

5.4.2 CAPACITY, LEVEL OF SERVICE, AND DEGREE OF SATURATION

5.4.2.1 CAPACITY

Capacity, as defined in HCM 2016, is the maximum sustainable hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under the prevailing roadway, environmental, traffic and control conditions. The concept applies equally to motorised vehicular traffic and to bicycle and pedestrian traffic.

The following points should be noted with respect to the above definition:

i. The time period used in capacity analyses should be one hour, but in practice analysis typically focuses on a 15-minute period (specifically, the peak 15 minutes of the peak hour) which is usually accepted as being the shortest interval during which stable flow exists.
ii. The prevailing roadway, traffic and control conditions should be reasonably uniform for the section of facility being analysed.
iii. Roadway conditions refer to the geometric characteristics of the road, including the type of facility and its development environment, the number of lanes, lane disciplines and lane types in each direction, lane and shoulder widths, lateral clearances, design speed and horizontal and vertical alignments.
iv. Traffic conditions refer to the characteristics of the traffic stream using the road, including vehicle type and the lane and directional distribution of the traffic.
v. Control conditions refer to the types and specific design of the control devices and traffic regulations applicable to the particular section of road.
vi. Driver characteristics play a central role in determining such key parameters as saturation flow rates at signals, and critical gap and follow-up headways at roundabouts and sign-controlled intersections, used in capacity calculations.

Capacity analysis typically focuses on vehicle capacity, in terms of vehicles (or passenger car equivalents) per hour. In comparisons between different transport modes and systems designed to increase vehicle occupancy (e.g. high-occupancy vehicle lanes) it is relevant to also consider the number of persons per hour passing a point (Monash University 2003).

5.4.2.2 LEVEL OF SERVICE

LOS is a qualitative stratification of the performance measure or measures representing quality of service. A LOS definition is used to translate complex numerical performance results into a simple stratification system representative of road users' perceptions of the quality of service provided by a facility or service (HCM 2016). These service measures include speed and travel time, delay, density, freedom to manoeuvre, traffic interruptions, comfort and convenience, and safety. In general, there are six levels of service, designated A to F, with LOS A representing the best operating condition and service quality from the users' perspective (i.e. free-flow) and LOS F the worst (i.e. forced or breakdown flow or having reached a point that most users would consider unsatisfactory, as described by a specific service measure value or a combination of service measure values). **Table 5.8** shows the general operating conditions represented by these levels of service. The specific definitions of level of service differ by facility type. The HCM presents a more thorough discussion of the level-of-service concept.

Table 5.8 General Definitions of Levels of Service

Level of Service	General Operating Conditions
A	Free flow
B	Reasonably free flow
C	Stable flow
D	Approaching unstable flow
E	Unstable flow
F	Forced or breakdown flow

A key issue is determining the LOS that is deemed acceptable, and whether that level should be a projected level for future operation of a facility, or the level existing at the current operation of the facility. The appropriate LOS for a particular jurisdiction will be determined in the context of the policies indicating what are regarded as acceptable levels.

The LOS concept may be used as the basis of capacity and operational analysis for all types of road facilities and for multi-modal road users. While there is a range of parameters that could be used to define LOS for each type of road facility, for practical purposes certain quantitative performance measures have been developed for the different types of facility to assist in defining levels of service. These are summarised in **Table 5.9** (adapted from HCM 2016).

Table 5.9 Performance measures used for defining LOS

Element	LOS measure
1. Vehicular	
Interrupted flow	
• Urban street	Speed
• Signalised intersection	Delay
• Two-way-stop intersection	Delay
• Roundabout	Delay
• Interchange ramp terminal	Delay
Uninterrupted flow	
• Two-lane highway	Speed, per cent time spent following
• Multi-lane highway	Density
• Expressway/motorway	
- basic segment	Density
- ramp merge or diverge	Density
- weaving	Density
2. **Other road users**	
Pedestrians	Speed, delay, space
Cyclists	Speed, event, delay

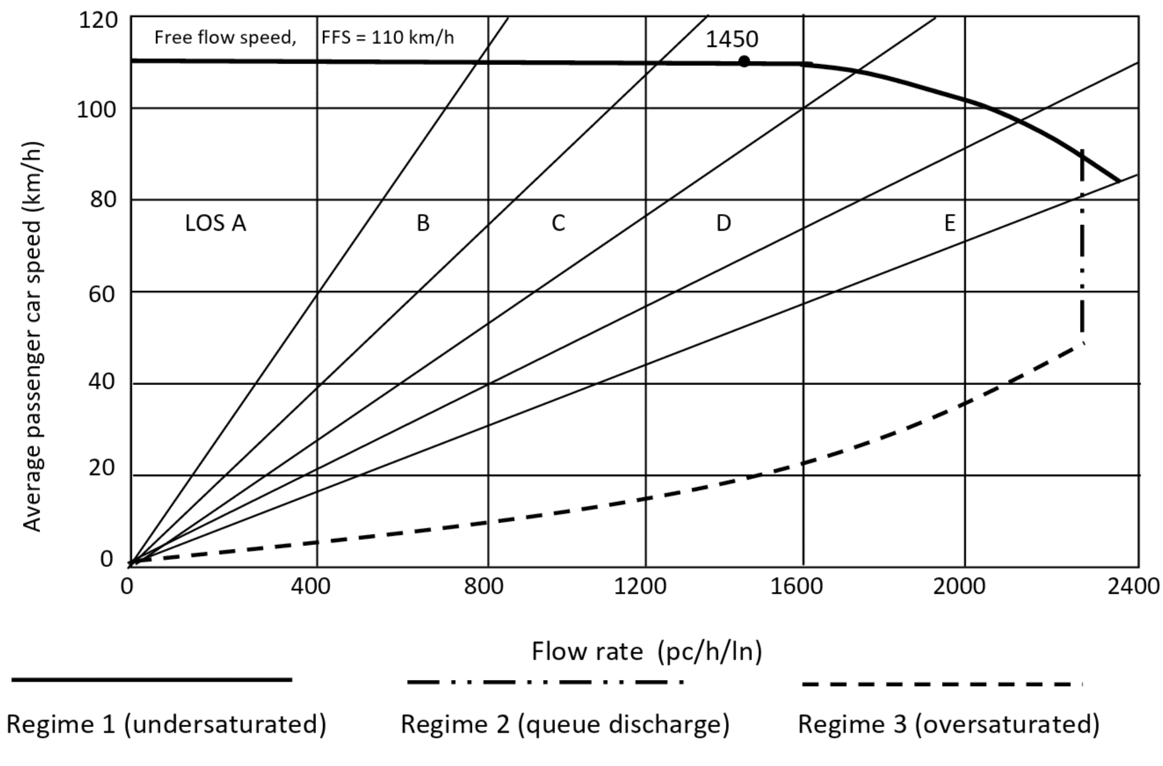
5.4.2.3 SERVICE FLOW RATE

Service flow rates indicate the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point under the prevailing roadway, traffic and control conditions while maintaining a designated LOS. They indicate the vehicle or person capacity for each LOS and are used to determine the LOS corresponding to actual traffic volumes. As with capacity, the service flow rate is generally taken over a 15-minute time period.

Each type of facility has five service flow rates, one for each of the levels of service A to E, as illustrated in **Figure 5.21**. At LOS F flow breakdown occurs, and a meaningful service flow rate cannot be specified.

Figure 5.21 depicts an example of a motorway/expressway operation curve for a free speed of 110 km/h. Other facilities and motorway/expressway with different operating speeds will have different curves. The different levels of service are described by the density (pc/km/ln).

At each LOS, the service flow rate is defined as the maximum for that level. Service flow rates are discrete values, whereas the LOS represents a range of conditions. Service flow rates therefore effectively define the flow boundaries between the levels of service.



Note: The LOS is defined by density as the slopes shown in the diagram. Highway Capacity Manual, TRB (2016)

Source: Adapted from Exhibits 12-3, 12-7 and 12-16 in HCM 2016 (TRB 2016).

Figure 5.21 LOS and service flow rates for expressway/motorway

5.4.2.4 DEGREE OF SATURATION

The concept of degree of saturation, or VCR, is used in the capacity and operational analysis of intersections.

The degree of saturation of a signalised intersection approach may be defined as the ratio of the arrival flow (demand) to the capacity of the approach during the same period.

The degree of saturation of an intersection approach ranges from close to zero for very low traffic flows up to 1.0 for saturated flow or capacity. A degree of saturation greater than 1.0 indicates oversaturated conditions in which long queues of vehicles build up on the critical approaches. In general, the lower the degree of saturation the better the quality of traffic service. However, the degree of saturation, delay and queue length parameters should always be used together to assess intersection performance.

In practice the target degrees of saturation of 0.90 for signals, 0.85 for roundabouts and 0.80 for unsignalized intersections are generally agreed to. These are usually called ‘practical degrees of saturation’.

5.4.3 FACTORS AFFECTING CAPACITY, LEVEL OF SERVICE, AND DEGREE OF SATURATION

5.4.3.1 IDEAL CONDITIONS

In the analysis of capacity, LOS or degree of saturation, the starting point is often to select values that are applicable to ideal conditions, and then to apply correction or adjustment factors that reflect the actual roadway, traffic and control conditions.

In general, an ideal condition is one for which further improvements will not result in any increase in capacity or LOS or decrease in the degree of saturation. More specific details of ideal conditions are given in later sections.

5.4.3.2 ROADWAY CONDITIONS

Roadway conditions that affect capacity, LOS and degree of saturation include:

i. type of facility and its development environment
ii. traffic lane widths
iii. shoulder widths and/or lateral clearances
iv. design speed
v. horizontal and vertical alignment.

Adjustment factors for these conditions are referred to in subsequent sections of this Guide, and details are given in the relevant sections of HCM 2016.

5.4.3.3 TERRAIN CONDITIONS

For capacity and related analyses, the general terrain of a roadway is classified into three categories, namely:

Level (flat) terrain – any combination of grades and horizontal and vertical alignment permitting heavy vehicles to maintain about the same speed as passenger cars.

Rolling (hilly) terrain – any combination of grades and horizontal and vertical alignment causing heavy vehicles to reduce their speeds substantially below those of passenger cars, but not causing them to operate at crawl speeds for any significant length of time.

Mountainous terrain – any combination of grades and horizontal and vertical alignment causing heavy vehicles to operate at crawl speeds for significant distances and/or at frequent intervals.

It is also necessary to consider pedestrian and cyclist needs as their presence in the traffic mix for the given terrain is likely to affect the capacity.

5.4.3.4 TRAFFIC COMPOSITION

For capacity and related analyses, road traffic is typically classified into three categories, namely:

Passenger cars – vehicles registered as passenger cars, plus other light vehicles and light delivery vans having no more than four single tyres.

Trucks – vehicles having more than four single tyres, and involved primarily in the transport of goods or services.

Buses – vehicles having more than four single tyres, and involved primarily in the transport of people. For capacity analysis, passenger cars towing caravans, boats and other similar recreation equipment should be included in this category.

5.4.3.5 PEDESTRIANS AND CYCLISTS

Pedestrians form the largest single road user group. Most individual trips, whatever the primary mode used, begin and/or finish with a walk section. Walking is a fundamental component of all travel. Pedestrian safety and mobility must be explicit factors in the planning, design, implementation and maintenance of road facilities.

Initiatives are being implemented at the planning and policy levels, which give explicit consideration to the walking mode and to pedestrian safety and amenity.

The framework for pedestrian planning and design lies within the practice of traffic engineering. Pedestrians may be considered as being similar to vehicles, operating on a transport network consisting of footways, stairs, travelators etc. Network capacity can be defined, demand measured or predicted, operational levels calculated, and areas of congestion and hazard identified. To achieve maximum safety, the pedestrian network should be separate from, but integrated with, the main road and public transport system. This will necessitate regular crossings in order to sustain the coverage and continuity of the network for walking.

Cyclists also form a significant road-user group with specific needs. Solutions addressing their capacity needs can significantly affect road capacity, LOS and degree of saturation.

Because of increasing environmental, physical and financial constraints, attention must be given to issues of traffic calming and demand management in order to encourage pedestrian, cycling and higher vehicle occupancy modes of transport. Planning, design and management of traffic facilities must cater for pedestrian or cyclist traffic along or across roads as this can affect road capacity.

5.4.3.6 DRIVER POPULATION

The traffic stream characteristics associated with the information provided are representative of regular weekday commuter drivers and other regular users of a facility. In situations where weekend or recreation, or even mid-day (off-peak) drivers are a significant proportion of the traffic stream, the capacity and/or LOS may be reduced.

An adjustment factor is provided where relevant, but the range of values given is relatively wide and engineering judgement and/or local data should be used in selecting the appropriate value.

5.4.3.7 CONTROL CONDITIONS

The control of the time available for individual traffic movements is a critical factor affecting the capacity, LOS and/or the degree of saturation of interrupted flow facilities. Typical forms of control include traffic signals, stop and give-way signs, turn restrictions, lane-use controls and parking restrictions.

5.4.4 UNINTERRUPTED FLOW FACILITIES

HCM 2016 provides detailed information on capacity analysis and is the primary reference document for this section.

5.4.4.1 SINGLE-LANE FLOW

In certain situations, traffic flow may be constrained to a single traffic lane without overtaking, typical examples being a single lane provided in one direction on an undivided urban road with reversible lane flow, in a tunnel or at a construction or maintenance site. In these types of situations, depending on the length of the single lane restriction and on the volume of traffic, the speed of all vehicles will tend to the speed of the slowest vehicle, and traffic will operate in accordance with car-following mechanisms.

Several of the earliest studies of traffic capacity examined single-lane traffic flow. A linear relationship between the average speed and the density of vehicles was assumed, and the capacity was related to the free speed (space mean speed at low flows, space mean speed is the arithmetic mean of the measured speeds of all vehicles within a given length of lane or carriageway, at a given instant of time) and the jam density (the maximum density for stopped traffic) as in **Equation 5.2**:

$$C = \frac{k_j V_f}{4} \tag{5.2}$$

Where:

C = capacity (passenger cars per hour - pc/h)
V_f = free speed (km per hour)
k_j = jam density (pc per km)

5.4.4.2 TWO-LANE TWO-WAY ROADS

Two-lane rural roads have one lane for use by traffic travelling in each direction. Overtaking of slower vehicles requires the use of the opposing traffic lane when sight distance and gaps in the opposing traffic stream permit. At low traffic volumes and under ideal conditions, drivers are able to travel at their desired speed without interference. As traffic volumes increase, and as roadway, terrain and traffic conditions become less than ideal, drivers are increasingly affected by the presence of other vehicles on the road, and bunches form in the traffic stream. Vehicles in these bunches are subjected to delay because of the inability to overtake slower-moving vehicles.

5.4.4.2.1 HCM METHOD

The analysis process outlined in this section is based on HCM 2016. The manual should be consulted for further explanation of the concepts and a detailed description of the process.

HCM 2016 distinguishes between three categories of two-lane highways, as follows:

i. **Class I** – These are two-lane highways on which motorists expect to travel at relatively high speeds. Two lane highways that are major intercity routes, primary arterials connecting major traffic generators, daily commuter routes, or primary links in state or national highway networks generally are assigned to Class I. Class I facilities most often serve long-distance trips or provide connecting links between facilities that serve long-distance trips.

ii. **Class II** – These are two-lane highways on which motorists do not necessarily expect to travel at high speeds. Two-lane highways that function as access routes to Class I facilities, serve as scenic or recreational routes that are not primary arterials, or pass through rugged terrain generally are assigned to Class II. Class II facilities most often serve relatively short trips, the beginning and ending portions of longer trips, or trips for which sightseeing plays a significant role.

iii. **Class III** – These are two-lane highways serving moderately developed areas. They may be portions of a Class I or Class II highway that pass through small towns or developed recreational areas. On such segments, local traffic often mixes with through traffic, and the density of unsignalized roadside access points is noticeably higher than in a purely rural area. Class III highways may also be longer segments passing through more spread-out recreational areas, also with increased roadside densities. Such segments are often accompanied by reduced speed limits that reflect the higher activity level.

The three classes of road perform markedly different functions and are not totally dependent on the highway's role in the hierarchy. A highway between major urban centres through mountainous terrain might be classified as Class II instead of Class I if it is established that drivers appreciate that higher speeds are inappropriate.

HCM 2016 provides a method to analyse the performance of a two-way segment: as directional segments, with each direction of travel considered separately; for sections with steep grades and for segments with passing lanes. The two-way analysis assumes roughly similar flows in each direction.

The performance of a particular road section is calculated from the predicted performance of base or ideal conditions which include:

i. lane widths greater than or equal to 3.6m
ii. clear shoulders wider than or equal to 18 m
iii. no no-overtaking zones
iv. all passenger cars
v. no impediments to through traffic, such as traffic control or turning vehicles
vi. level terrain.

The process estimates the free-flow speed (FFS) based on the base free-flow speed (from the base conditions) and termed the BFFS. The FFS is based on the lane widths, shoulder widths and the average number of access points per kilometre. Reduce the shoulder width or the lane width and free speed is reduced. Increase the frequency of access points and the free speed is reduced. The actual travel speed is a function of the FFS and the passenger car equivalent flow rate which is based on the proportion of heavy vehicles, the grade (or terrain type) and broad flow rates. The effect of an additional vehicle at low flows differs from the effect of an additional vehicle at higher flows.

The per cent time-spent-following is a measure of the level of opportunities to overtake. The per cent time spent-following approaches 100 asymptotically:

i. as the traffic demand increases
ii. as the percentage of no-overtaking zones increases

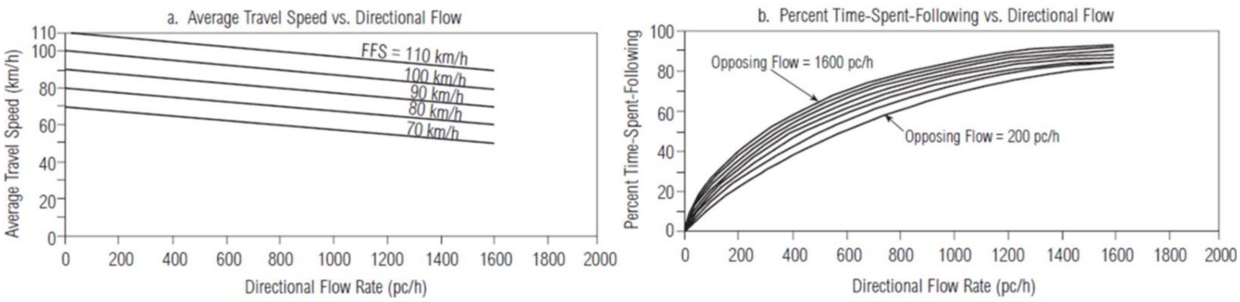
iii. as the directional split moves closer to 90/10.

HCM 2016 considers a no-overtaking zone to exist when the sight distance is below 300m. For the analysis of an existing roadway, a no-overtaking zone could be defined by a barrier line. This would make it easier for practitioners to measure the zone in the field, or from plans, than to determine where the sight distance is below 300m.

Figure 5.22 shows the expected average travel speed and the per cent time-spent-following for ideal or base conditions.

The LOS criteria used for the different highway classes differ. HCM 2016 notes that:

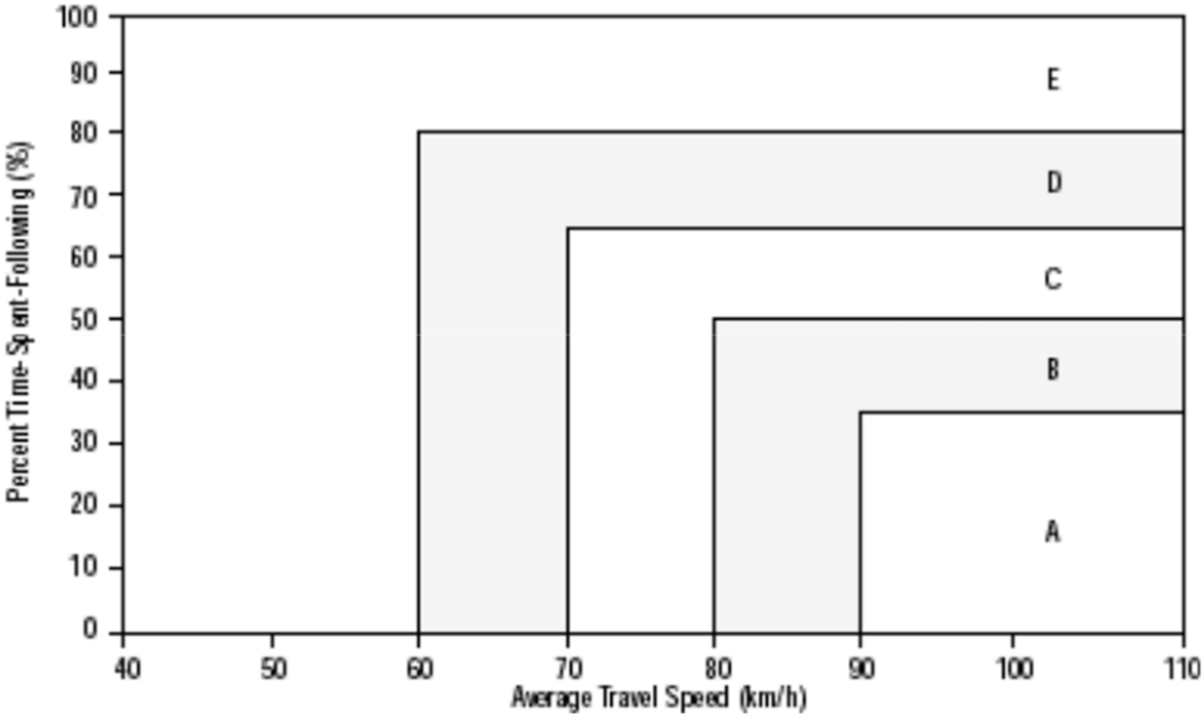
i. The LOS for Class I highways on which efficient mobility is paramount is defined in terms of both per cent time-spent-following and average travel speed.
ii. On Class II highways, mobility is less critical, and LOS is defined only in terms of per cent time-spent-following.
iii. On Class III highways, high speeds are not expected. Because the length of Class III segments is generally limited, passing restrictions are also not a major concern. In these cases, drivers would like to make steady progress at or near the speed limit. Therefore, on these highways, per cent free-flow speed (PFFS) is used to define LOS.



Source: Adapted from Exhibit 15-2 in HCM 2016 (TRB 2016).

Figure 5.22 Speed-flow and per cent time-spent-following relationships for directional segments with base conditions

Figure 5.23 shows the LOS criteria for Class I highways. The criteria for automobile two-lane highways are shown in **Table 5.10**. The LOS can be evaluated from field measurements on an existing road. The per cent of vehicles within 3 seconds, collected at fixed regular intervals and averaged for the section in question, is a good substitute for the per cent time-spent-following.



Source: Adapted from Exhibit 15-3 in HCM 2016 (TRB 2016).

Figure 5.23 LOS criteria for two-lane highways in Class I

Table 5.10 Motorized Vehicle LOS for two-lane highways

LOS	Class I highway		Class II highways PTSF (%)	Class III highways PFFS (%)
	Average travel speed ATS (km/h)	Per cent time-spent-following (PTSF) (%)		
A	> 90	≤ 35	< 40	> 91.7
B	> 80 – 90	> 35 – 50	> 40 – 55	> 83.3 – 91.7
C	> 70 – 80	> 50 – 65	> 55 – 70	> 75.0 – 83.3
D	> 60 – 70	> 65 – 80	> 70 – 85	> 66.7 – 75.0
E	≤ 60	> 80	> 85	≤ 66.7

Note: LOS F applies whenever the arrival flow exceeds the segment capacity.

Source: Adapted from Exhibit 15-3 in HCM 2016 (TRB 2016).

HCM 2016 indicates that the capacity of a two-lane highway is 1700 pc/h for each direction of travel and is nearly independent of the directional distribution of traffic. For extended lengths of two-lane highway, the capacity will not exceed 3200 pc/h for both directions of travel combined.

The conditions for the different levels of performance of two-lane highways are described in the following terms:

i. At LOS A, motorists experience high operating speeds on Class I highways and little difficulty in passing. Platoons of three or more vehicles are rare. On Class II highways, speed would be controlled primarily by roadway conditions. A small amount of platooning would be expected. On Class III highways, drivers should be able to maintain operating speeds close or equal to the FFS of the facility.

ii. At LOS B, passing demand and passing capacity are balanced. On both Class I and Class II highways, the degree of platooning becomes noticeable. Some speed reductions are present on Class I highways. On Class III highways, it becomes difficult to maintain FFS operation, but the speed reduction is still relatively small.

iii. At LOS C, most vehicles are travelling in platoons. Speeds are noticeably curtailed on all three classes of highway.

iv. At LOS D, platooning increases significantly. Passing demand is high on both Class I and II facilities, but passing capacity approaches zero. A high percentage of vehicles are now travelling in platoons, and PTSF is quite noticeable. On Class III highways, the fall-off from FFS is now significant.

v. At LOS E, demand is approaching capacity. Passing on Class I and II highways is virtually impossible, and PTSF is more than 80%. Speeds are seriously curtailed. On Class III highways, speed is less than two-thirds the FFS. The lower limit of this LOS represents capacity.

vi. LOS F exists whenever arrival flow in one or both directions exceeds the capacity of the segment. Operating conditions are unstable, and heavy congestion exists on all classes of two-lane highway.

5.4.4.3 MULTI-LANE ROADS

Multi-lane roads have two or more lanes for use by traffic in each direction. They may be classified as:

i. divided – when opposing directions of traffic are physically separated by a median

ii. undivided – when opposing directions of traffic are not physically separated.

Multi-lane roads have at-grade intersections including signalised intersections; this attribute distinguishes them from expressways/motorways. Multi-lane roads and urban roads have different traffic signal densities. HCM 2016 provides procedures for both classes that should be reviewed when analysing urban roads in suburban areas.

The recommended analysis procedure is based on HCM 2016, Chapter 12. The analysis is extended for road sections with higher grades over longer distances. The analysis involves a comparison with base conditions. The base represents the highest operating level of a multilane highway with the following characteristics:

i. 3.6m minimum lane widths
ii. 3.6m minimum total lateral clearance in the direction of travel – this represents the total lateral clearances from the edge of the travel lanes to obstructions along the edge of the road and in the median (in computations, lateral clearances greater than 1.8m are considered in computations to be equal to 1.8m)
iii. only passenger cars in the traffic stream
iv. no direct access points along the roadway
v. a divided highway
vi. FFS equal to 100 km/h.

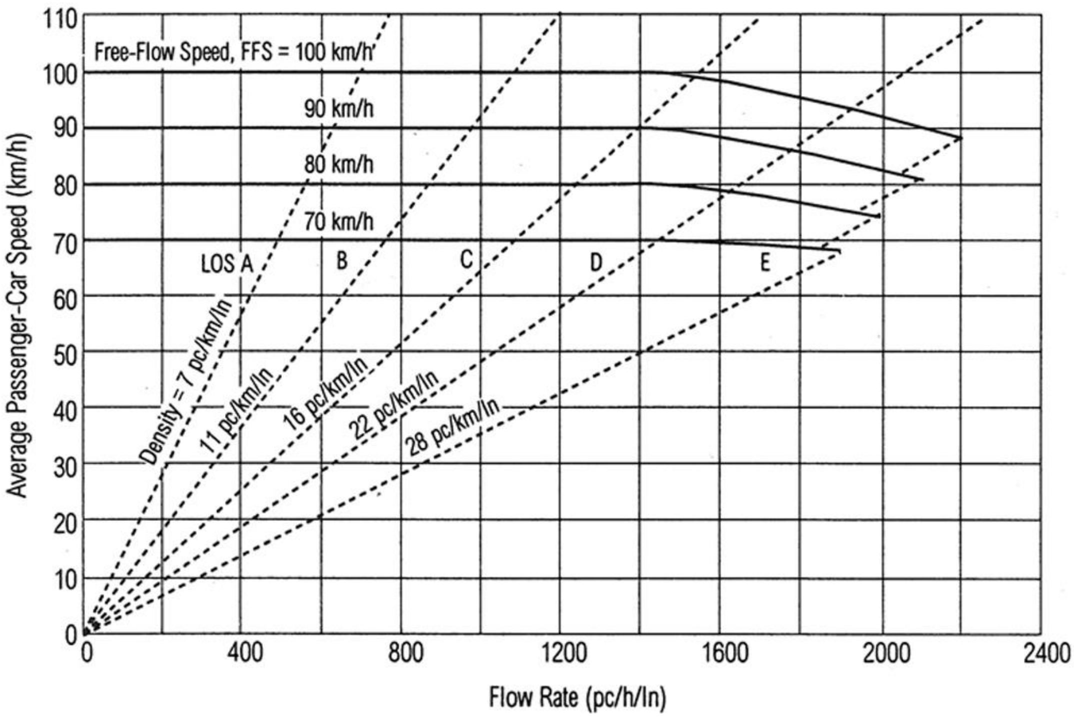
The FFS is estimated from the FFS for the base conditions. A value between 8 and 10 km/h above the speed limit could be used if there is no other evidence from similar roads. The FFS is reduced if lane width is reduced, if the total lateral clearance is reduced, and as the frequency of access points is increased. The free speed on an undivided road is 2.6 km/h lower than on a divided road with the same characteristics.

The design flow rate is calculated for a peak 15 minutes and accounts for the percentage and type of heavy vehicles, whether drivers are commuters or not, and on the variation of the traffic within a peak hour. HCM 2016 uses a peak hour factor (PHF) term in almost all procedures. The PHF is the ratio between the hourly flow and the peak 15-minute flow rate.

When using the HCM 2016 procedure, there may be a need to consider the vehicle equivalency of Ghana trucks. There is little evidence to indicate appropriate values to be used, but a sensitivity analysis could be used to establish if higher vehicle equivalency values are appropriate.

The average trip speed is then estimated using the FFS and the design flow rate. **Figure 5.24** shows the relationship between these parameters although HCM 2016 provides equations for a computerised solution.

A vehicle density, given by the design flow rate divided by the average trip speed, is the prime measure used in the evaluation of the LOS. Other characteristics for different LOS levels are shown in **Table 5.11**.



Source: Adapted from Exhibit 12-17 in HCM 2016 (TRB 2016); note that parts of the curves are obscured by grid lines.

Figure 5.24 Speed-flow curves with LOS criteria for multi-lane roads

Table 5.11 LOS criteria for multi-lane highways

Free-flow speed	Criteria	A	B	C	D	E
100 km/h	Maximum density (pc/km/ln)	7	11	16	22	25
	Average speed (km/h)	100.0	100.0	98.4	91.5	88.0
	Maximum volume to capacity ratio (v/c)	0.32	0.50	0.72	0.92	1.00
	Maximum service flow rate (pc/h/ln)	660	1080	1550	1980	2200
90 km/h	Maximum density (pc/km/ln)	7	11	16	22	26
	Average speed (km/h)	90.0	90.0	89.8	84.7	80.8
	Maximum volume to capacity ratio (v/c)	0.30	0.47	0.68	0.89	1.00
	Maximum service flow rate (pc/h/ln)	600	990	1430	1850	2100
80 km/h	Maximum density (pc/km/ln)	7	11	16	22	27
	Average speed (km/h)	80.0	80.0	80.0	77.6	74.1
	Maximum volume to capacity ratio (v/c)	0.28	0.44	0.64	0.85	1.00
	Maximum service flow rate (pc/h/ln)	550	900	1300	1710	2000
70 km/h	Maximum density (pc/km/ln)	7	11	16	22	28
	Average speed (km/h)	70.0	70.0	70.0	69.6	67.9
	Maximum volume to capacity ratio (v/c)	0.26	0.41	0.59	0.81	1.00
	Maximum service flow rate (pc/h/ln)	290	810	1170	1550	1900

5.4.4.4 EXPRESSWAY/MOTORWAY

An expressway/motorway is a divided road with two or more lanes for traffic travelling in each direction, with no at-grade intersections and with full control of access from abutting property.

Expressways/motorways have three types of elements, namely:

i. basic expressway/motorway segments – which are sections of expressway/motorway that are outside the influence of any ramp or weaving area
ii. ramps and ramp terminals – which provide access to and from the expressway/motorway
iii. weaving areas – which are sections of expressway/motorway on which two or more vehicle flows must cross, such as when a merge area is closely followed by a diverge area.

These elements can be combined to evaluate a expressway/motorway facility. HCM 2016 provides a method to evaluate the performance of a expressway/motorway that has congested elements for more than one 15-minute time period.

The recommended analysis procedure is the current procedure in HCM 2016, and the default values documented in it should be used.

5.4.4.4.1 BASIC EXPRESSWAY/MOTORWAY SEGMENTS

As for other uninterrupted facilities, the evaluation of a basic expressway/motorway segment relies on an evaluation of the average travel speed. The first consideration is the FFS, which could be measured in the field, or estimated using relationships in HCM 2016 (Chapter 12).

Basic expressway/motorway segments can be analysed for sections with a specific grade. This process is typically applied to sections with steeper grades over extended lengths. The process is not discussed here and HCM 2016 should be consulted for further information.

The FFS is a function of the base FFS and a number of adjustment parameters. The base conditions are:

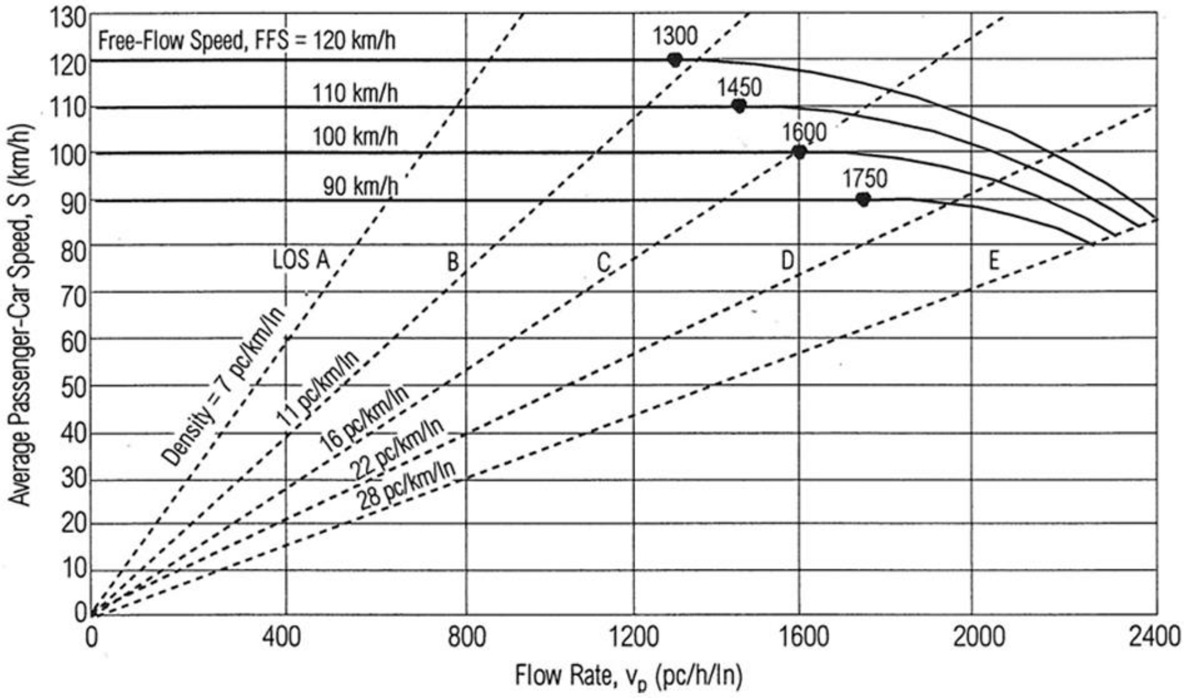
i. minimum lane widths of 3.6m
ii. minimum left-shoulder lateral clearance between the edge of the travel lane and the nearest obstacle or object that influences traffic behaviour of 1.8m
iii. minimum median lateral clearance of 0.6m
iv. traffic stream composed entirely of passenger cars
v. five or more lanes for one direction (in urban areas only)
vi. interchange spacing at 3km or greater
vii. level terrain, with grades no greater than 2%

viii. a driver population composed principally of regular users of the facility

ix. the free speed is 120 km/h in rural areas and 110 km/h in urban areas.

The FFS is reduced when the lane width is reduced, when the right shoulder width is reduced, when the number of lanes is reduced and when the interchange density is increased. The design flow rate is a function of the type and proportion of heavy vehicles and whether or not drivers are commuters. The effect of long combination vehicles such as road trains may need to be considered in this approach, but appropriate values for the vehicle equivalencies are not readily available.

The LOS is calculated from the vehicle density, being the design flow rate divided by the average passenger car speed. **Figure 5.25** shows the speed-flow curve for basic expressway/motorway segments. Results can be measured or estimated and plotted directly onto this curve to predict the LOS. **Table 5.12** is a list of the LOS criteria with density being the prime term.



Source: Adapted from Exhibit 12-16 in HCM 2016 (TRB 2016).

Figure 5.25 Speed-flow relationship for basic expressway/motorway segments

Table 5.12 **LOS criteria for basic expressway/motorway segments**

FFS (km/h)	Criteria	LOS				
		A	B	C	D	E
120	Maximum density (pc/km/ln)	7	11	16	22	28
	Minimum speed (km/h)	120	120	114.6	99.6	85.7
	Maximum (v/c)	0.35	0.55	0.77	0.92	1
	Maximum service flow rate (pc/h/ln)	840	1320	1840	2200	2400
110	Maximum density (pc/km/ln)	7	11	16	22	28
	Minimum speed (km/h)	110	110	108.5	97.2	83.9
	Maximum (v/c)	0.33	0.51	0.74	0.91	1
	Maximum service flow rate (pc/h/ln)	770	1210	1740	2135	2350
100	Maximum density (pc/km/ln)	7	11	16	22	28
	Minimum speed (km/h)	100	100	100	93.8	82.1
	Maximum (v/c)	0.3	0.48	0.7	0.9	1
	Maximum service flow rate (pc/h/ln)	700	1100	1600	2065	2300
90	Maximum density (pc/km/ln)	7	11	16	22	28
	Minimum speed (km/h)	90	90	90	89.1	80.4
	Maximum (v/c)	0.28	0.44	0.64	0.87	1
	Maximum service flow rate (pc/h/ln)	630	990	1440	1955	2250

Source: Adapted from Exhibit 23-2 in HCM 2000 and updated from Exhibit 12-15 in HCM 2016 (TRB 2016).

Engineering judgement is necessary in applying the above table for local practices. The maximum (service) flow rate at a LOS E is a geometric or physical capacity suitable for strategic planning purposes when the future demand would operate at a design LOS. If the design LOS for the future year is C, then the maximum volume-to-capacity ratio (VCR) is 0.70 at an FFS of 100km/h and this VCR determines the number of lanes required for the future

On basic expressway/motorway segments, the LOS definitions from HCM 2016 are also applicable to multi-lane highways as follows:

i. LOS A describes free-flow operations. FFS prevail on the expressway/motorway or multi-lane highway, and vehicles are almost completely unimpeded in their ability to manoeuvre within the traffic stream. The effects of incidents or point breakdowns are easily absorbed.

ii. LOS B represents reasonably free-flow operations, and FFS on the expressway/motorway or multi-lane highway is maintained. The ability to manoeuvre within the traffic stream is only slightly restricted, and the general level of physical and psychological comfort provided to drivers is still high. The effects of minor incidents are still easily absorbed.

iii. LOS C provides the flow conditions with speeds near the FFS of the expressway/motorway or multi-lane highway. Freedom to manoeuvre within the traffic stream is noticeably restricted, and lane changes require more care and vigilance on the part of the driver. Minor incidents may still be absorbed, but the local deterioration in service will be substantial. Queues may be expected to form behind any significant blockage.

iv. LOS D is the level at which speeds begin to decline slightly with increasing flows, with density increasing more quickly. Freedom to manoeuvre within the traffic stream is seriously limited, and the drivers experience reduced physical and psychological comfort levels. Even minor incidents can be expected to create queuing, because the traffic stream has little space to absorb disruptions.

v. LOS E describes operation at or near capacity. Operations on the expressway/motorway or multi-lane highway at this level are highly volatile because there are virtually no usable gaps within the traffic stream, leaving little room to manoeuvre within the traffic stream. Any disruption to the traffic stream, such as vehicles entering from a ramp or an access point or a vehicle changing lanes, can establish a disruption wave that propagates throughout the upstream traffic stream. Towards the upper boundary of LOS E, the traffic stream has no ability to dissipate even the most minor disruption, and any incident can be expected to produce a serious breakdown and substantial queuing. The physical and psychological comfort afforded drivers is poor.

vi. LOS F describes unstable flow. Such conditions exist within queues forming behind bottlenecks. Breakdowns occur for a number of reasons as follows:

- traffic incidents can temporarily reduce the capacity of a short segment, so that the number of vehicles arriving at a point is greater than the number of vehicles that can move through it
- points of recurring congestion, such as merge or weaving segments and lane drops, experience very high demand in which the number of vehicles arriving is greater than the number of vehicles that can be discharged
- in analyses using forecast volumes, the projected flow rate can exceed the estimated capacity of a given location.

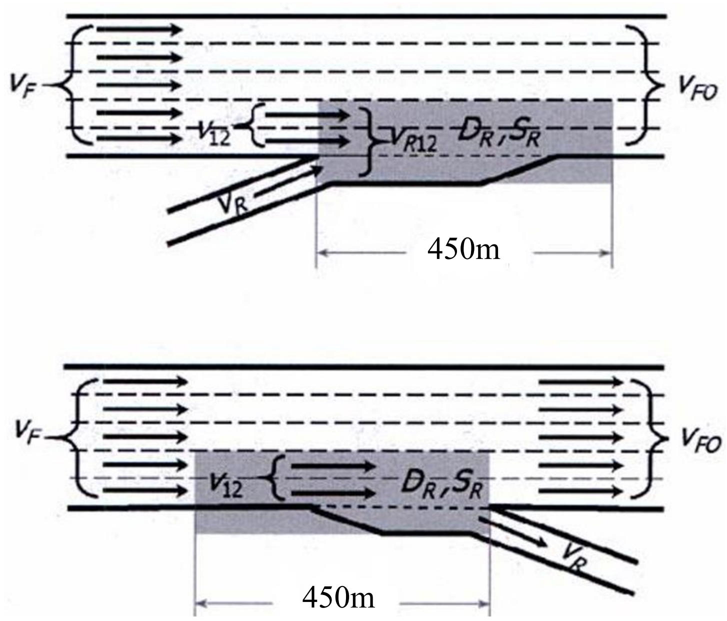
Note that in all cases, breakdown occurs when the ratio of existing demand to actual capacity, or of forecast demand to estimated capacity, exceeds 1.0. LOS F operations within a queue are the result of a breakdown or bottleneck at a downstream point. Operations immediately downstream of such a point, however, are generally at or near capacity, and downstream operations improve (assuming that there are no additional downstream bottlenecks) as discharging vehicles move away from the bottleneck.

5.4.4.4.2 RAMPS AND RAMP JUNCTIONS

The recommended procedure for the evaluation of ramps is given in HCM 2016 (Chapter 14). The evaluation of ramps considers vehicle interactions that occur within 450m of the ramp, shown as the influence area in **Figure 5.26**. This figure also shows important traffic-flow variables used in the analysis. The traffic flows are recorded in passenger car units per hour and are a function of the type and proportion of heavy vehicles and the attributes of the driving population.

The terminology for the peak 15-minute traffic flows is as follows:

i. V_F is the total traffic across all lanes of the carriageway entering the ramp area
ii. V_R is the ramp traffic
iii. V_{12} is the traffic in the two kerb-side lanes of the carriageway entering the ramp area
iv. V_{R12} is the traffic in the two kerb lanes and on the ramp
v. V_{FO} is the total traffic across all lanes of the carriageway exiting the ramp area.



Source: Adapted from Exhibit 14-7 in HCM 2016 (TRB 2016).

Figure 5.26 Influence area at ramps

The process to evaluate ramps is to predict the traffic in the two kerb-side lanes (lane 1 and lane 2). (If there are only two lanes on the carriageway, then this flow is equal to the total flow.) The proportion of traffic in the two kerb-side lanes depends on the proximity and type of the previous upstream ramp and the proximity and type of the next downstream ramp.

The density of the merge area is then calculated using a linear relationship with the peak 15-minute ramp flow, V_R, the flow in the two kerb-side lanes, V_{12}, and the acceleration lane length L_A. HCM 2016 defines the acceleration-lane length measured from 'the point at which the left edge of the ramp lane or lanes and the right edge of the expressway/motorway lanes converge to the end of the taper segment connecting the ramp to the expressway/motorway.

The point of convergence can be defined by painted markings or physical barriers or by some combination of the two. Note that both taper area and parallel ramps are measured in the same way'.

Equation 5.3 is used to estimate the density in the merge influence area, replicated from HCM 2000 **Equation 25-5**. Note that the equation for density applies only to undersaturated flow conditions.

$$D_R = 3.402 + 0.00456V_R + 0.0048V_{12} - 0.01278L_A \qquad (5.3)$$

Where,

D_R = density of merge influence area (pc/km/ln)
V_R = on-ramp peak 15-min flow rate (pc/h)
V_{12} = flow rate entering ramp influence area (pc/h)
L_A = Length of acceleration lane (m).

The calculated densities for on-ramp traffic onto a three-lane expressway/motorway carriageway are shown in **Figure 5.27**. The three lines represent the merging traffic being 5%, 10% and 15% of the total traffic. The densities are reasonably independent of the ramp flows when plotted against the total downstream traffic (expressway/motorway and ramp flows). This plot is based on **Equation 5.3** with an acceleration lane length of 100m, three lanes on the expressway/motorway and upstream and downstream ramps have no influence on the proportion of traffic in the first two lanes. By increasing the acceleration lane by 100m, the density in the influence area will decrease by 1.3 pc/km/ln.

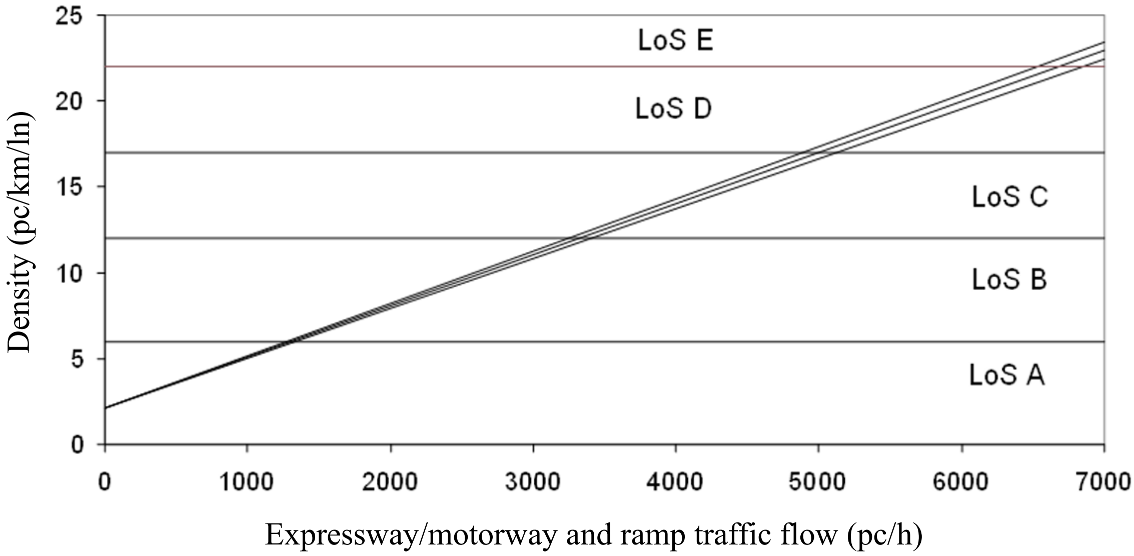


Figure 5.27 Influence-area density for on-ramp flows of 5%, 10% and 15% of the expressway/motorway flows; the acceleration lane length is 100m

Equation 5.4 is used to estimate the density within the diverge influence area, replicated from HCM 2000 **Equation 25-10**.

$$D_R = 2.624 + 0.0053V_{12} - 0.0183L_D \quad (5.4)$$

Where,

D_R = density of diverge influence area (pc/km/ln),
V_{12} = flow rate entering ramp influence area (pc/h), and
L_D = length of deceleration lane (m)

For an off-ramp, the calculated densities in the influence area of a three-lane expressway/motorway are shown in **Figure 5.28**. The three lines represent the diverging traffic being 5%, 10% and 15% of the total traffic. Again, changing the off-ramp traffic flow has only a marginal effect on the traffic densities. This plot is based on **Equation 5.4** with a deceleration lane length of 100m, three lanes on the expressway/motorway and upstream and downstream ramps have no influence on the proportion of traffic in the first two lanes. By increasing the deceleration lane length by 100m, the density in the influence area will decrease by 1.8 pc/km/ln.

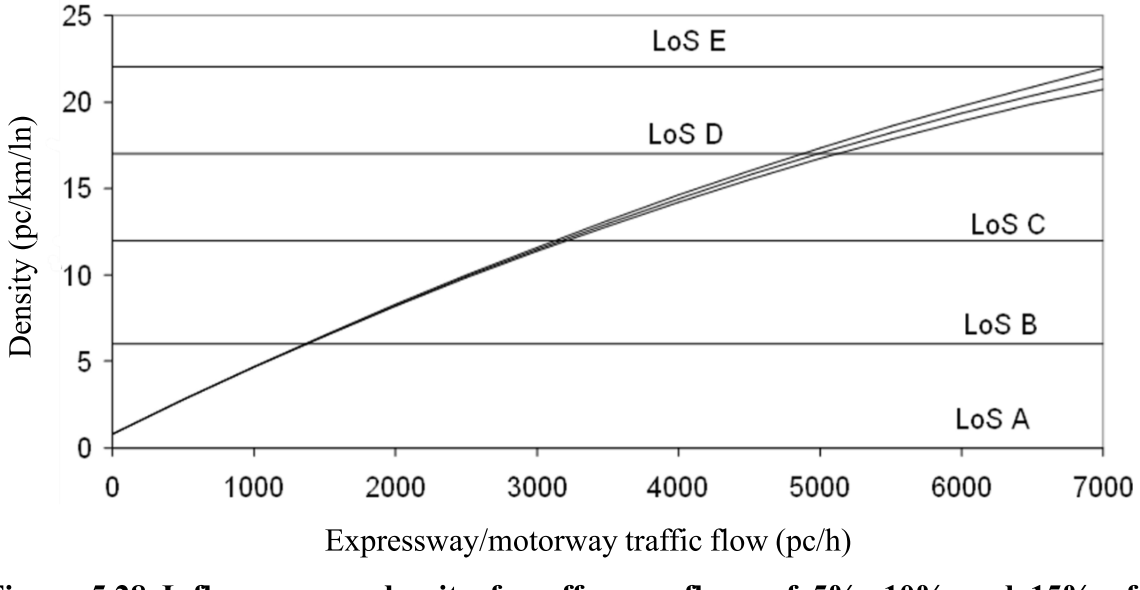


Figure 5.28 Influence-area density for off-ramp flows of 5%, 10% and 15% of the expressway/motorway flows; the deceleration lane length is 100 m

Capacities of the expressway/motorway approaching the diverge area, departing from the merge or diverge area, and of the ramp and the influence area, are given in HCM 2016 (Chapter 14). The HCM 2016 lists the capacity of ramp roadways and ramp expressway/motorway, as reproduced in **Table 5.13** and **Table 5.14**. It is assumed in HCM 2016 that the capacity of a basic expressway/motorway segment under base conditions is 2400 pc/h/ln (FFS = 120 km/h) and 2300 pc/h/ln (FFS = 100 km/h).

Table 5.13 Approximate capacity of ramp roadways in passenger cars/hour

Free-flow speed of ramp, SFR (km/h)	Capacity (pc/h)(1)	
	Single-lane ramps	Two-lane ramps
> 80	2200	4400
> 65–80	2100	4200
> 50–65	2000	4000
≥ 30–50	1900	3800
< 30	1800	3600

1 The operational capacity is less than the value indicated above when flow breakdown occurs.

Source: Exhibit 14-12 in HCM 2016 (TRB 2016).

Table 5.14 Capacity values for merge and diverge areas in passenger cars/hour

Expressway/ motorway free-flow speed (km/h)	Capacity of upstream/downstream expressway/motorway segment (pc/h)(1)(3)				Max desirable flow entering merge influence area V_{R12} (pc/h)(2)	Max desirable flow entering diverge influence diverge area V_{12} (pc/h)(2)
	Number of lanes in one direction					
	2	3	4	> 4		
120	4800	7200	9600	2400/ln	4600	4400
110	4700	7050	9400	2350/ln	4600	4400
100	4600	6900	9200	2300/ln	4600	4400
90	4500	6750	9000	2250/ln	4600	4400

1 Demand in excess of these capacities results in LOS F.

2 Demand in excess of these values does not result in LOS F; operations may be worse than predicted by this methodology.

3 The operational capacity is less than the value indicated above when flow breakdown occurs.

Source: Exhibit 14-10 in HCM 2016 (TRB 2016).

The LOS is estimated from the densities of vehicles in the influence area. Appropriate densities from HCM 2016 are given in **Table 5.15**.

Table 5.15 LOS criteria for expressway/motorway merge and diverge segments

LOS	Density (pc/km/ln)
A	≤ 6
B	> 6–12
C	> 12–17
D	> 17–22
E	> 22
F	Demand exceeds capacity

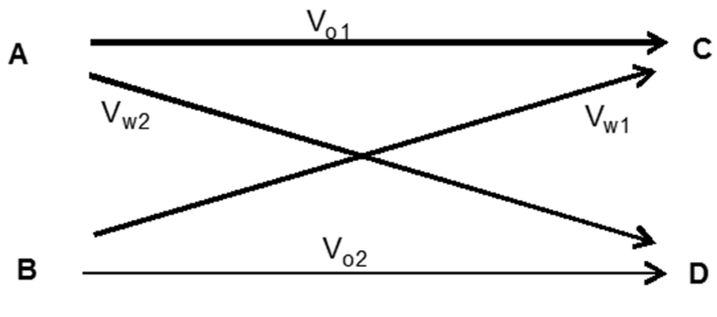
HCM 2016 notes that LOS in merge and diverge influence areas are defined in terms of density for all cases of stable operation, from LOS A to E. LOS F exists when the demand exceeds the capacity of upstream or downstream expressway/motorway sections or the capacity of ramps. The different LOS are as follows:

i. LOS A represents unrestricted operations. Density is low enough to permit smooth merging and diverging, with virtually no turbulence in the traffic stream.
ii. At LOS B, merging and diverging manoeuvres become noticeable to through drivers, and minimal turbulence occurs. Merging drivers must adjust speeds to accomplish smooth transitions from the acceleration lane to the expressway/motorway.
iii. At LOS C, speed within the influence area begins to decline as turbulence levels become noticeable. Both ramp and expressway/motorway vehicles begin to adjust their speeds to accomplish smooth transitions.
iv. At LOS D, turbulence levels in the influence area become intrusive and virtually all vehicles slow to accommodate merging and diverging. Some ramp queues may form at heavily used on-ramps, but expressway/motorway operation remains stable.
v. LOS E represents conditions approaching capacity. Speeds reduce significantly, and turbulence is felt by virtually all drivers. Flow levels approach capacity, and small changes in demand or disruptions within the traffic stream can cause both ramp and expressway/motorway queues to form.

The HCM 2016 process allows for closely spaced ramps, and ramps with more than one lane. The manual should be consulted for details about the analysis process.

5.4.4.4.3 WEAVING SECTIONS

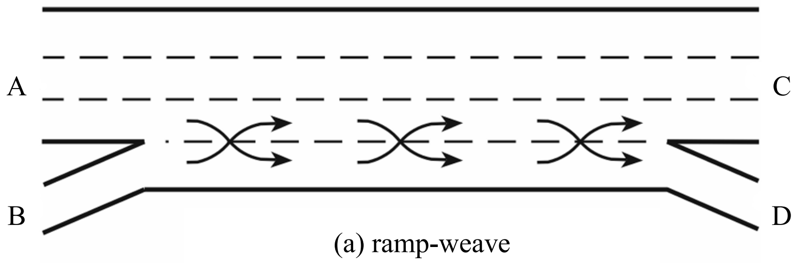
The analysis of weaving sections is less developed than other expressway/motorway analysis procedures. The analysis process requires the weaving and the non-weaving traffic to be estimated. This is shown diagrammatically in **Figure 5.29**.



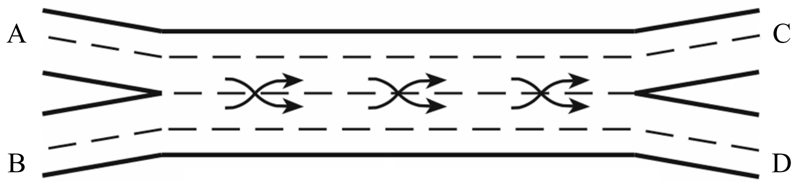
Source: Adapted from Exhibit 13-9 in HCM 2016 (TRB 2016).

Figure 5.29 Diagrammatic representation of traffic on expressway/motorway weaving areas

In this figure, V_{o1} and V_{o2} are the outer, non-weaving flow rates and V_{w1} and V_{w2} are the two weaving flow rates. The number of lanes crossed by both weaving traffic flows is used to classify weaving sections. HCM 2016 (Chapter 13) defines the different types of weaving sections as follows, and these are illustrated in **Figure 5.30**.

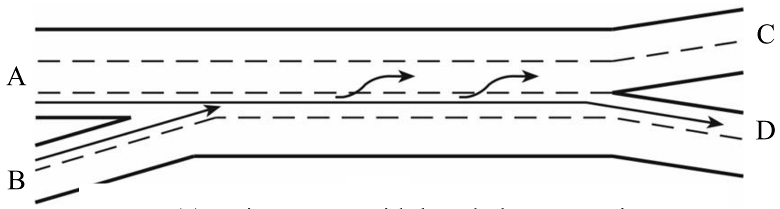


(a) ramp-weave

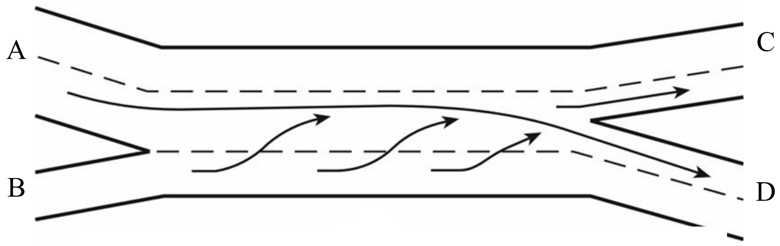


(b) major weave with crown line

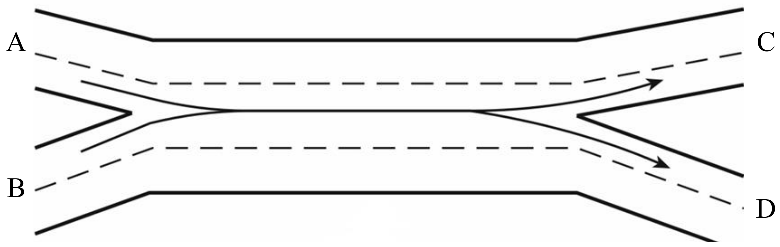
Type A weaves



(a) major weave with lane balance at exit

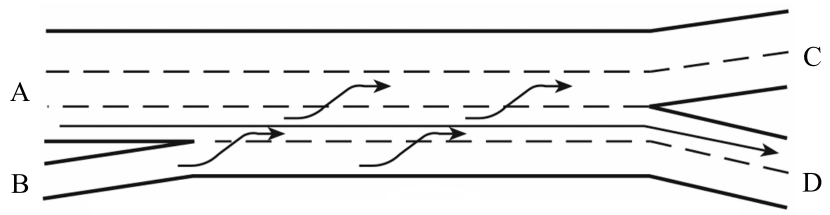


(b) major weave with merging at entrance

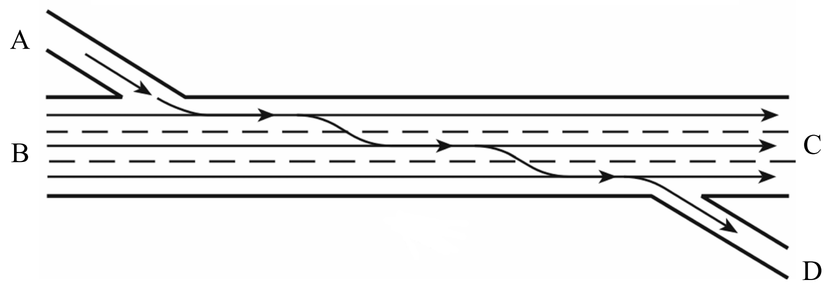


(c) major weave with merging at entrance and lane balance at exit

Type B weaves



(a) major weave without lane balance



(b) two-sided weave

Type C weaves

Figure 5.30 Types of weaves

i. Type A weaving segments – all weaving vehicles must make one lane change to complete their manoeuvre successfully. All of these lane changes occur across a lane line that connects from the entrance gore area directly to the exit gore area. The most common form of Type A weaving segment is formed by a one-lane on-ramp followed by a one-lane off-ramp, with the two connected by a continuous auxiliary lane. All on-ramp vehicles entering the expressway/motorway must make a lane change from the auxiliary lane to the shoulder lane of the expressway/motorway. All expressway/motorway vehicles exiting at the off-ramp must make a lane change from the shoulder lane of the expressway/motorway to the auxiliary lane. This type of configuration is also referred to as a ramp-weave.

ii. Type B weaving segments – one weaving movement can be made without making any lane changes, and the other weaving movement requires at most one lane change.

iii. Type C weaving segments – similar to those of Type B in that one or more through lanes are provided for one of the weaving movements. One weaving movement requires a minimum of two-lane changes for successful completion of a weaving manoeuvre while the other movement can be made without making a lane change.

The configuration of the weaving segment has a marked effect on operations because of its influence on lane-changing behaviour. A weaving segment with 1000 veh/h weaving across 1000 veh/h in the other direction requires at least 2000 lane changes per hour in a Type A segment, since each vehicle makes one lane change. In a Type B segment, only one movement must change lanes, reducing the number of required lane changes per hour to 1000. In a Type C segment, one weaving flow would not have to change lanes, while the other would have to make at least two-lane changes, for a total of (at least) 2000 lane changes per hour.

Configuration has a further effect on the proportional use of lanes by weaving and non-weaving vehicles. Since weaving vehicles must occupy specific lanes to efficiently complete their manoeuvres, the configuration can limit the ability of weaving vehicles to use outer lanes of the segment. This effect is most pronounced for Type A segments because weaving vehicles must primarily occupy the two lanes adjacent to the crown line. It is least severe for Type B segments since these segments require the fewest lane changes for weaving vehicles, thus allowing more flexibility in lane use.

The length and width of the weaving segment are two geometric parameters that describe the area used by weaving vehicles. The weaving length is defined in HCM 2016 as ‘measured from a point at the merge gore where the right edge of the freeway shoulder lane and the left edge of the merging lane(s) are 0.6 m apart to a point at the diverge gore where the two edges are 3.7m apart’. All weaving vehicles must make their lane changing within the length of the weaving segment. If the length of a weaving segment decreases, the intensity of lane changing and the resulting turbulence increases. The weaving width is the number of lanes affected or influenced by the weaving traffic.

The analysis process estimates the average speed of the weaving and non-weaving traffic assuming that the weave is unconstrained. This assumption is later tested and adjusted if necessary. Given the average speeds and flows, the traffic density is calculated and compared with values listed in **Table 5.16**.

Table 5.16 LOS criteria for weaving segments

LOS	Density (pc/km/ln)	
	Freeway weaving segment	Multi-lane and collector-distributor weaving segments
A	≤ 6.0	≤ 8.0
B	> 6.0–12.0	> 8.0–15.0
C	> 12.0–17.0	> 15.0–20.0
D	> 17.0–22.0	> 20.0–23.0
E	> 22.0-27	> 23.0-25
F	> 27 or demand exceeds capacity	> 25 or demand exceeds capacity

Source: Exhibit 13-6 in HCM 2016 (TRB 2016a).

Figure 5.31 illustrates the predicted densities when the weaving traffic is 40% of the total traffic. The different curves relate to different total traffic. The weaving section has four lanes and is Type A. The densities and the LOS are largely dependent on the total traffic and to a lesser extent on the length of the weaving section. **Figure 5.32** demonstrates that the effect of changing the proportion of weaving traffic is marginal.

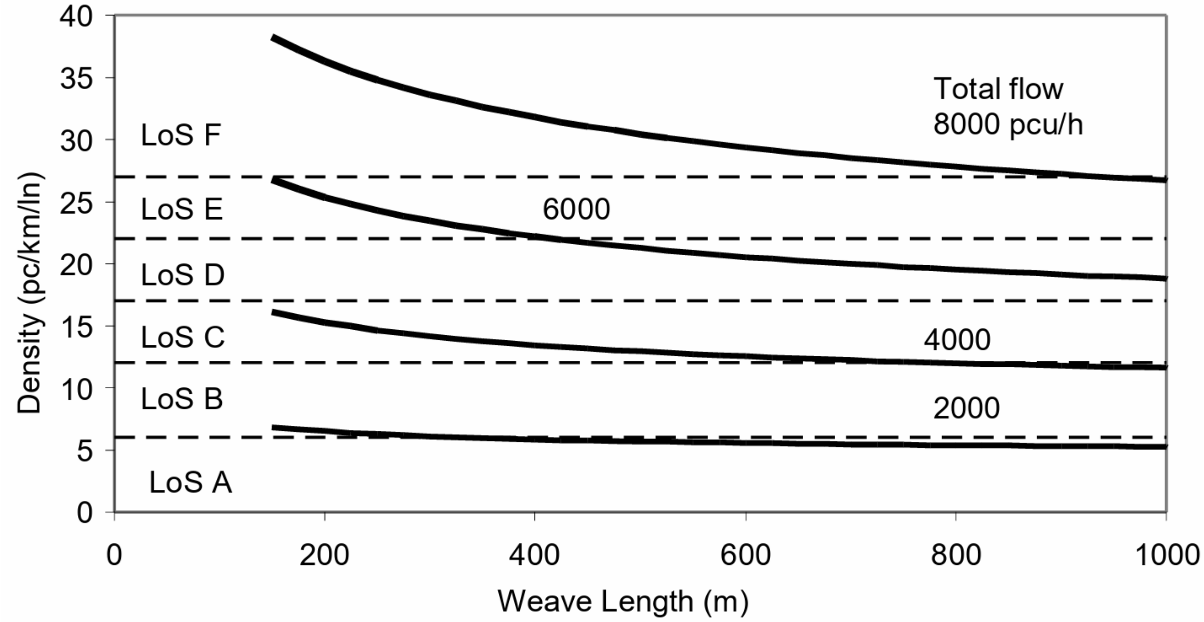


Figure 5.31 LOS for weaving sections of Type A, with 40% weaving traffic and varying total traffic

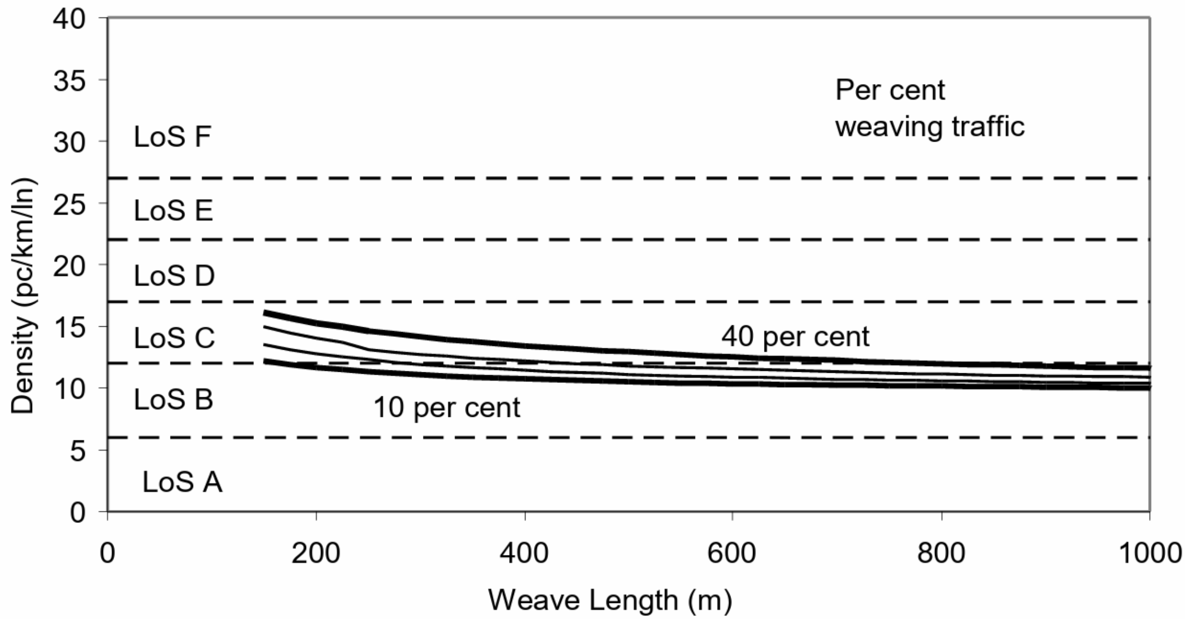


Figure 5.32 LOS for weaving sections of Type A, with 4000 pc/h total traffic and with different proportions of weaving traffic

5.4.5 INTERRUPTED FLOW FACILITIES

In general, the more important unsignalized and signalised intersections determine the overall capacity and traffic performance of interrupted flow facilities. Analysis of these types of intersections usually involves consideration of their capacity and of the mid-block or route capacity of the approach roads. Intersection selection and analysis involves consideration of safety, operational performance or other factors in order to maximise safe mobility (i.e. the safest practicable treatment that also provides an acceptable level of mobility).

The capacity analysis of an intersection may be based on existing traffic volumes or on estimated future traffic volumes depending on the particular problem being addressed. For example, if the objective is to correct an existing deficiency or to review existing traffic signal timings, present-day peak-period volumes would be appropriate. However, if the objective is to prepare an ultimate design in order to decide possible future land use requirements, estimated future traffic volumes should be used.

If future traffic volumes are to be used, they may be obtained as follows:

i. use traffic modelling techniques to produce estimates of future traffic volumes and turning movements
ii. alternatively, if sufficient information is not available for modelling, or if it is thought to be inappropriate
    - determine present-day peak-period traffic volumes and turning movements
    - determine the critical approach road
    - estimate the potential future capacity of the critical approach road when fully developed
    - factor up the existing traffic volumes and turning movements at the intersection by a factor equal to the potential future capacity of the critical approach road divided by the existing volume on it.

Even if the first method is used, it should be checked by the second to ensure that estimated future approach-road volumes at the intersection are realistic.

In certain circumstances, the installation of traffic signals may improve capacity, traffic operation and/or safety at an intersection.

5.4.5.1 MOTOR VEHICLE LEVEL OF SERVICE

Motor vehicle level of service (LOS) at an intersection is defined by the Highway Capacity Manual in terms of delay experienced by a motor vehicle traveling through the intersection during the busiest (peak) 15 minutes of traffic of the day. Typically, delay is averaged over all approaches with traffic controls (STOP, YIELD, or signal). It can also be computed separately for each approach or each lane group (adjacent lanes with at least one movement in common; for example one lane with through movement adjacent to a lane with through/right-turn movement). **Table 5.17** provides motor vehicular level-of-service criteria at intersections.

Table 5.17 LOS criteria for two-way stop-controlled intersections

LOS	Average delay per vehicle (d) in seconds [(1)]		
	Unsignalized intersections	**Signalised intersections**	**Roundabouts**
A	$d \leq 10$	$d \leq 10$	$d \leq 10$
B	$10 < d \leq 15$	$10 < d \leq 20$	$10 < d \leq 15$
C	$15 < d \leq 25$	$20 < d \leq 35$	$15 < d \leq 25$
D	$25 < d \leq 35$	$35 < d \leq 55$	$25 < d \leq 35$
E	$35 < d \leq 50$	$55 < d \leq 80$	$35 < d \leq 50$
F	$50 < d$	$80 < d$	$50 < d$

Source: Highway Capacity Manual, (HCM 2000) Transportation Research Board, 2000

1 Delay is "control delay" as defined in HCM 2000, and includes time for slowing, waiting in queues at the intersections, and accelerating back to free-flow speed.

5.4.5.2 IMPROVING VEHICULAR LEVEL OF SERVICE AT INTERSECTIONS

When attempting to improve the motor vehicular level-of-service at intersections, the designer should work to ensure that the measures to improve motor vehicular level of service do not have a disproportionately negative impact on other intersection users. There are several techniques commonly used to achieve this objective as described in the following paragraphs.

Changing the type of traffic control (for example, transitioning from STOP control to signalization or to a roundabout) may add motor vehicular capacity at intersections. At intersections already signalized, more capacity may be gained from replacing fixed-time signal control with motor vehicle, bicycle and pedestrian-actuated control.

Auxiliary left-turn and right-turn lanes (see **Section 8.8**) increase intersection capacity by removing slowing or stopped vehicles from lanes otherwise usable by through traffic. Auxiliary through lanes can be appropriate at isolated signalized intersections and increase intersection capacity. However, the length of the auxiliary lanes for the receiving leg will determine the ability of this extra through traffic to merge. If auxiliary lanes are too short, they may congest the intersection and block the minor street traffic, and fail to reduce delay.

The designer should also note that adding auxiliary lanes increases the crossing distance for pedestrians. The designer should ensure that the level of service increases provided for motor vehicles do not result in large degradations in LOS for other users. Where widening to provide auxiliary lanes is planned, the designer should consider crossing islands and other features to ensure the ability for pedestrians to cross.

At roundabouts, capacity can be increased by an additional approach lane and a corresponding section of additional circulating lane.

Adding parallel links of street network may reduce traffic volumes at an intersection, thereby eliminating or postponing the need to increase its capacity.

5.4.5.3 PEDESTRIAN LEVEL OF SERVICE

Pedestrian level of service is defined by the delay experienced by the pedestrian at the intersection. **Table 5.18** summarizes pedestrian level of service for signalized and unsignalized intersections, and roundabouts. The table also summarizes, for the various levels of service, the propensity for pedestrians to engage in unsafe crossing behaviour by accepting dangerously small gaps in traffic for crossing, or ignoring traffic signal indications.

Table 5.18 Pedestrian Level of Service (LOS) Criteria at Intersections

LOS	Average Delay to Pedestrian (seconds)			Likelihood of Risk Taking Behaviour
	Unsignalized intersections	Signalised intersections	Roundabouts	
A	d ≤ 5	d ≤10	d ≤ 5	Low
B	5 < d ≤ 10	10 < d ≤ 20	5 < d ≤ 10	
C	10 < d ≤ 20	20 < d ≤ 30	10 < d ≤ 20	Moderate
D	20 < d ≤ 30	30 < d ≤ 40	20 < d ≤ 30	
E	30 < d ≤ 45	40 < d ≤ 60	30 < d ≤ 45	High
F	45 < d	60 < d	45 < d	

Source: Highway Capacity Manual, 2000

At unsignalized intersections, the delay in crossing the major road (i.e., approaches not controlled by STOP control) is the time needed for pedestrians to receive a gap in traffic adequate to cross safely. Gaps are, in turn, related to the volume of traffic and the likelihood of driver's yielding the right of way to a pedestrian in the crosswalk. Pedestrians crossing STOP controlled or YIELD controlled approaches do not have to wait for a gap in traffic, but wait for the first vehicle in line to yield right of way. Pedestrian crossings across STOP controlled or YIELD controlled approaches are likely to have a significantly better level of service than crossings at the uncontrolled approaches.

At signalized intersections, the delay to pedestrians is that time spent waiting for the next signal phase permitting safe crossing. Where pedestrian indications are present, this signal phase begins with the WALK display. Where pedestrian indications are not present, the signal phase permitting crossing begins with the red signal indication on the intersection approach to be crossed.

The average delay to pedestrians (i.e., the average time spent waiting for the next signal phase permitting safe crossing) is less than one-half the total signal cycle length. Typically, these cycle lengths are 60 to 90 seconds, resulting in pedestrian delay of 30 to 45 seconds. Longer signal cycles, such as the 120-180 second cycles on major arterials, result in corresponding higher delays (60-90 seconds respectively) for pedestrians. Typically, short signal cycle lengths, therefore, provide better pedestrian level of service than long cycle lengths.

At roundabouts, pedestrians may walk further than at a signalized intersection due to the diameter of the circulating roadway. However, pedestrians cross only a single lane of traffic at a time, taking refuge in the splitter island. Actual delay is likely to be comparable or less than at a normally situated crosswalk.

5.4.5.4 BICYCLE LEVEL OF SERVICE

Where there is no bicycle lane or shoulder being used by bicyclists, bicycles are considered to be part of the stream of vehicular traffic and they experience the same control delay that would accrue to a motor vehicle in their position in traffic. For roads without bicycle lanes or shoulders, therefore, the bicycle level of service is computed the same as for motor vehicles.

Bicyclists in their lane (or shoulder) "bypass" stopped motor vehicles, and therefore seldom experience delay due to queuing. Delay due to queuing of bicycles is a factor only with extraordinary volumes. Therefore, for bicyclists in bicycle lanes or shoulders at signalized intersections, the average delay can be estimated as one-half of the signal red and yellow time facing that approach. This reflects bicycle arrivals at random, with average delay therefore one-half of the maximum. Level of service for bicycles at signalized intersections is summarized in **Table 5.19**.

Table 5.19 Bicycle Level of Service (LOS) Criteria at Signalized Intersections

LOS	Average Delay to Bicyclist (seconds)
A	$d \leq 10$
B	$10 < d \leq 20$
C	$20 < d \leq 30$
D	$30 < d \leq 40$
E	$40 < d \leq 60$
F	$60 < d$

Source: Highway Capacity Manual, 2000

Delay can be estimated as 0.5 (red and yellow signal time) on bicyclist's approach.

Bicyclists can experience substantial delay at intersections when they are not detected by the traffic signal system. This failure to be detected may result in longer waits for a green signal, inability to obtain a green arrow for a left turn, or a decision to proceed on red.

At unsignalized locations, bicycles on the major street are not likely to be delayed because they have priority over minor street vehicles. Bicyclists crossing or entering the major street from a STOP controlled minor street are delayed by the amount of time required to find an acceptable gap. Field measurement of this time, during peak as well as off-peak periods, is the preferred method of establishing this delay.

At roundabouts, bicycles generally experience the same delays as motor vehicles as they “take the lane” in approaching the circulating roadway.

5.5 CALCULATION OF DESIGN STANDARD TRAFFIC VOLUME

The design standard traffic volume is calculated upon considering the potential traffic capacity, level of service (LOS), hourly distribution rate (K factor), Directional distribution rate (D factor), and correction for intersections with traffic signals. **Figure 5.33** shows the flowchart for determining the design standard traffic volume.

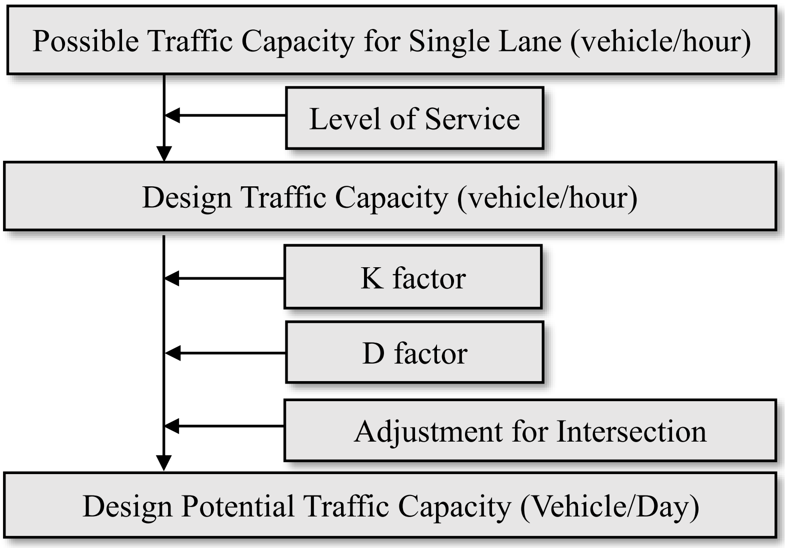


Figure 5.33 Design standard traffic volume

5.5.1 LEVEL OF SERVICE

Level of Service is introduced to denote the level of quality one can derive from a road under different operation characteristics and traffic volume. The level of service is established with a view to preserving traffic in the planned target year at a certain level or higher. This is classified into the three types as given in **Table 5.20**.

Table 5.20 Coefficient of level of service

Design Class	Flat, Hilly Mountainous	Urban
A	0.75	0.80
B, C, D, E	0.85	0.90
Special value	1.00	1.00

The level of service is used when seeking the design traffic capacity from the potential traffic capacity, and it needs to be applied upon considering the character and importance of the road.

5.5.2 K FACTOR

K factor is hourly distribution ratio of the 30th hour traffic volume in relation to the annual average daily traffic volume (AADT). The K factor observed from an existing road is expressed as a percentage and is usually around 10%, although it may vary between 7 - 25% depending on the design class and area characteristics. In road design, the K factor has conventionally been used for referring to the design hourly traffic volume (30th hour traffic volume) in relation to the design traffic volume (annual average daily traffic volume) of the project road. It is desirable to set it appropriately based on the traffic volume data for the route and local area, however, the standard values in **Table 5.21** are used if such data cannot be obtained.

Table 5.21 Standard value of K factor

Design Class	Urban	Rural	
		Flat	Hilly, Mountainous
A1	—	12%	14%
A2	9%	—	—
B1, C1	—	12%	14%
D1, E (rural)	—	12%	14%
B2, C2, D2, E (urban)	9%	—	—

5.5.3 D FACTOR

D Factor is directional distribution ratio of one directional traffic volume in relation to the outbound and inbound total hourly traffic volume. The directional split refers to the traffic volume of a road in one direction that may be greater than in the other direction during any particular hour on a road. In road planning and design, the D factor is used when seeking the design hourly traffic volume. This was originally calculated from the 30th hour traffic volume. However it is commonly sought from the peak hourly value on an average day. It is desirable to set the D factor appropriately based on the traffic volume on the road and in the local area, however, if sufficient data cannot be obtained, a standard value of 60% is used.

5.5.4 CORRECTION FOR SIGNALIZED INTERSECTIONS

The correction for signalized intersections is used to adjust the design standard traffic volume used for deciding the number of lanes in consideration of the impact of traffic signals. The correction coefficient is set based on the ratio of time that traffic signals are on green. It is desirable to decide the correction for signalized intersections according to the green light ratio of traffic signals on the target road and in the local area, however, if sufficient data cannot be obtained, a standard value of 0.8 on 2-lane roads and 0.6 on roads with more than 4 lanes is adopted as the correction for signalized intersections.

5.6 NUMBER OF LANES

5.6.1 DETERMINATION OF NUMBER OF LANES

When deciding the number of lanes, it is desirable to seek the road's traffic capacity and determine the number according to the ratio of this and the design traffic volume. However, because the traffic capacity changes according to the lane width, marginal spacing (distance from the edge of carriageway to retaining walls, protective walls and other obstructions on the roadside), and roadside conditions (degree of urbanization and terrain), and may even differ within the same design section as shown in **Figure 5.34**, this method is not the general approach. It is determined from the design standard traffic volume, which is obtained by assuming the standard road geometry and traffic conditions.

The design standard traffic volume signifies the road traffic volume. The design standard traffic volume per lane signifies the traffic volume used as standard for deciding the number of lanes on a road. To obtain this, the traffic capacity, which is calculated based on the standard envisaged structural conditions (especially width composition and gradient) and traffic conditions according to the design class and terrain is converted into a daily unit. Also, the level of service regarding the traffic volume, economy of road construction, and various administrative judgments are taken into account.

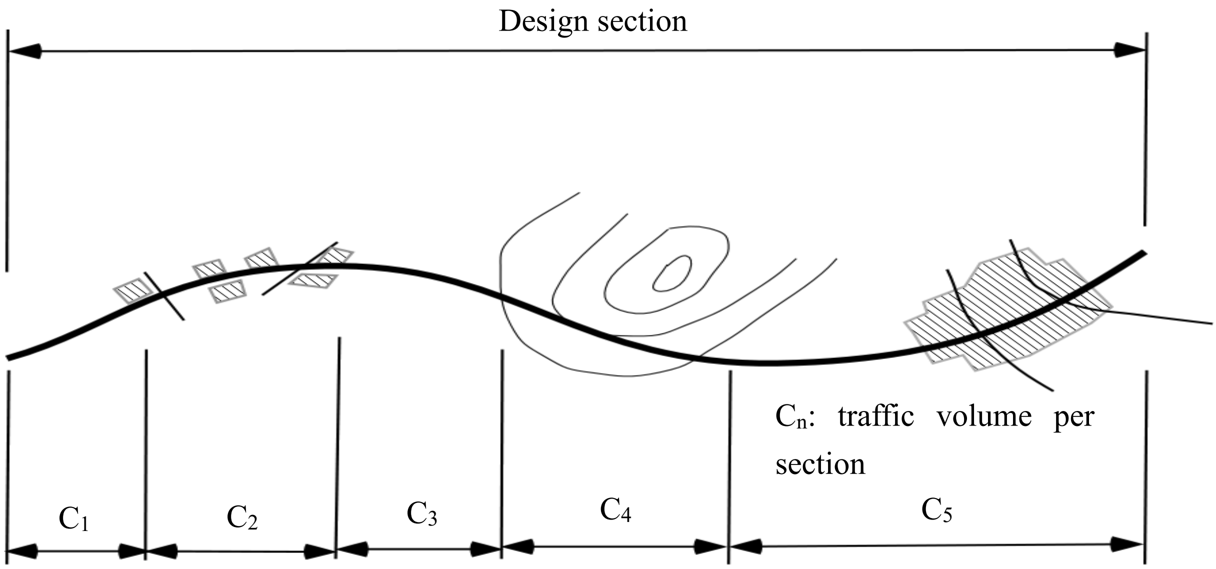


Figure 5.34 Fluctuations in traffic capacity according to roadside conditions

5.6.2 PROCEDURE FOR DETERMINING THE NUMBER OF LANES

The number of lanes is determined according to the following procedure:

i. The relationship between design standard traffic volume and design traffic volume corresponding to the class of the road. Considering that both the design standard traffic volume and design traffic volume are rough values, there is a certain degree of flexibility in deciding this, so it is permissible to discard fractions when setting the number of lanes. For roads passing through urbanised areas with 4 or more lanes having 2 or more intersections per kilometre, the design standard traffic volume per lane given in **Table 5.22** is multiplied by 0.6.

ii. Basically, the number of lanes should be an even number of 2 or more (2, 4, 6, 8, etc.). However, odd numbers of 5 or more may be adopted in cases where the need arises due to differences in the outbound and inbound traffic volume etc. Moreover, in cities, on ring roads etc., one-way roads having 2 or more lanes (2, 3, 4, 5, etc.) may sometimes be adopted without having lanes going in the opposite direction.

5.6.3 VERIFICATION OF THE NUMBER OF LANES

When determining the number of lanes according to area and route characteristics, it is necessary to implement verification of the hourly traffic volume that takes the route's peak characteristics (peak coefficient), direction characteristics (heavy direction rate) and large vehicle mixing rate into account and strive to decide the appropriate number of lanes. In particular, when reconstructing existing roads that have large fluctuations in traffic volume according to the month, day and hour, it is necessary to grasp the actual traffic volume situation in relation to these fluctuation characteristics.

In determining the number of lanes of a road, the design traffic volume (daily traffic volume) is used, while also taking the daily design standard traffic volume into account. For the design traffic volume, it is common to use the annual average daily traffic volume. However, there may be cases where the number of lanes obtained in verification based on the hourly traffic volume does not match. In such cases, for example, it is necessary to decide the appropriate number of lanes by conducting a renewed examination concerning whether to use the annual average as the design traffic volume.

5.6.4 DESIGN STANDARD TRAFFIC VOLUME

The number of lanes, which is an important element of a road cross section, is determined by the ratio between the design standard traffic volume, which is a daily unit established assuming the standard road geometry and traffic conditions, and the design traffic volume. The design standard traffic volume signifies the road traffic volume, which is the traffic capacity converted to a daily unit according to each design class. Accordingly, the values in **Table 5.22** are the standard values for deciding the number of lanes, but it is not appropriate to use it for other purposes. **Table 5.22** shows the design standard traffic volume per lane.

Table 5.22 Design standard traffic volume per lane

Design Class		Terrain	With median separator Design standard traffic volume per lane (veh/day)	Without median separator Design standard traffic volume per lane (veh/day)
A	A1 - Rural Expressway/Motorway A2 - Urban Expressway/Motorway	Flat	14,000	-
		Hilly	10,000	-
		Mountainous	8,000	-
		Urban	16,000	-
B	B1 - Rural Major Arterial B2 - Urban Major Arterial	Flat	10,000	7000
		Hilly	5,000	5000
		Mountainous	3,000	3000
		Urban	10,000	7000
C	C1 - Rural Minor Arterial C2 - Urban Minor Arterial	Flat	3,000	3,000
		Hilly	2,000	2,000
		Mountainous	1,000	1,000
		Urban	3,000	3,000
D	D1 - Rural Collector D2 - Urban Collector	Flat	1,000	1,000
		Hilly	1,000	1,000
		Mountainous	300	300
		Urban	1,000	1,000
E	E-Local/Access	Urban	300	300

Note: For urban roads, which have a lot of intersections, the design standard traffic volume per lane given in this table is multiplied by 0.6.

5.7 SPEED

Speed is one of the most important factors considered by travellers in selecting alternative routes or transportation modes. Travellers assess the value of a transportation facility in moving people and goods by its reliability, convenience, and economy, which are generally related to its speed.

The attractiveness of a public transportation system or a new roadway are each weighed by the travellers in terms of time, convenience, and money saved. Hence, the desirability of rapid transit may well rest with how rapid it actually is. In addition to driver and vehicle capabilities, the speed of vehicles on a road depends on five general conditions:

i. physical characteristics of the roadway,
ii. amount of roadside interference,
iii. weather,
iv. presence of other vehicles, and
v. speed limitations (established either by law or by traffic control devices).

Although any one of these factors may govern travel speed, the actual travel speed on a facility usually reflects a combination of these factors.

The objective in design of any engineered facility used by the public is to satisfy the public's demand for service in an economical manner, with efficient traffic operations and with low crash frequency and severity. The facility should, therefore, accommodate nearly all demands with reasonable adequacy and also should only fail under severe or extreme traffic demands. Because only a small percentage of drivers travel at extremely high speed, it is not economically practical to design for them. They can use the roadway, of course, but will be constrained to travel at speeds less than they consider desirable. On the other hand, the speed chosen for design should not be that used by drivers under unfavourable conditions, such as inclement weather, because the roadway would then be inefficient, might result in additional crashes under favourable conditions, and would not satisfy reasonable public expectations for the facility.

There are important differences between design criteria applicable to low- and high-speed designs. To implement these differences, the upper limit for low-speed design is 70 km/h and the lower limit for high-speed design is 80 km/h.

5.7.1 OPERATING SPEED

Operating speed is the speed at which drivers are observed operating their vehicles during free-flow conditions. The 85th percentile of the distribution of observed speeds is the most frequently used measure of the operating speed associated with a particular location or geometric feature.

The following geometric design and traffic demand features may have direct impacts on operating speed:

i. horizontal curve radius,
ii. grade,
iii. access density,
iv. median treatments,
v. on-street parking,

vi. signal density,
vii. vehicular traffic volume, and
viii. pedestrian and bicycle activity.

5.7.2 RUNNING SPEED

The speed at which an individual vehicle travels over a highway section is known as its running speed. The running speed is the length of the highway section divided by the time for a typical vehicle to travel through the section. For extended sections of roadway that include multiple roadway types, the average running speed for all vehicles is the most appropriate speed measure for evaluating level of service and road user costs. The average running speed is the sum of the distances travelled by vehicles on a highway section during a specified time period divided by the sum of their travel times.

5.7.3 POSTED SPEED

Posted speed is a speed limitation set for reasons of safe traffic operations rather than for geometric design considerations and is aimed at encouraging drivers to travel at appropriate speeds for all prevailing conditions.

When determining the most appropriate speed limit for a length of road a number of factors are taken into consideration, in the context of a Safe System. Such factors include:

i. the crash history of the road
ii. the function of the road (e.g., arterial, residential and residential collector)
iii. the level of pedestrian and cyclist activity
iv. traffic volume and composition
v. type of abutting development (e.g., commercial, residential, industrial, rural and semi-rural)
vi. the geometry of the road (e.g., divided or undivided, number of lanes, presence of sealed shoulders, and vertical and horizontal road alignments).

5.7.4 DESIRED SPEED

The term ‘desired speed’ in this guide refers to the speed that drivers want to operate at and is a fundamental component of the Operating Speed Model. The desired speed for a section of road is influenced by the following elements:

i. roadside environment – topography in rural areas, development density and type (i.e. built environments) in urban areas
ii. road characteristics – geometric standard (predominately horizontal alignment; to a lesser extent, vertical alignment; and lane widths), frequency of intersections and

accesses, sight distance, parking provisions etc.

iii. speed limit

iv. road function – to the extent that on important roads such as motorways and highways, drivers are less willing to accept reductions in desired speed.

On intermediate and low speed rural roads, the principal factors controlling the desired speed are topography and horizontal alignment. All other factors listed above only have a secondary effect.

On motorways and rural roads with a high geometric standard, speed limit primarily influences the desired speed, particularly if regular enforcement is provided. On urban arterial roads, the primary factors influencing the desired speed are roadside environment and speed limit, since these will usually limit desired speed before the standard of the horizontal alignment.

If a driver is impeded in any way, such as having to slow for another driver or to travel around a ‘tight’ curve, they will quickly accelerate back to the desired speed once the impediment is removed/passed.

Desired speed is equal to the speed that drivers will adopt on less constrained alignment elements (i.e., longer straights and large radius horizontal curves) of a reasonably uniform section of road when not constrained by other vehicles. On high standard roads where speed is expected to be uniform, desired speed will often equal operating speed. On roads where operating speeds are expected to be variable, desired speed will often be higher than operating speed.

Desired speed does not vary over a section of roadway that has a similar roadside environment, road characteristics and speed limit but the section may still have isolated geometric features inconsistent with the desired speed. Isolated geometric features can include the following:

i. a ‘tight’ horizontal curve (or a short section of road containing a few ‘tight’ curves)

ii. an intersection controlled by a stop or give-way sign

iii. a roundabout

iv. an overtaking lane/climbing lane.

In the case of an overtaking lane, it is possible that the operating speed of the overtaking lane will exceed the desired speed for the section of road.

If the roadside environment, road characteristics and speed limit of the roadway are similar before and after the isolated feature, the desired speed will remain the same. The desired speed will only change if the roadside environment or road characteristics change over a significant length of roadway.

Typically, reductions in desired speed take longer to come into effect than increases in desired speed.

Therefore, on two-way roads, there may be locations where the desired speed is different for each direction.

5.7.5 DESIGN SPEED

Design Speed is a selected speed used to determine the various geometric design features of the roadway (AASHTO). The design speed is the speed at which drivers with average skill can drive safely and pleasantly in conditions of good weather and low traffic density and without any other constraints. If the alignment is a straight line, it is possible to run safely more than the design speed. However, drivers generally select their speed according to conditions of intersections, parked vehicles, road congestion, and presence of pedestrians and bicycles.

Design speed is closely related to the traffic volume that can be handled by the road, the required speed differs according to the type and grade of road. Moreover, from the viewpoint of vehicle traffic safety and smoothness, speed is closely related to alignment elements (curve radius, gradient, sight distance, etc.). Accordingly, in the Road Design Guide, design speed is stipulated according to each type and grade as a basic condition necessary for the road design. The values shown in **Table 5.23** shall be adopted as road design speed according to each design class.

Table 5.23 Design speed

Design Class	Terrain	Design Speed (km/h)	
		(Desirable)	(Absolute)
A1/A2	Flat	120	100
	Hilly	100	80
	Mountainous	80	60
B1/B2	Flat	100	80
	Hilly	80	60
	Mountainous	60	50
C1/C2	Flat	80	60
	Hilly	60	40
	Mountainous	50	30
D1/D2	Flat	60	40
	Hilly	50	30
	Mountainous	40	20
E	-	40	20

Where it is difficult to use the desirable design speed for maximum safety of road users, the absolute values are recommended. Cases where the absolute values may be used are given as follows;

i. Mountainous areas that are subject to topographical constraints.
ii. Urban areas that are subject to roadside constraints.

It should be noted that the design speed differs from traffic control speed settings and other actual operating speeds.

5.7.5.1 SECTIONAL DESIGN

A route may have sections with different characteristics, such as terrain, traffic volume and importance. It is recommended the designer adopts the same design speed for sections with similar characteristics to ensure a more harmonious alignment.

5.7.5.1.1 MINIMUM LENGTHS FOR SECTIONAL DESIGN

Geometric design of a route has to be as continuous as possible, therefore selection of lengths and change points have to be considered wisely in sectional designs. **Table 5.24** gives the minimum lengths for sectional design.

Table 5.24 Minimum lengths for sectional design

Design Class	Desirable (km)	Unavoidable case (km)
A1, A2, B1, C1	20 - 30	5
D1, E	10 - 15	2
B2, C2, D2	From principal intersection to the next principal intersection	

5.7.5.1.2 CONNECTION OF SECTIONAL DESIGNS

To achieve a good connection between different design sections, the difference in design speeds must not exceed 20 km/h. **Figure 5.35** shows an example of connection of sectional design.

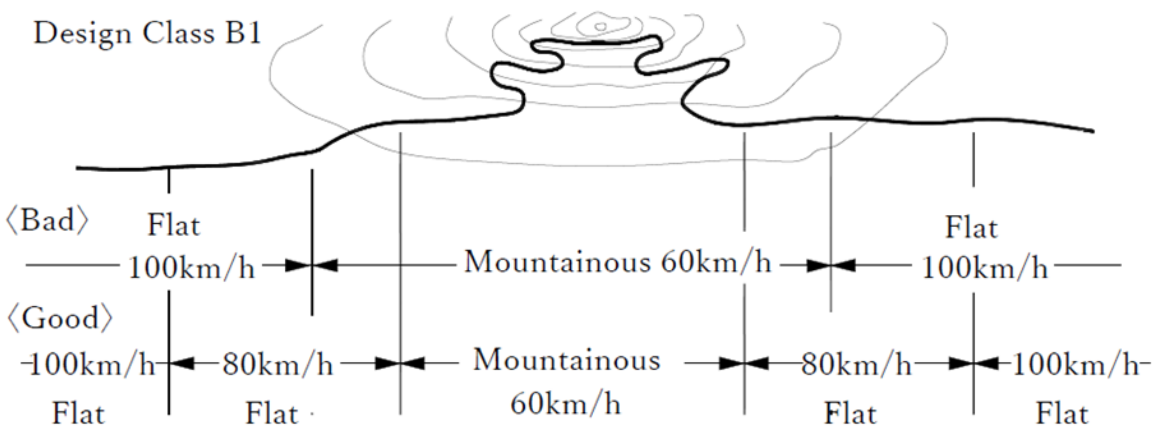


Figure 5.35 Example of connection of sectional design

5.7.5.1.3 CHANGE POINTS OF SECTIONAL DESIGNS

The change points of sectional designs are usually determined by features such as the change in topography, principal intersection, long bridge (>200m) and other features that will change traffic volume over sections of roads.

5.8 ACCESS CONTROL

To preserve the operational efficiency and safety of major roads, access control becomes important. Unrestricted accesses to developments along major roads lead to increased accident and reduced capacity.

On motorway/expressway, complete entry and exit restriction shall be implemented to secure continuity of travel. On other roads, partial entry and exit restriction shall be implemented depending on the character of the route, conditions of vehicular traffic etc.

When vehicles travel for long distances, it is desirable that they can maintain a certain high speed from the perspectives of safety, smoothness and comfort. Also, when encountering large volumes of vehicular traffic, it is desirable to limit the interference of traffic flow due to occurrence of stops, left turns and crossing vehicles at intersections as much as possible.

Entry and exit restrictions are methods of enhancing traffic functions on roads where emphasis is placed on the continuity of travel. There are three (3) types of entry and exit restrictions and these should be examined according to the character of the road. They are:

i. Full Access Control
ii. Partial Access Control
iii. No Access Control

5.8.1 FULL ACCESS CONTROL

This is adopted on roads that are dedicated to mobility functions and have completely restricted entry and exit. The restriction is realized through adopting grade-separated intersections with other roads and permitting entry and exit only from limited access roads as shown in **Figure 5.36**. In full access control, service roads are sometimes adopted. Such service roads are intended to concentrate services and crossing structures to the roadside separate from the main road and to direct traffic from roadside areas to specified grade-separated intersections, thereby making it possible to reduce the number of grade-separated intersections with other major roads. **Figure 5.37** shows full access control with service roads. In the case of full access control, median strip openings should not be provided.

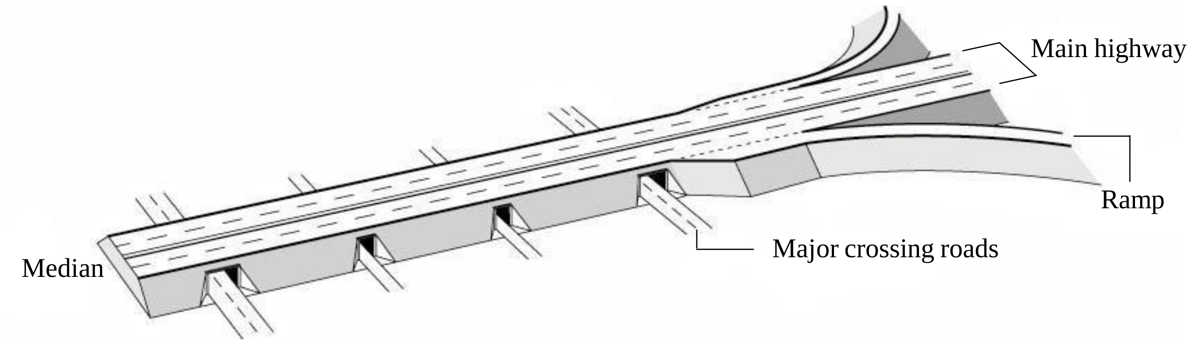


Figure 5.36 Full access control

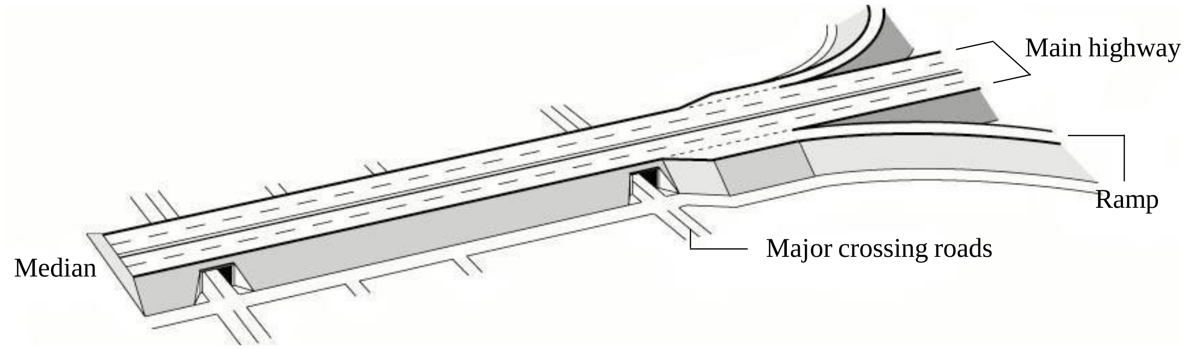


Figure 5.37 Full access control (with service road)

5.8.2 PARTIAL ACCESS CONTROL

Partial access control is adopted on roads with mobility functions where entry and exit are partially restricted. In the partial access control cases shown in **Figure 5.38** and **Figure 5.39**, a road having partial connection with residential area roads, and a road having partial at-grade intersections are shown.

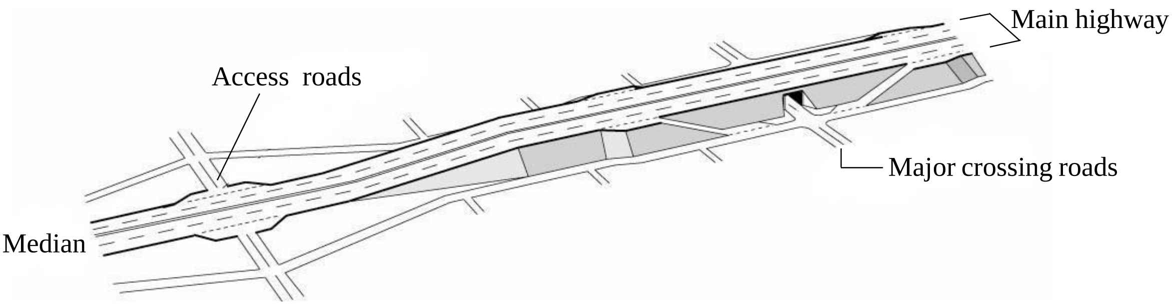


Figure 5.38 Partial access control (no median strip opening)

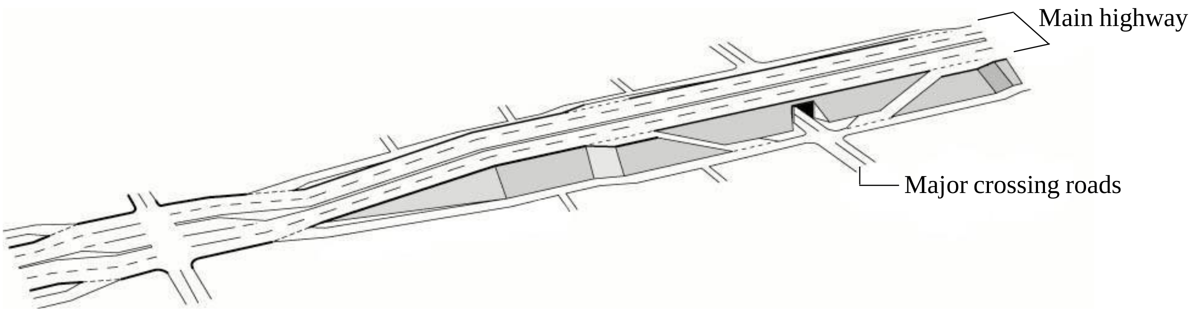


Figure 5.39 Partial access control (with median strip opening)

5.8.3 NO ACCESS CONTROL

No access control is adopted on roads that are not only with mobility functions but also access, stopping and parking functions and pedestrian traffic functions. In such cases, at-grade road connections are permitted not only with the existing roads but also with private roads. **Figure 5.40** and **Figure 5.41** show examples of No Access control roads

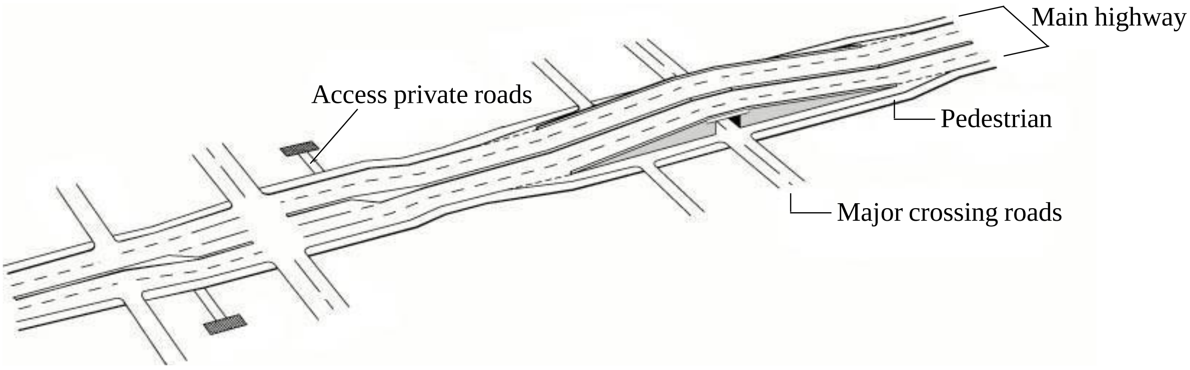


Figure 5.40 No access control (grade separated intersection with the main highway)

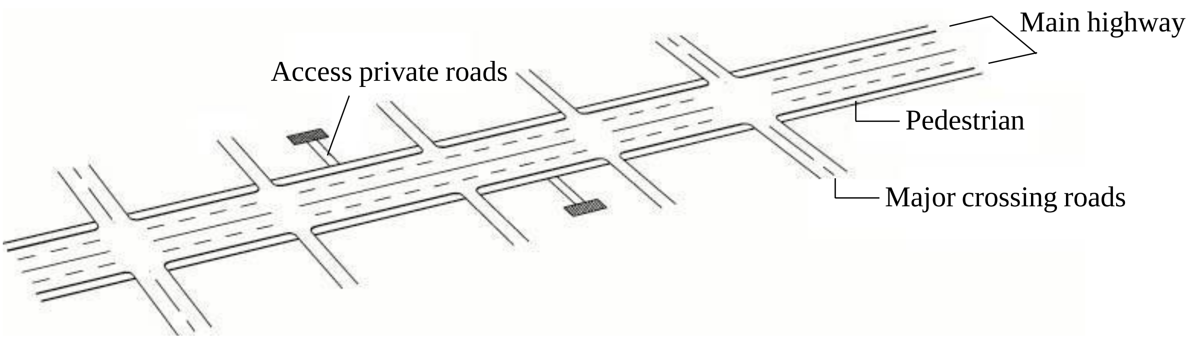


Figure 5.41 No access control (with at-grade intersections)

5.8.4 ACCESS CONTROL BASED ON DESIGN CLASS

Access control is determined according to the character and importance of the road. Access control by design class is shown in **Table 5.25**.

Table 5.25 Access control by design class

Design class	Access control
A1/A2	Full
B1/B2	Partial
C1/C2	Partial/Not required
D1/D2/E	Not required

i. **Class A1 (Rural motorway/expressway)**
Grade-separated intersections are adopted with other roads to ensure that entry and exit is completely prohibited, however, exceptions may be permitted in cases where the intersecting roads have low traffic volume and other cases that cannot be avoided due to terrain. However, even in such exceptional cases, entry and exit is only permitted with other roads by connecting road, while median strip openings and intersections are not adopted.

ii. **Class A2 (Urban motorway/expressway)**
In cities, grade-separated intersections are adopted for the purpose of smooth traffic flow of large volumes. Entry and exit with other roads are limited to interchanges only.

iii. **Class B and C (Major and minor arterials)**
At crossings with Motorway/Expressway, grade-separated intersections are adopted. Especially on Major Arterials that have large volumes of medium- and long-distance traffic, consideration is given to restricting roadside entry and exit according to demand to ensure safety and smooth traffic flow. Moreover, in cases of Major Arterials Roads passing through urban areas where it is necessary to implement entry and exit restriction, ample consideration should be given to ease of access with roadside houses and convenience for bicycles and pedestrians.

iv. **Class D and E**
At Class D and E, the access control is not required.

5.9 SPECIFICATIONS OF DESIGN VEHICLES

For safety conscious road design, vehicles must be carefully classified into design vehicles. They form the basis of road design because of their various sizes, capacities, and turning radii which have influence on parameters such as lane width, curve widening, sight distance, design of intersection etc.

Vehicle classification usually adopted in the country for design are six, namely:

i. Small vehicle
ii. Medium vehicle
iii. Large Vehicle Type 1
iv. Large Vehicle Type 2
v. Trailer Type 1
vi. Trailer Type 2

Since trailer have large geometric constraints, it would be too expensive and impractical to let them pass on all roads; accordingly, they are only targeted for application on high-specification roads.

To ensure safe and smooth flow of traffic along major roads, design is conducted using the specifications of small vehicles, large vehicle and trailer, while other minor roads are designed using the specifications of small vehicles. The small vehicle specifications are especially used to determine the width composition and sight distance standards, while large vehicle and trailer specifications are used to determine the width composition, curve widening, intersection design, and longitudinal gradient. The specifications of design vehicles are shown in **Table 5.26.** Minimum turning paths for the various design vehicles are shown in **Figure 5.42** to **Figure 5.48**.

The height of large vehicle and trailer is used as an element for determining vertical clearance. Medium vehicle is adopted on minor roads with restrictions on large vehicle and trailer. **Table 5.26** shows how design vehicles affect road design or serve as controls.

Table 5.26 Specifications of design vehicles

Design vehicle	Specifications (m)								Design Class**	Relation between design vehicle and road characteristics					
Name	Symbol*	Length	Width	Height	Overhang		Axial distance	Minimum turning radius		Width composition	Widening of curved section	Design of intersection	Vertical gradient	Sight distance	Clearance
					Front	Rear									
Small Vehicle (Saloon/Sedan)	S-5	4.7	1.7	2.0	0.8	1.2	2.7	6.0	A, B, C	✓				✓	
Medium Vehicle (SUV, van, pick-up)	M-6	6.0	2.0	2.8	1.0	1.3	3.7	7.3	D, E	✓					
Large Vehicle Type 1 (Medium bus, light/medium truck)	L-9	9.1	2.4	4.1	1.2	1.8	6.1	12.7		✓	✓	✓	✓		✓
Large Vehicle Type 2 (Heavy truck/ large Bus)	L-12	12.1	2.5	4.0	1.5	4.0	6.5	12.8(bus) 15.6 (truck)	A, B, C	✓	✓	✓	✓		✓
Trailer Type 1 (Light semi-trailer truck)	T-17	16.5	2.5	4.1	1.3	2.2	Front 4.0 Rear 9.0	13.7		✓	✓	✓	✓		✓
Trailer Type 2 (Heavy semi-trailer truck)	T-21	21.0	2.6	4.1	1.2	1.4	Front 5.9 Rear 12.5	13.7		✓	✓	✓	✓		✓

*The preceding alphabet stands for the design vehicle. The succeeding number is the approximate length (m) of the design vehicle.
**These design classes are to serve as a guide. Traffic survey should be conducted to confirm the design vehicle.

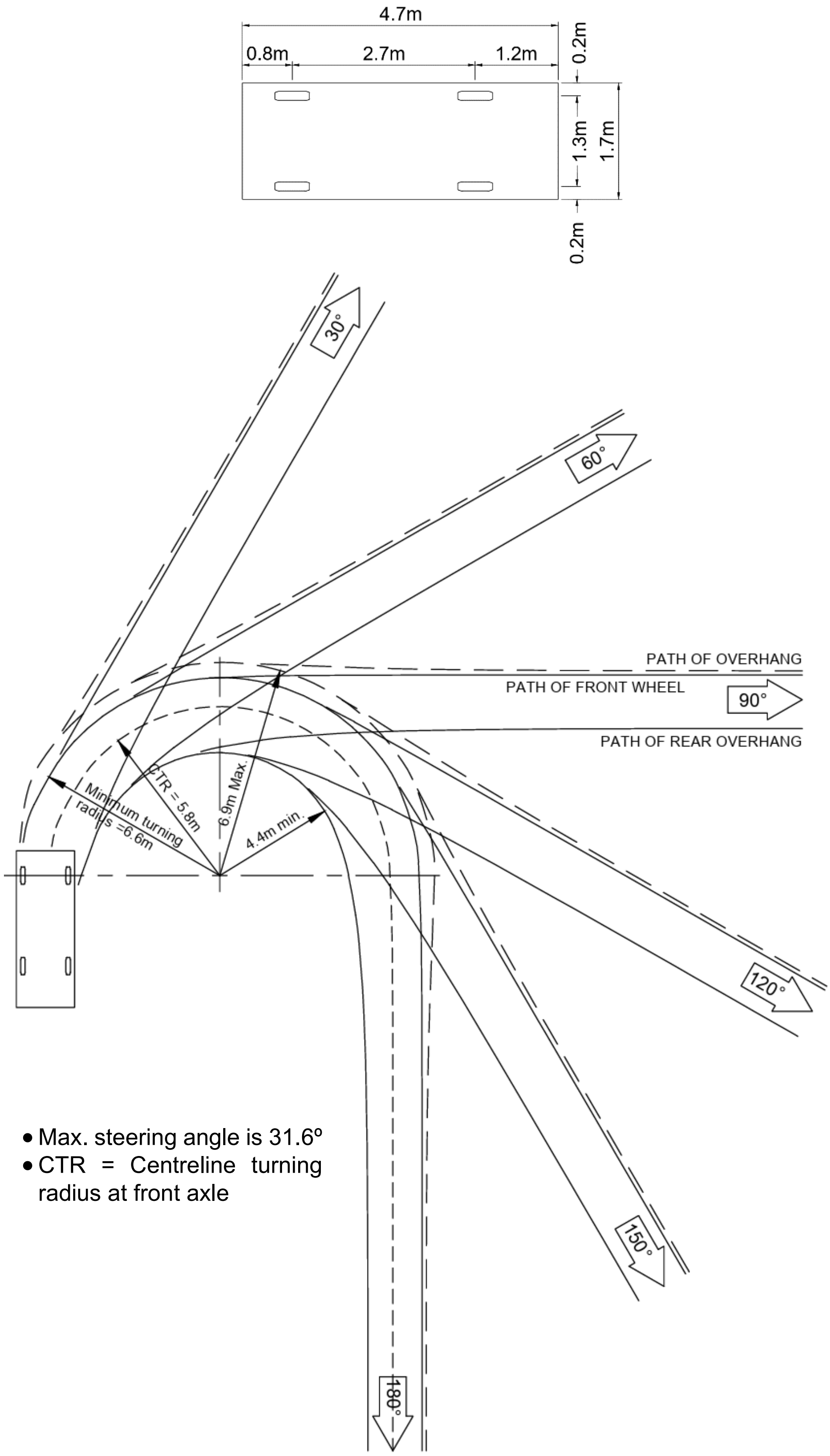


Figure 5.42 Minimum Turning Path for S-5 Design Vehicle

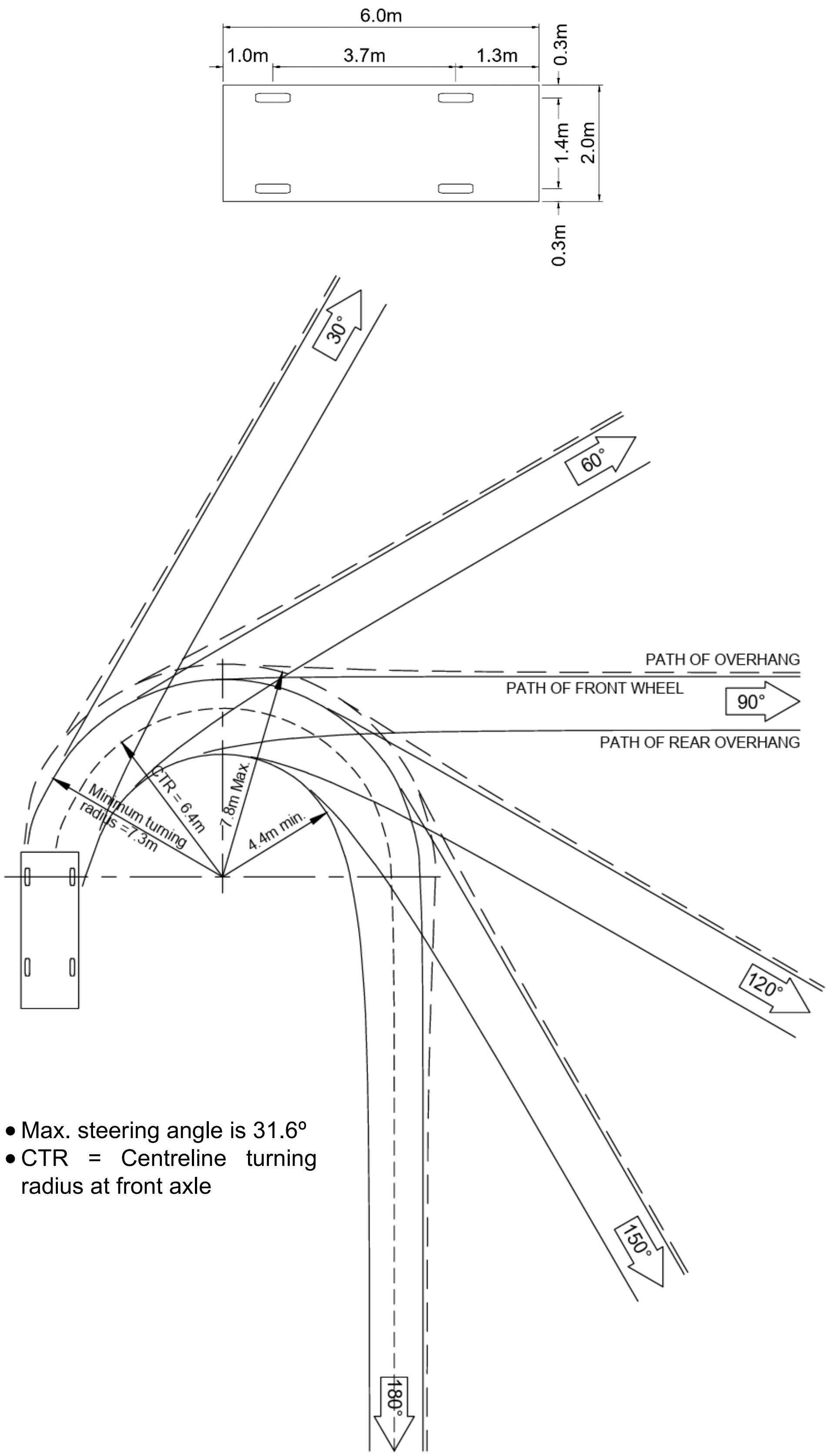


Figure 5.43 Minimum Turning Path for M-6 Design Vehicle

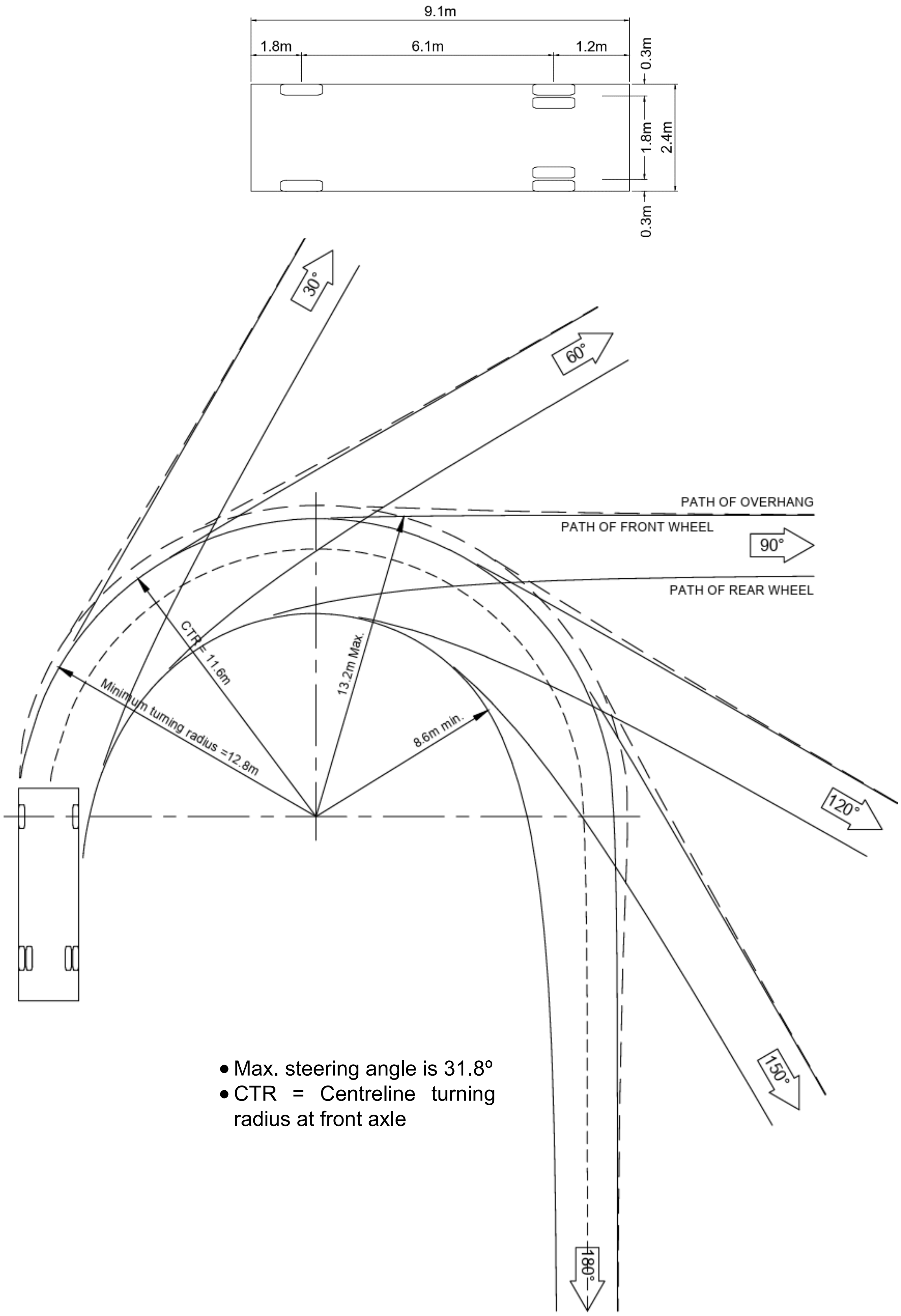


Figure 5.44 Minimum Turning Path for L-9 Design Vehicle

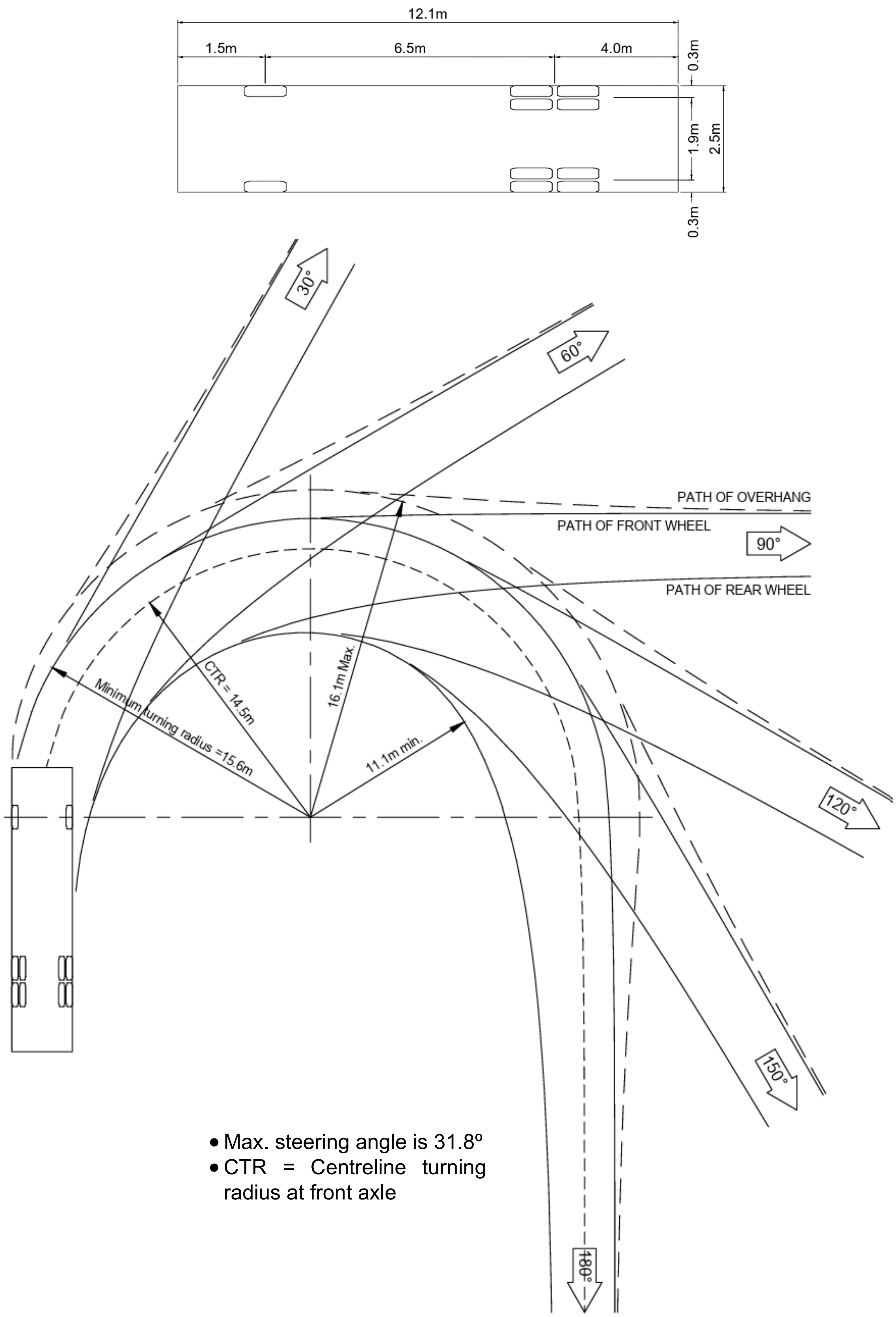


Figure 5.45 Minimum Turning Path for L-12 (truck) Design Vehicle

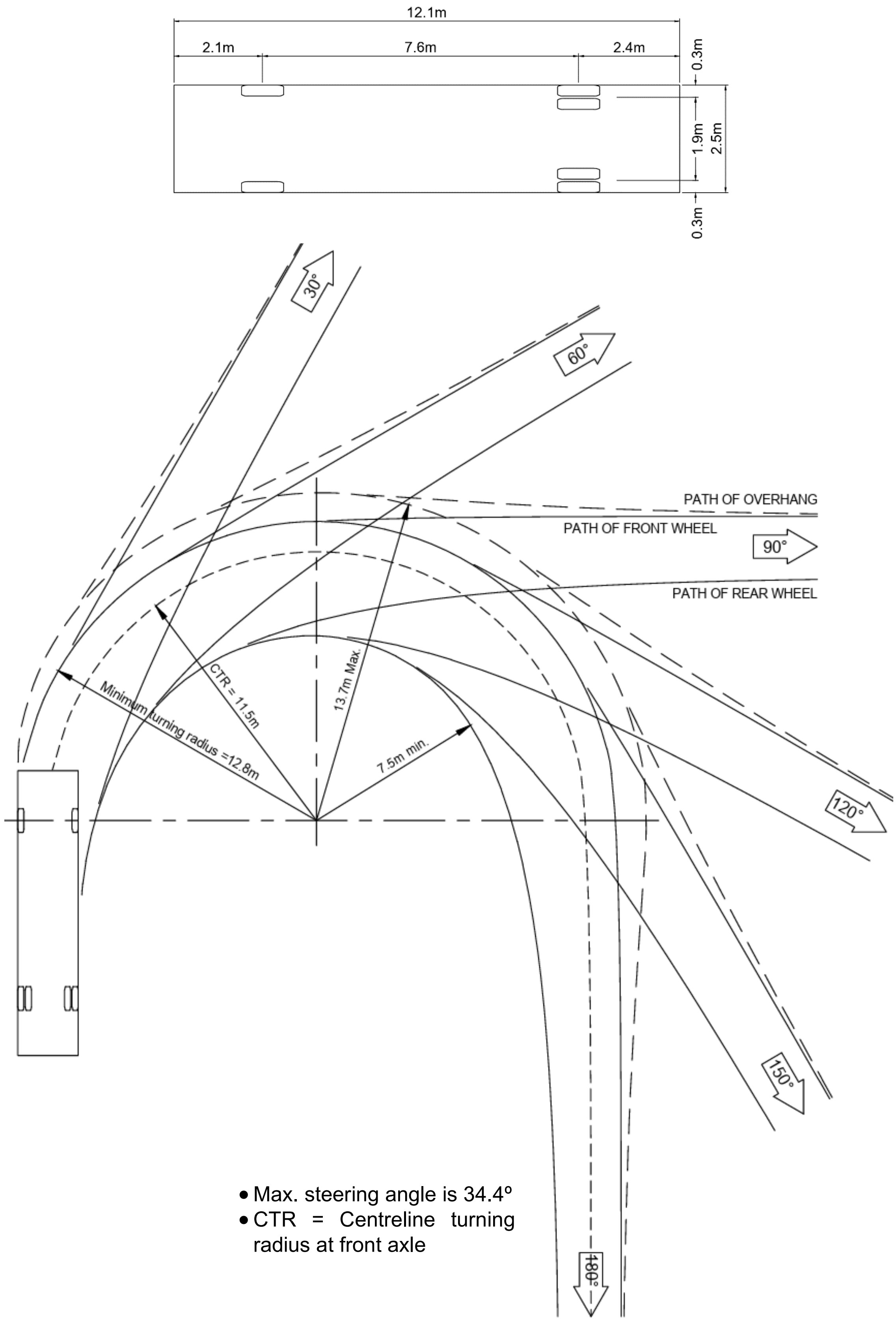


Figure 5.46 Minimum Turning Path for L-12 (bus) Design Vehicle

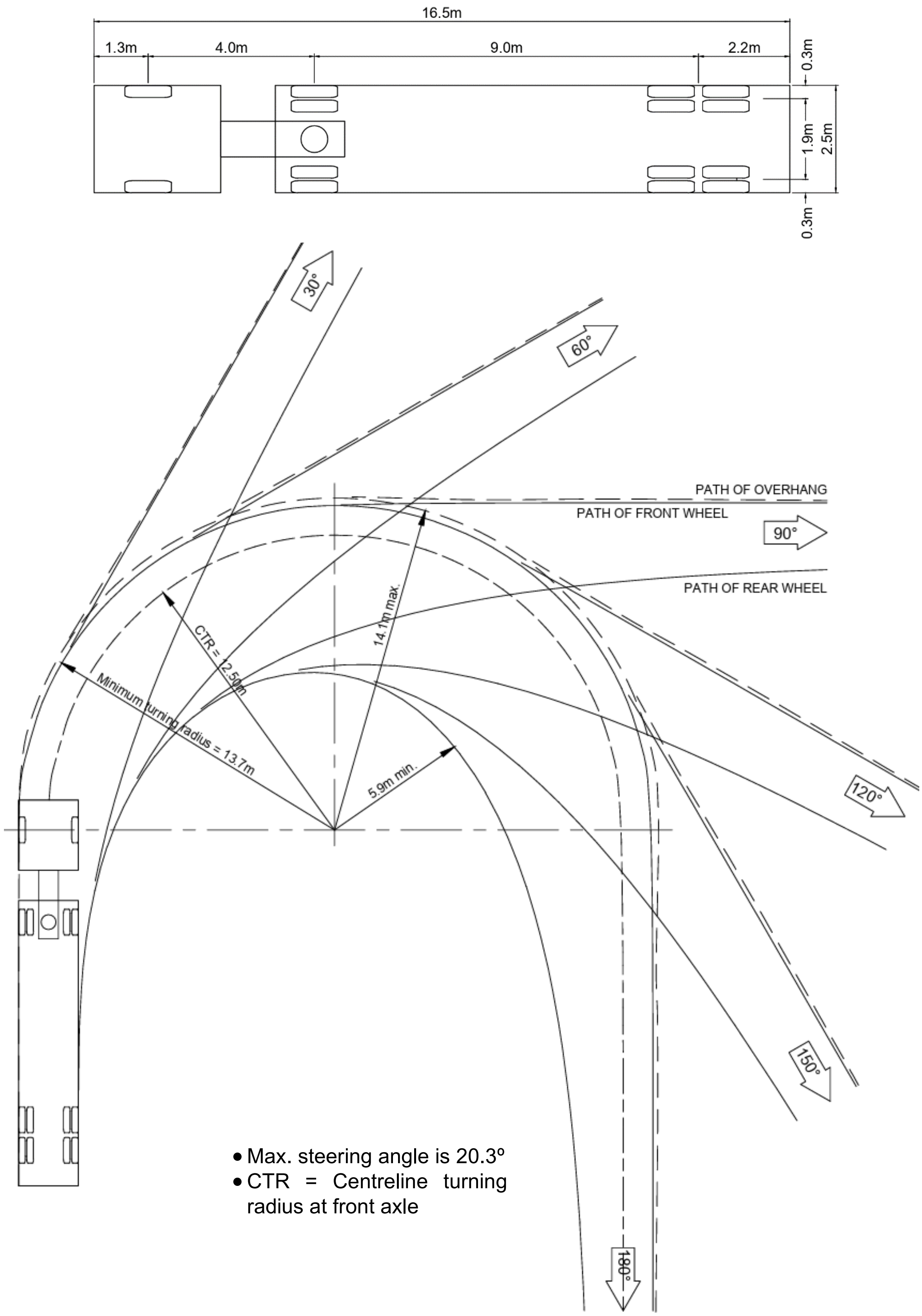


Figure 5.47 Minimum Turning Path for T-17 Design Vehicle

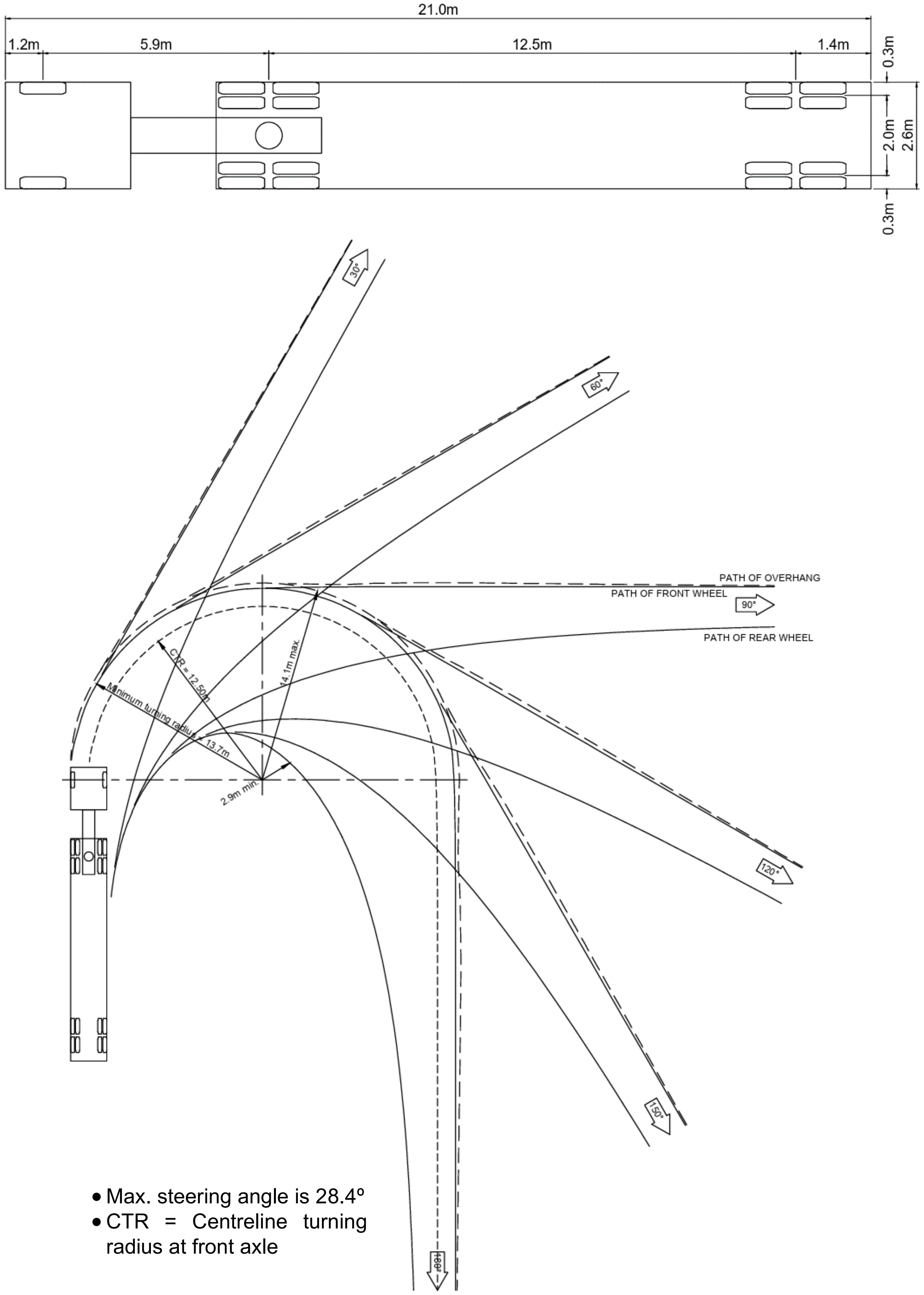


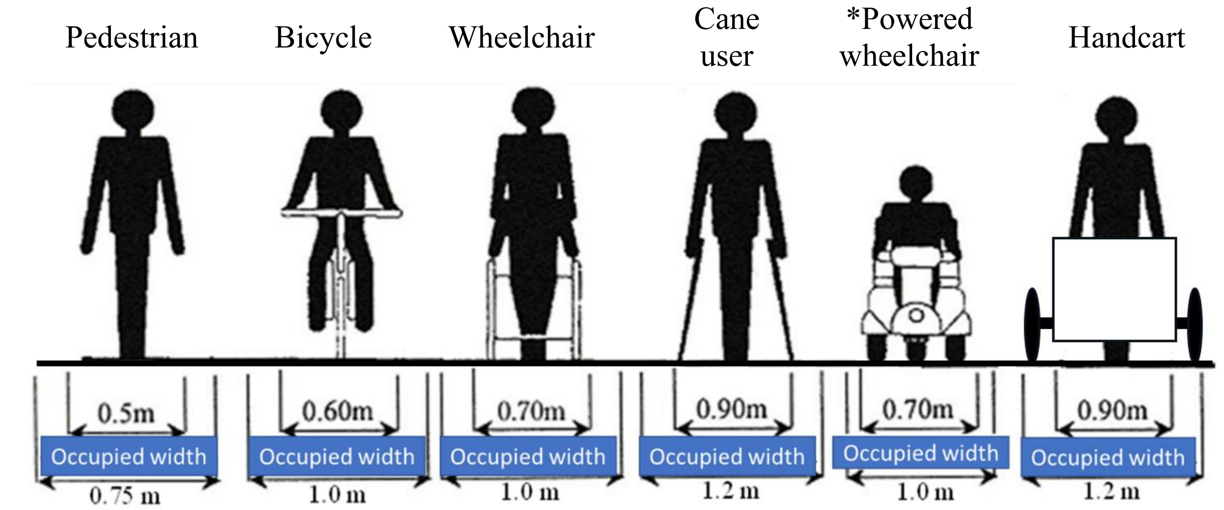
Figure 5.48 Minimum Turning Path for T-21 Design Vehicle

5.10 DESIGN SPECIFICATIONS OF NON-MOTORISED ROAD USERS

In designing walkways and bicycle lane, the occupied widths and other specifications of non-motorised road users are shown in **Table 5.27** and **Figure 5.49**.

Table 5.27 Specifications of non-motorised road users

Non-Motorised Road Users	Specifications (m)			
	Occupied width	Height	Length	Pedal height
Pedestrian	0.75	-	-	-
Bicycle	1.00	2.25	1.90	0.05
Wheelchair	1.00	-	-	-
Cane user	0.90	-	-	-
*Powered wheelchair	0.70	-	-	-
Handcart	1.2	-	-	-



* A person in any type of wheelchair is considered to be a pedestrian and must follow pedestrian rules.

Figure 5.49 Basic dimensions of non-motorised road users

The actual occupied width of a pedestrian is around 0.5m and 0.7m or less for wheelchairs, however, a certain degree of allowance is also added. Similarly, the actual width of a bicycle is around 0.6m, however, since bicycles waver from side to side as they travel, allowance is added. The bicycle height is the head height of a cyclist when riding, so vertical clearance alongside roads used by cyclists must be higher than this. The pedal height is the allowance height for ensuring that pedals do not collide with the road surface.

When planning walkways based on the values stated in the preceding paragraph, in addition to the standard width, it is desirable to consider the behaviour and volume of pedestrian traffic traversing the section.

5.11 REFERENCES

1. World Health Organization/WHO. 2018. Global Status Report on Road Safety. WHO: Geneva
2. Building and Road Research Institute. 2022. Road Traffic Crashes in Ghana Statistics 2021
3. Stigson, H. Krafft, M., and Tingvall, C. 2008. Use of Fatal Real-Life Crashes to Analyze a Safe Road Transport System Model, Including the Road User, the Vehicle, and the Road. Traffic Injury Prevention ,9:5, 463–471
4. Austroads. 2015. Guide to Road Design Part 1, AGRD01-15, Austroads, Sydney, Australia.
5. Austroads. 2019. Guide to Road Safety Part 6: Road Safety Audit, AGRS06-19, Austroads, Sydney, Australia.
6. Stigson, H. 2009. A safe road transport system - factors influencing injury outcome for car occupants. Thesis for doctoral degree. Stockholm, Karolinska Institutet.
7. FHWA. 2002. Context Sensitive Design/Thinking Beyond the Pavement, Federal Highway Administration (www.fhwa.dot.gov/csd)
8. Transport Association of Canada. 1999. Geometric design guide for Canadian roads: parts 1 and 2, TAC, Ottawa, Ontario, Canada
9. Reynolds, C. C. et al. 2009. The impact of transportation infrastructure on bicycling injuries and crashes: a review of the literature. Environmental Health 2009, 8:47.
10. Jurewicz, C. 2017. Innovation and Safe System Road Infrastructure, Proceedings of the 2017 Australasian Road Safety Conference, Perth, Australia
11. https://roadsafety.piarc.org/
12. Damsere-Derry, J., Ebel, B. E., Mock, C. N., Afukaar, F., Donkor, P., and Kalowole, T. O. 2019. Evaluation of the effectiveness of traffic calming measures on vehicle speeds and pedestrian injury severity in Ghana. Traffic Injury Prevention, 20(3), 336–342.
13. Knoblauch R. L., et al. 1988. Investigation of exposure-based pedestrian accident areas: crosswalks, sidewalks, local streets, and major arterials. Washington, DC, Federal Highway Administration
14. Elvik, R. et al. 2009. Handbook of Road Safety Measures, 2nd ed.
15. Duduta, N. et al. 2015. Traffic Safety on Bus Priority Systems, EMBARQ WRI
16. Carrigan, A. et al. 2013. Social, Environmental and Economic Impacts of Bus Rapid Transit, EMBARQ WRI.
17. Dumbaugh, E., and Rae, R. 2009. Safe urban form: revisiting the relationship between community design and traffic safety. Journal of the American Planning Association, 75(3), 309–329.
18. Noland, R. B. 2003. Traffic fatalities and injuries: the effect of changes in infrastructure and other trends. Accident Analysis & Prevention, 35(4), 599–611.

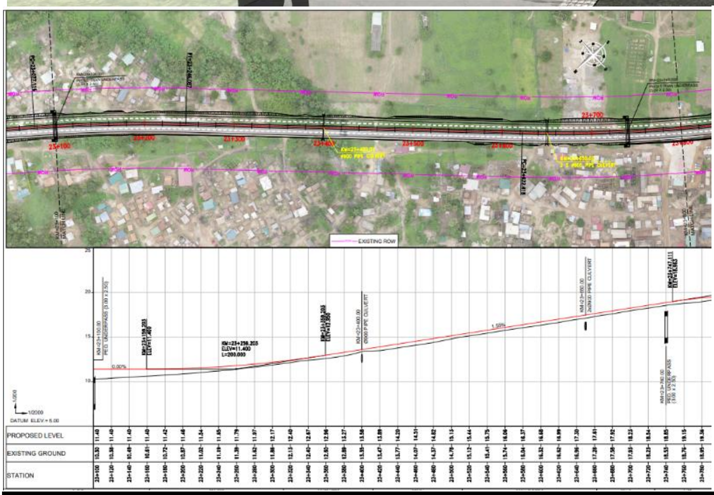
19. Hauer, E. 2000. Safety of horizontal curves. Draft prepared in the course of project for UMA Engineering (for the new Canadian Geometric Design Guide) and for DELCAN.
20. Wolhuter, K. M. 2015. Geometric design of roads handbook. CRC Press.
21. Zegeer, C. V., and Council, F. M. 1995. Safety relationships associated with cross-sectional roadway elements, Transportation Research Record Issue: 1512.
22. Alina Burlacu, Carmen Răcănel, and Adrian Burlacu. 2018. Preventing aquaplaning phenomenon through technical solutions. Građevinar 12/2018. Accessed at http://casopis-gradjevinar.hr/assets/Uploads/JCE-70-2018-12-4-1578-EN.pdf.
23. Olson, P. L., Cleveland, D. E., Fancher, P. S., Kostyniuk, L. P., and Schneider, L.W. 1984. Parameters affecting stopping sight distance, NCHRP Report 270, Transportation Research Board.
24. Federal Highway Administration. 2000. Prediction of the expected safety performance of rural two-lane highways, US. FHWA-RD-99-207
25. Austroads Guide to Road Design. 2016. Part 3 section 7.6.1
26. Austroads Guide to Road Design. 2016. Part 3 section 7.6.1.
27. Espada, I., Stokes, C., Cairney, P., Truong, L., Bennett, P., Tziotis, M., and Tate, F. 2019. Passing Lanes: Safety and Performance (No. AP-R596-19).
28. 85 Austroads. 2018. Research Report AP-R560-18-Towards Safe System Infrastructure: A Compendium of Current Knowledge.
29. Wang, C., Quddus, M. A., and Ison, S. G. 2013. The effect of traffic and road characteristics on road safety: A review and future research direction. Safety Science, 57, 264–275
30. Gothie, M. 1996. Relationship between Surface Characteristics and Accidents, in Proceedings of 3rd International Symposium on Pavement Surface Characteristics, 271–281.
31. AASHTO. 2010. Highway Safety Manual. American Association of State Highway and Transportation Officials, Washington, DC
32. Paul J. Carlson, Eun Sug Park, and Carl K. Andersen. 2008. The Benefits of Pavement Markings: A Renewed Perspective Based on Recent and Ongoing Research. US Federal Highway Administration.
33. Rea, M. S., J. D. Bullough, C. R. Fay, J. A. Brons, J. Van Derlofske, and E. T. Donnell. 2009. Review of the Safety Benefits and Other Effects of Roadway Lighting (report to the National Cooperative Highway Research Program). Washington, DC: Transportation Research Board.
34. FHWA. 2019. The Manual on Uniform Traffic Control Devices.
35. Pedestrian Safety Guide and Countermeasure Selection System. 2019. http://www.pedbikesafe.org/pedsafe/index.cfm
36. Vicroads. 2019. Road design notes: Raised Safety Platforms (RSPs)

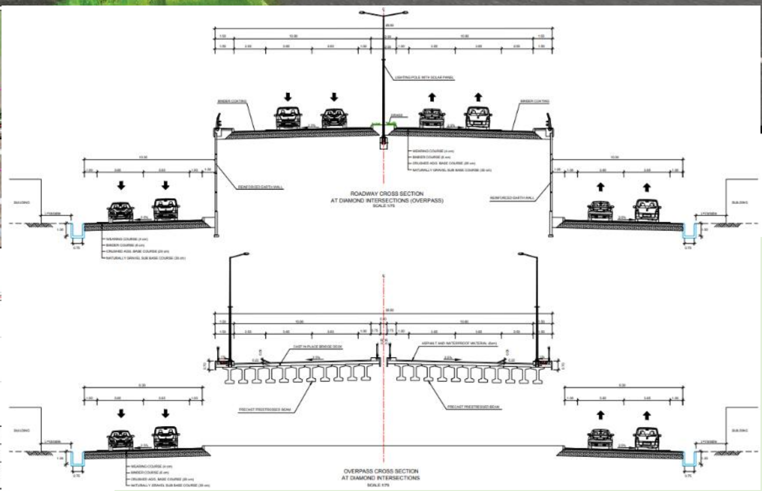
37. Gluck, J., H. S. Levinson, and V. Stover. 1999. National Cooperative Highway Research Program Report 420: Impacts of Access Management Techniques. NCHRP, Transportation Research Board, Washington, DC.
38. Elvik, R., Hoye, A., Vaa, T., and Sorensen, M. 2009. The Handbook of Road Safety Measures, Second edition. Emerald Group Publishing Limited. ISBN 978- 1-84855-250-0.
39. Harwood, D. W., M. T. Pietrucha, M. D. Wooldridge, R. E. Brydia, and K. Fitzpatrick. 1995. National Cooperative Highway Research Program Report 375: Median Intersection Design. NCHRP, Transportation Research Board, Washington, DC
40. Austroads. 2012. Effectiveness of road safety engineering treatments, AP-R422-12.
41. Simodynes, T., Welch, T., and Kuntemeyer, M. 2000. Effects of Reducing Conflict Points on Reducing Accidents (abstract), Third National Access Management Conference, p. 141, Federal Highway Administration, Washington, DC.
42. National Highway Institute. 1992. Access Management, Location and Design: Participant Notebook, NHI Course No. 15255, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.
43. Sarath Chandra Gorthy. 2017. Analysis of Right-in, Right-out Commercial Driveway Safety, Operations and Use of Channelization as Compliance Countermeasure. MSc Thesis, Clemson University
44. Elvik, R., Hoye, A., Vaa, T., and Sorensen, M. 2009. The Handbook of Road Safety Measures, 2nd ed., Emerald Group, United Kingdom]
45. Rune Elvik, Handbook of Road Safety Measures, p. 236
46. Australian Transport Safety Bureau. 2004. “Level crossing accident fatalities.”
47. Ghana Highway Authority Road Design Guide, 1991.
48. Japanese Road Structure Ordinance, April 2021.
49. Explanation and Operation of the Road Structure Ordinance.
50. Geometric Design Manual of Uganda, (2005).
51. A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.
52. Austroads 2016c, Guide to road design part 3: geometric design, AGRD03-16, Austroads, Sydney, NSW.
53. Highway Capacity Manual (HCM), 2016.

Volume II

Chapter 6 Cross Section Elements

Chapter 7 Elements of Design





GHANA ROAD DESIGN GUIDE 2023

CHAPTER 6

CROSS SECTION ELEMENTS

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CHAPTER 6 CROSS-SECTION ELEMENTS

6.1 INTRODUCTION

6.1.1 BASIC CONSIDERATIONS

The type of cross-section to be used in the development of any road project depends on:

i. location of the road (urban or rural)
ii. the functions of the road, for example, through route or local access
iii. new road or treatment of an existing road
iv. traffic volume and mixture
v. number and type of trucks
vi. provision for public transport
vii. walking and cycling needs
viii. creating accessible environments for all
ix. place function and associated space requirements (for example outdoor dining)
x. environmental constraints, for example, topography, existing public utility services, existing road reserve widths, significant vegetation, geology

6.1.2 DETERMINATION OF CROSS-SECTION ELEMENTS AND WIDTHS

To secure the inherent objective of traffic functions, the level of mobility is set according to network characteristics such as the road type and traffic characteristics such as traffic volume. Examination is conducted with a view of designing the necessary cross section composition for securing the aforementioned road functions and setting the necessary width of each element. Moreover, in order to secure the traffic and spatial functions, examination is conducted to provide appropriate services according to roadside conditions and other area characteristics and secure the width for the cross-section elements. The total width is determined according to the combined width of these elements. **Figure 6.1** is a flowchart for the determination of cross section elements and widths.

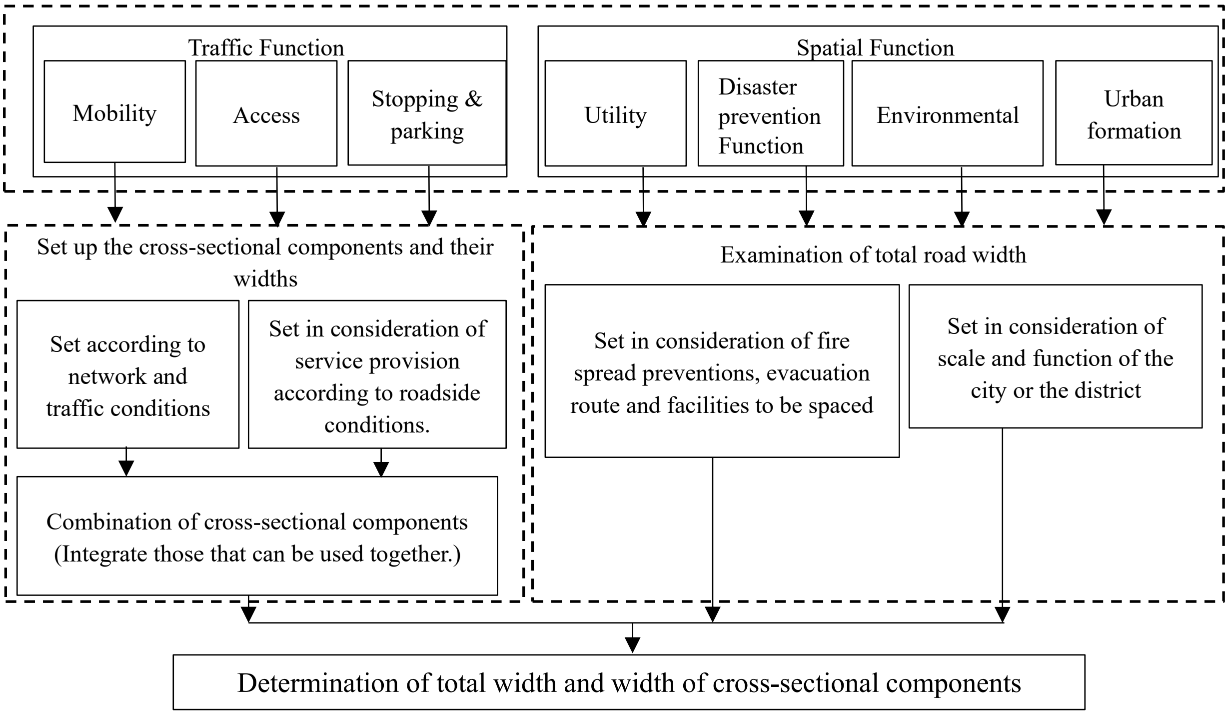


Figure 6.1 Flowchart for determination of cross-section based on road functions

6.1.3 EXAMINATION OF TOTAL WIDTH

To secure disaster prevention space for preventing the spread of disaster and ensuring traffic flow, installation space for utilities, and environmental space for road greening, preservation of roadside and environment etc., the necessary total width should be determined according to the respective spatial functions. Moreover, to ensure urban formation, the total width should be examined in a manner that is suited to the scale and functions of green districts among others. Then, the cross-section elements should be allotted to the total width to ensure that the necessary spatial functions are secured on each road.

6.1.4 DETERMINATION OF CROSS SECTION COMPOSITION BASED ON COMPREHENSIVE JUDGEMENT

As shown in **Figure 6.1**, when determining the cross-section composition of a road, it is necessary to ensure that the necessary width for traffic functions and environmental space functions are secured, and that the total width can also accommodate the necessary spatial functions, while making adjustments to ensure that the necessary road functions are secured. In this way, it is necessary to determine the total width of cross-section elements upon making a comprehensive judgment.

When reconstructing existing road spaces, it is necessary to adjust the total width of cross-section elements while placing emphasis on securing as far as possible the necessary road functions within the limited total width. Moreover, it is necessary to review the necessary road functions in light of the spatial restrictions.

6.1.5 CROSS SECTION ELEMENTS AND ROAD FUNCTIONS

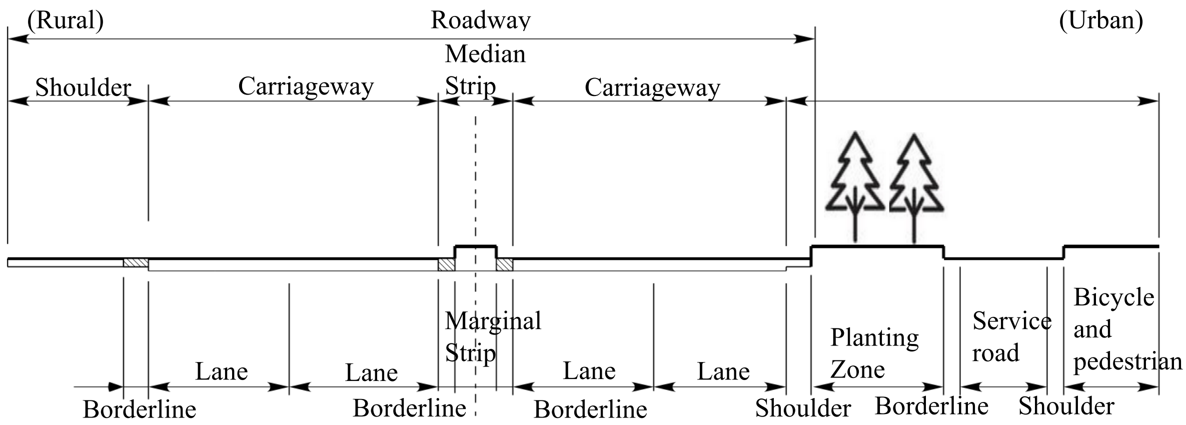
Table 6.1 shows the cross-section elements that need to be considered for securing the respective traffic functions and spatial functions.

Table 6.1 Cross section elements that should be considered for securing road functions

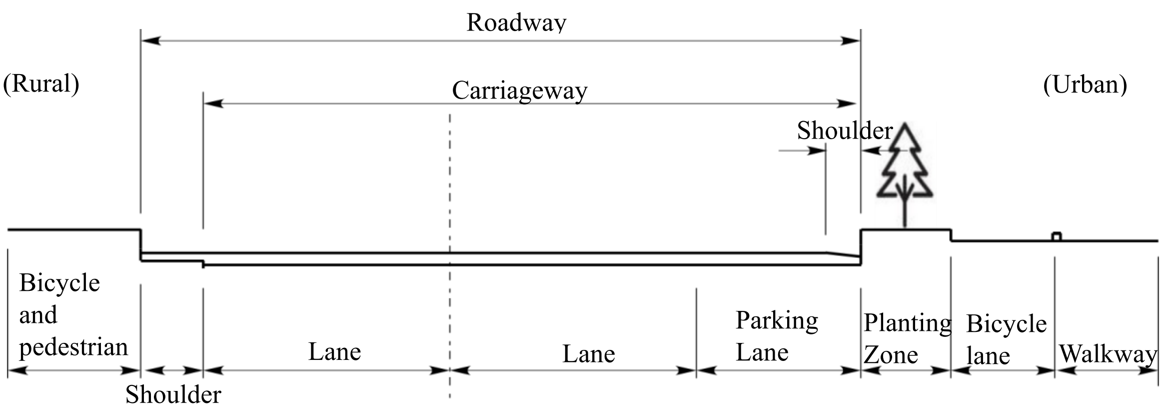
Cross-section elements	Traffic functions	Spatial functions
Carriageway	✓	
Median strip	✓	✓
Shoulder	✓	
Stopping lane	✓	✓
Walkway	✓	✓
Bicycle and pedestrian lanes	✓	✓
Bicycle lanes	✓	✓
Planting zone	✓	✓
Service road	✓	✓

Note) ✓ Denotes items that need to be considered

Figure 6.2 shows examples of cross-section elements and their combinations.



(a) Typical Dual Carriageway (2 lane)



(b) Typical Single Carriageway

Figure 6.2 Examples of cross section elements and their combination

6.1.6 IMPORTANT POINTS TO CONSIDER WHEN LIMITED BY RIGHT OF WAY

If it is necessary to reduce the width of each cross-sectional element, consideration should be given to minimizing the impacts imparted on safety and driving. Generally speaking, reduction should be conducted within the median strip, planting zone, shoulder and stopping lane. If it is still necessary to downsize, it should be applied to traffic lanes. In regard to downsizing of walkways and bicycle lanes, separate judgment must be made from the carriageway in consideration of the volume of bicycle and pedestrian traffic.

6.2 CARRIAGEWAY

The carriageway is that part of the road that is solely intended to enable the movement of vehicles. It is the space required for traffic functions. The carriageway width and number of lanes are determined according to the road class, traffic volume, design speed and terrain among others.

6.2.1 NUMBER OF LANES

The number of lanes of the road is determined according to the road's design traffic volume in relation to the design standard traffic volume per lane and the corresponding design class shown in **Table 5.22** of **Chapter 5**.

6.2.1.1 PROCEDURE FOR DETERMINING THE NUMBER OF LANES

The number of lanes is determined according to the following procedure:

i. The relationship between design standard traffic volume and design traffic volume corresponding to the class of the road. Considering that both the design standard traffic volume and design traffic volume are rough values, there is a certain degree of flexibility in deciding this, so it is permissible to discard fractions when setting the number of lanes. For roads passing through urbanised areas with 4 or more lanes having 2 or more intersections per kilometre, the design standard traffic volume per lane given in **Table 5.22** is multiplied by 0.6.

ii. Basically, the number of lanes should be an even number of 2 or more (2, 4, 6, 8, etc.). However, odd numbers of 5 or more may be adopted in cases where the need arises due to differences in the outbound and inbound traffic volume etc. Moreover, in cities, on ring roads etc., one-way roads having 2 or more lanes (2, 3, 4, 5, etc.) may sometimes be adopted without having lanes going in the opposite direction.

6.2.1.2 VERIFICATION OF THE NUMBER OF LANES

When determining the number of lanes according to area and route characteristics, it is necessary to implement verification of the hourly traffic volume that takes the route's peak characteristics (peak coefficient), direction characteristics (heavy direction rate) and large vehicle mixing rate into account and strive to decide the appropriate number of lanes. In particular, when reconstructing existing roads that have large fluctuations in traffic volume according to the month, day and hour, it is necessary to grasp the actual traffic volume situation in relation to these fluctuation characteristics.

In determining the number of lanes of a road, the design traffic volume (daily traffic volume) is used, while also taking the daily design standard traffic volume into account. For the design traffic volume, it is common to use the annual average daily traffic volume. However, there may be cases where the number of lanes obtained in verification based on the hourly traffic volume does not match. In such cases, for example, it is necessary to decide the appropriate number of lanes by conducting a renewed examination concerning whether to use the annual average as the design traffic volume.

6.2.2 LANE WIDTH

Among the elements that comprise a road’s cross-section, lane width has a major impact on travel speeds and comfort. It is deemed rational to decide the necessary width of lanes according to the following:

i. Route designs speed
ii. Traffic volume
iii. Terrain
iv. Percentage of large vehicles

The lane width must be sufficient to allow passing of vehicles traveling in opposite directions, overtaking, and parallel running. However, if the lane width is made too wide, situations will arise where vehicles run in three (3) lines on a 2-lane road, and it becomes difficult to keep traffic in orderly lines; hence making lanes too wide is not advisable in terms of preventing accidents.

The lane width comprises the physical width of vehicles plus a necessary allowance for passing and overtaking. **Figure 6.3** shows the results of experimentally seeking the relationship between travel speed and carriageway width in line with the travel speed on a 2-lane road. The width will be approximately 6m (lane width 3m) when the travel speed of passenger vehicles is 40 km/h and 7-7.5m (lane width 3.5-3.75m) when the travel speed is 100km/h. **Table 6.2** shows the relationship between design speed and lane width.

Table 6.2 Relationship between design speed and lane width

Design Speed (km/h)	Desirable lane width (m)
>80	3.65
60 – 80	3.50
40 – 60	3.25
<40	3.00

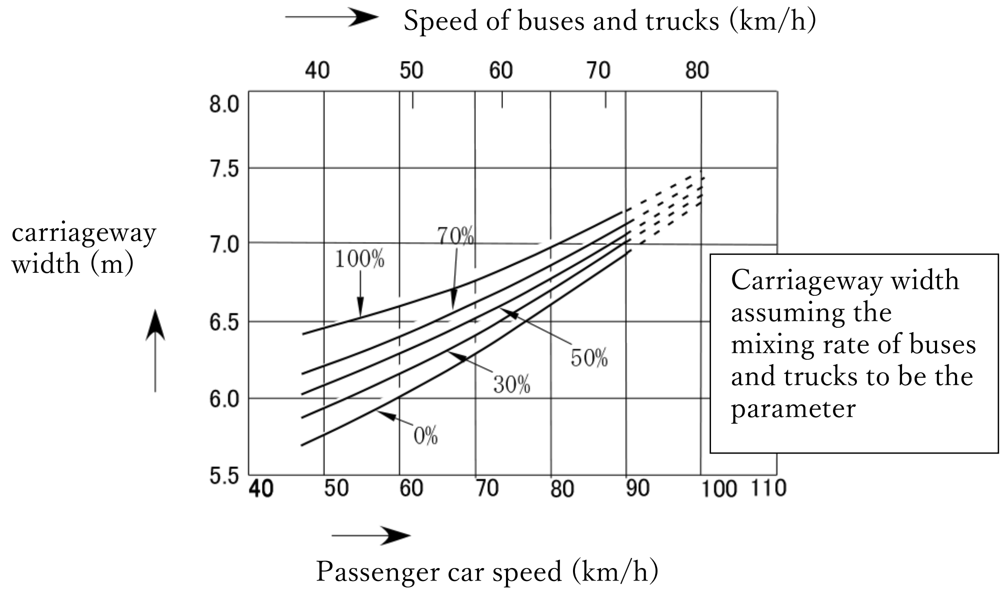


Figure 6.3 Carriageway width of 2 lanes based on experiment

Standard lane widths are shown in **Table 6.3**.

Table 6.3 Standard lane widths

Design Class	Terrain	Standard width (m)	
		Desirable	Absolute
A1 - Rural Expressway/Motorway A2 - Urban Expressway/Motorway	Flat	3.65	3.50
	Hilly	3.65	
	Mountainous	3.50	
B1 - Rural Major Arterial B2 - Urban Major Arterial	Flat	3.65, 3.50	3.25
	Hilly	3.50	
	Mountainous	3.25	
C1 - Rural Minor Arterial C2 - Urban Minor Arterial	Flat	3.65, 3.50	3.25
	Hilly	3.50, 3.25	3.00
	Mountainous	3.25	3.00
D1 - Rural Collector D2 - Urban Collector	Flat	3.50, 3.25	3.00
	Hilly	3.25	3.00
	Mountainous	3.00	2.75
E-Local/Access	Flat	3.25	3.00
	Hilly	3.00	2.75
	Mountainous	3.00	2.75

6.3 CROSS SLOPE

Two-lane and wider undivided carriageway on tangent sections or on very flat curves have a crown or a high point in the middle and slope downward towards both edges. These are also called as cross slope or camber. Cross slope or camber slope should be sufficient to provide adequate surface drainage whilst not being so great as to make steering difficult. Steeper cross slopes requires a conscious effort in steering and increases susceptibility to lateral skidding when vehicles brake on wet pavements and even on dry pavements when stops are made under emergency conditions. The ability of a surface to shed water varies with its smoothness and integrity. On unpaved roads, the minimum acceptable value of cross slope should be related to the need to carry surface water away from the pavement surface effectively, with the maximum value above which erosion of material starts to become a problem.

6.3.1 CROSS SLOPE OF CARRIAGEWAY

The cross slopes of various types of carriageways are shown in **Table 6.4**. However, to secure cross slope on roads in rainy areas or on wide roads that have high design speed or other features that require consideration of drainage, regardless of the standard values shown in **Table 6.4**, it is necessary to conduct cross slope examination. According to cases and experience in various countries, it is generally desirable to set the cross slope at 2.5% or more.

Table 6.4 Cross slopes of various types of road surface

Surface type	Cross slope (%)
Cement concrete	2.0
Asphalt concrete	2.5
Surface dressing	3.0
Gravel	4.0

6.3.2 CROSS SLOPE OF SHOULDER

The cross slopes of various types of shoulders are shown in **Table 6.5**.

Table 6.5 Cross slopes of various type of shoulder

Surface	Cross slope (%)
Hard surface	2.0 -5.0
Gravel	4.0 – 6.0

6.3.3 CROSS SLOPE OF WALKWAY AND BICYCLE LANE

The cross slopes of walkways and bicycle lanes are from 1.5% to 2.5% but in unavoidable situations values just below 4% can be adopted.

6.4 MEDIAN

A median is the portion of the roadway separating opposing directions of the travelled way. Medians are highly desirable on roadways with four or more lanes. Median can be defined by pavement markings, raised island, barriers, or vegetation.

In addition, it is a space for providing spatial functions of urban area formation, disaster prevention, and environmental improvement. The width and type of the median is determined according to the class of road, traffic volume, traffic functions in the local area, and spatial functions such as fire prevention, greening, landscaping, and housing of traffic facilities.

6.4.1 FUNCTIONS OF MEDIAN STRIP

Functions of the median strip comprise traffic and spatial functions. These are described below and shown in **Figure 6.4**.

6.4.1.1 TRAFFIC FUNCTIONS

i. It prevents traffic accidents by separating opposing traffic streams.
ii. It promotes smooth traffic by preventing uncontrolled U - turn movements.
iii. It enables easy management of left turning traffic.
iv. It is used to prevent light dazzling effects. It provides space for setting up anti-glare

nets and planting for intercepting light.

v. Pedestrians can make use of it as a refuge area to cross the road safely and easily.

vi. It provides space for the installation of streetlights and other traffic control facilities.

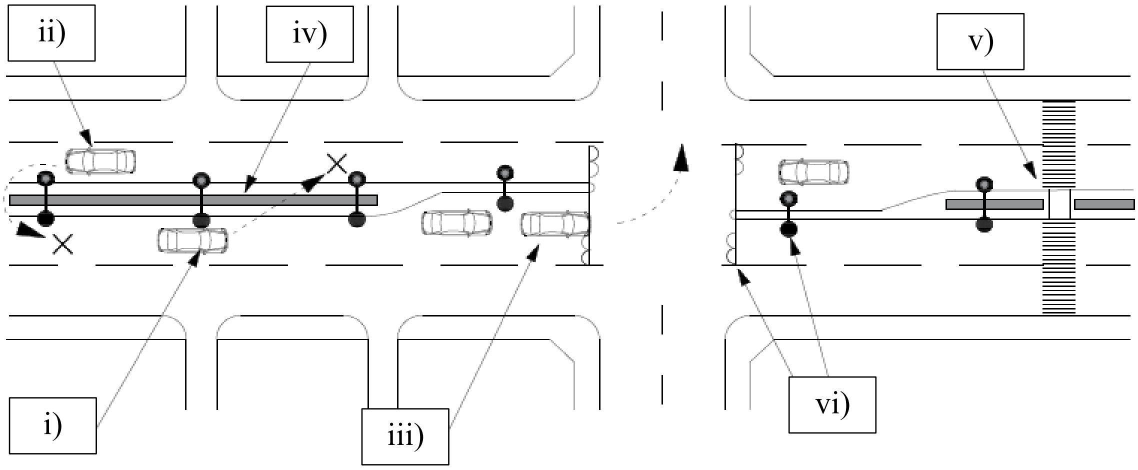


Figure 6.4 Functions of median strip

6.4.1.2 SPATIAL FUNCTIONS

The median strip is a part of a street that is endowed with urban area formation, disaster prevention, and landscaping functions.

It provides space for utilities.

It becomes a greening space for environment preservation functions such as noise attenuation, atmospheric purification, and green shade as it can be used for planting trees.

For future road expansion

6.4.2 PROVISION OF MEDIAN STRIP

On roadways with 4 or more lanes, a median strip is always established, except in cases where outbound and inbound routes are independently established. The median strip should be installed to secure safe and smooth traffic in consideration of the aforementioned median strip functions, traffic conditions and roadside conditions.

Moreover, even on a 2-lane single carriage, in cases where safe traffic is seriously hindered by the existence of sharp curves etc., it is advisable to provide a median strip to separate traffic going in opposite directions. In such cases, it may be necessary to expand the shoulder width to secure marginal spacing. In the event of accidents and disaster, this will ensure that the separation does not hinder rescue activities and passage of vehicles.

6.4.3 COMPOSITION OF MEDIAN STRIP

The median strip consists of a central reserve and marginal strip as shown in **Figure 6.5**.

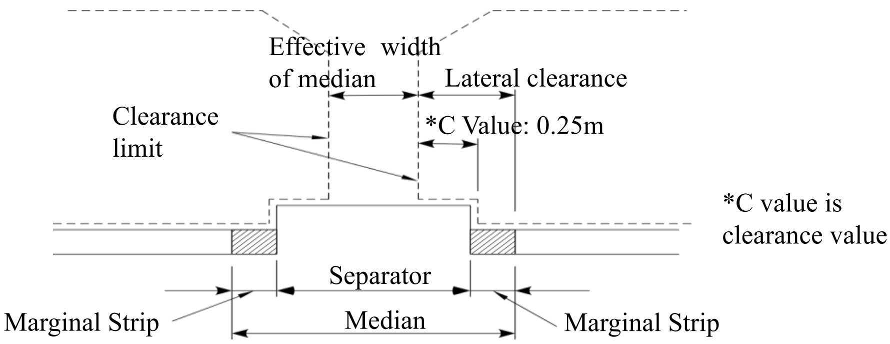


Figure 6.5 Components of median

The central reserve is the part other than the marginal strip where a protective fence or kerb may be installed to clearly separate the traffic going in the opposite directions. The marginal strip is installed on both sides of the central reserve. Functions of the marginal strip are as follows:

i. To clearly define the inner edge of the carriageway thereby enhancing safety by guiding the sightline of drivers
ii. To secure the necessity marginal spacing width for traveling and preserve the effective width of the carriageway

6.4.4 WIDTH OF MEDIAN STRIP AND BORDERLINE

The functions of the median strip are enhanced the wider it becomes. For example, if a vehicle deviates and enters a median strip with sufficient width, there is a greater chance of it recovering control thereby preventing accident. Wide medians also provide space for maintenance work. A width of 4m is desirable to allow for the introduction of left turning movements in built-up areas. **Table 6.6** shows the recommended median and borderline widths and C values.

Table 6.6 Recommended median & borderline width and C value

Design Class		Median (m)	Borderline (m)	C value (m)
A	A1	10.0 (min)	1.2 (min)	0.50
	A2	2.0 – 4.0	0.50 – 0.75	0.25
B, C, D & E		1.0 - 4.0	0.30 – 0.50	0.25

6.4.5 TYPES OF MEDIAN STRIP

There are two main types of median strips:

i. Crest type

The crest type is mostly used for narrow median strip. Safety fence is a form which is mostly used on motorways with narrow median strip. New Jersey barrier, Kerbs and double borderlines may also be used.

ii. Sag type

The sag type is mostly used for wide median strip. Vegetation cover such as shrubs and grass are recommended. Examples of median strips are shown in **Plate 6.1**.

	
(i) Crest type with New Jersey Concrete Barrier	(ii) Crest type with kerbs
	
(i) Sag type with vegetation	(ii) Sag type with median drainage

Plate 6.1 Types and structures of median strip

6.5 SHOULDER

A shoulder is the portion of the roadway contiguous with the carriageway that accommodates stopped vehicles, provides space for emergency use and lateral support of road pavement. In some cases, the shoulder can accommodate cyclists and pedestrians. The width of the shoulder is determined according to the class of road and traffic volume. Where shoulders are intended to be used as pedestrian facilities, the shoulder must be accessible to and usable by individuals with physical challenges.

It may be desirable that the colour and texture of shoulders be different from those of the travelled way. This contrast serves to clearly define the travelled way at all times, particularly at night and during inclement weather, while discouraging the use of shoulders as additional through lanes.

6.5.1 FUNCTIONS OF SHOULDER

Except in cases where a median strip or stopping lane is provided, a shoulder is attached to the carriageway. The function of the shoulder is as follows:

i. It improves traffic safety by enhancing visibility on curves especially on cut sections.
ii. It enhances the aesthetic of roads.
iii. It protects the road pavement.
iv. It provides parking space at least partly off the carriageway for distressed vehicles
v. It ensures safety and comfort of motorist.
vi. It can be utilized as walkways/cycle lane
vii. It provides space for maintenance works
viii. It provides additional space for emergency manoeuvres.
ix. It provides space for accommodating storm water in other to minimize encroachment (spread) of water on the carriageway.
x. It enhances the discharge of water from the carriageway

6.5.2 SHOULDER WIDTH

The desirable shoulder width for the various design classes is shown in **Table 6.7.**

Table 6.7 Desirable shoulder width

Design Class	Shoulder (m)		
	Desirable	Absolute minimum	*Width for Partial shoulders
A	3.0	2.5	1.75
B	2.5	2.0	1.25
C	2.5	1.5	0.75
D	2.0	1.0	0.5
E	1.5	0.5	-

*Partial shoulders are sometimes used where full shoulders are unduly costly, such as on long bridges (over 60m) or in mountainous terrain.

6.5.3 BORDERLINE OF SHOULDER

It is necessary to set up borderline of shoulder (Refer to **Figure 6.6)** on high-speed roads for the following reasons:

i. The border of carriageway should be well defined by marking for the guidance of motorists.

ii. To keep lateral clearance width for running vehicles.

Borderline structures are of various types. These include:

i. Concrete block
ii. Kerb
iii. Line marking

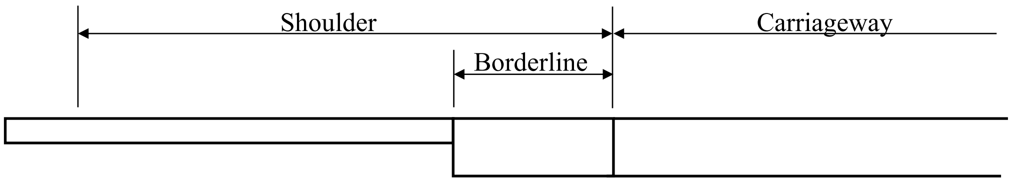


Figure 6.6 Borderline and shoulder

6.6 VERGE

A verge is a hardened and/or stabilised ground surface which may be in earth, grassed or laid with crushed rocks or gravels forming the remaining section of the road formation that joins the shoulder with the batter as shown in **Figure 6.7**.

The main functions of the verge are to provide:

i. a traversable transition between the shoulder and the batter slope to assist controllability of errant vehicles
ii. a firm surface for stopped vehicles at a safe distance from traffic lanes
iii. support for the boxing edge, shoulder material and kerb and channel
iv. space for installation of road agency infrastructure such as road signs and road safety barriers
v. space to accommodate infrastructure belonging to other services such as telecommunication, electricity, water and public transport
vi. Accommodation of drainage system
vii. Clear zone
viii. Refuge area for stranded road-users
ix. Visibility needs
x. reduced scouring due to road storm water run-off.

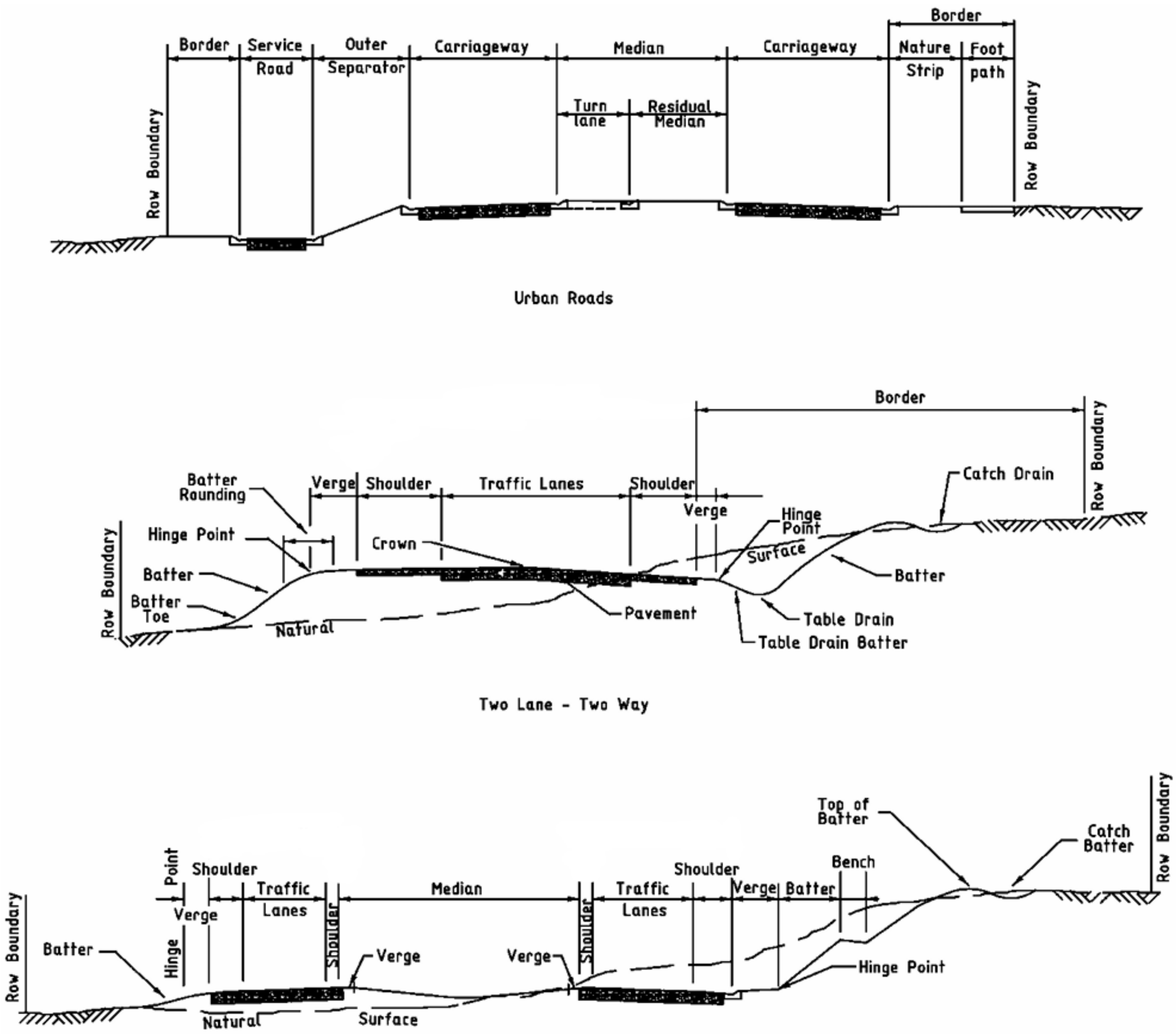


Figure 6.7 Cross-section terminology

It should be noted that verges are not usually provided in urban areas where kerb and channel is constructed except:

i. along expressway/motorway
ii. where guard fence is provided
iii. along a short fill between adjacent cuts where continuity of kerb and channel is required.

6.6.1 VERGE WIDTHS

Examples of verge widths for various functions of verges are shown in **Table 6.8**. It is not intended that verge widths should vary continuously. Designers should apply long sections of appropriate minimum verge width to facilitate construction practices, with short transitions where greater or lesser widths are required, e.g., construction of run out areas for roadside barrier terminals.

Table 6.8 Verge width

Function	Width (m)
Shoulder support and space for installation of road signs	1.0
Traversable transition between the shoulder and the batter slope (depending on how steep the superelevation and/or batters might be and what batter rounding is required)	1.0 - 6.0
Behind kerb and channel (measured to line of kerb)	1.5
Cut and fill	
To provide a space for installation of road safety barrier (extra for terminals)	1.5
To achieve horizontal sight distance, or to balance cut and fill	Calculated where required (Refer **Section 7.2.7**)

The verges under a structure affect the structure length and cost. A minimum verge of 0.8m should be used except:

i. where additional clearance is required for sight distance, such as at diamond interchanges when a ramp terminal is located close to the structure
ii. where additional space is required for underground services or underground drainage
iii. in long cuts where the change in verge width would create visual discontinuity.

The reinforced concrete surface is desirable because of the likelihood that trucks will park in the shade under structures, and some will drive onto the verge to maximise the clearance from the traffic lane.

The verge slope should be the same as the pavement abutting it.

6.7 BATTERS

Batters are surfaces, commonly but not always of uniform slope, which connect carriageways or other elements of cross-sections to the natural surface. Batters may:

i. provide a recovery area for errant vehicles
ii. be used as part of the landscaped area
iii. be used for access by maintenance vehicles.

There are three basic types of batter:

i. The cut or fill batter is the surface which connects the design earthworks surface to the natural surface.
ii. The table drain batter is the slope which connects the right-hand shoulder or verge to the table drain.
iii. The median batter is the slope which connects the left side verge to the median drain.

Examples of batters are shown in **Figure 6.7**. Batter slopes are usually defined as the ratio of one vertical to 'x' horizontal and are shown as, for example, 1:4. The following factors should be considered when selecting batter slopes:

i. the results and recommendations of geotechnical investigation
ii. batter stability; no batter should be designed steeper than the stable slope in the natural material, unless slope reinforcement and drainage is properly catered for
iii. batter safety (economics of eliminating safety barriers, e.g. maintaining a smooth face for rock cuttings which will not snag a vehicle)
iv. safety, as fill batters with the following slopes can be considered to be:
    - recoverable for cars with 1:4 or flatter batter slopes
    - non-recoverable for cars with batter slopes from 1:3 to 1:4, but they are considered to be traversable. Cars are likely to continue to the bottom of the slope
    - non-recoverable (and non-traversable) for cars with batter slopes steeper than 1:3
    - recoverable for trucks with batter slopes of 1:10
v. future costs of maintaining the adopted slope and accessibility requirements considering occupational health and safety issues
vi. appearance and environmental effects (e.g. erosion)
vii. earthworks balance
viii. available width of road reserve
ix. landscaping requirements.

Table 6.9 shows desirable maximum batter slopes for cut and fill situations. Slopes flatter than the desirable maximum should be used where possible to improve roadside safety for motorists, batter stability and erosion control. Where steeper slopes are unavoidable and barriers are required, designers should consult **Section 12.5**.

Table 6.9 Typical design batter slopes

Cut			Fill	
	Depth of cut (m)	**Cut Slope (V:H)**	**Height of Fill (m)**	**Fill Slope (V:H)**
Earth	0.0 - 1.0	1:3	0.0 - 1.0	1:4
	1.0 - 3.0	1:2	1.0 - 3.0	1:4 - 1:2
	>3.0	1:1.5	>3.0	• 1:2 (Desirable max. slope with guard rail) • 1:1.5 (Absolute max. slope with guard rail) • New jersey barrier shall be used in lieu of guard rails wherever warranted and approved
Medium Hard	0.0 - 1.0	1:2		
	1.0 - 3.0	1:1.5		
	>3.0	1:1		
Rock	0.0 - 1.0	1:1		
	1.0 - 3.0	1:1 - 3:1		
	>3.0	Variable 3:1 - 10:1		

6.7.1 BENCHES

On high batters (generally exceeding 5-6m vertical height) or where batters are constructed on unstable material, consideration should be given to the provision of benches. Benches can have the beneficial effects of:

i. eliminating the need to flatten the batter slope in the interests of stability
ii. minimising the possibilities of rock falling onto the pavement
iii. reducing scour on the batter face
iv. reducing the amount of water in cuttings to be carried by the table drain
v. providing easier access for maintenance of the batter face
vi. improving the appearance of the cutting
vii. assisting the re-establishment of vegetation
viii. improving sight distance on horizontal curves.

Benches should be sloped away from the roadway and longitudinally so that stormwater can be drained towards the ends of the bench and discharged on to the natural ground. In some instances, the invert so formed may require lining.

The minimum width of bench should be 2m (**Figure 6.8**) with a maximum crossfall of 10%.

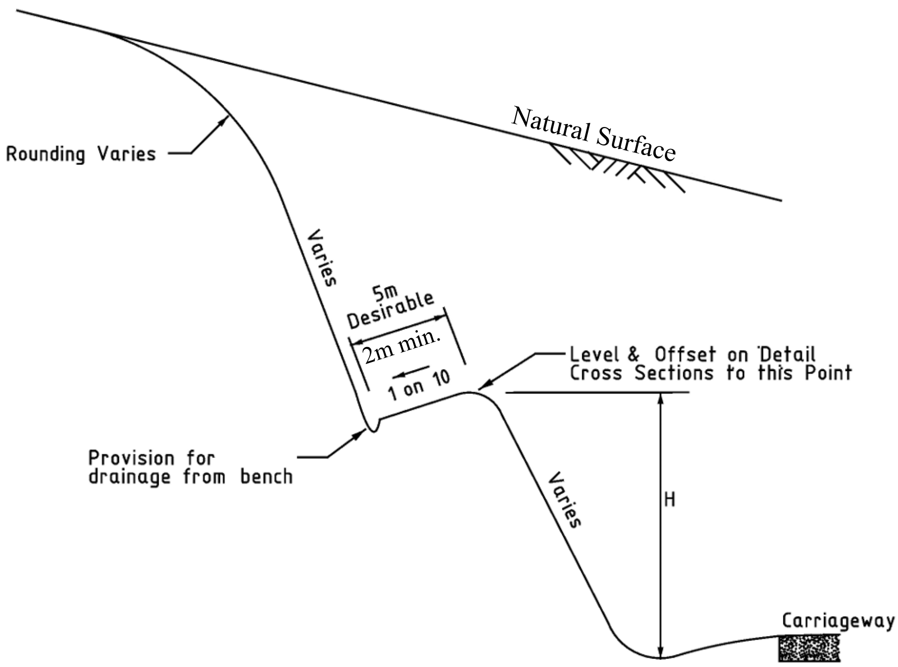


Figure 6.8 Benches

6.8 TURNOUT

A turnout is a widened, unobstructed shoulder area that allows slow-moving vehicles to pull out of the through lane to give passing opportunities to following vehicles to ensure the smoothness and safety of traffic on the road. Turnouts are most frequently used on lower volume roads and in difficult terrain with steep grades and tight shoulders where more than 10% of the vehicle volumes are large trucks and recreational vehicles, and construction of an additional lane may not be cost-effective.

On bus routes, turnouts should be provided at bus stops as much as possible to also serve as bus bays. Turnout should be provided at entrance to and exit of long bridge / viaduct (> 50m) and tunnel as well as approach and departure of curvature less than radius of 1,000m. **Plate 6.2** shows an example of a turnout.

Plate 6.2 Examples of road turnout

6.8.1 SPACING

The minimum intervals between turnouts are shown in **Table 6.10**.

Table 6.10 Intervals between turnouts

Design Class	Interval (m)
A & B	300
C – E	500

6.8.2 LENGTH

The recommended length of turnouts including taper is shown in **Table 6.11**. Turnouts shorter than 60m are not recommended even for very low approach speeds. Turnouts longer than 185m are not recommended for high-speed roads to avoid use of the turnout as a passing lane.

The lengths are based on the assumption that slow-moving vehicles enter the turnout at 8km/h slower than the mean speed of the through traffic, allowing the entering vehicle to coast to the mid-point without braking, and then brake if necessary or merge back into the through lane.

The minimum width of a turnout is 3.50m, with 5.0m desirable. The available sight distance should be at least 300m on the approach to the turnout. Proper signing and pavement marking are also needed to maximize turnout usage and reduce crashes.

Table 6.11 Recommended length of turnouts including taper

Approach Speed (km/h)	Length (m)
0 - 40	60
50	65
60	85
70	105
80	135
90	170
≥100	185

6.8.3 LAYOUT

The parallel length of a turnout should not be less than 30m, and width of the carriageway over that section should not be less than 5.5m. Details of a turnout is shown in **Figure 6.9.**

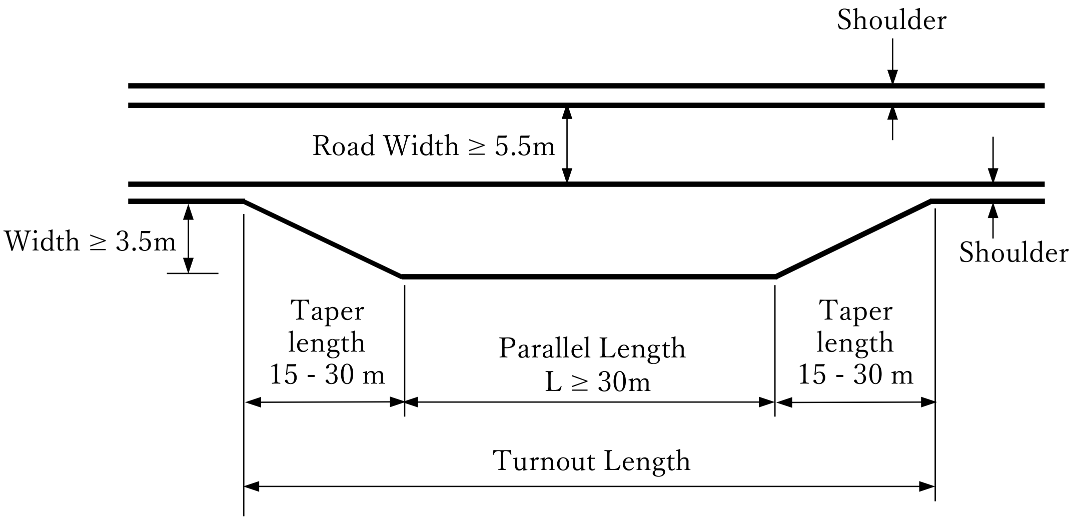


Figure 6.9 Turnout layout

6.9 LAY-BY

Lay-by (Bus Bay or Bus Stop) may be defined as an area along or by the main roadside dedicated for vehicles to pick up and drop off passengers.

6.9.1 TYPES OF LAY-BY LAYOUT

The main types of lay-by layouts are:

i. kerbside lay-by
ii. in-lane lay-by
iii. indented lay-by

The type of layout to adopt should be determined in consideration of the traffic conditions, road cross section elements, etc.

6.9.1.1 KERBSIDE LAY-BY

This type, which entails establishing a bus stop on a walkway without changing the walkway width, is adopted in cases where the total width of the road is not sufficient to make a cutting in the sidewalk and install a stopping lane. Issues with this type are that impacts are imparted to following vehicles, and it becomes difficult to accurately park at the bus stop if other vehicles are parked on the road. **Figure 6.10** depicts an example of a kerbside lay-by.`

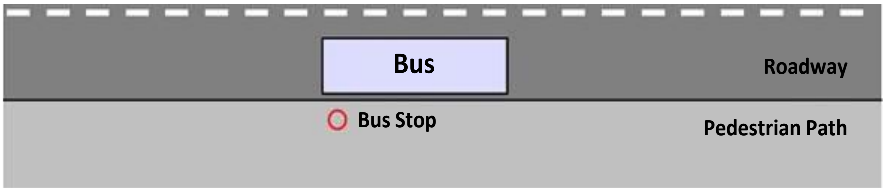


Figure 6.10 Kerbside lay-by

Entry and exit space are required where the bus needs to pull out of and back into the kerbside traffic lane because of an obstruction, usually on-street parking (see **Figure 6.11**). When on-street parking is too close to a kerbside bus stop, the bus may have trouble entering and exiting the stop and aligning close and parallel to the kerb.

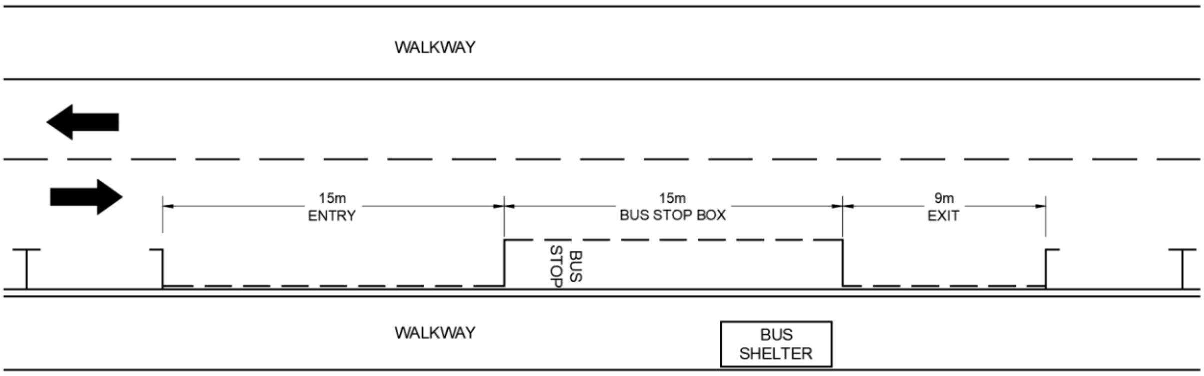


Figure 6.11 Kerbside bus stop with parking on either side

6.9.1.2 IN-LANE LAY-BY

In this type, to resolve the issue where buses cannot park at a bus stop due to vehicles parked on the road, the bus stop is provided on a protrusion on the carriageway side (shoulder, stopping lane, or carriageway), making it possible to establish the bus stop without narrowing the effective width of the walkway. However, it is difficult to apply this type on a road that does not have a wide shoulder or stopping lane. **Figure 6.12** depicts an example of an in-lane lay-by.

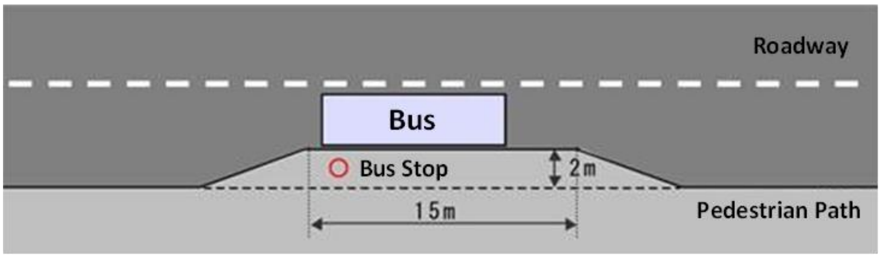


Figure 6.12 In-lane lay-by

6.9.1.3 INDENTED LAY-BY

This type entails establishing a bus stopping lane by cutting into the walkway. In addition to expediting boarding and alighting, it has the characteristic of making it easier for following vehicles to overtake.

However, depending on the shape of the walkway cut and situation regarding road parking in the local area, it may be difficult for passenger vehicles to park accurately at such a bus stop. Moreover, when providing this type, additional land needs to be acquired to achieve the effective width of the walkway around the bus stop. **Figure 6.13** depicts an example of an indented lay-by.

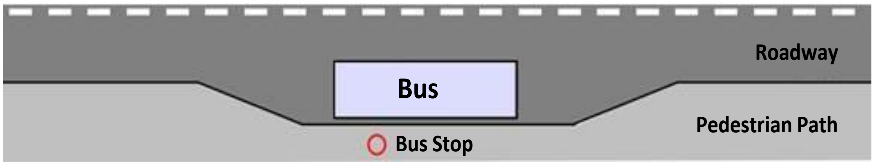


Figure 6.13 Indented lay-by

6.9.1.3.1 TYPES OF INDENTED LAY-BYS

Indented lay-bys may be with separator (Type-A) or without separator (Type-B) depending on the function of the road. The former is recommended for roads with high traffic volumes and speeds where the risk of accidents is high, whilst the latter may serve well for roads with low traffic volumes and speeds. It is advisable to make bus bays with separators two lanes since the 'first come, first go’ principle associated with single lane bus bays with separators can be irritating and inconvenient. **Figure 6.14** and **Figure 6.15** illustrate Type-A and Type-B lay-bys respectively, while the recommended types of lay-bys for the various road classes are shown in **Table 6.12**. **Plate 6.3** shows a Type-B lay-by without walkway.

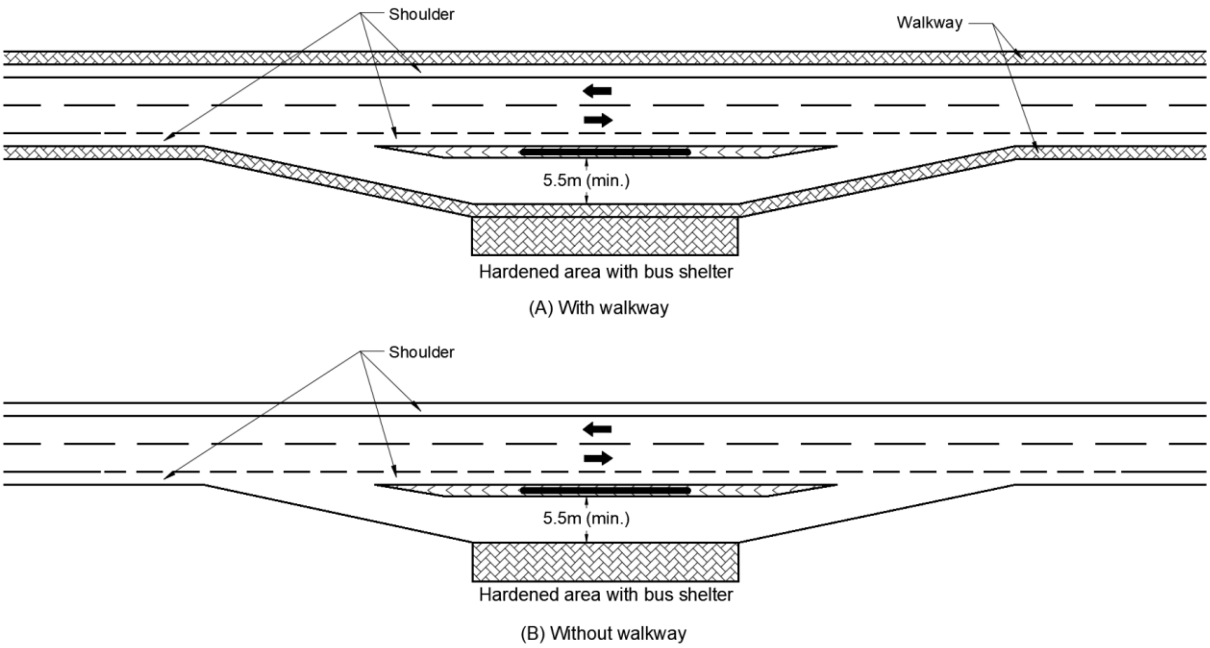


Figure 6.14 Lay-by with separator (Type-A)

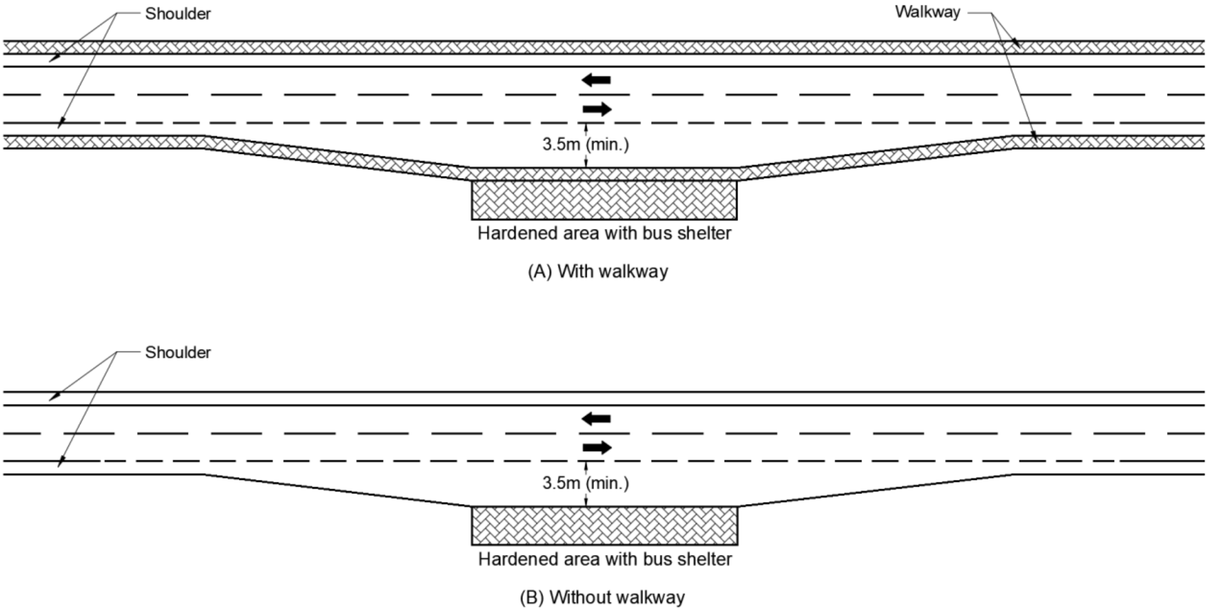


Figure 6.15 Lay-by without separator (Type-B)

Table 6.12 Recommended type of lay-by for various Design Classes

Design Class	Type of Lay-by
A	Type-A
B and C	Type-A (Desirable), Type-B (absolute)
D and E	Type-B

Plate 6.3 Type-B lay-by without walkway

6.9.2 SITING OF LAY-BYS

Lay-by should be provided at locations that meet the following conditions:

i. Where there are frequent passenger vehicle services, and the stopping of such vehicles leads to extreme decline in traffic capacity.
ii. Where the stopping of vehicles badly impedes the safe and smooth passage of other vehicles.
iii. Where accidents related to the arrival and/or departure of vehicles occur or there is a risk of such accidents occurring.
iv. Where there are many boarding and alighting passengers and it is desirable to secure the safety of users and convenience for connecting services, etc.

6.9.3 GEOMETRIC CONSIDERATIONS FOR SITING LAY-BYS

The following geometric criteria should be considered when providing a lay-by:

i. The horizontal alignment of the carriageway should be straight or have a curve radius not less than the standard value (refer to **Table 7.1**); moreover, the section concerned must have a reasonable vertical gradient (refer to **Table 7.17**).
ii. If placing near an intersection, the lay-by should be positioned at the intersection exit side with a separation of at least 3.75V metres (V= speed in km/h).
iii. If placing close to a pedestrian crossing, the lay-by should be positioned at least 10m beyond the pedestrian crossing.
iv. Along urbanised sections, lay-bys should be spaced 200 to 400m apart.
v. A lay-by should not be placed at the following locations:

- Crest profile.
- Between an intersection and the associated advance direction sign.
- Horizontal curves, both on the inside and outside, of radii smaller than values given in **Table 6.13** or a place where the stopping of buses could impede visibility.

Table 6.13 Recommended minimum radius for positioning lay-bys

Speed Limit (km/h)	Radius (m)
≥100	1440
80	950
60	510
≤50	360

- A steep slope or within 40m of such a slope.
- Within 30m of a railway crossing.

vi. To minimize traffic conflicts on single carriageways, lay-bys should not be placed opposite each other. Lay-bys must be staggered tail-to-tail (with a minimum gap of 10m) so that departing vehicles move away from each other as shown in **Figure 6.16**. Where it is not feasible to comply with this arrangement, a head-to-head stagger can be adopted with a minimum gap of 150m provided between the two lay-bys as shown in **Figure 6.17**.

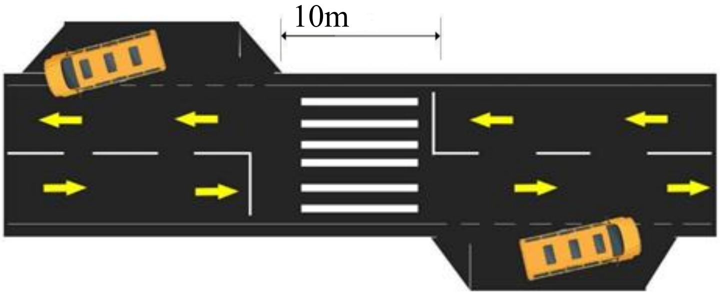


Figure 6.16 Tail-to-tail staggering of lay-by

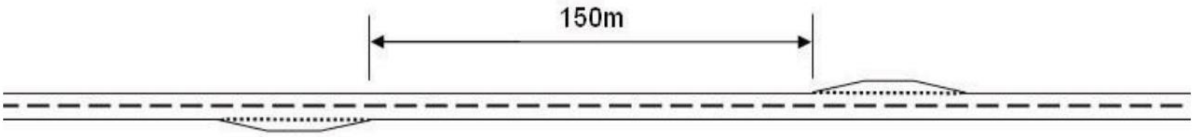


Figure 6.17 Head-to-head staggering of lay-by

When siting lay-bys, the designer should bear in mind that the existing lay-bys will generally be located where they are most convenient for the passengers, and it is usually very difficult to persuade passengers and bus drivers to move to new stops, especially if they are more than 50m away. The provision of a bus shelter to protect the waiting passengers against the sun and rain will encourage passengers to use the bus stop. Such shelters should be robust enough to be difficult to vandalise and should have a depth of at least 1.6m. Also, they should have a minimum width of 2.0 m.

All lay-bys should be indicated by an informatory sign at the entrance as well as one or more advance informatory sign.

6.9.4 GENERAL ARRANGEMENT OF TYPE-A LAY-BY

Figure 6.18 and **Table 6.14** show the general arrangement and dimensions of Type-A lay-by respectively.

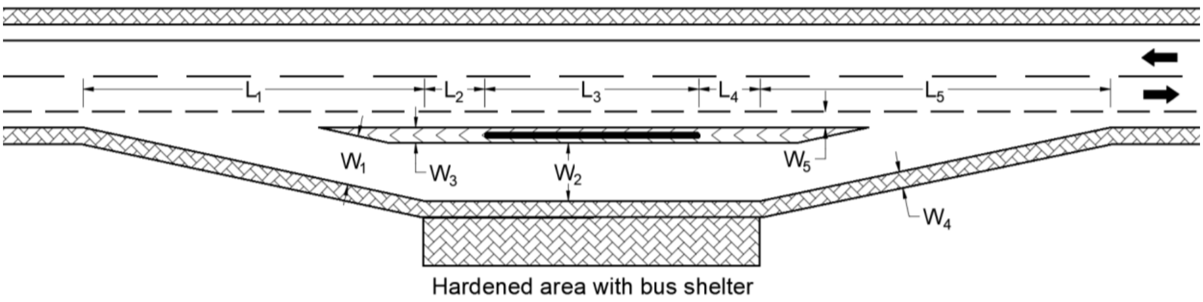


Figure 6.18 General arrangement of Type-A lay-by

Table 6.14 Dimension of Type-A lay-by

Design Class	Design speed (km/h)	Dimension (m)								
		L_1	L_2	L_3	L_4	L_5	W_1	W_2	W_3	W_4
A	120	180	40 - 50	30 -100	30 - 40	220	3.5	5.5	1.0 - 2.0	1.5 (min.)
	100	160	40 - 50		30 - 40	190				
	80	140	30 - 40		25 - 30	160				
	$<$ 80	65	15		15	65				
B & C	$\geq$ 80	75	30		30	75				
	$<$ 80	65	15		15	65				

Where,

L_1 and L_5 are tapper lengths

$L_2 + L_3 + L_4$ = parallel length

W_1 and W_2 are widths of lay-by

W_3 is effective width of separator

W_4 is width of walkway

W_5 is width of shoulder

6.9.5 GENERAL ARRANGEMENT OF TYPE-B LAY-BY

Figure 6.19 and **Table 6.15** show the general arrangement and dimensions of Type-B lay-by respectively.

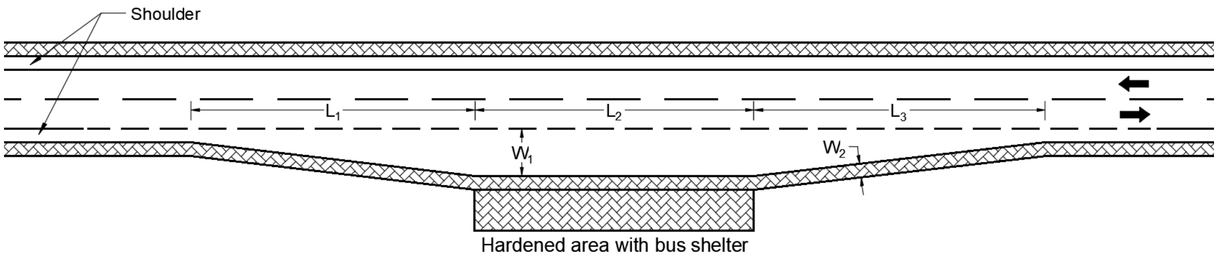


Figure 6.19 General arrangement of Type-B lay-by

Table 6.15 Dimensions of Type-B lay-by

Road Class	Design speed (km/h)	Dimension (m)				
		L_1	L_2	L_3	W_1	W_2
B, C, D & E	≥80	35	30 – 100	40	3.5 - 5.5	1.5 (min.)
	60	25		30		
	≤50	20		25		

Where,

L_1 and L_3 are tapper lengths

L_2, is parallel length

W_1 is width of lay-by

W_2 is width of walkway

6.9.6 DIMENSION OF WALKWAY IN PLACES WITH BUS STOP FACILITIES

The height of a walkway section on which a bus stopping lane or bus stop is provided should be 15cm in respect to the carriageway. However, in cases where buses cannot accurately park due to the road structure, the height may be adjusted to other than 15cm, and a slope for descending to the carriageway should be provided for wheelchair users.

The vertical gradient from a general walkway to the walkway of a bus stop should have a runoff not greater than 5%.

6.10 STOPPING LANE

The stopping lane is a necessary space for enabling motorized traffic functions (access functions) such as stopping, and also for providing spatial functions such as urban area formation. The width of the stopping lane is determined in consideration of traffic functions according to the design class, traffic volume and types of stopping, and spatial functions for provision of stopping space for utilization of roadside facilities.

6.10.1 PROVISION OF STOPPING LANE

Development along rural roads are scattered so few lay-bys may be adequate for stopping activities there. The shoulders can conveniently serve the purpose. However, in the urban area there are much roadside developments and regular stopping places may not be adequate, especially where there are walkways, and it is necessary to provide stopping lanes on the right side of the carriageways. This ensures safety and convenience.

On inter-city roads and city roads, irrespective of entry and exit restrictions, there is a need for stopping lanes on roads as shown in **Plate 6.4**.

However, the purpose of the stopping lane differs between that on inter-city roads and city roads (especially dedicated roads subject to entry and exit restriction), where it is intended to remove distressed vehicles from the carriageway and to enable stopping for access to roadside facilities respectively.

City roads form the urban road network, and if the roadside is a commercial or business district, it is expected that there will be a high demand to stop vehicles on the road for accessing roadside commercial facilities, delivery of goods etc. Therefore, providing stopping lanes will not only make it possible to secure smooth traffic flow, but also make overall roads wider and thus enhance the urban landscape. Moreover, on roads planned inside areas that are expected to be developed as commercial or business districts in the future, where emphasis should be placed on vehicular access functions, it is desirable to actively establish stopping lanes from the planning stage in anticipation of demand for stopping vehicles.

Moreover, if no stopping lane is provided, since the stopping of buses can be expected to cause great disruption, it is necessary to provide bus stopping areas. If a service road is provided, it is natural to expect that the demand for stopping vehicles on the main road will be low. In such instances the stopping lanes are provided on the right side of the service road. **Table 6.16** shows the recommended width of stopping lanes.

Table 6.16 Recommended width of stopping lane

Vehicle type	Stopping lane width (m)	
	Desirable	Absolute minimum
Large vehicles	3.0	2.5
Small vehicles	2.0	1.5

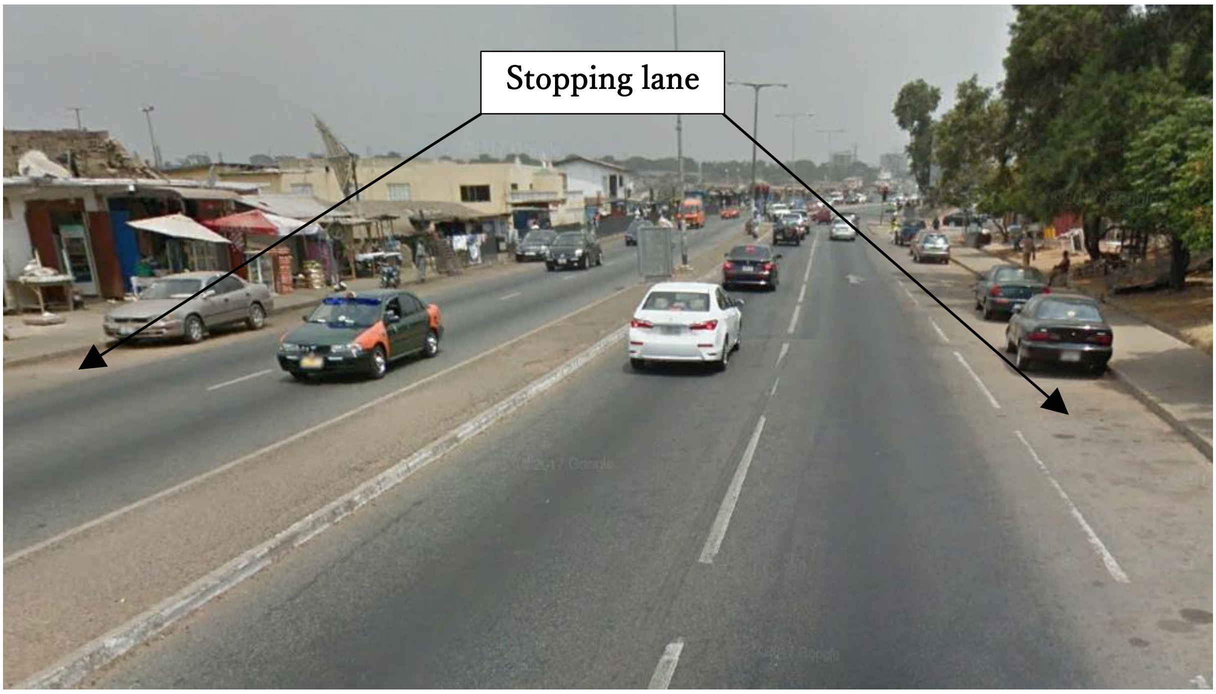


Plate 6.4 Stopping lane along a multilane arterial road

6.10.2 STOPPING LANE OPERATION METHOD

Naturally, a stopping lane is used for its inherent objective of letting vehicles temporarily stop, however, other methods of operation can also be considered depending on the conditions. By permitting only temporary stopping by vehicles, an advantage can be gained in enabling bicycles, motor-assisted bicycles, etc. to use the stopping lane.

Around intersections, since it is not a good idea to allow parking or stopping of vehicles on intersection entry and exit lanes, it is prudent to provide an additional lane or convert stopping lanes into wide walkways or bicycle and pedestrian lanes.

6.11 WALKWAY AND BICYCLE LANE

Walkways (sidewalk) and bicycle lanes are necessary spaces for enabling passage by pedestrians and bicycles and access to roadside facilities, stopping and staying and other pedestrian traffic functions. In addition, it is a space for providing spatial functions of urban area formation, disaster prevention, environmental improvement, and housing of facilities. The widths of walkways and bicycle lanes are determined in consideration of traffic functions according to the class of road, traffic volume and conditions of passage, and spatial functions for provision of evacuation paths, lighting, landscaping, and housing of utilities.

6.11.1 FUNCTIONS OF WALKWAYS/BICYCLE LANE

The functions of walkways and bicycle lanes include passage, access to roadside facilities, stopping and staying functions such as standing and chatting by pedestrians. They also have spatial functions in terms of urban area formation and provision of environmental space.

i. Traffic functions

Traffic function is secured in the same area by respectively separating pedestrians, bicycles and vehicles and the safe passage of each is enhanced and the smooth flow of vehicular traffic is ensured (by preventing slow-moving bicycles from encroaching on the carriageway and impacting the traffic capacity and travel speeds). Stopping and staying functions are also secured for pedestrians who are waiting at traffic signals, for buses or standing and chatting.

ii. Spatial functions

The overall road space including walkways and bicycle lanes helps form an expansive urban environment and fulfil spatial functions by offering tree planting space and forming environmental space and housing utilities such as sewers, water, and communication cables. It is important to consider the total balance of the entire road.

6.11.2 SECURING SPACE FOR WALKWAYS AND BICYCLE LANE

i. Passage space of pedestrians and bicycle lane

From the viewpoint of traffic functions, walkways and bicycle passage space should be planned in consideration of unique network needs. When planning networks for such spaces, it is important to examine from the viewpoint of comprehensive district traffic planning and to coordinate plans with local related agencies, residents etc.

ii. Separation of pedestrians, bicycles and vehicles

To ensure safety on roads that have a lot of fast-moving vehicular traffic, as a rule, the vehicle passage space should be separated from the pedestrian and bicycle passage space. Moreover, in cases where there is a lot of pedestrian and bicycle traffic, separate passage spaces should be provided for pedestrians and bicycles.

iii. Catering for diverse modes of use

It is important to build a society in which all persons including the vulnerable and physically challenged persons can safely participate with peace of mind. Concerning transit, to reduce the burden and enhance convenience and safety, it is important to advance the accessibility, and improvement of pedestrian spaces. Also, it is necessary to cater for diverse modes of use of pedestrian spaces by providing stopping and staying functions that permit families to walk shoulder to shoulder and pedestrians to stand, chat and wait for buses and so on.

6.11.3 PROVISION OF BICYCLE LANE AND WALKWAY

6.11.3.1 WIDTH SETTING

In this Guide, the minimum widths are specified respectively for bicycle lanes, bicycle and pedestrian lanes, and walkways. Concerning bicycle and pedestrian paths and walkways, minimum widths are specified for general cases and cases of heavy pedestrian traffic volume, and additional widths are prescribed for provision of pedestrian bridges, benches, roadside trees, etc.

When setting the width of walkways, it is necessary to assume various modes of use on the roads in question while also considering the area characteristics and type of road. Moreover, in cases of roads in inner cities or close to bus terminals, it is sometimes necessary to provide width for walkways, within the overall road width to secure pedestrian space with emphasis on spatial functions as necessary, and to adopt broad walkways, bicycle and pedestrian paths endowed with roadside trees, etc. in consideration of the balance between the vehicular space and pedestrian and bicycle space. **Figure 6.20** is a flow chart for setting up walkway and bicycle lanes.

6.11.3.2 WIDTH SETTING IN CONSIDERATION OF TRAFFIC FUNCTIONS

From the viewpoint of traffic functions, it is necessary to consider various modes of use by various users. When projecting such uses, it is necessary to consider land uses in the local area, roadside location conditions, locations of bus terminals, bus stops, schools, hospitals, shopping districts, etc., other local conditions, and the role of the target road within the local road network. As an example of diverse modes of use for reference purposes, necessary widths and lengths for vulnerable persons are specified in **Table 6.17.**

Table 6.17 Recommended width and length for the vulnerable

User	Width (m)	Length (m)
Visually impaired persons using a long cane	1.2	
Visually impaired persons using a guide dog	1.2	
Visually impaired persons with a companion	1.3	
Cane users	0.85	
Armrest users	1.0	
Wheelchair users	1.1	
Stroller users	1.0	2.0
Wheelchair users with a companion	1.0	2.5

6.11.3.3 WIDTH SETTING IN CONSIDERATION OF SPATIAL FUNCTION

Generally speaking, the width of walkways, bicycle lanes, etc. is set in consideration of traffic functions, however, in cases of symbolic roads in inner city areas and other roads that are necessary for securing a pleasant urban landscape, it is necessary to also consider spatial functions when deciding width. Spatial functions include isolated functions such as paving, bicycle lanes, etc. and also overall road functions combined with the carriageway, and it is necessary to set the width appropriately upon considering both aspects separately. Moreover, on roads where consideration is given to spatial functions, since the overall width tends to become large, it is necessary to adopt broad walkways and bicycle lanes while considering balance within the overall width.

Figure 6.20 is a flow chart for setting up walkway and bicycle lanes. The recommended widths and typical cross sections of Types 1, 2 and 3 are shown in **Figure 6.21** and **Figure 6.22**.

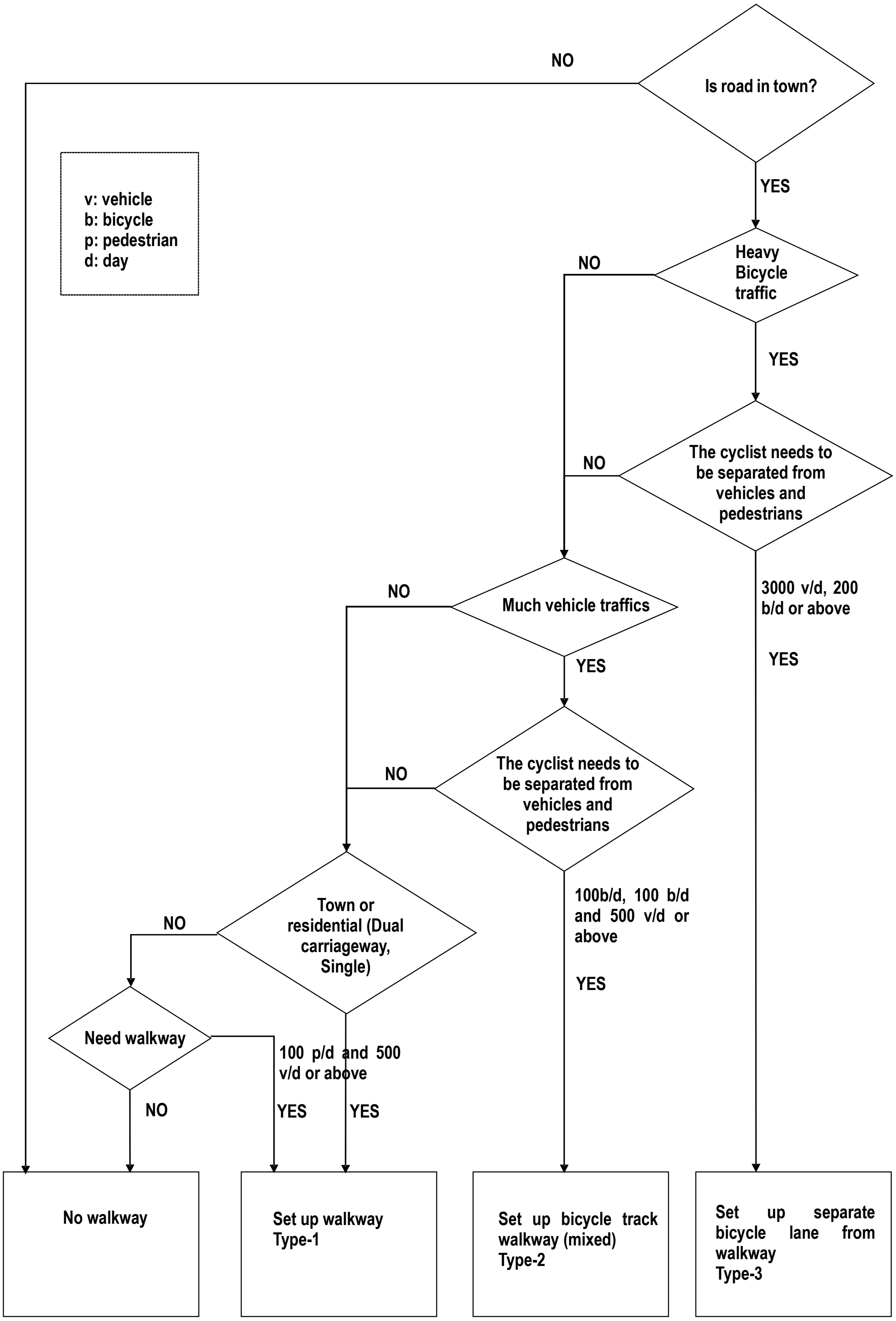


Figure 6.20 Flow chart for setting up walkway and bicycle lanes

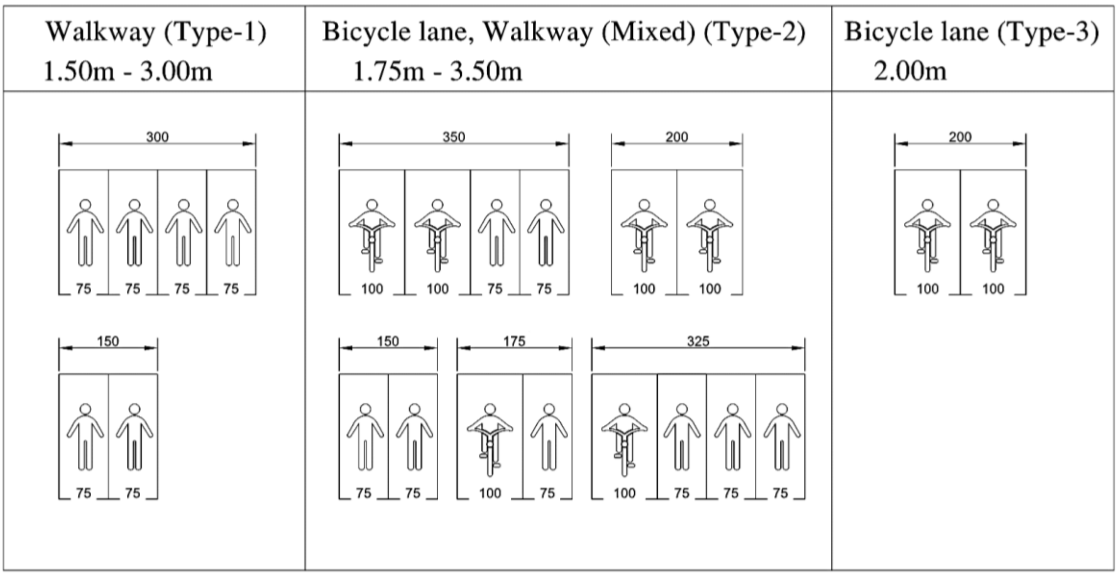


Figure 6.21 Width of walkway and bicycle lane

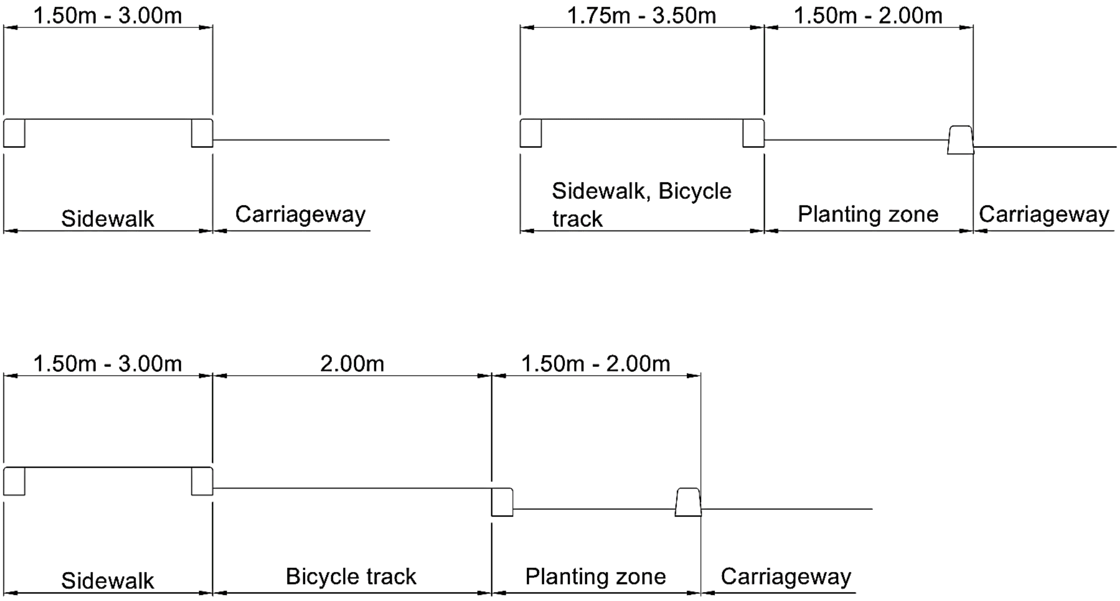


Figure 6.22 Typical cross section of walkway and bicycle lane

6.11.3.3.1 WALKWAY (TYPE-1)

Width is set between 1.5m-3.0m according to the pedestrian traffic volume. On roads with a lot of pedestrian traffic, the width is set at 3.0m to allow four pedestrians (occupation width 0.75m each) or a wheelchair (occupation width 1.0m) and two pedestrians (occupation width 0.75m each) to pass each other, while on roads that have little traffic volume, the width is set at 1.5m to allow two pedestrians to pass by each other.

Moreover, stopping and staying space is sometimes secured at bus stops and pedestrian crossings, etc. to ensure that people waiting for a bus or traffic signals do not obstruct the passage of other pedestrians. Furthermore, to secure comfort and a spacious feel to roads in consideration of various types of pedestrians and the need to stop and chat etc. when walking, it is also effective to consider gathering spaces upon making sure that the safe and smooth passage of pedestrians and bicycles is not hindered.

6.11.3.3.2 BICYCLE AND PEDESTRIAN LANE (TYPE-2)

Width is set between 1.5m-3.5m in anticipation of the mixed passage of pedestrians and small number of bicycles. On roads with a lot of pedestrian traffic, the width is set at 3.5m to allow two wheelchairs (occupation width 1.0m each) and bicycles/pedestrians to pass each other or overtake, while on roads that have little traffic volume, the width is set at 1.5m to allow two pedestrians to pass by each other.

A bicycle and pedestrian lane is adopted to separate bicycles from the carriageway, however, depending on the traffic conditions, coupled with collisions that may arise between bicycles and pedestrians, it is necessary to secure sufficient space for passing and overtaking in places where a certain degree of pedestrian traffic can be anticipated, thereby ensuring that the safety and comfort of pedestrian traffic are preserved.

In structural terms, a bicycle and pedestrian lane can be regarded as a bicycle lane and walkway without any boundary, however, to ensure some separation between bicycles and pedestrians, plants and road signs are placed at specific intervals along the bicycle and pedestrian lane in some cases.

Moreover, stopping and staying space is sometimes secured at bus stops and pedestrian crossings, etc. to ensure that people waiting for a bus or traffic signals do not obstruct the passage of other pedestrians. Furthermore, to secure comfort and a spacious feel to roads in consideration of various types of pedestrians and the need to stop and chat etc. when walking or riding, it is also effective to consider gathering spaces upon making sure that the safe and smooth passage of pedestrians and cyclist is not hindered.

6.11.3.3.3 BICYCLE LANE

Standard width is set at 1.0m as the dedicated width for one bicycle, and 2.0m is adopted to allow bicycles to pass by or overtake each other.

In setting the width of a bicycle lane, in cases where heavy bicycle traffic volume is anticipated, it is necessary to secure sufficient width and space to accommodate this traffic. The traffic capacity of a bicycle lane of width 2m (two directions) is 1,600 bicycles/hour and if the design traffic volume is greater than this, it is necessary to adopt an appropriate width according to the design traffic volume.

6.11.3.4 IMPORTANT POINTS TO DETERMINE WIDTH

When deciding the widths of walkways, bicycle lanes, etc., in addition to satisfying the minimum width requirements in this Guide, it is necessary to set the appropriate width upon also paying attention to the following requirements.

i. To secure the safe and comfortable passage of pedestrians, sufficient width should be secured in consideration of diverse modes of use. In other words, in commercial districts and areas around bus terminals etc., it is necessary to adopt the appropriate width upon fully considering the traffic volume of pedestrians, etc., peak time characteristics, walking conditions, etc.

ii. To facilitate the smooth movement of pedestrians on walkways, it is desirable to secure a broad width for as continuously as possible (except when there are road facilities, kerbstones, etc.).

iii. Around bus stops, it is necessary to secure space for boarding and alighting and make sure that there is no conflict with pedestrians, etc.

iv. On roads in built-up areas, it is necessary to secure the width required to, for example, enhance the road landscape, preserve the roadside environment, house road facilities and buried structures etc. Especially on streets in inner city parts, where walkways, bicycle lanes, etc. play an important role in securing spatial functions, it is desirable to adopt broad walkways, bicycle lanes, etc. with a lot of greenery.

v. On urban roads that have short intervals between intersections, in addition to the primary objective of facilitating the passage of pedestrians, etc., it is necessary to consider that walkways, etc. have a large secondary effect in terms of improving the sight distance to intersecting roads and contributing to traffic safety.

vi. In cases of access roads connecting regional communities that have few pedestrians, walkways may have the minimum width of 2m stipulated in this Guide, and walkway can be installed on one side only if there is no need to access roadside areas. Also, in mountainous areas where roadside uses are limited and the pedestrian traffic volume is very low, walkways are sometimes not provided at all.

vii. On tunnel sections, since having wide walkways lead to larger cross-section and greater costs, one sided walkways or even only inspection passages, etc. may be provided if the pedestrian traffic volume is low. Generally, the minimum necessary width should be adopted.

viii. On rural roads with low volume of pedestrian and bicycle traffic, passage space for pedestrians and bicycles can be secured through the shoulder. **Figure 6.23** shows an example of walkway on a street.

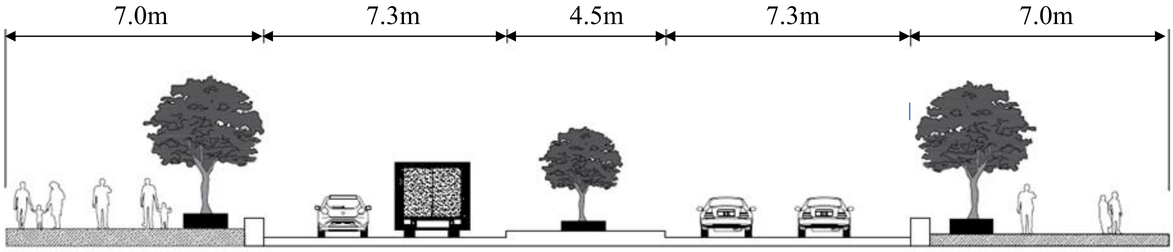


Figure 6.23 Example of walkway on a street

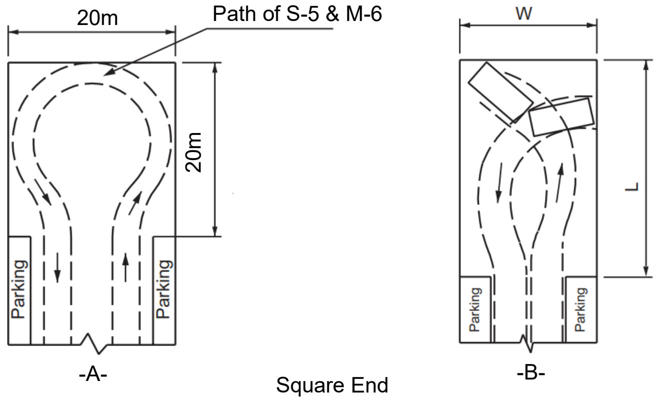
6.12 CUL-DE-SACS AND TURNAROUNDS

A local street/road open at one end only should have a special turning area at the closed end. This turning area desirably should be circular and have a radius appropriate to the vehicle types expected. Minimum outside radii of 10 m in residential areas and 15 m in commercial and industrial areas are commonly used.

A dead-end street narrower than 12 m usually should be widened to enable passenger vehicles, and preferably delivery trucks, to make U-turns or at least turn around by backing only once. The design commonly used is a circular pavement symmetrical about the centreline of the street sometimes with a central island, as shown in **Figure 6.24C**, which also shows minimum dimensions for the design vehicles. Although this type of cul-de-sac operates satisfactorily, improved operations may be obtained if the design is offset so that the entrance-half of the pavement is in line with the approach-half of the street, as shown in **Figure 6.24D**. One steering reversal is avoided on this design. Where a radius of less than 15 m is used, the island should be bordered by sloping kerbs to permit the manoeuvring of an occasional oversized vehicle.

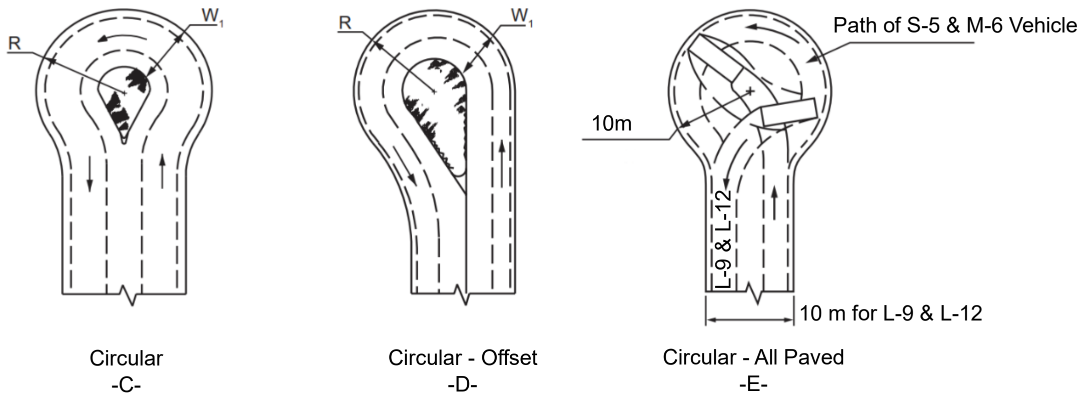
An all-paved plan, as opposed to an island configuration, with a 10m outer radius, shown in **Figure 6.24E**, needs little additional paving. If the approach pavement is at least 10 m wide, the result is a cul-de-sac on which small/medium vehicles can make the customary U-turn and large vehicles can turn by backing only once.

Other variations or shapes of cul-de-sacs that include right-of-way and site controls may be provided to permit vehicles to turn around by backing only once. Several types (**Figure 6.24F**, **Figure 6.24G**, **Figure 6.24H**, and **Figure 6.24I**) may also be suitable for access roads. The geometry of a cul-de-sac should be altered if adjoining residences also use the area for parking.



Square End

Design Vehicle	W (m)	L (m)
S-5 & M-6	10	20
L-9 & L-12	15	30



Design Vehicle	R (m)	W (m)
S-5 & M-6	10	6
L-9 & L-12	13	8
T-17	15	10

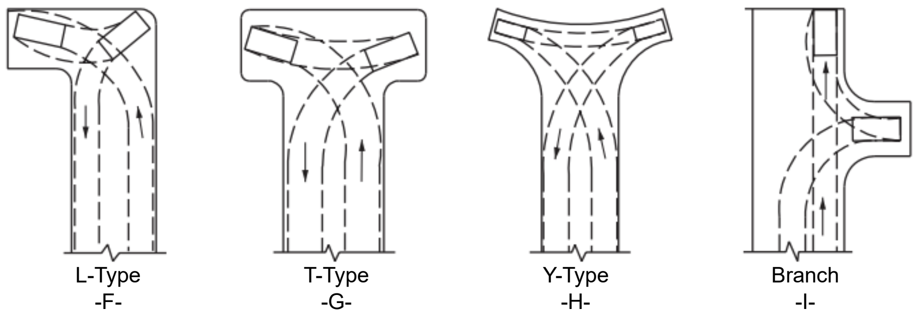


Figure 6.24 Types of Cul-de-Sacs and Dead-End Streets

Dead-end of access roads should include a turning area in accordance with **Figure 6.25**. This dead-end turning area design may be suitable for application on some very low-volume roads.

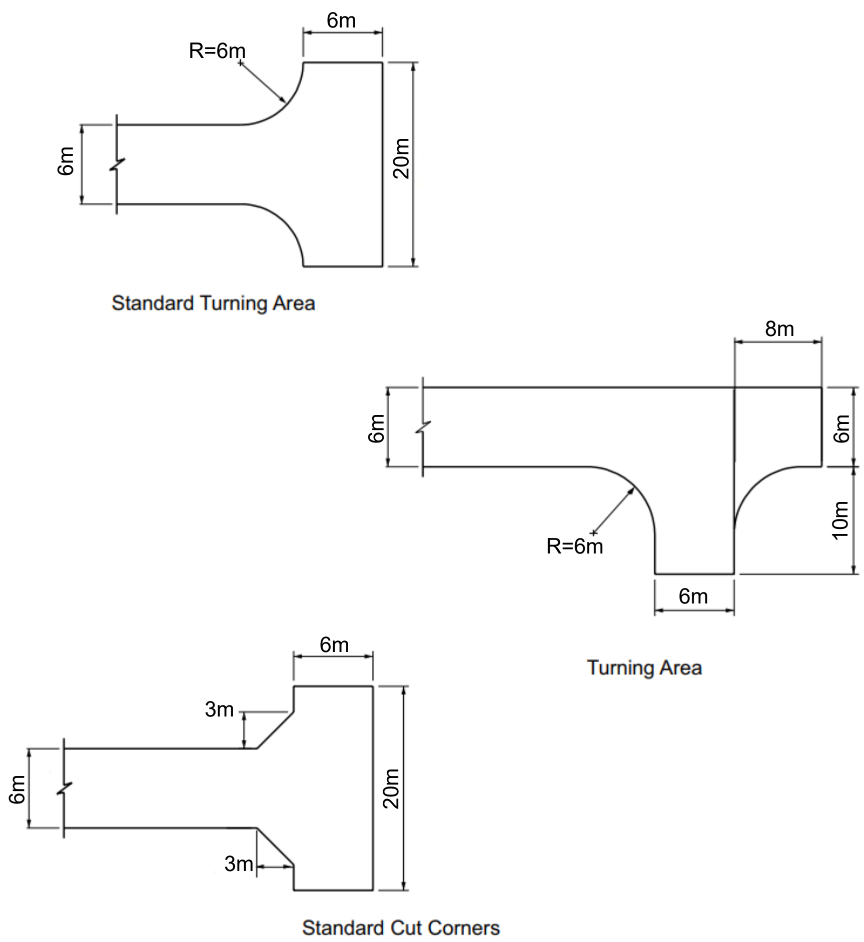


Figure 6.25 Access Road Turnarounds

6.13 PLANTING ZONE

Planting zones should be considered for roads through built-up area with high vehicular, bicycle and pedestrian traffic. They ensure safety and comfort of road users by creating the following:

i. Visual guidance effect
ii. Light interception effect
iii. Wind breaking effect
iv. Shading effect
v. Excellent aesthetics effect
vi. By separating vehicles, bicycles and pedestrians

Wider planting zones (buffer zones) can be used in heavily trafficked residential districts to prevent deterioration of the surroundings through air and noise pollution and vibration**.**

Figure 6.26 and **Figure 6.27** show Typical Road Cross-section with Planting Zone and Buffer Zone respectively.

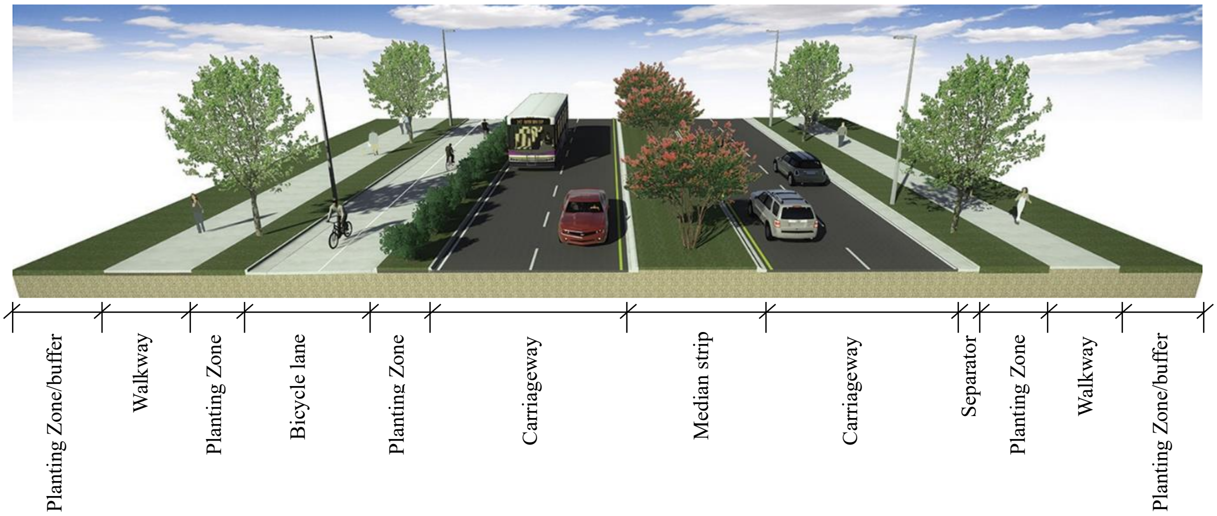


Figure 6.26 Typical road cross section with planting zone

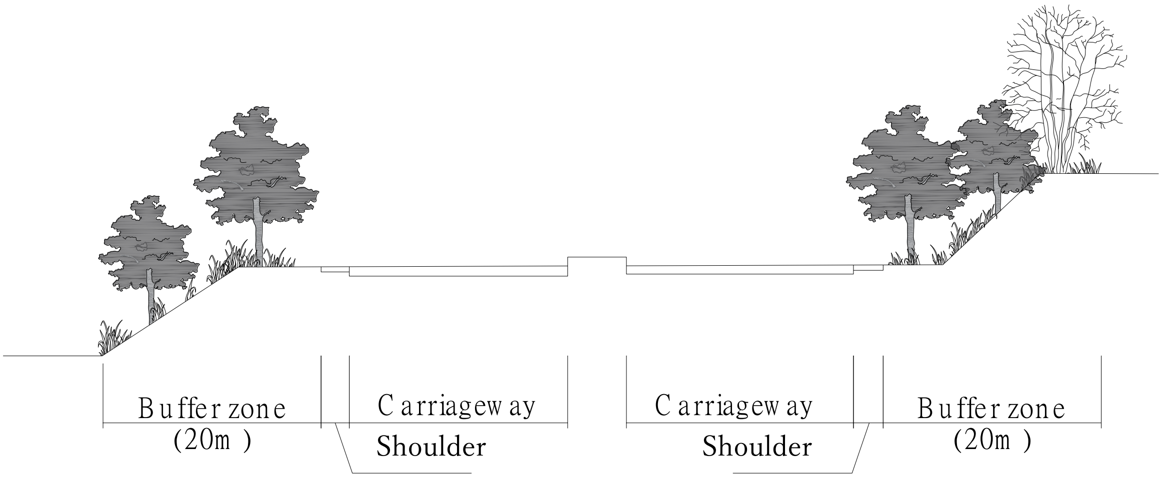


Figure 6.27 Typical road cross section with buffer zone

6.14 SERVICE ROAD

A service road is a subsidiary road running parallel to a main road and giving access to houses, shops, or businesses. It is also a space for providing spatial functions such as urban area formation.

6.14.1 WIDTH OF SERVICE ROAD

The width of the service road is determined in consideration of traffic functions according to the traffic volume, demand for stopping of vehicles, passage of large vehicles, etc., and spatial functions for area development, provision of parking spaces for utilizing roadside facilities.

The absolute minimum standard width of service roads is 5.5m including shoulders. This is prescribed to ensure the safe and smooth passage of vehicles. In deciding the width of service roads, in addition to demand for stopping and traffic functions such as passage of large vehicles, it is necessary to fully consider spatial functions such as urban area formation, disaster prevention, etc. according to necessity.

6.14.2 SHOULDER

Concerning the shoulder that is provided on the right side of a service road adjacent to the carriageway, since it is not necessary to have a shoulder with the same level of specifications as the main carriageway, a uniform width of 2.0m or more is adopted regardless of the design class. Width of 0.5m is also provided for inner shoulders as shown in **Figure 6.28**.

If road facilities are installed on the outer shoulder, the width should be 1.5m plus the width required to install the facilities.

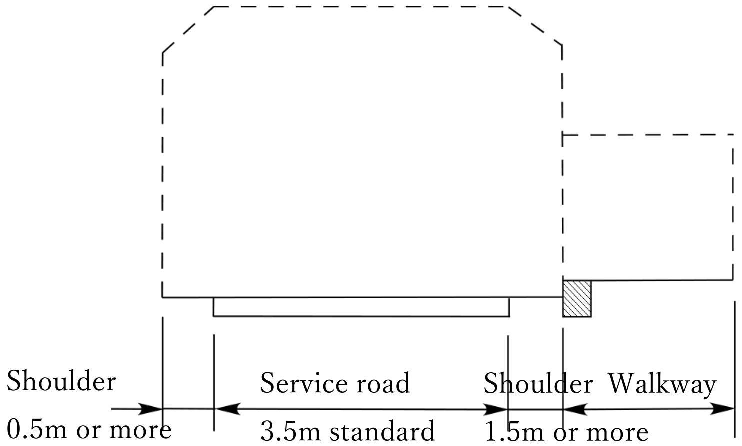


Figure 6.28 Cross section of service road

6.14.3 WALKWAY AND BICYCLE PASSAGE SPACE

When providing service roads, it is necessary to provide walkways and bicycle lanes to ensure the safe passage of pedestrians and bicycles. **Plate 6.5** is example of walkway and bicycle lane perspective view.

Plate 6.5 Examples of roads with walkway/bicycle lane

6.15 CLEARANCE

Lateral and vertical clearances are additional spaces besides normal width and height of vehicles provided for the safety of road users.

Accordingly, road furniture like safety fences and signposts, bridges, telephone, and electricity poles, etc. must be provided outside the clearances.

The recommended vertical clearance is 5.5m for carriageways and 2.5m for walkways and bicycle lanes (**Figure 6.31**). Any limitation in height should be indicated by a warning sign.

The recommended lateral and vertical clearances are shown in **Figure 6.29**, **Figure 6.30** and **Figure 6.33**.

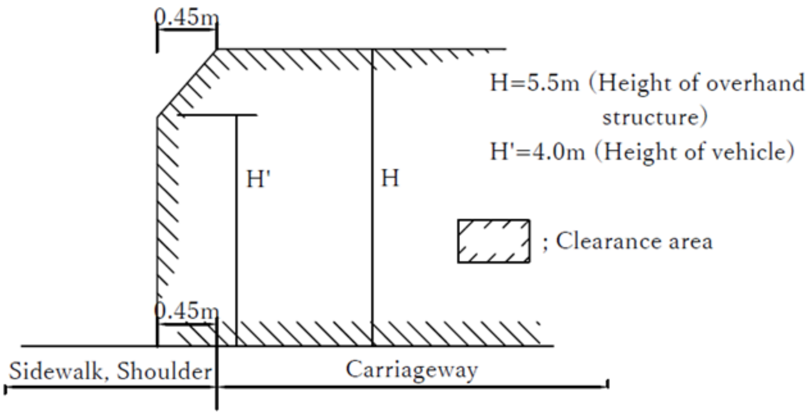


Figure 6.29 Carriageway clearance, carriageway with shoulder or walkway

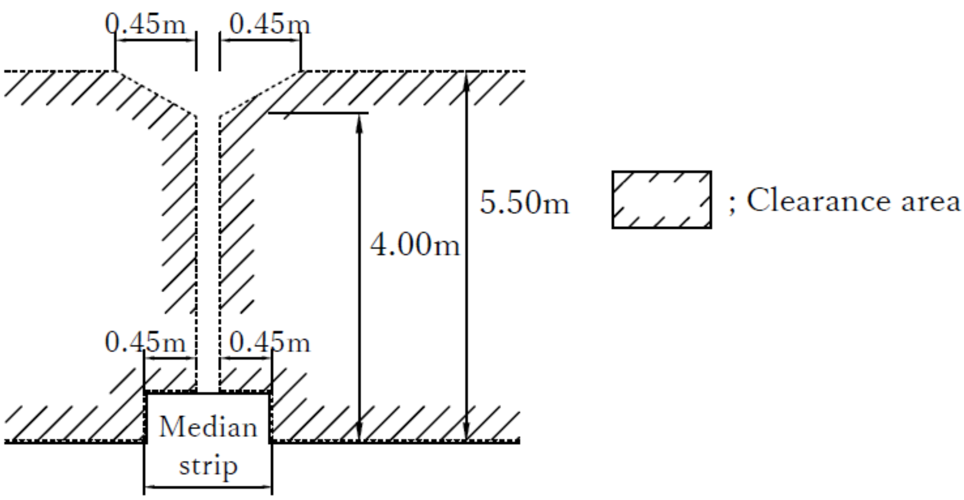


Figure 6.30 Carriageway clearance

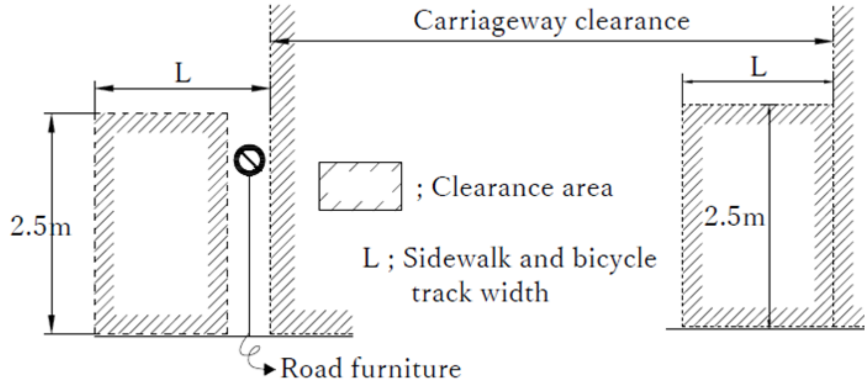


Figure 6.31 Walkway and bicycle lane clearance

An illustration of vertical clearances for a normal cross slope and superelevated carriageways are shown in **Figure 6.32**.

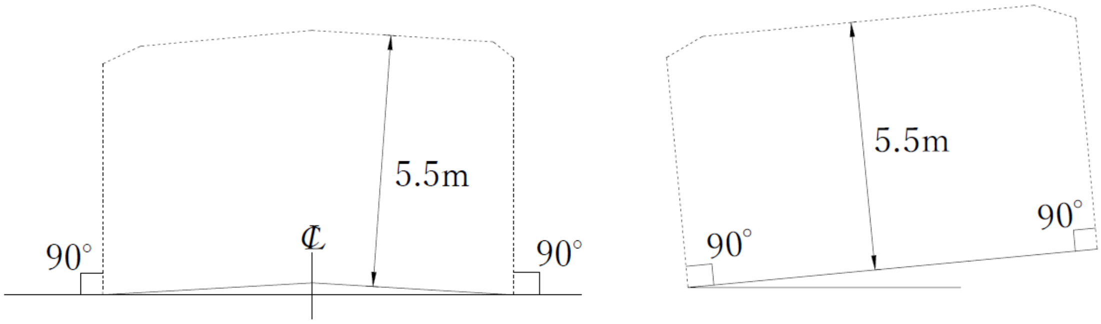


Figure 6.32 Vertical clearances for normal cross slope and superelevated carriageways

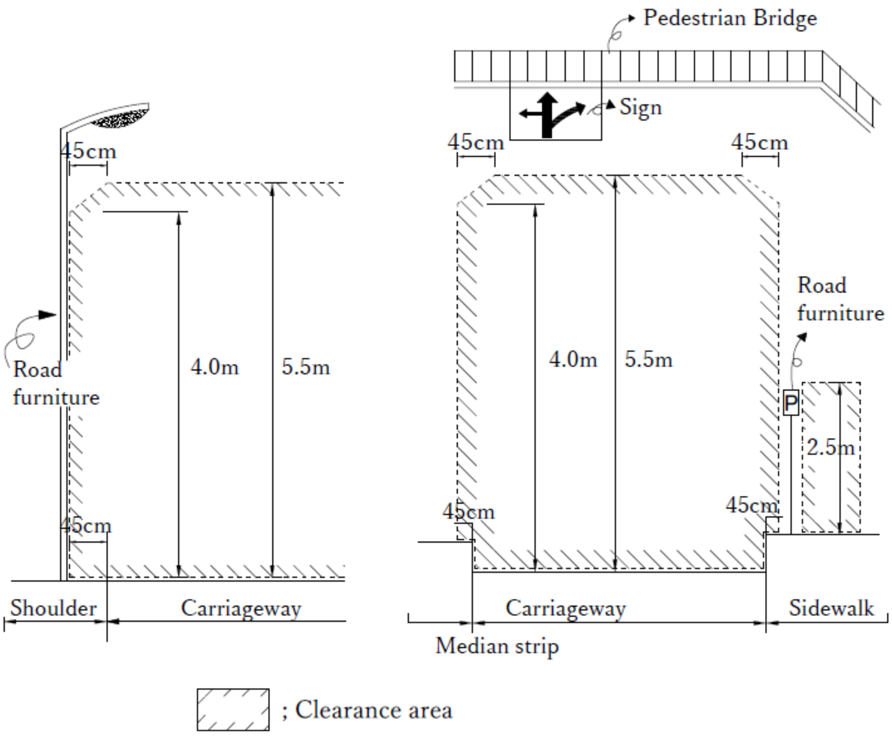


Figure 6.33 Example of clearance

6.16 RECOVERY ZONE

Vehicles need a safe recovery or clear zone to stop when they accidentally leave the road. This area is called the safety recovery zone or clear zone. Roads designed for higher traffic speed and density have wider safety recovery zones than roads designed for slower speeds and less traffic. Cut slopes require narrower safety recovery zones than do flat areas. Steep fills require the widest safety recovery zones. Since many vehicles continue on to the bottom of steep slopes, a clear area beyond the edge of the slope is desirable. A wider recovery zone is necessary at curves because cars are likely to leave the road at a sharper angle.

The clear zone falls within an area called the recovery zone. The recovery zone is the total unobstructed traversable area available along the edge of the road and, by convention, it is measured from the edge of the closest travel lane. The recovery zone may have recoverable slopes, non-recoverable slopes and a clear run-out area.

Recoverable slopes are those on which a driver may, to a greater or lesser extent, retain or regain control of a vehicle. A non-recoverable slope may be traversable, but a vehicle will continue to the bottom. A clear run-out area is located at the toe of a non-recoverable slope and is available for safe use by an errant vehicle. There is also provision for a smooth transition between slopes to allow for the safe passage of vehicles.

The clear zone is the total, fixed-object-free area available to the errant vehicle.

Trees and other fixed objects such as advertising signs and utility poles are the major problem in the safety recovery zone because they are the greatest cause of fatalities from vehicle collisions with fixed objects. The size and height of trees permitted in the recovery zone, ranges from no trees allowed at all to trees allowed that are less than four inches in diameter at one foot above ground. Shrubs that do not block visibility are usually acceptable. A vehicle leaving the road can be funnelled along the ditch bottom, or onto the backslope. Therefore, fixed objects should not be located in or near the bottom of a ditch or on the backslope near the ditch. In situations where it is inevitable to provide fixed objects such as sign posts, utility poles and trees refer S**ections 12.8**.

Figure 6.34 illustrates a recoverable slope followed by a non-recoverable slope. Since the clear zone distance extends onto a non-recoverable slope, the portion of the clear zone distance on such a slope may be provided beyond the non-recoverable slope if practical. This clear runout area would then be included in the total recovery area. The clear runout area may be reduced in width based on existing conditions or site investigations. Such a variable slope typical section is often used as a compromise between roadside safety and economics. By providing a relatively flat recovery area immediately adjacent to the roadway, most errant motorists can recover before reaching the steeper slope beyond. The slope break may be liberally rounded so an encroaching vehicle does not become airborne. It is suggested that the steeper slope be made as smooth as practical and rounded at the bottom.

Table 6.18 can be used to determine the suggested clear-zone distance for selected traffic volumes and speeds. However, **Table 6.18** provides only a general approximation of the needed clear-zone distance. These data are based on limited empirical data that were extrapolated to provide information for a wide range of conditions. The designer should keep in mind site-specific conditions, design speeds, rural versus urban locations, and practicality. The distances obtained from **Table 6.18** should suggest only the approximate centre of a range to be considered and not a precise distance to be held as absolute. For roadways with low traffic volumes, it may not be practical to apply even the minimum values found in **Table 6.18**.

Table 6.18 Suggested clear zone distance from edge of carriageway

Design speed (km/h)	Design ADT	Width of foreslope (m)			Width of backslope (m)		
		1V:6H or flatter	1V:5H to 1V:4H	1V:3H	1V:3H	1V:5H to 1V:4H	1V:6H or flatter
≤60	< 750[c]	2.0-3.0	2.0-3.0	[b]	2.0-3.0	2.0-3.0	2.0-3.0
	750-1500	3.0-3.5	3.5-4.5	[b]	3.0-3.5	3.0-3.5	3.0-35
	1500-6000	3.5-4.5	4.5-5.0	[b]	3.5-4.5	3.5-4.5	3.5-4.5
	> 6000	4.5-5.0	5.0-5.5	[b]	4.5-5.0	4.5-5.0	4.5-5.0
70-80	< 750[c]	3.0-3.5	3.5-4.5	[b]	2.5-3.0	2.5-3.0	3.0-3.5
	750-1500	4.5-5.0	5.0-6.0	[b]	3.0-3.5	3.5-4.5	4.5-5.0
	1500-6000	5.0-5.5	6.0-8.0	[b]	3.5-4.5	4.5-5.0	5.0-5.5
	> 6000	6.0-6.5	7.5-8.5	[b]	4.5-5.0	5.5-6.0	6.0-6.5
90	< 750[c]	3.5-4.5	4.5-5.5	[b]	2.5-3.0	3.0-3.5	3.0-3.5
	750-1500	5.0-5.5	6.0-7.5	[b]	3.0-3.5	4.5-5.0	5.0-5.5
	1500-6000	6.0-6.5	7.5-9.0	[b]	4.5-5.0	5.0-5.5	5.0-6.5
	> 6000	6.5-7.5	8.0-10.0[a]	[b]	5.0-5.5	6.0-6.5	6.5-7.5
100	< 750[c]	5.0-5.5	6.0-7.5	[b]	3.0-3.5	3.5-4.5	4.5-5.0
	750-1500	6.0-7.5	8.0-10.0 [a]	[b]	3.5-4.5	5.0-5.5	6.0-6.5
	> 6000	8.0-9.0	10.0-12.0 [a]	[b]	4.5-5.5	5.5-6.5	7.5-8.0
	Over 6000	9.0-10.0[a]	11.0-13.5 [a]	[b]	6.0-6.5	7.5-8.0	8.0-8.5
> 110	< 750[c]	5.5-6.0	6.0-8.0	[b]	3.0-3.5	4.5-5.0	4.5-5.0
	750-1500	7.5-8.0	8.5-11.0[a]	[b]	3.5-5.0	5.5-6.0	6.0-6.5
	1500-6000	8.5-10.0[a]	10.5-13.0[a]	[b]	5.0-6.0	6.5-7.5	8.0-8.5
	> 6000	9.0-10.51	11.5-14.0[a]	[b]	6.5-7.5	8.0-9.0	8.5-9.0

Notes:

a. When a site-specific investigation indicates a high probability of continuing crashes or when such occurrences are indicated by crash history, the designer may provide clear-zone distances greater than the clear zone shown in **Table 6.18**. Clear zones may be limited to 9 m for practicality and to provide a consistent roadway template if previous experience with similar projects or designs indicates satisfactory performance.

b. Because recovery is less likely on the unshielded, traversable 1V:3H foreslope on a fill section, fixed objects should not be present in the vicinity of the toe of these slopes. Recovery of high-speed vehicles that encroach beyond the edge of the shoulder may be expected to occur beyond the toe of slope. Determination of the width of the recovery area at the toe of slope should consider right-of-way availability, environmental concerns, economic factors, safety needs, and crash histories. Also, the distance between the edge of the carriageway and the beginning of the 1V:3H slope should influence the recovery area provided at the toe of slope. While the application may be limited by several factors, the foreslope parameters that may enter into determining a maximum desirable recovery area are illustrated in **Table 6.18**. A 3m recovery area at the toe of slope should be provided for all traversable, non-recoverable fill slopes. When design speeds are greater than the values provided, the designer may provide clear-zone distances greater than those shown in **Table 6.18**.

c. For roadways with low volumes, it may not be practical to apply even the minimum values found in **Table 6.18.**

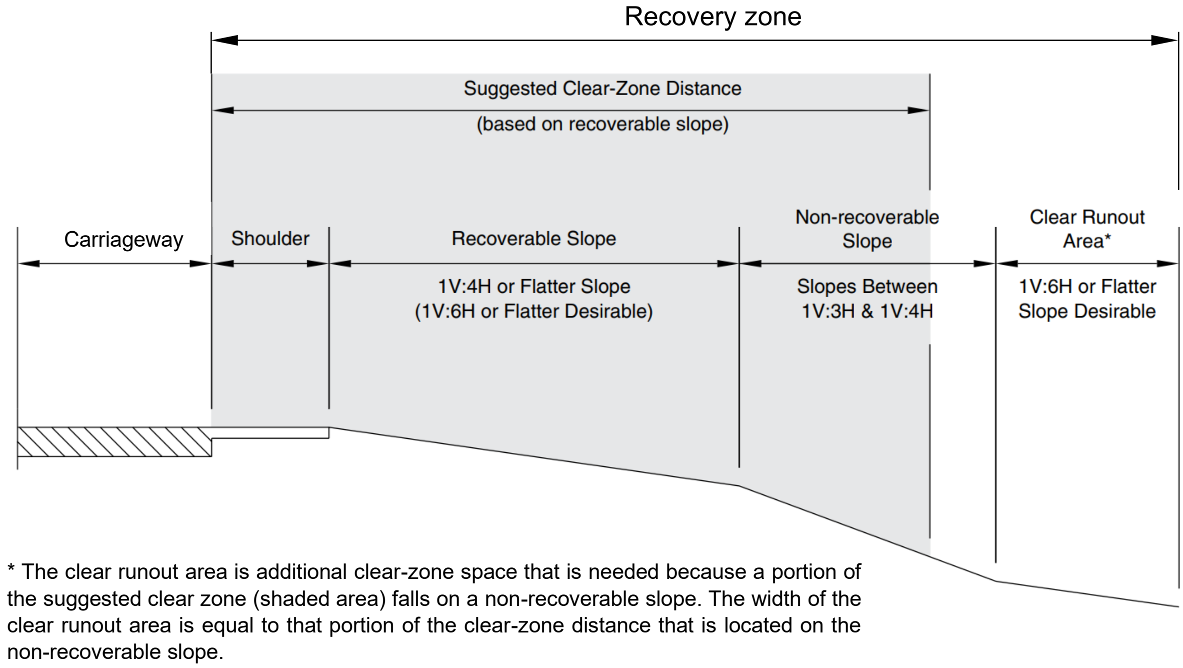


Figure 6.34 Roadside recovery zone

The designer may choose to modify the clear-zone distances in **Table 6.18** with adjustment factors to account for horizontal curvature, as shown in **Table 6.19**. These modifications normally are considered only when crash histories indicate such a need, when a specific site investigation shows a definitive crash potential that could be significantly lessened by increasing the clear zone width, and when such increases are cost-effective. Horizontal curves, particularly for high-speed facilities, are usually superelevated to increase safety and provide a more comfortable ride. Increased banking on curves where the superelevation is inadequate is an alternate method of increasing roadway safety within a horizontal curve.

For relatively flat and level roadsides, the clear-zone concept is simple to apply. However, it is less clear when the roadway is in a fill or cut section where roadside slopes may be positive, negative, or variable, or where a drainage channel exists near the through carriageway.

Consequently, these features should be discussed before a full understanding of the clear zone concept is possible.

Table 6.19 Horizontal Curve Adjustment Factor

Radius m	Design Speed km/h					
	60	70	80	90	100	110
900	1.1	1.1	1.1	1.2	1.2	1.2
700	1.1	1.1	1.2	1.2	1.2	1.3
600	1.1	1.2	1.2	1.2	1.3	1.4
500	1.1	1.2	1.2	1.3	1.3	1.4
450	1.2	1.2	1.3	1.3	1.4	1.5
400	1.2	1.2	1.3	1.3	1.4	-
350	1.2	1.2	1.3	1.4	1.5	-
300	1.2	1.3	1.4	1.5	1.5	-
250	1.3	1.3	1.4	1.5	-	-
200	1.3	1.4	1.5	-	-	-
150	1.4	1.5	-	-	-	-
100	1.5	-	-	-	-	-

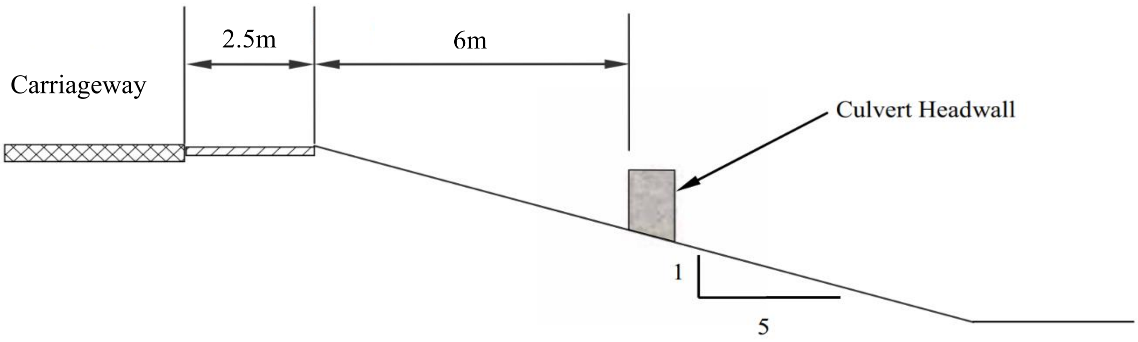
6.16.1 EXAMPLES OF THE CLEAR-ZONE CONCEPT TO RECOVERABLE FORE SLOPES

EXAMPLE 6-A

Design ADT: 4000

Design Speed: 100 km/h

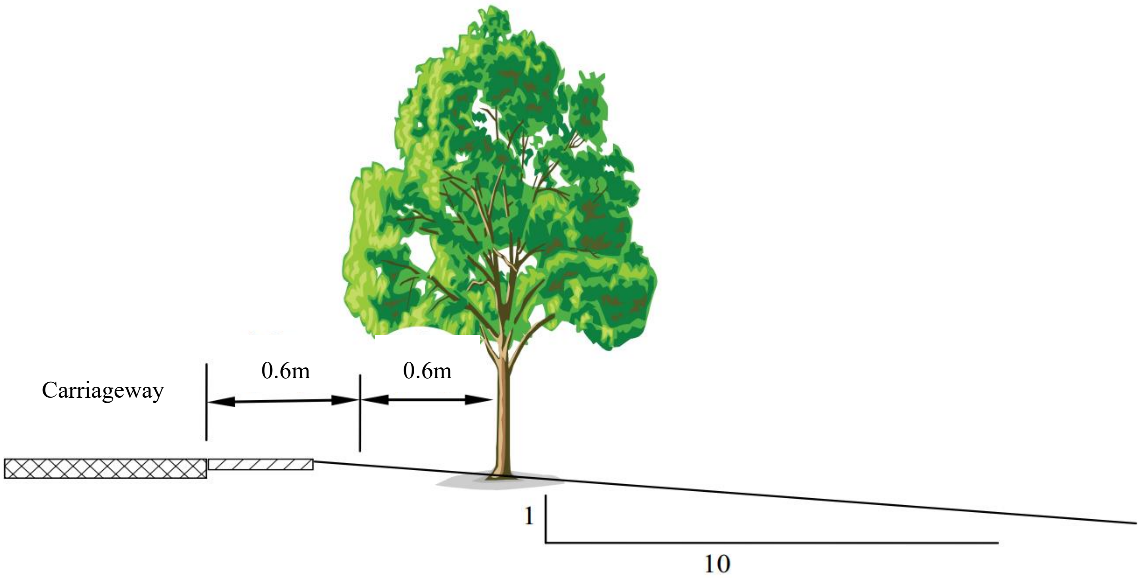
Suggested clear-zone distance for 1V:5H foreslope: 10 to 12 m (from **Table 6.18**)



Discussion—The available recovery area of 8.5 m is 1.5 m to 3.5 m less than the suggested clear-zone distance. If the culvert headwall is greater than 100 mm in height and is the only obstruction on an otherwise traversable foreslope, it should be removed and the inlet modified to match the 1V:5H foreslope. If the foreslope contains rough outcroppings or boulders and the headwall does not significantly increase the obstruction to a motorist, the decision to do nothing may be appropriate. A review of the highway’s crash history, if available, may be made to determine the nature and extent of vehicle encroachments and to identify any specific locations that may require special treatment.

EXAMPLE 6-B

Design ADT: 300
Design Speed: 60 km/h
Suggested clear-zone distance for 1V:10H slope: 2 to 3 m (from **Table 6.18**).

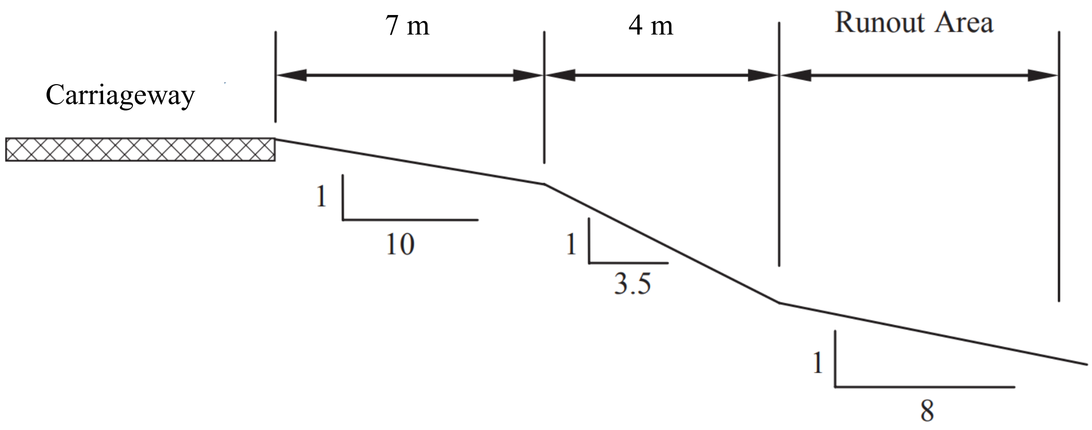


Discussion—The available recovery area of 1.2m is 0.8 to 1.8 m less than the suggested clear-zone distance. If this section of road has a significant number of run-off-the-road crashes, it may be appropriate to consider shielding or removing the entire row of trees within the crash area. If this section of road has no significant history of crashes and is heavily forested with most of the other trees only slightly farther from the road, this tree would probably not require treatment. If, however, none of the other trees are closer to the roadway than, for example, 4.0m, this individual tree represents a more significant obstruction and should be considered for removal. If a tree were 3.0m from the edge of the carriageway, and all or most of the other trees were 5 m or more, its removal may still be appropriate. Also, as this road is very low volume (ADT ≤400), where constraints of cost, terrain, right-of-way, or potential socio/environmental impacts make the provision of a 2m clear recovery area impractical, clear recovery areas less than 2m in width may be used. This example emphasizes that the clear-zone distance is an approximate number at best and that individual objects should be analysed in relation to other nearby obstacles.

EXAMPLE 3-C

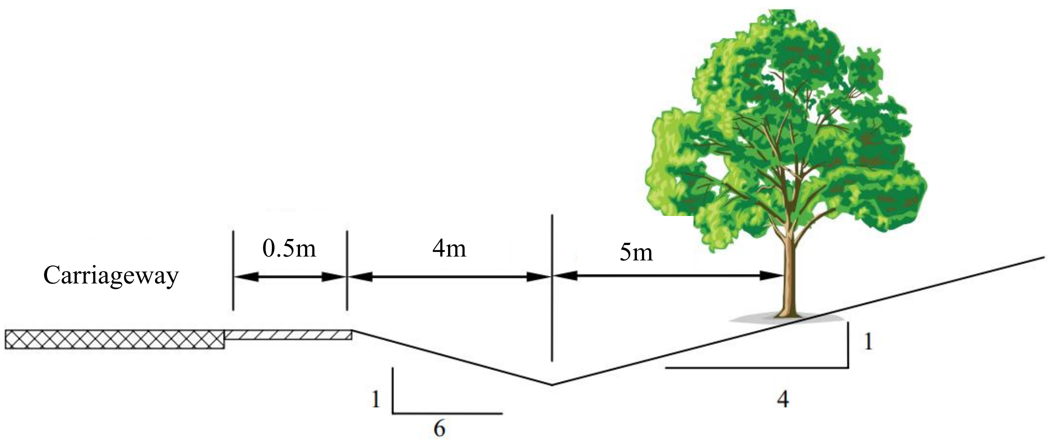
Design ADT: 7000
Design Speed: 100 km/h
Suggested clear-zone distance for 1V:10H foreslope: 9 to 10 m (from **Table 6.18**)
Suggested clear-zone distance for 1V:8H foreslope: 9 to 10 m (from **Table 6.18**)

Available recovery distance before breakpoint of non-recoverable foreslope: 7m
Clear runout area at toe of foreslope: 9 to 10 m minus 7m or 2 to 3 m



Discussion—Since the non-recoverable foreslope is within the recommended suggested clear-zone distance of the 1V:10H foreslope, a runout area beyond the toe of the non-recoverable foreslope is desirable. Using the steepest recoverable foreslope before or after the non-recoverable foreslope, a clear-zone distance is selected from **Table 6.18**. In this example, the 1V:8H foreslope beyond the base of the fill dictates a 9 to 10 m clear-zone distance. Since 7m are available at the top, an additional 2 to 3 m could be provided at the bottom. Since this is less than the 3m recovery area that should be provided at the toe of all the non-recoverable slopes the 3m should be applied. All foreslope breaks may be rounded and no fixed objects would normally be built within the upper or lower portions of the clear-zone or on the intervening foreslope.

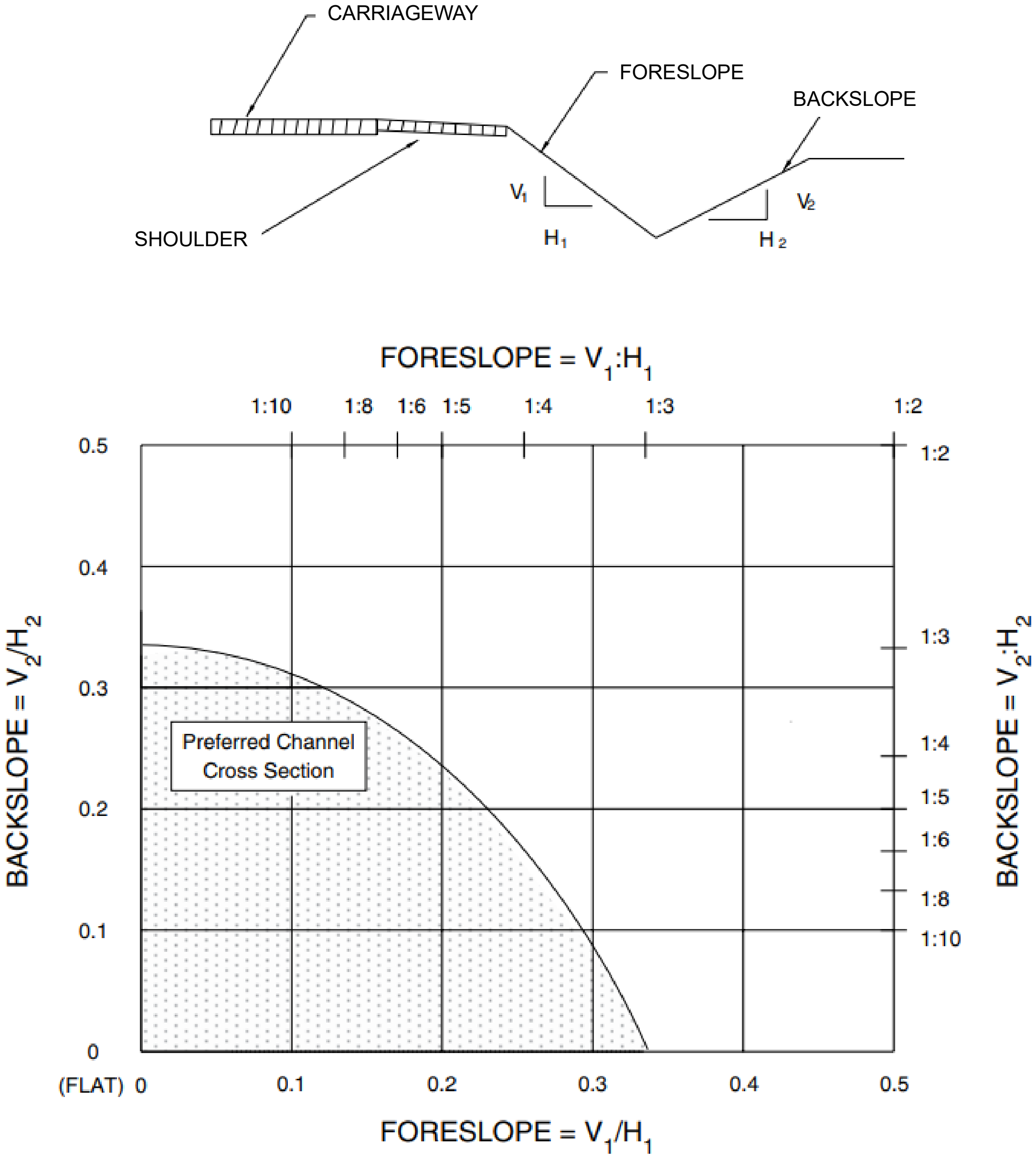
EXAMPLE 3-D
Design ADT: 1400
Design Speed: 100 km/h
Suggested clear-zone distance for 1V:6H foreslope (fill): 6 to 7.5 m (from **Table 6.18**)
Suggested clear-zone distance for 1V:4H backslope (cut): 5 to 5.5 m (from **Table 6.18**)



Discussion—For channels within the preferred cross-section area of **Figure 6.35** and **Figure 6.36**, the clear-zone may be determined from **Table 6.18**. However, when the suggested clear-zone exceeds the available recovery area for the foreslope, the backslope may be considered as additional available recovery area. The range for the suggested clear zone for the foreslope of 6 to 7.5 m extends past the slope break onto the backslope slope. Since the backslope (cut) has a suggested clear-zone of 5 to 5.5 m which is less than the foreslope the larger of the two values should be used. In addition, fixed objects should not be located near the centre of the channel where the vehicle is likely to funnel. An appropriate range for this combination slope could be 6 to 7.5 m.

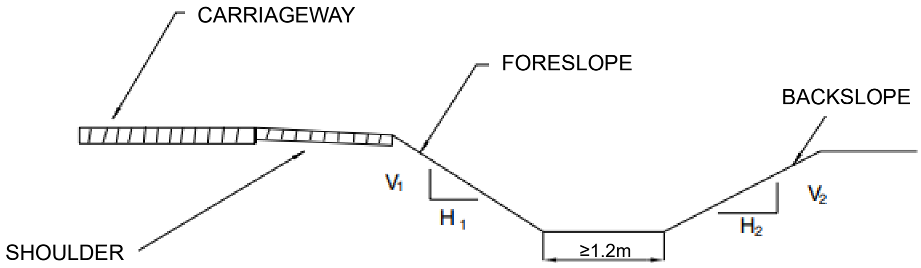
Because the tree is located beyond the suggested clear zone, removal is not required. Removal should be considered if this one obstacle is the only fixed object this close to the carriageway along a significant length.

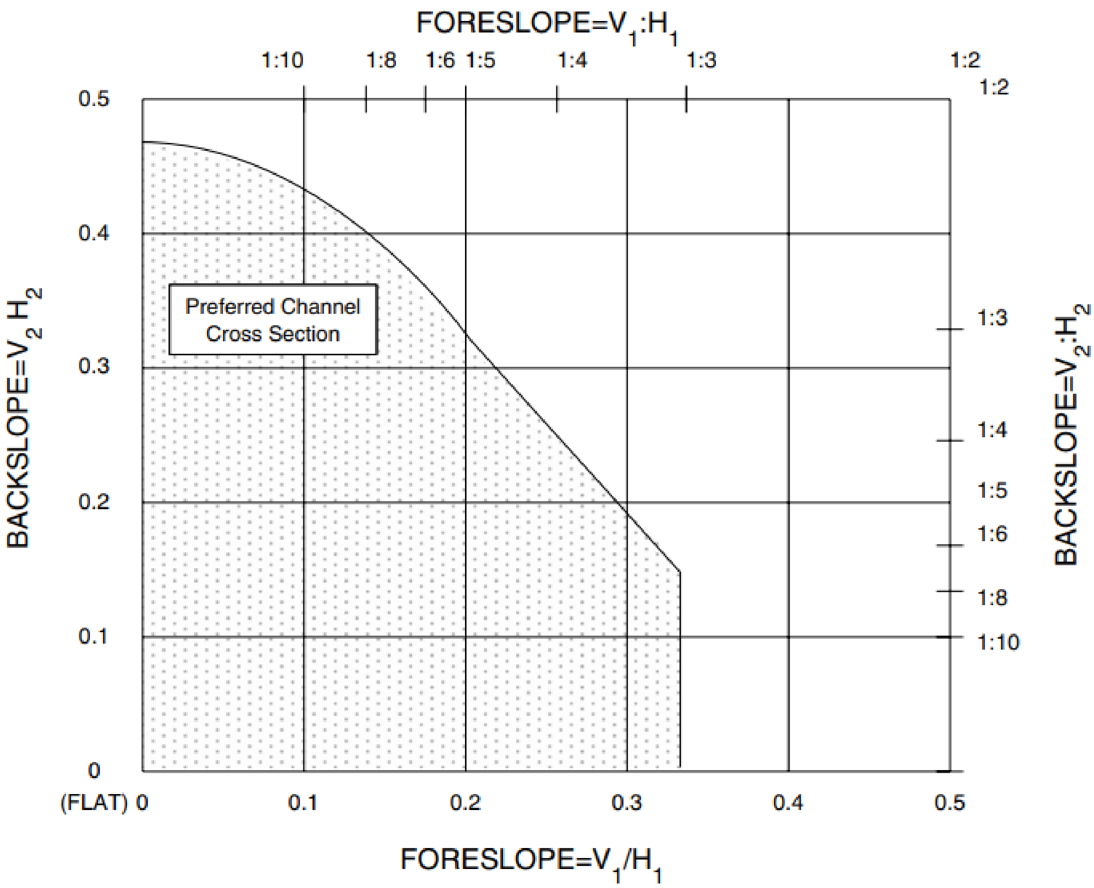
Drainage channels not having the preferred cross section (see **Figure 6.35** and **Figure 6.36**) should be located at or beyond the suggested clear zone. However, backslopes steeper than 1V:3H are typically located closer to the roadway. If these slopes are relatively smooth and unobstructed, they present little safety problem to an errant motorist. If the backslope consists of a rough rock cut or outcropping, shielding may be warranted as discussed in **Section 12.5**.



*This chart is applicable to all Vee ditches, rounded channels with a bottom width less than 2.4 m and trapezoidal channels with bottom widths less than 1.2 m.

Figure 6.35 Preferred Cross Sections for Channels with Abrupt Slope Changes





*This chart is applicable to rounded channels with bottom widths of 2.4 m or more and to trapezoidal channels with bottom widths equal to or greater than 1.2 m.

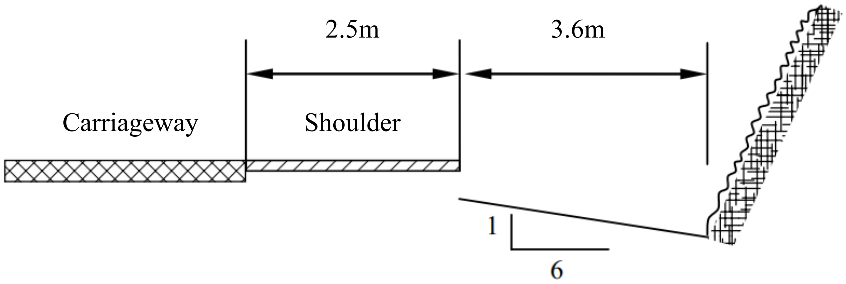
Figure 6.36 Preferred Cross Sections for Channels with Gradual Slope Changes

EXAMPLE 3-E

Design ADT: 3000

Design Speed: 100 km/h

Suggested clear-zone distance for 1V:6H foreslope: 8.0 to 9.0 m (from **Table 6.18**)



Discussion—The rock cut is within the given suggested clear-zone distance but would probably not warrant removal or shielding unless the potential for snagging, pocketing, or overturning a vehicle is high. Steep backslopes are clearly visible to motorists during the day, thus lessening the risk of encroachments. Roadside delineation of sharper than average curves through cut sections can be an effective countermeasure at locations having a significant crash history or potential.

6.17 TUNNELS

6.17.1 GENERAL CONSIDERATIONS

Development of streets or highways may include sections constructed in tunnels either to carry the streets or highways under or through a natural obstacle or to minimize the effect of the roadway on the community. General conditions under which tunnel construction may be warranted include:

i. Long, narrow terrain ridges where a cut section may either be costly or carry environmental consequences
ii. Narrow rights-of-way where all of the surface area is needed for street purposes
iii. Large intersection areas or a series of adjoining intersections on an irregular or diagonal street pattern
iv. Grade-separated pedestrian and/or bicycle facilities are needed
v. Railroad yards, airport runways, or similar facilities
vi. Parks or similar land uses, existing or planned
vii. Locations where right-of-way acquisition costs exceed cost of tunnel construction and operation.

Although the costs of operation and maintenance of tunnels are beyond the scope of this guide, these costs should nevertheless be considered.

General construction and design features of tunnel sections are discussed in the following sections. It is not intended that these sections be considered comprehensive on the subject of the design of highway tunnels. Specific design issues such as soil conditions, construction phasing, ventilating, lighting, pumping, and other mechanical or electrical considerations require specialized engineering. For further information, see the AASHTO LRFD Road Tunnel Design and Construction Guide Specifications.

6.17.2 TYPES OF TUNNELS

Tunnels can be classified into two major categories:

i. tunnels constructed by mining methods, and
ii. tunnels constructed by cut-and-cover methods.

The first category refers to those tunnels that are constructed without removing the overlying rock or soil. Usually, this category is subdivided into two very broad groups according to the appropriate construction method. The two groups are named to reflect the overall character of the material to be excavated: hard rock and soft ground.

Of particular interest to the highway designer are the structural requirements of these construction methods and their relative costs. As a general rule, hard-rock tunnelling is less expensive than soft-ground tunnelling. A tunnel constructed through solid, intact, and homogeneous rock will normally represent the lower end of the scale with respect to structural demands and construction costs. A tunnel located below water in material that needs immediate and heavy support will involve extremely expensive soft-ground tunnelling techniques such as shield and compressed air methods.

The shape of the structural cross section of the tunnel varies with the type and magnitude of loadings. In those cases where the structure will be subjected to roof loads with little or no side pressures, a horseshoe-shaped cross section is used. As side pressures increase, curvature is introduced into the sidewalls and invert struts added. When the loadings approach a distribution similar to hydrostatic pressures, a full circular section is usually more efficient and economical. All cross sections are dimensioned to provide adequate space for ventilation ducts.

The second category of tunnel classification deals with the two types of tunnels that are constructed from the surface: trench and cut-and-cover tunnels. The latter are used exclusively for subaqueous work. In the trench method, prefabricated tunnel sections are constructed in shipyards or dry docks, floated to the site, sunk into a dredged trench, and joined together underwater. The trench is then backfilled. When conditions are favourable with respect to subsurface soil, amount of river current, volume and character of river traffic, availability of construction facilities, and type of existing waterfront structures, the trench method may prove more economical than alternative methods.

The cut-and-cover method is by far the most common type of tunnel construction for shallow tunnels, which often occurs in urban areas. As the name implies, the method consists of excavating an open cut, building the tunnel within the cut, and backfilling over the completed structure. Under ideal conditions, this method is the most economical for constructing tunnels located at a shallow depth. However, it should be noted that surface disruption and challenges in managing utilities generally make this method very expensive and difficult.

6.17.3 GENERAL DESIGN CONSIDERATIONS

Tunnels should be as short as practical because the feeling of confinement and magnification of traffic noise can be unpleasant to motorists, and tunnels are the most expensive highway structures. Pedestrian tunnels should be wide enough to let natural light enter and to maximize the sense of security for pedestrians. The horizontal alignment through the tunnel is an important design consideration. Keeping the tunnel length on tangent as much as practical will not only minimize the length but also improve operating efficiency. Tunnels designed with extreme curvature may result in limited stopping sight distance. Therefore, sight distance across the face of the tunnel wall should be carefully examined. In general, interchanges should be located outside tunnel with merging or diverging lanes located at least 1.5*SSD from the tunnel portal. Particular attention should be given to the adequacy of advance directional signs if a tunnel is located within the normal directional signing systems.

Interchanges in tunnels should only be permitted with a very high standard merging and diverging lanes characterized by:

i. location on gentle alignment
ii. very good visibility
iii. parallel merging layout
iv. buffer zones after merging
v. adequate, well designed and maintained directional signs
vi. Enhanced delineation
vii. crash cushion at diverge gores

In addition, these tunnels should be equipped with sophisticated traffic control and surveillance systems.

The vertical alignment through the tunnel is another important design consideration. Grades in tunnels should be determined primarily on the basis of driver comfort while striving to reach a point of economic balance between construction costs and operating and maintenance expenses. Gradients should be limited to 3% wherever possible to limit both emissions and safety risks. Toll booths should be located at least 300m from a tunnel. Many factors have to be considered in tunnel lengths and grades and their effects on tunnel lighting and ventilation. For example, lighting expenses are highest near portals and depend heavily on availability of natural light and the need to make a good light transition. Ventilation costs depend on length, grades, natural and vehicle-induced ventilation, type of system, and air quality constraints.

The overall roadway design should avoid the need for guide signs within tunnels, because normal vertical and lateral clearances are usually insufficient for such signing and additional clearance can be provided only at very great expense. Exit ramps should be located a sufficient distance downstream from the tunnel portal to allow for any guide signs that may need to be placed between the tunnel and the point of exit. This distance should be a minimum of 300 m.

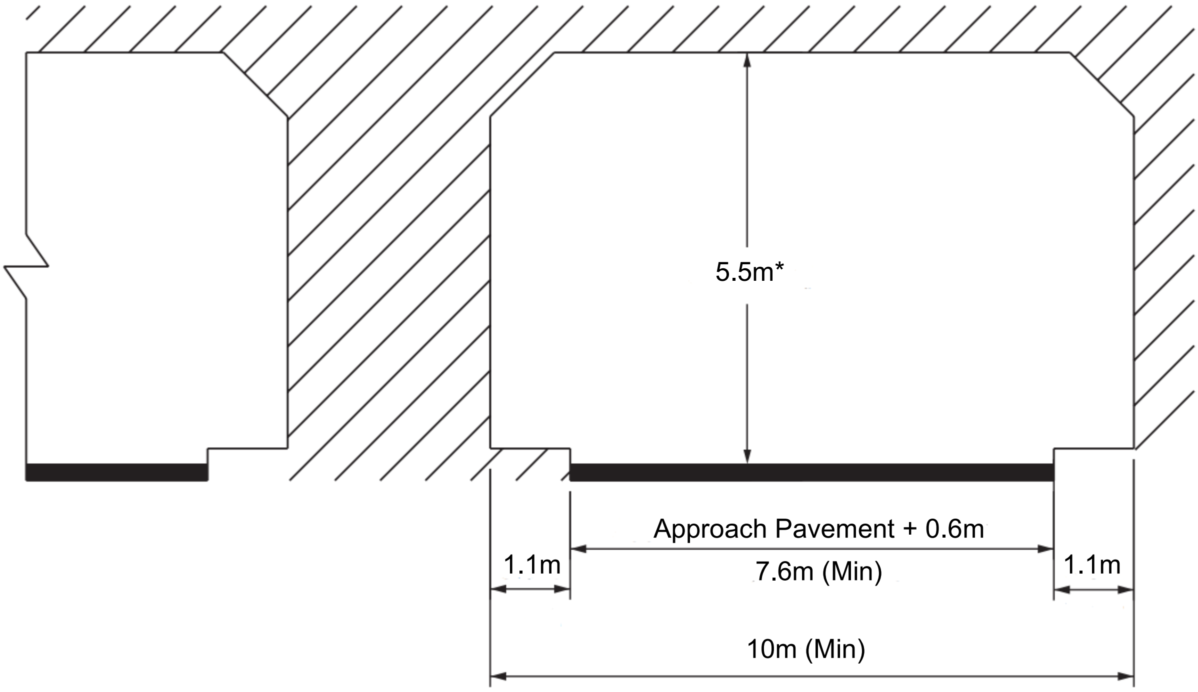
It is also highly undesirable that traffic be expected to merge, diverge, or weave within a tunnel, as might be the case if the tunnel is located between two closely spaced interchanges. Therefore, forks and exit or entrance ramps should be avoided within tunnels, where practical.

6.17.4 TUNNEL SECTIONS

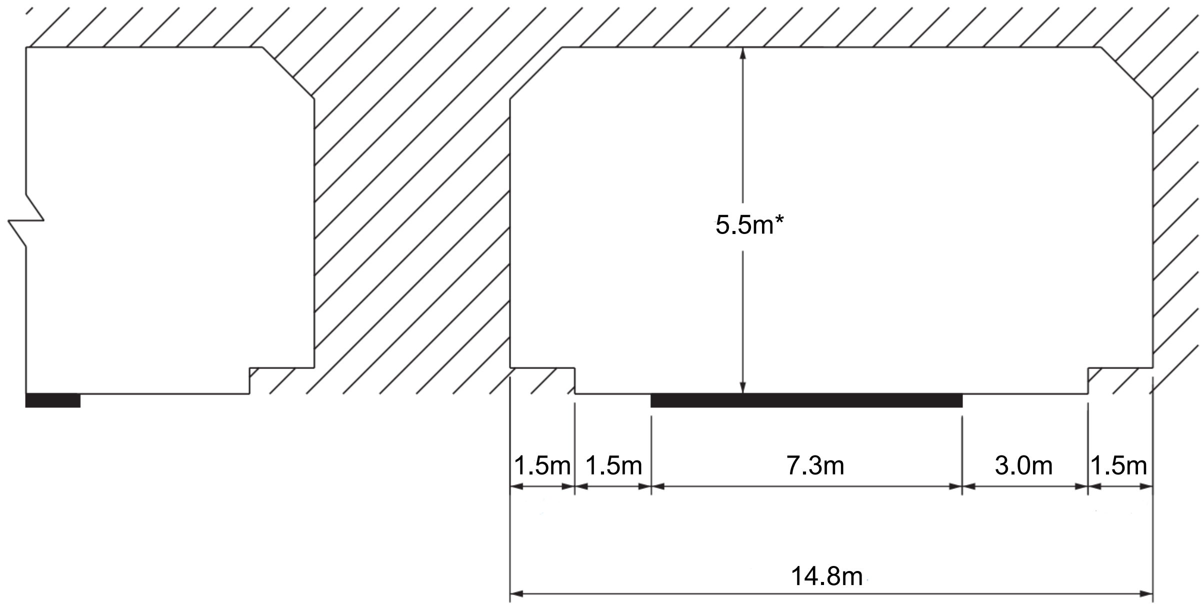
From the standpoint of service to traffic, the design criteria used for tunnels should not differ materially from those used for grade separation structures. The same design criteria for alignment and profile and for vertical and horizontal clearances generally apply to tunnels except that minimum values are typically used because of high cost and restricted right-of-way.

Full left- and right-shoulder widths of the approach road desirably should be carried through the tunnel. Actually, the need for added lateral space is greater in tunnels than under separation structures because of the greater likelihood of vehicles becoming disabled in the longer lengths. If shoulders are not provided, intolerable delays may result when vehicles become disabled during periods of heavy traffic. However, the cost of providing shoulders in tunnels may be prohibitive, particularly on long tunnels that are constructed by the boring or shield-drive methods. Thus, the determination of the width of shoulders to be provided in a tunnel should be based on thorough analyses of all factors involved. Where it is not practical to provide shoulders in a tunnel, arrangements should be made for around-the-clock emergency service vehicles that can promptly remove any stalled vehicles.

Figure 6.37 illustrates typical tunnel cross sections for two-lane tunnels. The minimum roadway width between kerbs, as shown in **Figure 6.37A**, should be at least 0.6 m greater than the approach travelled way, but not less than 7.2m. Where walkways are provided for emergency egress by pedestrians, they must be designed to be accessible to and usable by pedestrians with disabilities. For short tunnels, less than 60m in length, the walkway may have a minimum width of 1.1 m. In long tunnels, 60m or more in length, the walkway width must be at least 1.2m with passing sections at least 1.5m wide.



A. Minimum Cross Section for Short Tunnels (less than 60 m in length)



B. Desirable Cross Section for Long Tunnels (greater than or equal to 60 m in length)

* Note: An allowance should be added to the vertical clearance for future repaving.

Figure 6.37 Typical Two-Lane Tunnel Sections

Since varying the tunnel cross section to provide passing sections may be impractical, walkways with a continuous width of 1.5m must generally be provided. The total clearance between walls of a two-lane tunnel should be a minimum of10m for short tunnels and 10.5m for long tunnels.

The roadway width and the kerb or walkway width can be varied as needed within the total tunnel width; however, each width should not be less than the minimum value stated above.

The minimum clear height for all tunnels should not be less than that on the road (5.5m) leading to the tunnel, and it is desirable to provide an allowance for future repaving of the roadways.

Raised walkways are provided for through pedestrian movements in some non-expressway tunnels. Normally, pedestrians are not permitted in expressway/motorway tunnels; however, raised walkways should be provided for emergency walking and for access by maintenance personnel. A walkway or shoulder area in a tunnel also serves as a buffer that prevents the overhang of vehicles from damaging the wall finish or the tunnel lighting fixtures. The minimum design criteria to provide pedestrian accessibility for walkways in tunnels have been addressed above. Separate tunnels may be warranted for pedestrians or other special uses, such as bikeways.

6.17.5 SAFETY FACILITIES

6.17.5.1 EMERGENCY LAYBYS AND CROSS PASSAGES

For tunnels exceeding 1500m in length and without an emergency lane, emergency laybys should be provided at intervals not more than 1000m. They should be located on straight sections or large radius curves. Approach traffic should be able to see a stopped vehicle in the layby with visibility distance of SSD.

To reduce the risk of an errant vehicle colliding frontally with the end wall of emergency laybys, the tunnel wall or safety barrier should have a flared layout.

Cross-passages for emergency services or diversion should be provided at spacing not exceeding 1,500m for tunnels longer than 1,500m in length.

6.17.5.2 DANGEROUS GOODS TRANSPORT

The suitability of a tunnel for the transport of dangerous goods or hazardous materials should be established on the basis of risk and availability of alternative routes. Transport under escort is also a possibility.

Adequate consideration should be given to the containment and management of spillage. If dangerous goods is permitted, the drainage system should be specially designed for the collection of flammable or toxic liquid to minimise the risk of ignition and spread of fire, liquids or vapours through the tunnel tubes or between tubes.

To avoid trapping of potentially hazardous spillage in the road pavement, it is not advisable to use porous asphalt within a tunnel.

6.17.5.3 EMERGENCY STATION

Emergency stations should be provided at every layby and at spacing not more than 150m. Each station should be equipped with an emergency telephone and at least two fire extinguishers.

6.17.5.4 EMERGENCY ESCAPE FACILITIES

An emergency walkway not less than 0.75m in width should be provided on either side of a tunnel tube.

This requirement may be waived if:

i. an emergency lane not less than 2m is provided.
ii. the tunnel is unidirectional and is equipped with a sophisticated surveillance and control system.

Emergency exits should be provided for all new tunnels to enable a tunnel user to walk on foot and reach a safe place in the event of a fire or other serious incidents. They also provide an alternative access for emergency services to reach an incident site. The maximum spacing between emergency exits is 500m.

Emergency exits may be in one of the following forms:

i. Cross-passage between tunnel tubes
ii. Exit to an escape tunnel system
iii. Exit to an outdoor area or safe location
iv. Shelters with an escape connection

Shelters without an exit to escape routes should not be provided.

6.17.5.5 MONOTONY

Monotony in long tunnels could be mitigated by:

i. Tunnel walls in colours at regular intervals
ii. Special colour schemes
iii. Informatory signs showing the remaining length of the tunnel

6.17.6 TUNNEL APPROACH ZONE

Tunnel approach zone is the transition area between a tunnel and open roads. The zone needs to satisfy requirements for operation, maintenance, emergency response and landscaping design. Major tunnels may consist of the following facilities in this zone:

i. Ventilation building and fire control point
ii. Management centre with parking
iii. Access openings for staff, vehicles and emergency
iv. Laybys for inspection or retention of vehicles

v. Crossovers and U-turn facilities
vi. Facilities for surveillance and traffic control

Simplicity should be attained for tunnel approach zones from the visual perspective and in terms of facilities, accesses and staff activities. Tunnel portals (tunnel entrance) on high-speed roads should be visible to drivers a few hundred metres ahead.

6.17.6.1 TRAFFIC CONTROL FACILITIES

Laybys should be provided at the entrance and exit of tunnel portals for the purpose of inspection, escort and temporary parking of broken down vehicles, unauthorised vehicles or operational vehicles.

Laybys are desirably 4 to 5m wide inclusive of paved shoulders. The length should permit parking of at least one long vehicle and a tow truck.

A vehicle crossover facility should be provided for twin tube tunnels to allow for single tube contraflow operation.

6.17.6.2 ACCESS FOR OPERATION, MAINTENANCE AND EMERGENCY RESPONSE

Depending on the required response time for incidents, emergency response facilities should be provided in the vicinity of one or both tunnel portals. Consideration should also be given to provision of emergency access points for the use of the police and fire service.

In principle, direct frontage accesses for facilities should be minimised at tunnel approach zones. U-turn facilities are preferably grade-separated with acceleration lanes instead of at-grade design if they are likely to be used frequently.

Adequate visibility should be provided for any accesses or U-turn facilities. They should be closed off with traffic cones or barrier gates to deter unauthorised use.

Walkways and grade-separated crossings should be provided for operational and maintenance staff to access facilities and to implement routine traffic control. Walkways and working compounds should as far as practical be located outside clear zones or guarded by safety barriers.

6.17.7 ROADSIDE SAFETY FOR TUNNELS

Due to the need to satisfy a variety of operational, maintenance and emergency response functions, roadside areas of tunnel tubes and tunnel approach zones are more complicated in terms of roadside safety design.

6.17.7.1 TUNNEL TUBE

Tunnel walls raised walkways or safety barriers should be smooth, continuous and free of aggressive protrusions. Corners of walkway kerbs should be smooth without sharp edges which could puncture the tires of vehicles.

At the termination of emergency laybys, the tunnel wall and walkway should be tapered or equipped with crash cushions. Attention should also be given to the details for crossovers and cross passages inside tunnel tubes to minimise the risk of collision with the ending of walls and walkways.

6.17.7.2 TUNNEL PORTAL

Roadside areas at the interface between open road and tunnel tube should not consist of hazards which could cause an errant vehicle to stop abruptly, penetrate into a vehicle or launch the vehicle airborne. This requirement applies to tunnel entrances and tunnel exits.

Attention will need to be given to the detailed design of transition among tunnel walls, walkways and safety barriers on the approach zone. Examples of treatments are given in **Plate 6.6** and **Plate 6.7**. It is also necessary to ensure that the upper part of an errant tall vehicle will not collide with the tunnel portal of circular or arch cross-section.

The safety risks of tunnel portals may be reduced with better delineation and longitudinal rumble strips on the immediate approach.

Plate 6.6 Safety barrier overlapping tunnel entrance

Plate 6.7 Double safety barriers and access opening at tunnel entrance

Terminations of raised tunnel walkways are preferably contiguous with the approach safety barrier. Low level walkway may be flared and ramped down at a shallow angle.

At the exit portal of tunnels, safety barriers on the open road should be continuous with the tunnel wall or safety barrier systems inside the tunnel. Alternatively, they may be further set back and anchored without any exposed upstream end terminals. For bidirectional tunnels and tunnels regularly operated for bidirectional flows, transitions should be designed for safety in both traffic directions. Refer to **Section 12.8.3.3** for further information on tunnel lighting.

6.18 ROAD RESERVATIONS

Utility reserve should be defined within the road reservations of sufficient width to accommodate all utilities and allow for future expansions. **Table 6.20** shows the road reservations (right-of-way) for the various design classes in Ghana.

Table 6.20 Road reservations

Design Class		Right-of-Way (ROW) (m)	Functional Class	Administrative Class
A	A1 - Rural Expressway/Motorway	90	National	Trunk Road
	A2 - Urban Expressway/Motorway			
B	B1 - Rural Major Arterial	90	National	Trunk Road
	B2 - Urban Major Arterial			
		60	Major Arterial	Urban Road
C	C1 - Rural Minor Arterial	60	Inter-Regional & Regional	Trunk Road
	C2 - Urban Minor Arterial		Minor Arterial	Urban Road
D	D1 - Rural Collector	45	Inter-District Connector	Feeder Road
	D2 - Urban Collector		Collector/Distributor	Urban Road
E	Local/Access	30	Local/Access	Urban Road Feeder Road

Figure 6.38 to **Figure 6.41** shows a typical cross section of road reservation for the various design classes.

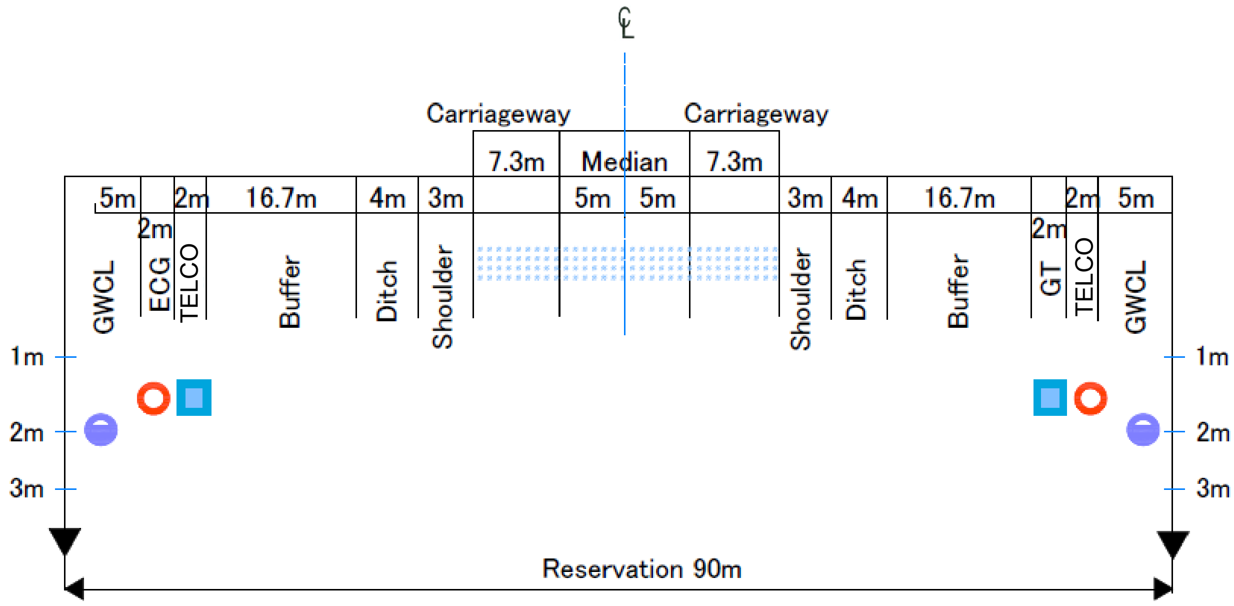


Figure 6.38 Road reservation for design class A1, A2, B1 and B2 (trunk road)

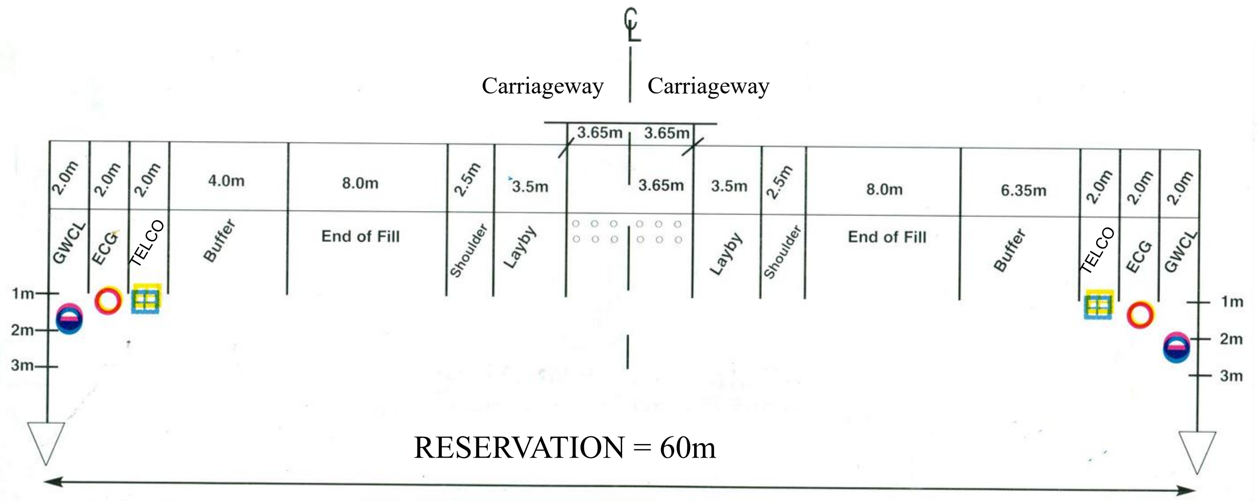


Figure 6.39 Road reservation for design class B2 (urban road), C1 and C2

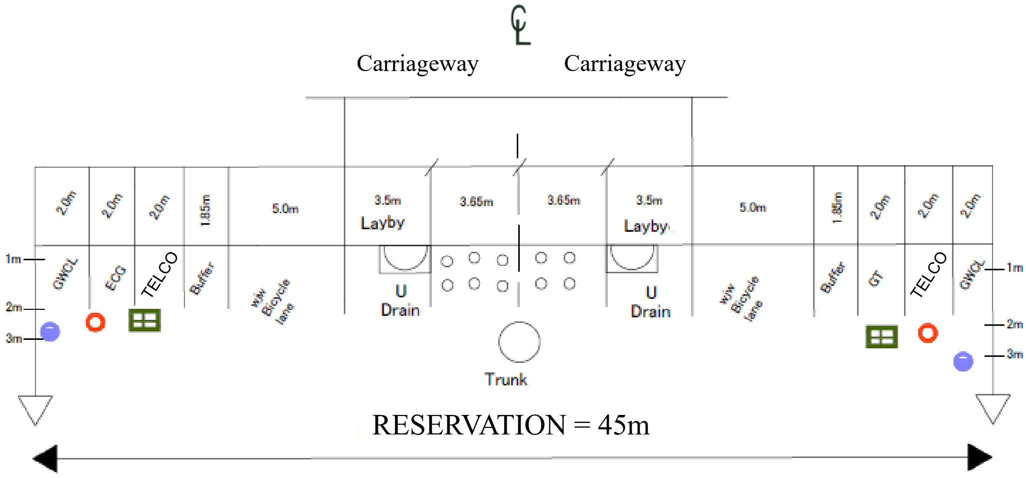


Figure 6.40 Road reservation for design class D1 and D2

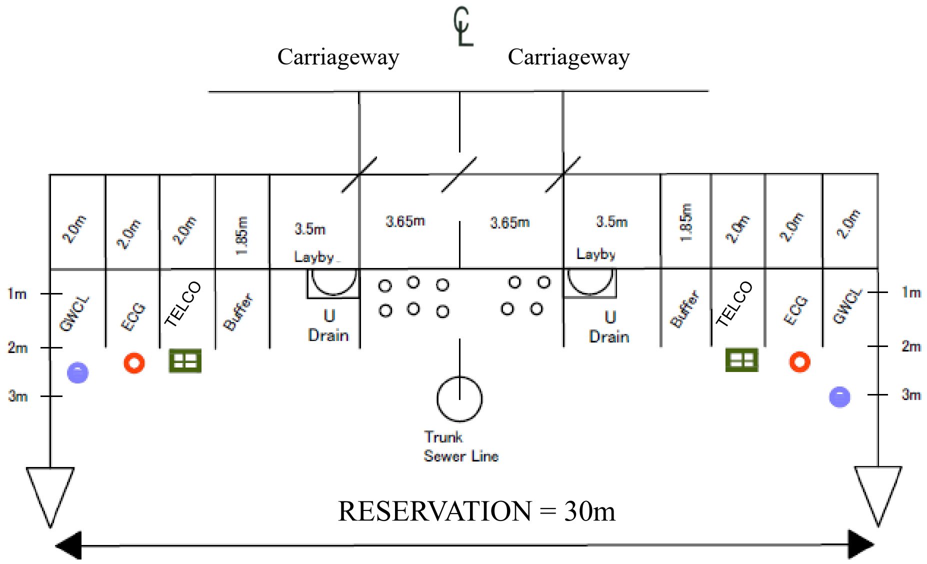


Figure 6.41 Road reservation for road design class E

6.19 REFERENCES

1. Ghana Highway Authority Road Design Guide, 1991.
2. Japanese Road Structure Ordinance, April 2021.
3. Explanation and Operation of the Road Structure Ordinance.
4. Geometric Design Manual of Uganda, (2005).
5. A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.
6. AASHTO LRFD Road Tunnel Design and Construction Guide Specifications, 2017
7. Asian Highway Design Standard for Road Safety, 2017.
8. Roadside Design Guide. United States: American Association of State Highway and Transportation Officials, 2011.
9. Standard Details, Road Signs and Markings for Urban and Trunk Road (1991), Ministry

of Roads and Highways.

10. Road Reservation Management (Manual for Coordination), first edition.

CHAPTER 7

ELEMENTS OF DESIGN

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CHAPTER 7 ELEMENTS OF DESIGN

7.1 ALIGNMENT DESIGN

Alignment design is the positioning of the physical elements of the roadway according to standards and constraints.

Specific objectives related to geometric design are listed below:

i. Optimise safety by providing a road and roadside that is designed to minimise fatalities and serious injuries (FSIs) for all road users. Vision zero sets a target that eliminates FSIs over time.
ii. Optimise operational efficiency by providing a road that can carry the required volume of traffic at a speed that is consistent with the functional class of the road and road safety objectives.
iii. Maintain uniformity of design parameters along a route and/or within a network, particularly across administrative boundaries, to provide a consistent and operationally effective travel experience relative to the functional class of the road.
iv. Development of economically efficient designs to optimise the benefit of limited funds available for road construction and maintenance. Therefore, for a particular design, minimise costs associated with construction, maintenance, and operation of the road whilst meeting all other objectives.
v. Adequately provide for the future requirements of the road network by considering the ultimate road layout required to serve general traffic growth and adjacent development in the vicinity of the works. Also ensure that future expansion of the road can be accommodated with minimum reconstruction.
vi. Optimise opportunities to cater for the needs of all road user groups including all types of vehicles expected to use the road.
vii. Minimise adverse environmental impacts (during construction and operation) and enhance the environment where possible both in the immediate vicinity of the road and over a wider area. This includes integration of the road design into the surrounding environment to achieve a visually pleasing outcome.
viii. Where practicable, provide a design that takes account of the views of the community including local residents, businesses, community groups and road users.
ix. Optimise the opportunity for physical activity through transport by creating safe and attractive walking and cycling environments within communities. The most suitable alignment must be harmonious with the natural topography for the conservation of nature.

The alignment is defined as the combination of horizontal and vertical geometric elements giving the location of the road in the terrain. The horizontal elements used are circles defined by the radius R, straights (tangents) and transition curves, normally clothoids. The vertical elements used are grades and vertical radii R.

Alignment design therefore seeks to integrate these elements to produce a compatible speed with the road's function and location. Safety, operational quality, and project costs can be significantly influenced by coordinating the horizontal and vertical alignments.

The alignment, the cross-section, road furniture, vegetation and the surroundings create the framework for driver perception and behaviour such as choice of speed, choice of track and lateral position on the road. The main design decision for alignment requirements is the speed limit by sections of the road and associated design speeds.

7.1.1 BASIC ASSUMPTIONS OF ALIGNMENT DESIGN

The basic assumption for road alignment is that the driver at design speed should be able to perceive any possible road hazard on or close to the road to take action to avoid accident. This requires all the alignment elements to have good visual guidance and sufficient stopping sight distances. There is also a need to be able to overtake slower vehicles. Parts of the road therefore require overtaking sight distances.

It is judged important for traffic safety reasons to avoid, as far as possible dilemma sight distances, i.e. in between stopping and overtaking sight distances, where the driver cannot see quite far enough to be sure that it is safe to overtake. The driver should also be able to drive comfortably due to inertia forces at the design speed relevant for horizontal and vertical alignment.

These basic assumptions give minimum parameters for vertical and horizontal alignment elements. They also give a number of recommendations on how to combine elements to facilitate visual guidance and safe driving. A basic rule is to avoid sharp bends after long tangents and other surprises to the driver.

The alignment and the cross-section should also be adapted to the surroundings to create a stimulating driving task with a good rhythm, harmony and with varying views, trying to maximize the length of road with sufficient overtaking sight distances and also obviously to give a cost-effective design optimizing road user, investment and maintenance costs and other important social and environmental impacts.

7.1.2 ALIGNMENT CHOICE AND TERRAIN ADAPTATION

To locate a road in a landscape is a challenge with constraints and possibilities. It is a technical and also an architectural process.

The main principle is to adapt the road to the surroundings considering technical requirements on sight distances such as sufficient overtaking possibilities, minimum geometric elements and visual guidance. Three basic concepts unite and constitute the technical and the architectural process:

a. Scale
b. Space
c. Rhythm

The scale is the size of the landscape to locate the road. Typical landscape sizes are illustrated in **Figure 7.1**.

Large-scaled landscape

Small-scaled landscape

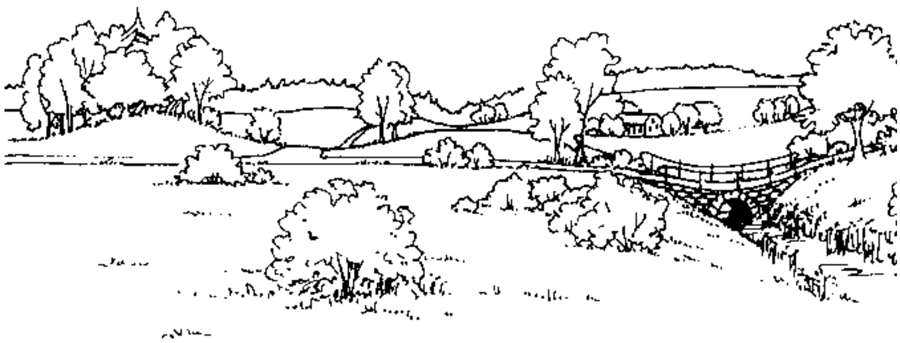


Figure 7.1 Description of the scale concept

The space or room is a defined part of the landscape – as far as you can overview from a specific point. The limitations of the space or room could be:

i. Terrain (mainly topography), vegetation, buildings;

ii. Road design, i.e., cross-section, horizontal and vertical alignment; and,

iii. Crossing bridges and road embankments.

This is illustrated in **Figure 7.2.**

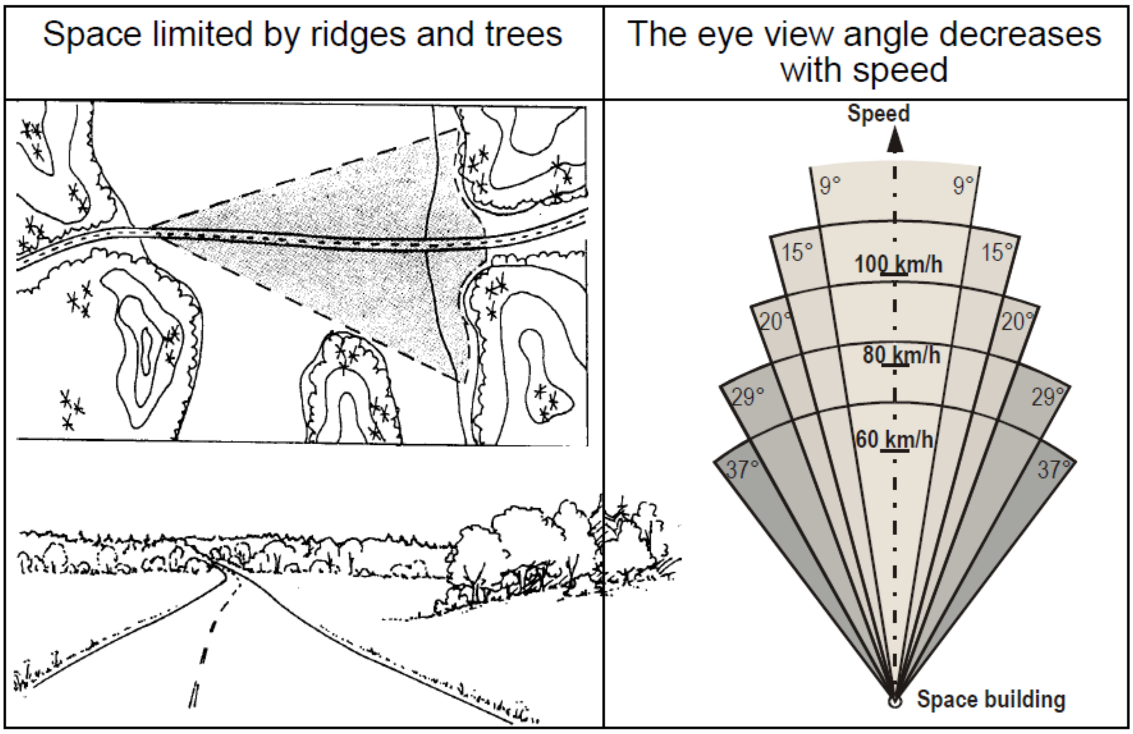


Figure 7.2 Driver's space or room concept

The rhythm of a trip along a road – the experience and enjoyment – depends on the design and how this design is located in the landscape. The designer should use the landscape combined with the road alignment, cross-section and road side area to create a variation, rhythm, in impressions and outlooks. The objective of alignment choice and terrain adoption is to design a road that is enjoyable to drive.

The alignment should together with the cross-section, the roadside area, and the surroundings create variation in outlooks for the driver and also support him in his driving task with visual guidance. Outlooks should be long enough to be comprehensible at the design speed. A simple rule of thumb is that outlooks should have at least the same length in metres as the design speed in km/h representing some 4 to 5 seconds driving time.

The alignment is three-dimensional. It is of utmost importance to look at and treat the alignment design as a space curve following as far as possible the laws of perspectives. The road should have an inner and an outer harmony. The inner harmony means that the road should have a satisfying, calm and graceful geometric form – considered only as a space curve without terrain. The outer harmony requires the space curve to be tuned with the terrain and in harmony with the landscape. The geometric elements should have the same scale as the surrounding terrain. Examples of landscape adaptation is shown in **Figure 7.3** and **Figure 7.4.**

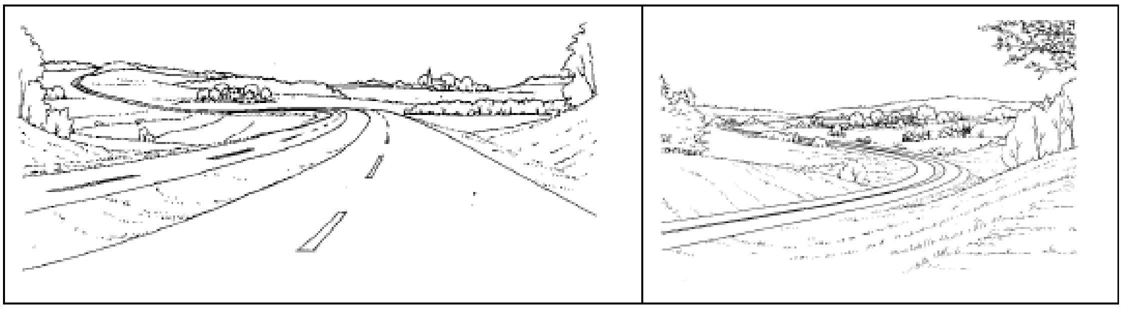


Figure 7.3 Some examples on rhythmical landscape adaptation

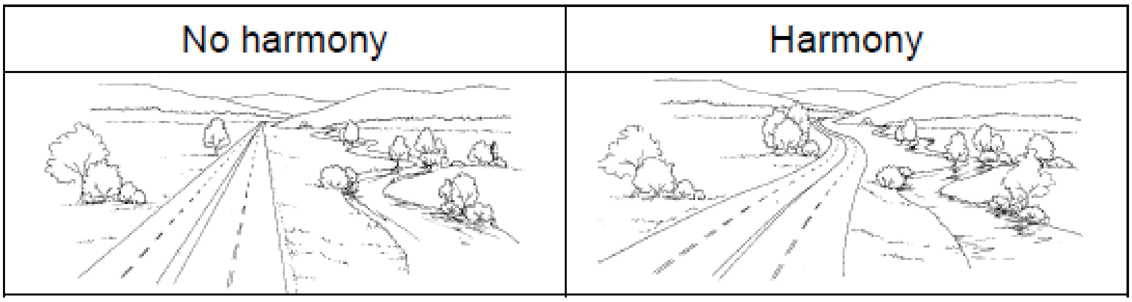


Figure 7.4 Example of adaptation to the landscape

7.2 HORIZONTAL ALIGNMENT

The horizontal alignment is the plan view of the road. The elements of horizontal alignment include straights, circular curves, and transition curves. Clothoid is used for transition curves.

Horizontal alignment should provide for safe and continuous operation of vehicles at a uniform design speed for substantial lengths of the road.

The major considerations in horizontal alignment are:

i. Topography
ii. Type of facility
iii. Design speed
iv. Profile grade
v. Subsurface conditions
vi. Existing highway and cultural development
vii. Likely future developments
viii. Location of the highway terminals
ix. Right of way
x. Safety
xi. Construction costs
xii. Environmental issues
xiii. Geological features
xiv. Drainage

All the above considerations should be balanced to produce an alignment that is appropriate for the location and functional classification of the road (Functional classification is explained in **Chapter 2**).

7.2.1 DESIGN PRINCIPLES OF HORIZONTAL ALIGNMENT

Horizontal alignment, combined with vertical alignment, serves as the primary controlling element associated with the design of all types of roads. Engineering judgment and experience plays a major role in selecting horizontal geometry that meets desired design criteria.

The alignment must allow smooth flow of traffic. A road with long sections of straights and gentle curves has a higher running speed than the design speed because drivers choose their running speeds from the alignment. Hence the designer must avoid sudden changes in a horizontal design and small curves at the end of gradients in a vertical design.

Excessive curvature or poor combination of curvature limit capacity, cause economic losses because of increased travel time and operating costs, and detract from a pleasing appearance. To avoid such poor design practices, when conducting horizontal alignment design, particular attention should be paid to the following principles.

i. Avoid having long straights as much as possible. Tangent lengths between curves are limited by the design speed. The maximum length in meters of tangent sections between two curves may not exceed twenty times the design speed of that roadway. To avoid driver fatigue, it is recommended that tangent sections be limited to a maximum of 40 to 60 percent of long roadway sections with maximum single tangent lengths between 2000 and 3000 metres.
ii. Existing environmental and other constraints should be identified on the base mapping to assist the designer in minimizing impacts to wetlands, historical and archaeological features, private and protected property, and permanent structures. To the extent possible, these constraints should serve as boundaries through which the designer must fit the geometry.
iii. The relationship of the roadway to wetlands and waterways and the interaction of different types of roadway drainage with these resources should be considered.
iv. For improvements to existing roadways, geometry should be concentric with and/or parallel to the existing roadway layout so that new impacts to the surrounding area are minimized.
v. High embankments need particular attention. In curve section, the horizontal radii must be larger and preferably facilities to give visual guidance like guardrails, vegetation, chevron boards etc should be introduced. In normal straight sections the shoulders are widened to minimize the sense of insecurity which arises out of illusion.
vi. Horizontal alignment should be as smooth and as direct as possible while responsive to the topography. Flatter curvature with shorter tangents is generally preferable to sharp curves connected by long tangents. Angle points should be avoided.
vii. The occurrence of abrupt reverse curves (having a short tangent between two curves in opposite directions) should be avoided since such alignment make it difficult for the driver to remain

within his lane. It is also difficult to superelevate both curves adequately, and this may result in erratic operation.

viii. The horizontal alignment should be in balance with the vertical profile and cross section rotation associated with superelevation.

ix. Horizontal curves should be avoided on bridges whenever possible. These cause design, construction, and operational problems. Where a curve is necessary on a bridge, a simple curve should be used on the bridge and any curvature or superelevation transitions placed on the approaching roadway.

x. The "broken-back" arrangement of curves (having a short tangent between two curves in the same direction) should be avoided except where very unusual topographical or right-of way conditions dictate otherwise. Drivers do not generally anticipate successive curves in the same direction. This also creates problems with superelevation and drainage.

xi. The use of compound curves affords flexibility in fitting the road to the terrain and other controls. However, caution should be exercised in the use of compound curves, because the driver does not expect to be confronted by a change in radius once he has entered a curve. Their use should also be avoided where curves are sharp.

iv. Compound curves with large differences in curvature introduce the same problems as are found at the transition from a tangent to a small-radius curve.

xii. It is desirable to avoid combinations of the following types of horizontal alignments.

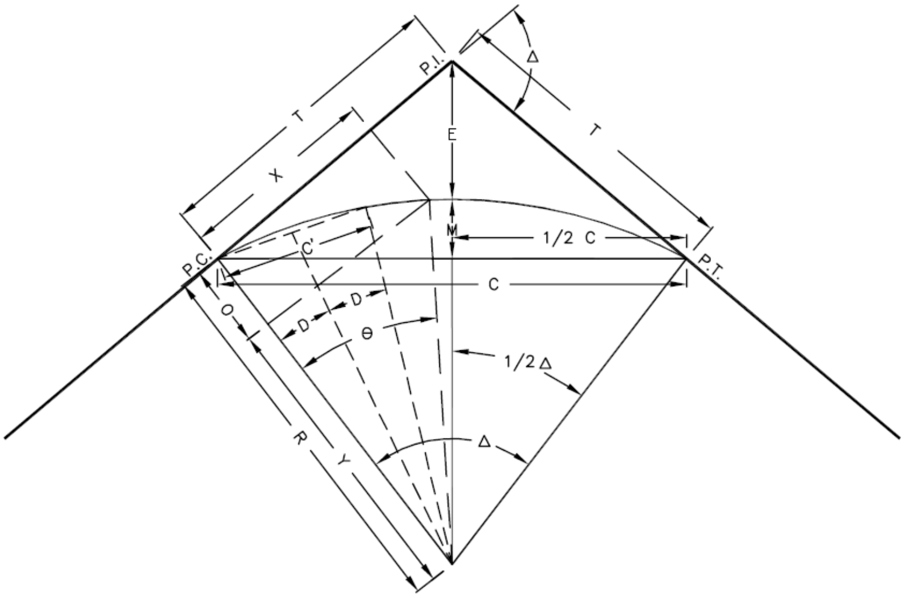
a. Insertion of circular curves with small curve radii at the end of straights.

b. Insertion of circular curves with short curve length in cases where the intersection angle is small.

7.2.2 TYPES OF HORIZONTAL CURVATURE

7.2.2.1 SIMPLE CURVE

A simple curve is a circular arc joining two tangents. A simple curve has a constant circular radius which achieves the desired deflection without using an entering or exiting transition. This is the most frequently used curve because of their simplicity for design, layout, and construction as shown in **Figure 7.5.**



R =	Radius	D =	Deflection Angle for Chord C'
C =	Long Chord	T =	Length of Tangent
C' =	Any Chord Length	O =	Tangent Offset
M =	Middle Ordinate	P.C. =	Point of Curvature
L =	Length of Arc	P.I. =	Point of Intersection
E =	External Distance	P.T. =	Point of Tangency
△ =	Intersection Angle = Central Angle	X =	Distance Along Tangent

FORMULAS

$T = R \tan(\triangle/2)$ $\qquad C = 2R \sin(\triangle/2)$ $\qquad E = R(\sec(\triangle/2) - 1)$

$M = R(1 - \cos\triangle/2)$ $\qquad L = \dfrac{\triangle}{360}(2\pi R)$ $\qquad \sin D = 1/2 \div C'R$

TANGENT OFFSET METHODS

$0 = R - (R^2 - x^2)^{0.5}$

$\sin\Theta = X/R$

$Y = R\cos\Theta$

$0 = R - Y$

Figure 7.5 Simple Circular Curve

Elements of a horizontal curve:

Δ	DELTA (Deflection Angle). The value of the central angle is equal to the intersection angle.
R	RADIUS. The radius of the circle of which the curve is an arc, or segment. The radius is always perpendicular to back and forward tangents.
PI	POINT OF INTERSECTION. The point of intersection is the theoretical location where the two tangents meet.
PT	POINT OF TANGENCY. The point of tangency is the point on the forward tangent where the curve ends. It is sometimes designated as EC (end of curve) or CT (curve to tangent).

PC	POINT OF CURVATURE. The point of curvature is the point on the back tangent where the circular curve begins. It is sometimes designated as BC (beginning of curve) or TC (tangent to curve).
POC	POINT ON CURVE. The point on curve is any point along the curve.
L	LENGTH OF CURVE. The length of curve is the distance from the PC to the PT, measured along the curve.
T	TANGENT. The length of tangent is the distance along the tangents from the PI to the PC or the PT. These distances are equal on a simple curve.
C	LONG CHORD. The long chord is the straight-line distance from the PC to the PT.
C’	Any chord distance between two points along a curve.
M	EXTERNAL DISTANCE. The external distance (also called the external secant) is the distance from the PI to the midpoint of the curve. The external distance bisects the interior angle at the PI.
D	MIDDLE ORDINATE. The middle ordinate is the distance from the midpoint of the curve to the midpoint of the long chord. The extension of the middle ordinate bisects the central angle.
D	DEFLECTION ANGLE for chord C’.

At a minimum, curve data shown on the drawings should include the radius, length of curve, deflection angle, and tangent length. Plan information should also include the stations at the PC and PT.

7.2.2.2 TRANSITION/SPIRAL CURVE

Spiral curves provide a gradual change in curvature from a straight to a circular path. The characteristic of transition (spiral or clothoid) curve is that it has a constantly changing radius. The Euler spiral, which is also known as the clothoid, is preferred to be used. The radius of clothoid varies from infinity at that tangent end of the spiral to the radius of the circular arc at the circular curve end as shown in **Figure 7.6**. By definition the radius at any point of the spiral varies inversely with the distance measured along the spiral.

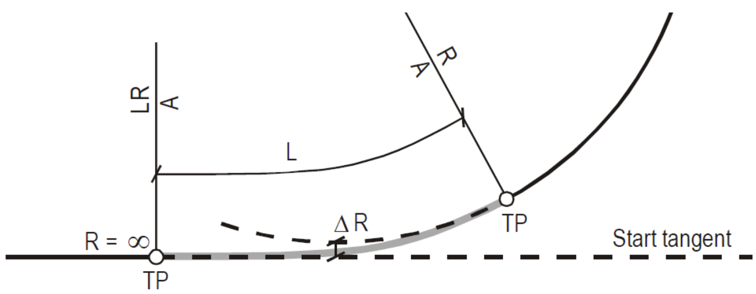


Figure 7.6 Spiral Curve

7.2.2.3 REVERSE CURVES

Two consecutive circular curves constitute a reverse curve if they join at a point of tangency where their centres are on opposite sides of the common tangent as shown in **Figure 7.7**.

For safety reasons, the use of this curve should be avoided when possible. As with broken back curves, drivers do not expect to encounter this arrangement on typical road geometry. Also, in design of reverse curves, to secure safe traveling, the radii of adjacent circular curves should be set with good balance to ensure they fit within a certain scope. In cases of reversing curves, a sufficient tangent should be maintained to avoid overlapping of the required superelevation runoff and tangent runout.

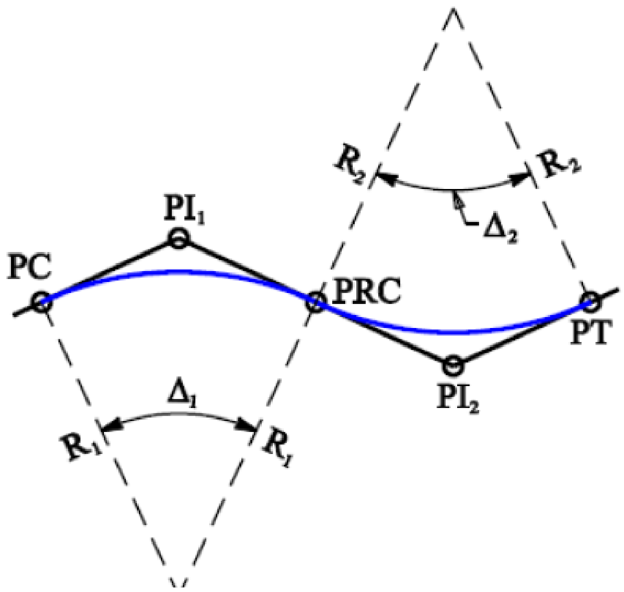


Figure 7.7 Reverse Curve

7.2.2.4 COMPOUND CURVES

Compound circular curves are two or more consecutive circular curves in the same direction with varying radii as shown in **Figure 7.8.** Compound circular curves are joined at a point of tangency and located on the same side of the common tangent. While simple curves are preferred, compound curves can be used to satisfy topographical constraints that cannot be as effectively balanced with simple curves.

Compound curves are used to transition into and from a simple curve and to avoid some control or obstacle which cannot be relocated. Compound curves are appropriate for intersection curb radii, interchange ramps, and transitions into sharper curves.

The following guidelines should be followed when using compound curves:

- For compound curves on open roads, it is generally accepted that the ratio of the flatter radius to the sharper radius should not exceed 1.5:1.
- For compound curves at intersections or on ramps, the ratio of the flatter radius to the sharper radius should not exceed 2:1
- When the above is not feasible, an intermediate simple curve or spiral should be used to provide the necessary transitions.
- Superelevating compound curves requires careful consideration.

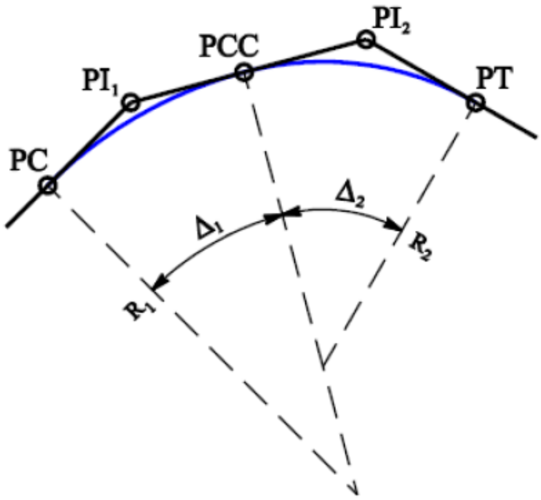


Figure 7.8 Compound Curve

7.2.2.5 BROKEN-BACK CURVE

A broken-back curve consists of two curves in the same direction joined by a short tangent as shown in **Figure 7.9**. Broken-back curves are undesirable and can typically be replaced by one simple curve as shown in **Figure 7.9**. If used, a simple curve, a compound curve or spiral transitions should be used to provide some degree of continuous superelevation. Lengths need to be adequate to transition superelevation correctly. The "broken-back" arrangement of curves should be avoided except where very unusual topographical or right of way conditions make other alternatives impractical.

The minimum tangent lengths must be at least six times the design speed. For a typical design speed of 100 km/h, this would correspond to a maximum tangent length of 2,000 meters and a minimum tangent length of 600 meters. Use of spiral transitions or compound curve alignments, in which there is some degree of continuous superelevation is preferable in such situation.

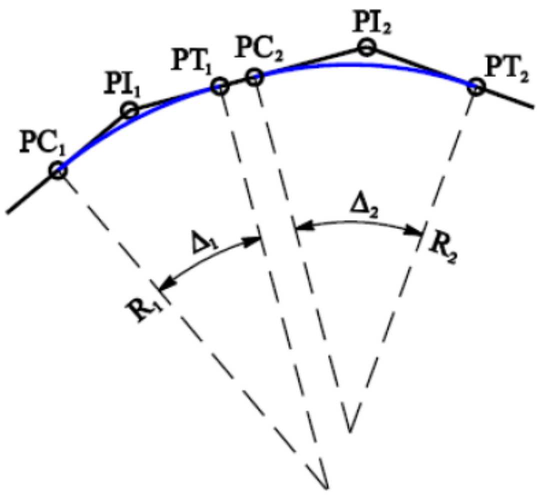
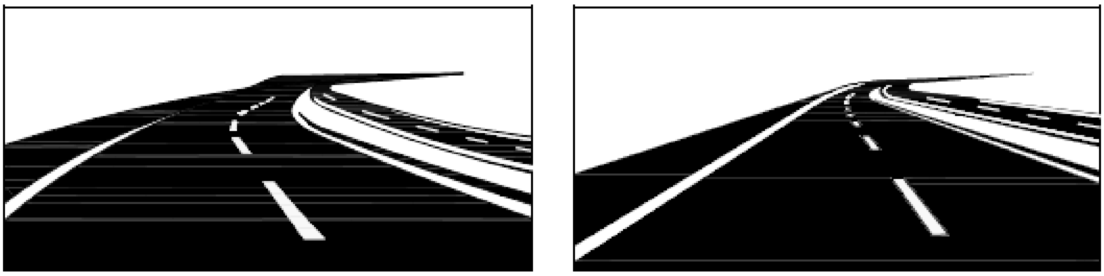


Figure 7.9 Back broken curve



Broken back curve | Replaced with a single curve

Figure 7.10 Correction of back broken curve

7.2.2.6 HAIRPIN/SWITCH BACK CURVE

A hairpin is a curve of very small radius where traffic reverses in direction. Drivers have to reduce speeds substantially, often to not more than 20km/h. They are used where necessary in traversing mountainous and escarpment terrain as shown in **Figure 7.11**. Employing a radius of 20m or less, with a minimum of 10m, they are generally outside of the standards for all design class of roads.

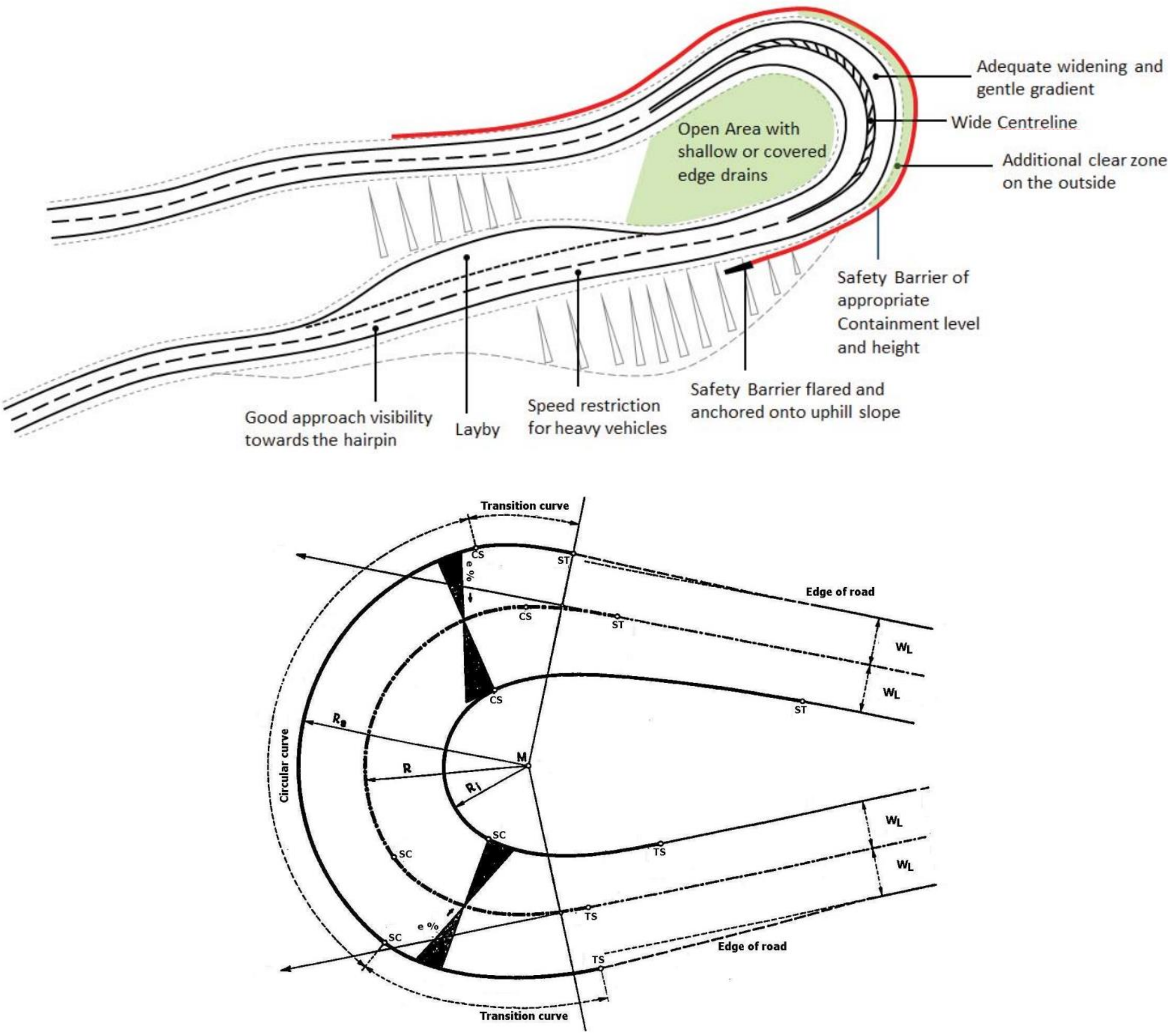


Figure 7.11 Switch back curve

Switchback curves require a careful design to ensure that all design vehicles can travel through the curve. They must therefore provide for the tracking widths of the design vehicles.

7.2.3 HORIZONTAL CURVE

7.2.3.1 CURVE RADII

Good horizontal alignments ensure smooth traffic flows on straight and curves sections. However, since it is necessary to consider the prevention of traffic accidents and comfort of traveling, continuously curved road alignments should be avoided. In curves, it is necessary to introduce superelevation to prevent turning vehicles from slipping or toppling as a result of lateral frictional forces. If the maximum friction is reached, the bend cannot be traversed safely. The superelevation enables the component of the vehicles' weight to reduce the lateral frictional forces.

Minimum curve radii for running vehicles are designed based on safety limits of friction as shown in **Equation 7.1**:

$$R_{min} = \frac{V^2}{127(e + f)} \tag{7.1}$$

Where,

R_{min}:	Minimum curve radius (m).
V:	Design speed (km/h).
e:	Superelevation (%).
f:	Side friction factor.

The absolute minimum curve radii of each design speed are calculated with e=9%, with set Radius gives maximum f=0.15. f=0.15 would be the maximum side friction to be applied for road design (refer to AASHTO 2018). The desirable minimum curve radii of each design speed are calculated in the same way with e=5%, with set Radius gives maximum f=0.10.

The recommended minimum curve radii are shown in **Table 7.1**.

Table 7.1 Minimum Curve Radii.

Design speed (km/h)	Absolute			Desirable		
	e (%)	f	Radius (m)	e (%)	f	Radius (m)
20	9	0.15	15	5	0.09	25
30	9	0.15	30	5	0.09	50
40	9	0.15	50	5	0.08	100
50	9	0.14	85	5	0.08	150
60	9	0.13	130	5	0.08	220
80	9	0.13	230	5	0.07	420
100	9	0.12	370	5	0.06	700
120	9	0.12	540	5	0.06	1,030

Table 7.2 shows the side friction factors recommended for design. According to the research result from AASHTO (2011) on side friction factor and speed, it shows f=0.116 to 0.19 at 50km/h, f= 0.09 at 120km/h would be limit from safety of the travel. The recommended side friction is plotted in **Figure 7.12**.

Table 7.2 Side friction factor used for design

Design Speed km/h	120	100	80	60	50	Less than 40
Side friction factor (f)	0.10	0.11	0.12	0.13	0.14	0.15

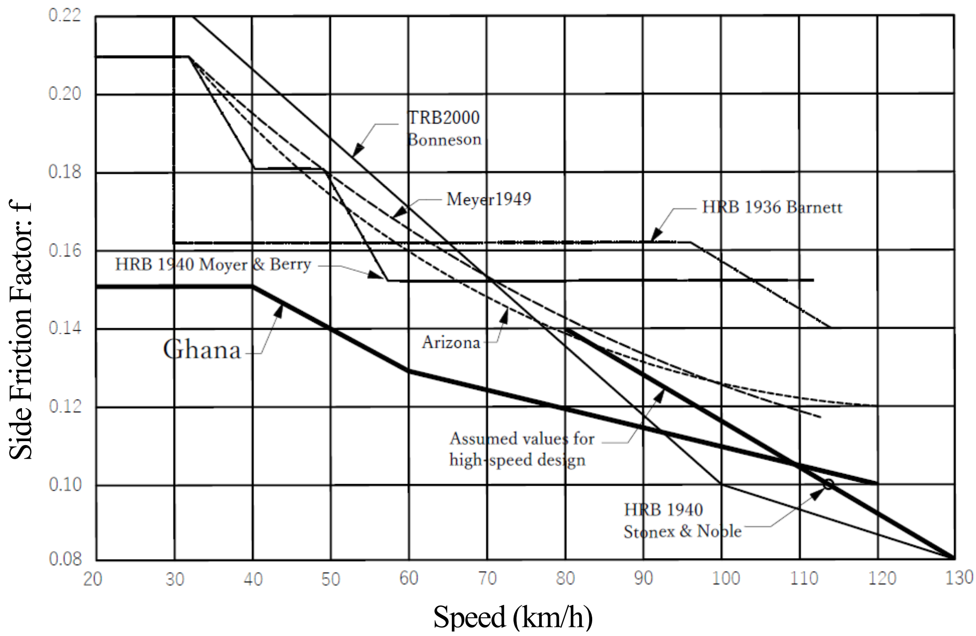


Figure 7.12 Side Friction Factor for High-Speed Streets and Highways

7.2.3.2 MINIMUM CURVE LENGTHS

When vehicles negotiate curves with short lengths, steering is abrupt and difficult for safe drive. Minimum lengths of curves are therefore used to eliminate the defect.

Generally, minimum curve length (including transition curves) is determined based on the following conditions:

a. Drivers should not feel any difficulty in operating the steering wheel.

b. Centrifugal acceleration due to curve change is kept within a certain value (0.3572m/sec^3).

When the road intersection angle is small, actual curve length and curve radii appear smaller than they actually are and tend to be mistaken by drivers. It has been established that 7° is the border angle to consider in the design as shown in **Figure 7.13**. Accordingly, intersection angle more than 7° and less than 7° should be considered to determine the minimum curve radii.

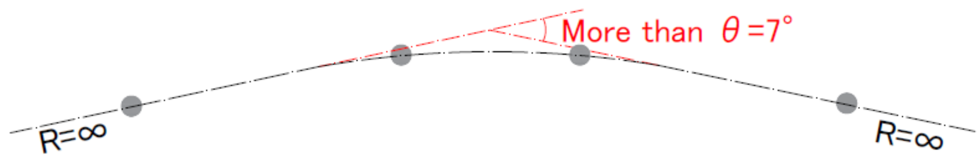


Figure 7.13 Intersection angle

7.2.3.2.1 Intersection angle more than 7°

Curves with intersection angles more than 7° will require a minimum time of 6 seconds for smooth steering. The minimum lengths of curves required for various design speeds based on the above three conditions are specified in **Table 7.3**.

Table 7.3 Minimum Curve Lengths of Intersection angle more than 7°

Design speed (km/h)	Calculated length (m)	*Minimum curve length (m)
120	200	200
100	168	170
80	133	140
60	100	100
50	83	90
40	67	70
30	50	50
20	33	40

*Values have been rounded to the nearest upper tenth.

The calculated curve length of intersection angle more than 7^0 is based on **Equation 7.2**.

$$L = V \times t \tag{7.2}$$

Where,

L: Curve length (m)

V: Design speed (km/h)

t: Time (s)

<Sample calculation>

Design speed 120km/h case.

L = 120km/h × 1000m/3600sec. × 6sec. = 200m

7.2.3.2.2 Intersection angle less than 7°

Curves with small intersection angles (less than 7°) are hardly visible to drivers and may give a false sense of security to drivers.

The minimum curve lengths to eliminate the defect in such cases are specified in **Figure 7.14** but for cases of difficult terrain or condition use **Table 7.3**.

Moreover, the sensory illusion that arises on road intersection angles of 7° or less becomes more pronounced the smaller the intersection angle. In this case, a long curve should be inserted to give drivers the impression that the road is curving smoothly.

Moreover, when inserting curves with application of the intersection angle of less than 7° shown in **Table 7.6** , it is desirable to insert a distance covered in around 2 seconds at the design speed.

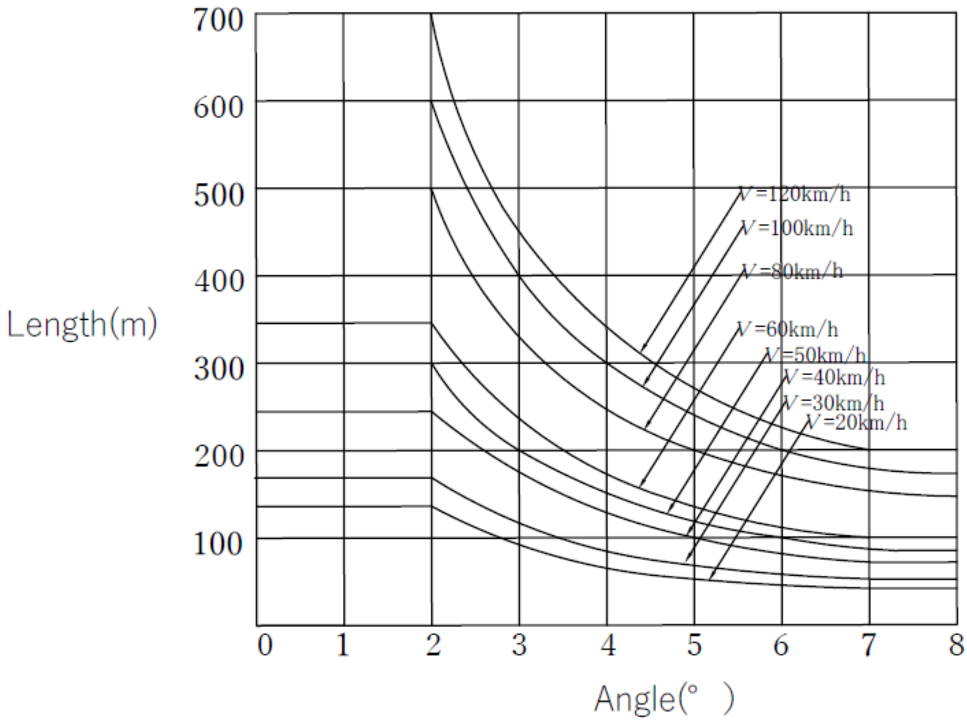
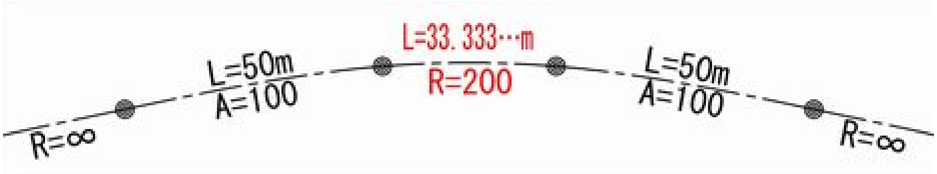


Figure 7.14 Minimum curve length and intersection angle

For curves with spirals, the minimum curve length is determined by the length of the spirals and the arc (spiral + arc + spiral) as shown in **Figure 7.15**. In such a case, the time to traverse the curve should be more than 6 seconds of which 2 seconds is used to design the arc.

Case of bad curve design (V=60km/h)



Case of good curve design (V=60km/h)

Figure 7.15 Example of curve design

7.2.3.3 SUPERELEVATION OF CURVE SECTION

Superelevation is the banking of a roadway around a curve as illustrated in **Figure 7.16**. The purpose of employing superelevation of the roadway cross section is to counterbalance the centrifugal force, or outward pull, of a vehicle traversing a horizontal curve. Side friction developed between the tires and the road surface also counterbalances the outward pull of the vehicle. A combination of these two concepts allows a vehicle to negotiate curves safely at higher speeds than would otherwise be possible.

Incorporating superelevation into a roadway's design may help avoid roadside obstacles that might otherwise be impacted by the alignment. In contrast, superelevation may not be desirable for low-speed roadways to help limit excessive speeds or in urban settings to limit impacts to abutting uses or drainage systems and utilities. Moreover, superelevation may not be desirable when considering pedestrian or bicycle accommodations along the roadway segment. Like other roadway design elements, designers must consider the trade-offs of introducing superelevation in a roadway's design.

When deciding how much superelevation to set in relation to a selected curve radius, the following are to be noted:

a. Examination of safety limit in relation to the side slipping of vehicles running through the curve.
b. Calculation of superelevation and side friction factor

Value of superelevation is related to curve radius and design speed according to **Equation 7.3**.

$$e = \frac{(0.7)^2 \cdot V^2}{127R} = 0.003858\frac{V^2}{R} \tag{7.3}$$

Where,

e: Superelevation.
V: Design speed (km/h).
R: Curve radius (m)

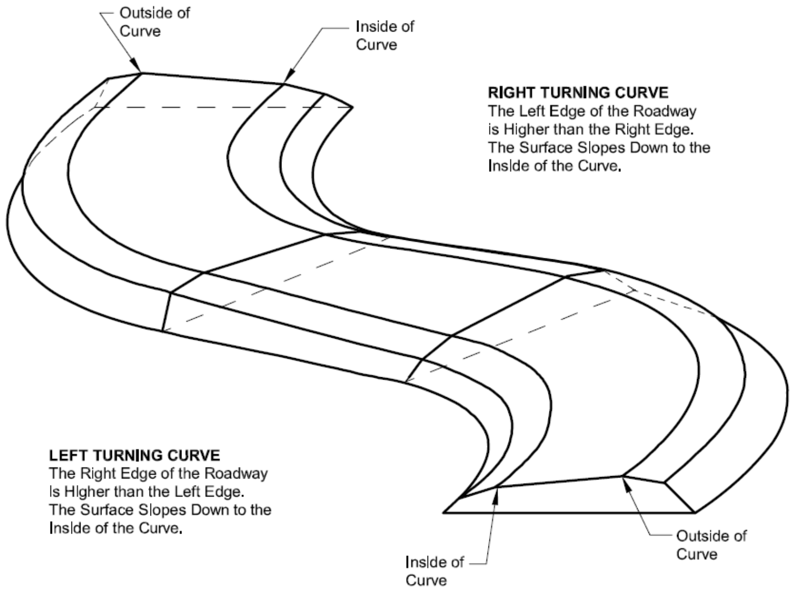


Figure 7.16 Superelevation for Left and Right Turning Curves

7.2.3.3.1 The maximum superelevation of curve section

The maximum useable rate for superelevation (e_{max}) is controlled by several factors: climate conditions, terrain conditions, type of area, and the frequency of slow-moving vehicles.

The selection of the appropriate value of e_{max} is at the discretion of the designer in terms of the design domain concept. The higher values of e_{max} are typically applied to rural areas and the lower values to the urban environment.

Apart from the factors already mentioned, traffic function also influences the superelevation. Roads with mobility functions can have superelevation as high as 9%. However, in urban areas where there are closely spaced at-grade intersections, accesses and traffic management facilities which cause stoppages on the roadway, high values of superelevations are undesirable. In such situations, a maximum superelevation of 5% is recommended to enhance the safety and convenience of motorists, cyclists, and other road users.

A lower value of e_{max} should also be considered in a road where steep gradients occur with any frequency. A superelevation of 9% would present trucks with some difficulties when they are climbing a steep grade at low speeds.

Table 7.4. shows the recommended maximum superelevation based on the design domain (environment) and traffic function.

Table 7.4 Maximum Superelevation

Maximum superelevation (%)	Design Domain	Traffic function
5.0	Urban roads	Access
9.0	Rural roads & High-speed urban roads	Mobility

Although superelevation is advantageous for traffic operation, various factors often combine to make its use impractical in many built-up areas (such as Suburban High Intensity, Suburban Town Centres and Urban Areas). Such factors include wide pavement areas, the need to meet the grade of adjacent property, surface drainage considerations, and frequency of cross streets, alleys, and driveways. Therefore, horizontal curves on low-speed roadways in urban areas may be designed without superelevation, counteracting the centrifugal force solely with side friction.

7.2.3.3.2 Non-superelevated curve sections

The centrifugal forces for large curves are small and the required superelevation becomes negligible. The superelevation becomes unnecessary at the radius shown in **Table 7.5** using **Equation 7.1** with f of 0.04 and e of -2.5% (cross slope of standard straight section)

Table 7.5 Curve Radii where Superelevation is Unnecessary.

Design speed (km/h)	120	100	80	60	50	40	30	20
Calculated Radius (m)	7,559	5,249	3,360	1,890	1,312	840	472	210
Design Radius (m)	7,600	5,300	3,500	2,000	1,300	850	500	200

7.2.4 TRANSITION CURVES

Transition curves are introduced between straights and circular curves or between two circular curves of significantly different radii for the following reasons:

a. For easy steering operation and riding comfort.
b. To provide lengths over which the superelevation and widening can be applied.
c. They improve the appearance of the road by avoiding sharp discontinuities in alignment at the start and end of circular curves.

7.2.4.1 TRANSITION LENGTH

The length of transition is a function of the design speed, and it is derived from the rate of gain of radial acceleration. This is given by **Equation 7. 4**.

$$\mathrm{Ls} = \frac{\mathrm{V}^3}{3.6^3\ \mathrm{C.R}} = \frac{0.06\mathrm{V}^3}{\mathrm{R}} \qquad (7.4)$$

Where:

Ls: Transition length (m)

V: Design speed (km/h)

R: Radius (m)

C: 0.3572m/sec^3 (The rate of gain of radial acceleration.)

7.2.4.2 MINIMUM TRANSITION LENGTHS

There are several methods of determining the minimum lengths of transition curves. These include:

a. The rate of pavement rotation method
b. The shift criterion method
c. The superelevation method

7.2.4.2.1 Rate of pavement rotation method

This is the change in crossfall divided by the time taken to travel along the length of transition at a given design speed. This is given by **Equation 7.5**:

$$L_s(\min) = \frac{V}{3.6} t \quad (7.5)$$

Where:

$L_s(\min)$: Minimum transition length (m).
V: Design speed (km/h).
t: Time taken to traverse the transition curve taken to be 2 seconds.

These minimum values are given in **Table 7.6**.

Table 7.6 Minimum Transition Lengths

Design speed (km/h)	120	110	100	80	60	50	40	30	20
Minimum transition length (m)	67	61	56	44	33	28	22	17	11

7.2.4.2.2 Shift criterion method

For the calculation of the minimum transition lengths, the shift criterion method can be adopted. The shift is the downward displacement of circular curve (**Figure 7.17**). The minimum limit of the shift recommended is 0.2m.

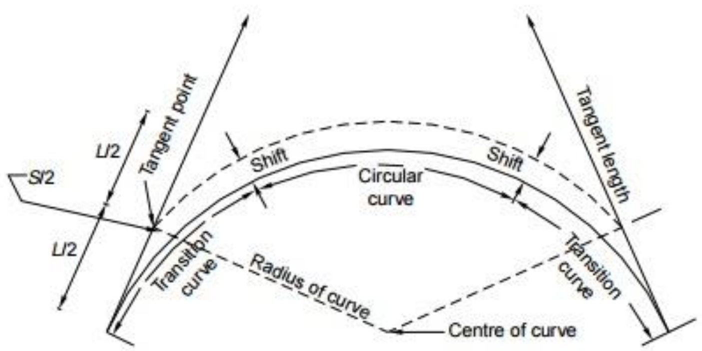


Figure 7.17 Shift

For values below 0.2m, transition curves are deemed not necessary. Generally, the formula for the shift is given by **Equation 7.6**:

$$S = \frac{{L_s}^2}{24R} \tag{7.6}$$

Where:

S: Shift (m).
L_s: Transition length (m).
R: Radius (m).

From **Equations 7.3 and 7.5** the relationship between the shift, design speed and the radius is given in **Equation 7.7**:

$$R = 0.053\frac{V^2}{\sqrt[3]{S}} \tag{7.7}$$

If the limit of the shift (0.2m) is put in **Equation 7.6**, values of radii obtained will not need any transitions. These are shown in **Table 7.7**.

Table 7.7 Radii where there is no need for transitions

Design speed (km/h)	120	100	80	60	50	40	30	20
Calculated Curve radius (m)	1,305	906	580	326	227	145	82	36
Design Curve radius (m)	1,310	910	580	330	230	150	85	40

7.2.4.2.3 Superelevation method

It is known that for superelevation below 3%, vehicles follow transitional paths within their own lanes without needing curves. To establish the overall minimum length of transition, a trade off of some of the minimum requirements of methods (i) and (ii) are made.

In the recommended transition tables in **Appendix A** the transition lengths have been omitted on the basis of the transition lengths being unnecessary below 3% superelevation. These give shift values below 0.2m and transition lengths lesser than the rate of pavement rotation method in **Table 7.7.**

7.2.5 WIDENING OF CURVE SECTIONS

On curve sections, because the rear wheels of vehicles run on the inside more than the front wheels, inner wheel difference arises. Accordingly, it is necessary to appropriately widen lanes on such sections.

The required amount of widening is dependent on the characteristics of the vehicles using the road, the radius and length of the curve and lateral clearances. Carriageway widening is also necessary to present a consistent level of driving task to the road users, to enable them to remain cantered in lane and reduce the likelihood of either colliding with an oncoming vehicle or driving onto the shoulder.

On high trafficked roads with significant proportions of trailers, the amount of curve widening is designed on the basis of the trailer. Similarly, on such roads with high proportions of large vehicles, the design is on the basis of the large vehicle.

Widening is not considered on low trafficked roads since the trailers and large vehicles can use opposite lanes for manoeuvres. Consideration of the small vehicle for design is not necessary in any traffic condition. The recommended amount of widening per lane on curves and on high fills is shown in **Table 7.8** and **Table 7.9**.

Table 7.8 Amount of widening

Curve radius (m)		Widening amount (m) (one lane unit)
Design by trailer	**Design by large vehicle**	
150 < R < 280	90 < R < 160	0.25
100 < R < 150	60 < R < 90	0.50
70 < R < 100	45 < R < 60	0.75
50 < R < 70	32 < R < 45	1.00
	26 < R < 32	1.25
	21 < R < 26	1.50
	19 < R < 21	1.75
	16 < R < 19	2.00
	15 < R < 16	2.25

R: Curve radius.

Table 7.9 Widening on high fill

Height of fill (m)	Widening (m)
0.0 – 3.0	0.3
3.0 – 6.0	0.6
6.0 – 9.0	0.9
Over 9.0	0.9

Widening for curve should be applied on the inside of a curve and be gradually introduced over the length of the transition. Fill widening shall be applied on both sides of the road.

Curve widening should be attained gradually over a length sufficient to make the whole of the carriageway fully usable, also to ensure a reasonably smooth alignment of the edge of carriageway and to fit the paths of vehicles entering or leaving the curve.

7.2.5.1 CALCULATION OF AMOUNT OF WIDENING

A. Trailer

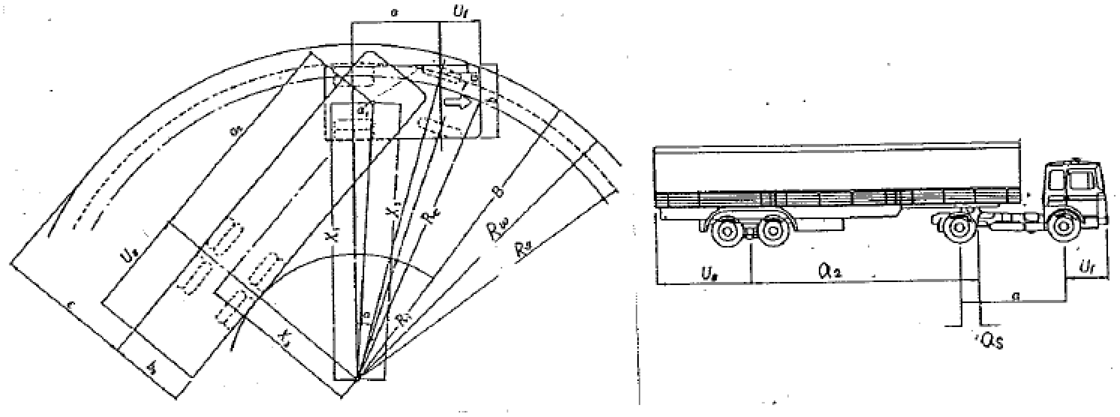


Figure 7.18 Trailer

B: Running width of vehicles.	Rc: Radius of centreline.
L: Length of vehicles.	b: Width of vehicles.
S: Interval of wheels.	a: Wheelbase.
Rw: Outer curve radius.	Rs: Radius of gyration of outer front wheel.
Ri: Inner curve radius.	α: Turning angle of outer front wheel.
U_B: Back end overhang.	U_f: Front end overhang.
ε: Widening amount of one lane.	as: Off-set

<Formulae>

$$\varepsilon = B - b$$

$$B = R_w + b_z/2 - \sqrt{Rc^2 - (a + U_f)^2 - a_z^2 + a_s^2}$$

$$R_w = \sqrt{\{\sqrt{Rc^2 - (a + U_f)^2} + b/2\}^2 + (a + U_f)^2}$$

<Sample calculation>

Condition

(a: 4m. b, b_2: 2.5m. U_f: 1.3m. a_z: 9m. a_s: 0m. R_c: 100m.)

$$B = R_w = 1.25 - \sqrt{100^2 - (4 + 1.3)^2 - 9^2}$$

$$R_w = \sqrt{\{\sqrt{100^2 - (4 + 1.3)^2} + 2.5/2\}^2 + (4 + 1.3)^2} = 101.248$$

$$B = 101.248 + 1.25 - 99.453 = 3.045$$

$\varepsilon = 3.045 - 2.500 = 0.545\text{m}$ → Amount of widening of one lane is 0.5m.

B. Large vehicle

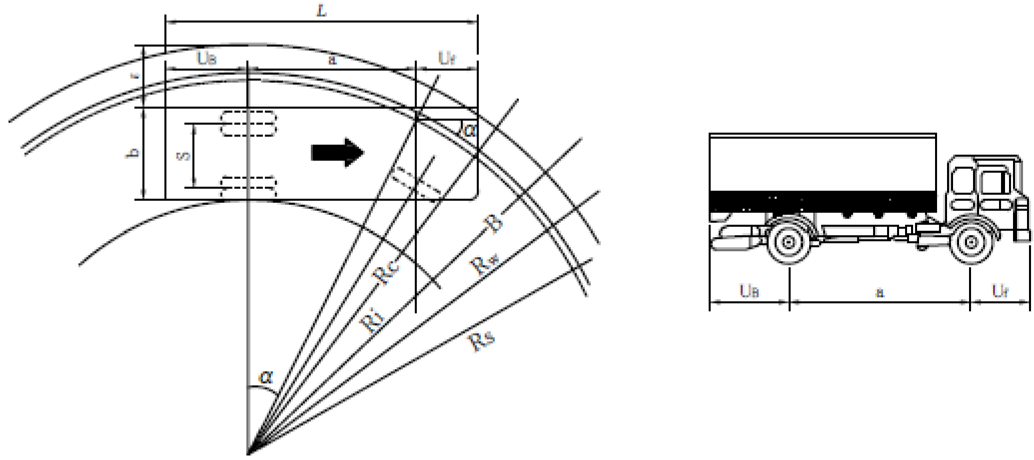


Figure 7.19 Large vehicle

B: Running width of vehicles.
L: Length of vehicles.
S: Interval of wheels.
Rw: Outer curve radius
Ri: Inner curve radius.
U_B: Back end overhang.
ε: Amount of widening of one lane.

Rc: Radius of centreline.
b: Width of vehicles.
a: Wheelbase.
Rs: Radius of gyration of outside front wheel
α: Turning angle of outside front wheel.
U_f: Front end overhang.

<Formulae>

$$\varepsilon = B - b$$

$$B = R_w + b/2 - \sqrt{Rc - (a + U_f)^2}$$

$$R_w = \sqrt{\{\sqrt{R_c{}^2 - (a + U_f)^2} + b/2\}^2 + (a + U_f)^2}$$

<Sample calculation>

Condition

(b: 2.5m, a: 6.5m, U_f: 1.5m, R_c, 100m.)

$$B = R_w + 1.25 = \sqrt{100^2 - (6.5 + 1.5)^2}$$

$$R_w = \sqrt{\{\sqrt{100^2 - (6.5 + 1.5)^2} + 2.5/2\}^2 + (2.5 + 1.5)^2} = 101.246$$

$$B = 101.246 + 1.25 - \sqrt{100^2 - (6.5 + 1.5^2)} = 2.817$$

$\varepsilon = 2.817 - 2.500 = 0.317$ →Amount of widening of one lane is 0.25m.

7.2.6 SIGHT DISTANCE

When designing roads, standards are provided for width, alignment, gradient, and sight distance. Among these, sight distance has an important role to play. This is because drivers operate vehicles while watching the road ahead. The ability to see ahead is of the utmost importance in the safe and efficient operation of vehicles on roads. Sight distances of sufficient length should be provided so that drivers are able to predict, react to and negotiate through road features, pavement conditions and the movements of other vehicles or road-users.

There are four (4) principal sight distances which are of particular interest in geometric design.

i. Sopping sight distance
ii. Passing sight distance
iii. Decision Sight Distance
iv. Intersection Sight Distance

7.2.6.1 STOPPING SIGHT DISTANCE

This is the distance required for a driver to bring his vehicle to a stop once he has seen a hazard on the road ahead. Stopping sight distance is the sum of two distances: (1) the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied, and (2) the distance needed to stop the vehicle from the instant brake application begins. These are referred to as brake reaction distance and braking distance, respectively.

Generally, a sufficiently safe value is adopted for the brake reaction distance and the braking distance upon fully taking running speed, friction force between the road and tires, the time it takes drivers to react and brake in response to obstructions, and wetness into account. In road design, sight distance must be set higher than the brake reaction distance and braking distance.

The derivation of stopping sight distance is based on assumed values for total driver reaction time (2-2.5 sec) and the rate of deceleration, the latter expressed in terms of the coefficient of longitudinal friction given in **Equation 7.8** and **Equation 7.9**.

$$S = \frac{V.t}{3.6} + \frac{V^2}{254(f + G)} \tag{7.8}$$

$$S = 0.694 \ + 0.00394 \frac{V^2}{f} \tag{7.9}$$

Where,

S: Sight distance (m)
t: Perception time + Reaction time (2.5 sec^2)
g: Gravitational acceleration (9.8m/sec^2)
V: Running speed (km/h)

f: Coefficient of longitudinal friction

G: percent grade, +ve for upgrade and –ve for down grade (%/100)

The recommended values obtained based on t=2.5sec and f=0.29-0.40 for level roadway and wet pavement using **Equation 7.8** are shown in **Table 7.10** based on Crons findings for developing countries.

Table 7.10 Minimum stopping sight distance on level roadways

Design speed (km/h)	Running speed (km/h)	f	0.694 V	0.00394 V^2/f	S (m)	Minimum S.S.D (m)
120	(85%) 102	0.29	70.8	141.4	212.2	210
100	(85%) 85	0.30	58.9	94.8	153.7	160
80	(85%) 68	0.31	47.1	58.7	105.8	110
60	(90%) 54	0.33	37.4	34.8	72.2	75
50	(90%) 45	0.35	31.2	22.8	54.0	55
40	(90%) 36	0.38	24.9	13.4	38.3	40
30	(100%) 30	0.40	20.8	8.9	29.7	30
20	(100%) 20	0.40	13.9	3.9	17.8	20

Effect of grade on stopping sight distances for 3%, 6% and 9% upgrades and downgrades using **Equation 7.8** are shown in **Table 7.11**.

Table 7.11 Minimum stopping sight distance on Grades

Design speed (km/h)	Upgrades (+ve)			Downgrades (-ve)		
	3%	6%	9%	3%	6%	9%
120	199	188	179	228	249	276
100	145	138	132	164	178	194
80	101	96	93	112	120	130
60	69	67	65	76	80	85
50	52	51	49	56	59	62
40	37	37	36	39	41	43
30	29	29	28	30	31	32
20	18	17	17	18	19	19

7.2.6.2 PASSING SIGHT DISTANCE

The Passing Sight Distance is the minimum sight distance on a two-way road that must be available to enable the driver of one vehicle to pass another vehicle safely without interfering with the speed of an oncoming vehicle travelling at the design speed. If passing is to be accomplished without interfering with an opposing vehicle, the passing driver should be able to see a sufficient distance ahead, clear of traffic, so the passing driver can decide whether to initiate and to complete the passing manoeuvre without cutting off the passed vehicle before meeting an opposing vehicle that appears during the manoeuvre. When appropriate, the driver can return to the right lane without completing the pass if he or she sees opposing traffic is too close when the manoeuvre is only partially completed. Many passing manoeuvres are accomplished without the driver being able to see any potentially conflicting vehicle at the beginning of the manoeuvre.

Passing Sight Distance is a desirable requirement for two-way single roadway roads. Sufficient visibility for passing increases the capacity and efficiency of a road and should be provided for as much of the road length as possible within financial limitations.

The passing sight distance is determined empirically, and the values recommended are given in **Table 7.12**. They are consistent with Crons values recommended for developing countries.

Table 7.12 Minimum Passing Sight Distance

Design speed (km/h)	120	100	80	60	50	40	30	20
Passing sight distance (m)	780	620	500	360	280	210	140	70

7.2.6.3 DECISION SIGHT DISTANCE

Normally, the stopping sight distance is an adequate sight distance for roadway design. However, there are cases where it may not be appropriate. In areas where information about navigation or hazards must be observed by the driver, or where the driver's visual field is cluttered, the stopping sight distance may not be adequate. In addition, there are avoidance manoeuvres that are far safer than stopping but require more planning by the driver. These may not be possible if the minimum stopping sight distance is used for design. In these instances, the proper sight distance to use is the decision sight distance.

The decision sight distance is the distance traversed while recognizing an object or hazard, plotting an avoidance course, and making the necessary manoeuvres. Unlike the stopping sight distance, the decision sight distance is quite complex.

Because decision sight distance gives drivers additional margin for error and affords them sufficient length to manoeuvre their vehicles at the same or reduced speed rather than to just stop, it is substantially longer than stopping sight distance. Drivers need decision sight distances whenever there is likelihood for error in information reception, decision-making, or control actions. Critical locations where these kinds of errors are likely to occur, and where it is desirable to provide decision sight distance include:

i. Approaches to interchanges and intersections.
ii. Changes in cross-section such as at toll plazas and lane drops.
iii. Design speed reductions.
iv. Areas of concentrated demand where there is likely to be 'visual noise', e.g., where sources of information, such as roadway elements, opposing traffic, traffic control devices, advertising signs and construction zones, compete for attention.

The minimum decision sight distances that should be provided for specific situations are shown in **Table 7.13**. If it is not feasible to provide these distances because of horizontal or vertical curvature or if relocation is not possible, special attention should be given to the use of suitable traffic control devices for advance warning. Although a sight distance is suggested for the left side exit, the designer should bear in mind that exiting to the left on a main road is in conflict with driver expectancy and is highly undesirable. The only reason for providing this value is to allow for the possibility that a left side exit has to be employed.

Table 7.13 Sight distance decision

Design Speed (km/h)	Situations				
	Interchanges. Sight distance to nose		Lane drop, closure, merge. Sight distance to taper area	Lane shift. Sight distance to beginning of shift	Intersections. Sight distance to turn lane
	Right exit	Left exit			
	Decision Sight Distances (metres)				
50	NA	NA	150	85	150
60	200	275	200	100	200
80	250	340	250	150	250
100	350	430	350	200	350
120	400	500	400	250	400

7.2.6.4 INTERSECTION SIGHT DISTANCE

The driver of a vehicle approaching an intersection should have an unobstructed view of the entire intersection and an adequate view of the intersecting highway to permit control of the vehicle to avoid a collision. When designing an intersection, the following factors should be taken into consideration:

i. Adequate sight distance should be provided along both roadway approaches and across corners.
ii. Gradients of intersecting roads should be as flat as practical on sections that are to be used for storage of stopped vehicles.
iii. Combination of vertical and horizontal curvature should allow adequate sight distance of the intersection.
iv. Traffic lanes and marked pedestrian crosswalks should be clearly visible at all times.
v. Lane markings and signs should be clearly visible and understandable from a desired distance.
vi. Intersections should eliminate, relocate or modify conflict points to the extent allowable in order to improve safety.
vii. Intersections should be evaluated for the effects of barriers, rails, and retaining walls on sight distance.

For selecting intersection sight distance, refer to **Section 8.6.1**. Sight distance criteria are provided for the following types of intersection controls:

a. Intersections with no control
b. Intersections with stop control on the minor road
c. Intersections with yield control on the minor road
d. Intersections with traffic signal control
e. Intersections with all-way stop control
f. Left turns from the major road

7.2.7 OBSTRUCTIONS TO SIGHT DISTANCE ON HORIZONTAL CURVES

Visibility within curve sections can be impaired because of obstacles such as buildings, cut faces of slopes, etc. Accordingly, designs need to be checked in both the horizontal and vertical planes for obstructions. Sight distances can be ensured by leaving clear the inner part of the curve to a distance equal to the setback distance as shown in **Figure.7.20**. The setback distance (E) is given by **Equation 7.10.**

Minimum radii of horizontal curvature are determined by application of vehicle dynamics and not through sight distance controls. It is, therefore, possible that the selected radius may not be adequate to ensure the safe stopping sight distance requirements. If the obstructions to sight distance are immovable, re- alignment may be necessary.

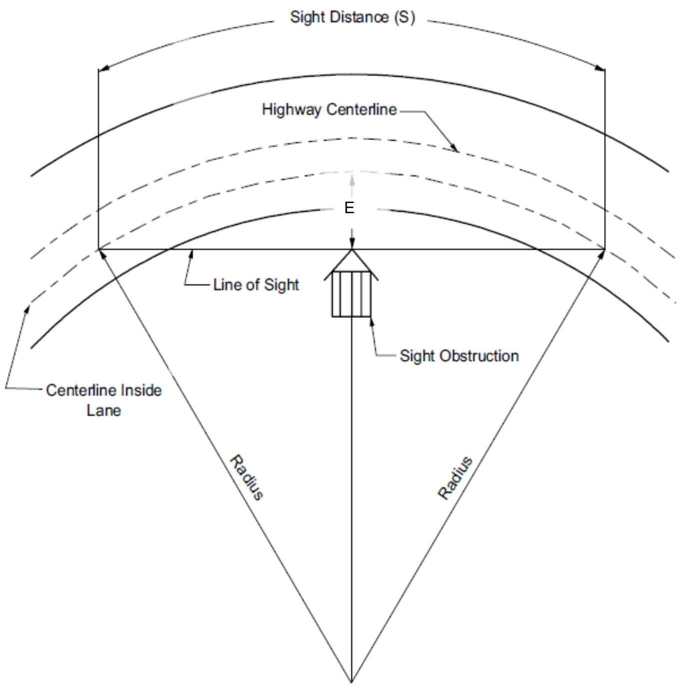


Figure 7.20 Sight distance on horizontal curves

$$E = \frac{S^2}{8R} \tag{7.10}$$

Where,

S: Stopping Sight distance (m)
R: Curve radius (m)
E: Setback distance (m)

7.2.8 SUPERELEVATION TRANSITIONING

The development of superelevation on a horizontal curve requires a transition from a normal crown section, which is accomplished by rotating the pavement. The pavement may be rotated about the centreline or either edge of the travel lanes.

There are five basic cross section controls — (-A-) through (-E-) — involved in transitioning the pavement to obtain full superelevation illustrated in **Figure 7.21**.

i. Cross section (-A-) is the normal crown section where the transitioning begins.
ii. Cross section (-B-) is reached by rotating half the pavement until it is level.
iii. Cross section (-C-) is attained by continuing to rotate the same half of pavement until a plane section is attained across the entire pavement section, at a cross slope equal to the normal crown slope.
iv. Cross section (-D-) is the rate of the cross slope at any intermediate cross section between (-C-) and (-E-) is proportional to the distance from Cross section (-E-).
v. Cross section (-E-) is achieved by further rotation of the planar section, the entire pavement section, to attain the full superelevation at a cross slope equal to (-E-).

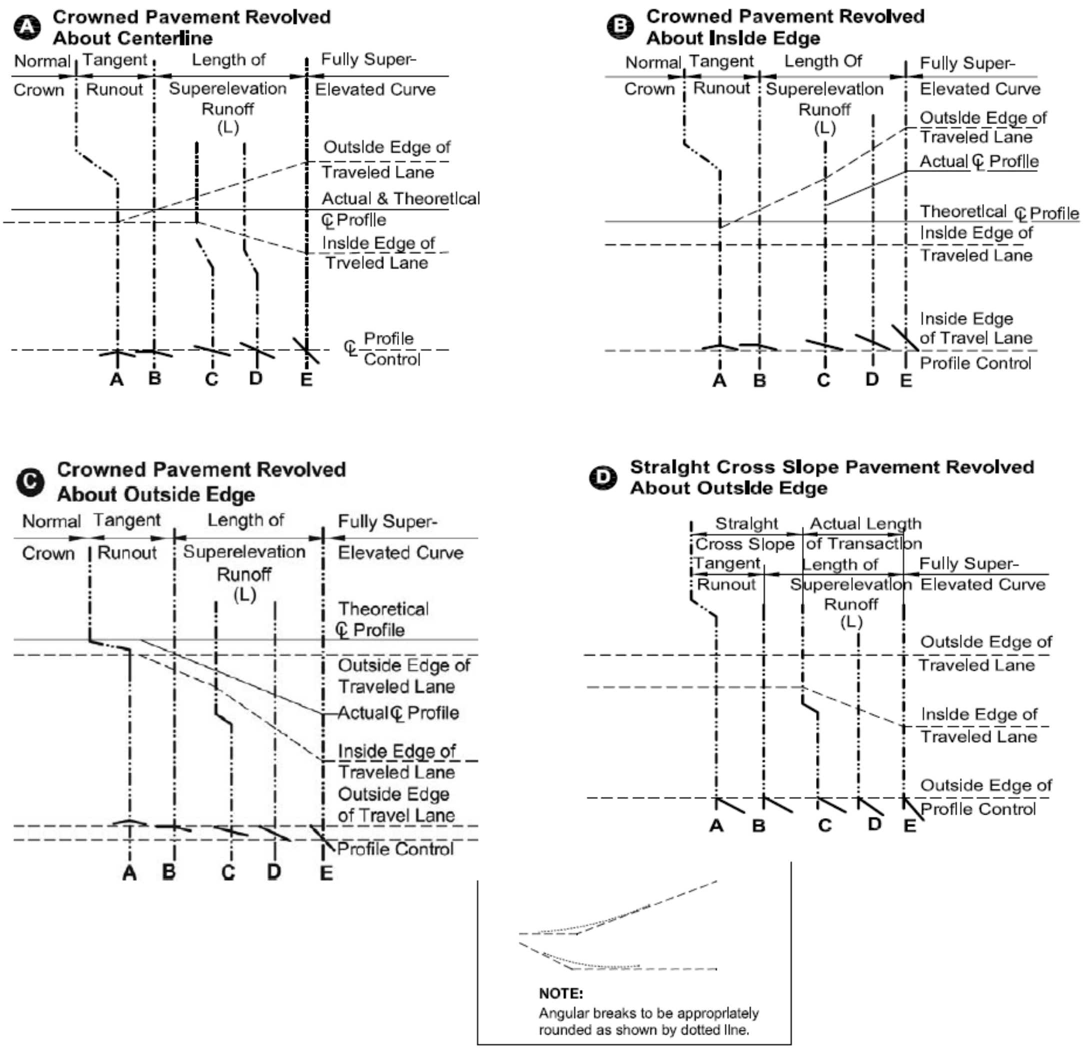


Figure 7.21 Methods of Attaining Superelevation

7.2.8.1 SUPERELEVATION RUNOFF

Superelevation runoff is the general term denoting the length of roadway needed to accomplish the change in cross slope from a section with adverse crown removed (-A-) to a fully superelevated section (-E-), or vice versa. It is necessary to consider limiting the turning angle velocity when the superelevation changes to a certain degree to ensure that drivers experience no discomfort when driving through such sections.

In transition curves, such runoffs are wholly applied within the transition and at the beginning and end of simple curves. In simple curves, the superelevation runoff is provided 2/3 in straight portions and 1/3 in circular curves. The runoff is calculated from **Equation 7.11**.

$$L_R = \frac{\mathrm{wn}_1 \cdot \mathrm{e}_d}{\mathrm{q}} \mathrm{b}_w \qquad (7.11)$$

Where,

L_R: Superelevation runoff, m

e_d: Design superelevation rate, m/m
w: Width of one traffic lane, m
n_1: Number of lanes rotated
q: Ratio of superelevation runoff
b_w: Adjustment factor for number of lanes rotated

Values of 'q' which are functions of the design speed are given in **Table 7.14**.

Table 7.14 Ratio of superelevation runoff

Design speed (km/h)	120	100	80	60	50	40	30	20
Ratio (q)	1/263	1/244	1/200	1/167	1/150	1/143	1/133	1/125

If the **Equation 7.8** is applied to cross-sections wider than two lanes, the length of the superelevation runoff could double or triple and there may simply not be enough space to allow for these lengths. On a purely empirical basis, it is recommended that the calculated lengths be adjusted downwards by the lane adjustment factors offered in **Table 7.15**.

Table 7.15 Adjustment factor for number of lanes rotated

Number of lanes rotated, n_1	Adjustment factor, b_w ($b_w = [1 + 0.5\,(n_1 - 1)]/n_1$	Length increases relative to one-laned rotated, ($n_1 b_w$)
1	1.00	1.0
1.5	0.83	1.25
2	0.75	1.5
2.5	0.70	1.75
3	0.67	2.0
3.5	0.64	2.25

7.2.8.2 TANGENT RUNOUT

Tangent runout is the general term denoting the length of roadway needed to accomplish the change in cross slope from a normal section (-A-) to a section with the adverse crown removed (-B-), or vice versa.

The length of tangent runout is determined by the amount of adverse cross slope to be removed and the rate at which it is removed. To achieve a smooth edge of pavement profile, the rate of removal should equal the relative gradient used to define the superelevation runoff length. Based on this rationale, the **Equation 7.12** should be used to compute the minimum tangent runout length.

$$L_t = \frac{e_{NC}}{e_d} L_R \tag{7.12}$$

Where,

L_t = minimum length of tangent runout, m

e_{NC} = normal cross slope rate, percent

e_d = design superelevation rate, percent

L_R = minimum length of superelevation runoff, m

7.2.8.3 METHODS OF ATTAINMENT OF SUPERELEVATION.

To attain superelevation an axis must be selected about which the pavement is rotated. In general, there are four methods that may be selected:

A. Rotation about the centreline profile of travelled way. This is generally the preferred method for two lane and undivided multilane roadways and when the elevations of the outside of roadway must be held within critical limits, such as in an urban area to minimize the impact on adjacent properties. This is also the method that distorts the edge line profiles the least. **Figure 7.22(A)** graphically demonstrates how the roadway superelevation is developed for this method.

B. Rotation about the inside-edge profile of travelled way. This is generally the preferred method when the lower edge profile is of concern, such as when the profile is flat, and the inside edge of the roadway needs to be controlled for drainage purposes. **Figure 7.22(B)** graphically demonstrates how the roadway superelevation is developed for this method. This method is suitable for ramps.

C. Rotation about the outside-edge profile of travelled way. This method is similar to inside edge rotation except that the change is effected below the outside-edge profile instead of above the inside edge profile. This method is used when the higher edge profile is critical, such as on divided highways where the median edge profiles are held. **Figure 7.22(C)** graphically demonstrates how the roadway superelevation is developed for this method.

D. Rotation about the outside-edge profile of travelled way when the roadway has a straight cross-slope at the beginning of transition (-A-). The outside-edge rotation is shown because this point is most often used for rotation of two-lane one-way roadways, with profile along the median edge of travelled way or for the travelled way having a typical straight cross-slope. **Figure 7.22(D)** graphically demonstrates how the roadway superelevation is developed for this method.

7.2.8.4 SHOULDER SUPERELEVATION

All outside shoulders and median (with median drain) shoulders should slope away from the travel lanes on superelevated curves.

Shoulders may slope in the same direction as the travel lane on superelevated curves when the wearing course runs across the travel lane and the shoulders.

Examples of superelevation transitioning.

1. Simple circular curves

<Conditions> For a 7.3m asphaltic road with design speed of 60 km/h and in a simple left-hand curve of radius 350m with superelevation of 4%, calculate the length of superelevation transitioning and draw the associated superelevation diagram. Assume that the chainage at TP1 is 2+250.

Solution

Step 1 Calculate the superelevation runoff using **Equation 7.11** and **Table 7.14**

$$L_R = \frac{\mathrm{wn_1} \cdot \mathrm{e}_d}{\mathrm{q}} \mathrm{b}_w$$

e_d:	0.04m/m
w:	3.65m
n_1:	1
q:	1/167
b_w:	1

$$L_R = \frac{3.65 \times 1 \times 0.04 \times 1}{1/167} = 24.382\mathrm{m}$$

$$\text{Length of } L_R \text{within curve} = \frac{1}{3} \times 24.382 = 8.13\mathrm{m}$$

Therefore, chainage at end of superelevation runoff (full superelevation) = CH2+250 + 8.13 = CH2+258.13

$$\text{Length of } L_R \text{within tangent} = \frac{2}{3} \times 24.382 = 16.25\mathrm{m}$$

Therefore, chainage at start of superelevation runoff (level crown) = CH 2+250 - 16.25 = CH 2+233.75

Step 2 Calculate the tangent runout using **Equation 7.12**

$$L_t = \frac{\mathrm{e}_{NC}}{\mathrm{e}_d} L_R$$

$$L_t = \frac{2.5}{4} 24.382 = 15.23875 \cong 15.239\mathrm{m}$$

Therefore, chainage at start of tangent runout (normal crown) = CH 2+33.75 - 15.239 = CH 2+218.511

Note that, the distance between level crown and reverse crown is equal to L_t

Therefore, chainage at reverse = CH 2+233.75 + 15.239 = CH 2+248.989

Therefore length of superelevation transitioning $= L_R + L_t = 39.621m$

Step 3 Plot x (chainage) vs crossfall (y-axis)

Section of Superelevation Transitioning	Cross section	Left Hand Side		Right Hand Side	
		Cross fall (%)	Chainage	Cross fall (%)	Chainage
Tangent Runout	Normal Crown	-2.5	2+219	-2.5	2+219
	Level Crown	-2.5	2+234	0	2+234
Superelevation Runoff	Reverse Crown	-2.5	2+249	2.5	2+249
	Full Superelevation	-4	2+258	4	2+258

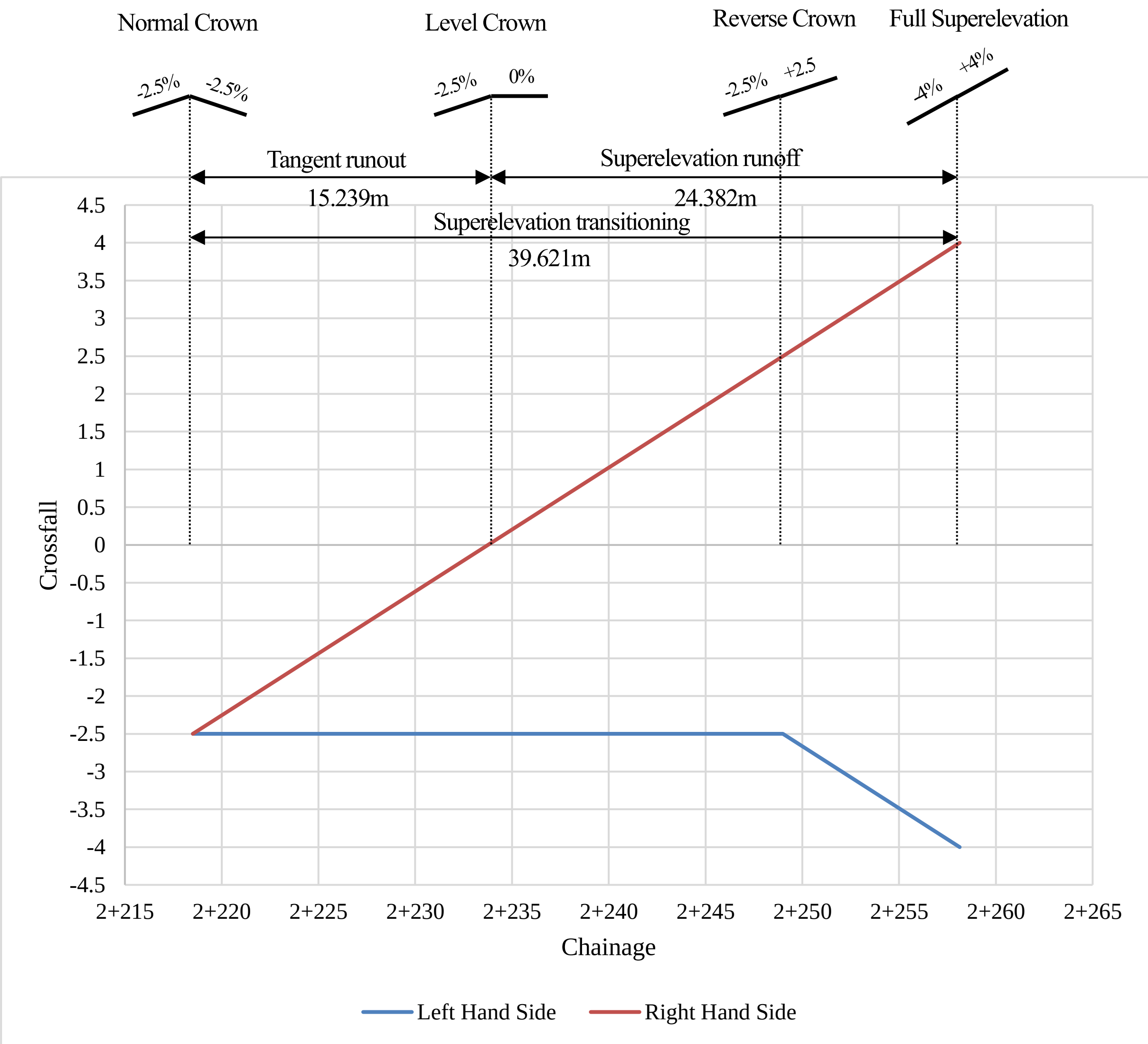


Figure 7.22 Diagram of profile and cross section (Simple circular curves)

2. Curve with transitions

<Conditions>
A curve with transitions on a 7.3m wide asphaltic carriageway has the following parameters: Design speed is 60 km/h, right-hand Curve radius is 133.246m, maximum superelevation is 9%, The transition length is 97.264m and chainage at TS1 is CH2+400.

Solution

Step 1 Determine length of superelevation runoff

For curve with transitions, transition length = length of superelevation runoff (L_R)= 97.264m

Chainage at start of superelevation runoff (normal crown) = CH2+400 - 97.264 = CH2+302.736.

Step 2 Determine length of tangent runout

$$L_t = \frac{2.5}{9} 97.264 = 27.018\text{m}$$

Chainage at start of tangent runout = CH2+302.736 - 27.018 = CH2+275.718

Chainage at reserve crown = Chainage at normal crown + L_t = CH2+302.736 + 27.018 = CH2+329.754

$Therefore$ length of superelevation transitioning = 97.264 + 27.018 = 124.282m

Step 3 Plot x (chainage) vs crossfall (y-axis)

Section of Superelevation Transitioning	Cross section	Left Hand Side		Right Hand Side	
		Cross fall (%)	Chainage	Cross fall (%)	Chainage
Tangent Runout	Normal Crown	-2.5	2+276	-2.5	2+276
	Level Crown	0	2+303	-2.5	2+303
Superelevation Runoff	Reverse Crown	2.5	2+330	-2.5	2+330
	Full Superelevation	9	2+400	-9	2+400

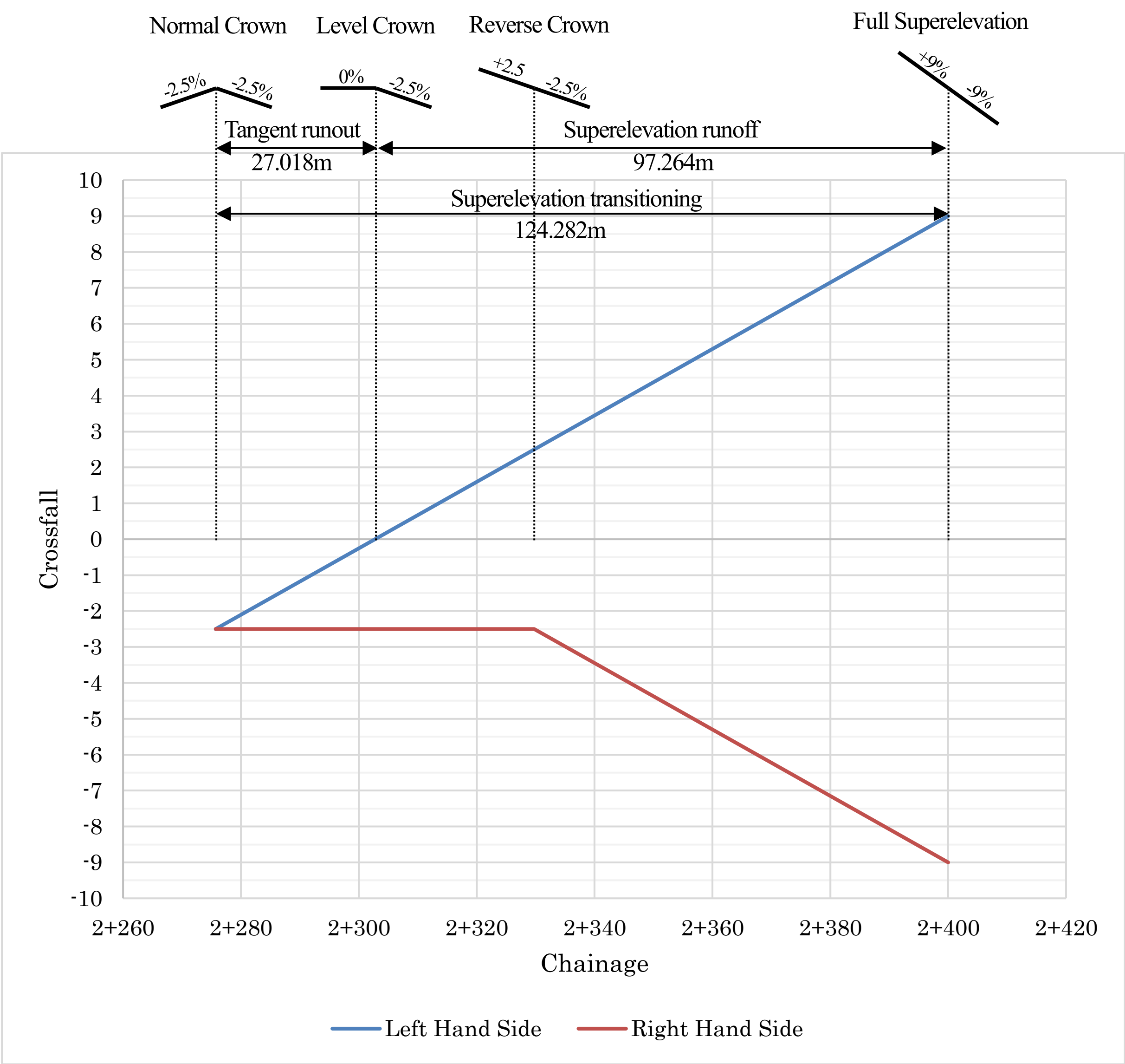


Figure 7.23 Diagram of profile and cross section (Curve with transitions)

7.2.9 WIDENING RUNOFF

On simple curves, widening is applied from zero to full widening over the superelevation runoff length and wholly on the inside edge. On transition curves, it may be placed wholly on the inside or divided equally between the inside and outside of the curve from zero to full widening over the transition length.

In all cases the centreline marking should be placed halfway between the edges of the widened carriageway. Calculation of runoff is illustrated in **Figure 7.24**.

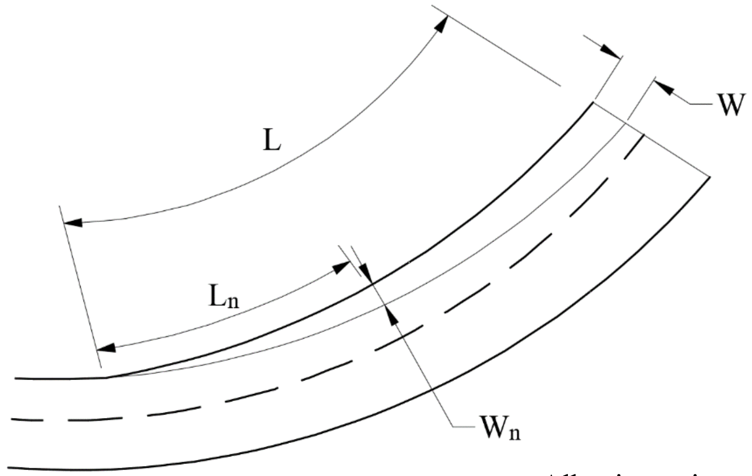


Figure 7.24 Widening runoff

The width of widening at any point on the runoff section is given by **Equation 7.13.**

$$W_n = \frac{L_n}{L} \times W \qquad (7.13)$$

Where:

L:	Runoff section.
W:	Width at L (lane width of widening × No of lanes)
W_n:	Width of widening at any point on the runoff section
L_n:	Length of runoff for every corresponding W_n

For values of lane width of widening refer to **Table** 7.8.

7.2.10 INCREASE AND DECREASE OF LANE RUNOFF

This is the tapering which ensures the smooth flow of traffic when increasing or decreasing the number of lanes. For uninterrupted flows, the runoffs producing the increase or decrease in the number of lanes on sections of road are designed on the basis of the design speed and the route location (rural or urban area).

Lane runoff may be needed in the following circumstances:

i. Interface between an undivided road and a divided road
ii. Changes in the number of traffic lanes
iii. Local narrowing to single lane
iv. Reduction of paved shoulder width
v. Reduction of traffic lane width
vi. Redistribution of cross-sections on urbanised sections

The runoff ratios are shown in **Table 7.16**.

Table 7.16 Increase and decrease of lane runoff

Design speed (km/h)	120	100	80	60	50	40	30	20
Rural area runoff ratio	1/70	1/60	1/50	1/40	1/30	1/25	1/20	1/15
Urban area runoff ratio	—	—	1/40	1/30	1/25	1/20	1/15	1/10

Runoff ratio is width/length

7.3 VERTICAL ALIGNMENT

The vertical alignment defines the position of the grade height of the roadway. Vertical alignment is composed of tangent sections called grades joined by vertical curves. The grades are equal to the vertical rise or fall over the distance traversed, expressed as a percent. The vertical curves can be a crest, curved downward going from a positive to a negative grade or a sag curved upwards going from a negative to a positive grade. The resulting grades and vertical curves are called the profile or gradeline.

7.3.1 DESIGN PRINCIPLES FOR VERTICAL ALIGNMENT

Roadway vertical alignment is controlled by design speed, topography, traffic volumes and composition, functional classification, safety, sight distance, typical cross sections, horizontal alignment, climate, vertical clearances, drainage, economics, and aesthetics.

When designing vertical alignment, while paying attention to grades and curves and their combinations, examination must also be conducted regarding the actual vertical gradient. It is especially necessary to avoid mutual combinations of the following kinds of vertical alignments:

a. A smooth gradeline with gradual changes, as consistent with the type of highway, road, or street and the character of terrain, should be sought in preference to a line with numerous breaks and short lengths of grades. Specific design criteria are the maximum grade and the critical length of grade, but the manner in which they are applied and fitted to the terrain on a continuous line determines the suitability and appearance of the finished product.

b. The “roller-coaster” or the “hidden-dip” type of profile should be avoided (**Figure 7.25c**). Such profiles generally occur on relatively straight, horizontal alignment where the roadway profile closely follows a rolling natural ground line. Examples of such undesirable profiles are evident on many older roads and streets; they are unpleasant aesthetically and difficult to drive. Hidden dips may create difficulties for drivers who wish to pass, because the passing driver may be deceived if the view of the road or street beyond the dip is free of opposing vehicles. Even with shallow dips, this type of profile may be disconcerting, because the driver cannot be sure whether or not there is an oncoming vehicle hidden beyond the rise. This type of profile is avoided by use of horizontal curves or by more gradual grades.

c. Undulating gradelines, involving substantial lengths of momentum grades, should be evaluated for their effect on traffic operation. Such profiles permit heavy trucks to operate at higher overall speeds than where an upgrade is not preceded by a downgrade but may encourage excessive speeds of trucks with attendant conflicts with other traffic.

d. A "broken-back" gradeline (two vertical curves in the same direction separated by a short section of tangent grade) as shown in **Figure 7.25a** generally should be avoided, particularly in sags where the full view of both vertical curves is not pleasing. This effect is particularly noticeable on divided roadways with open median sections.

e. On long grades, it may be preferable to place the steepest grades at the bottom and flatten the grades near the top of the ascent or to break the sustained grade by short intervals of flatter grade instead of providing a uniform sustained grade that is only slightly below the recommended maximum. This is particularly applicable to roads and streets with low design speeds.

f. Where at-grade intersections occur on roadway sections with moderate to steep grades, it is desirable to reduce the grade through the intersection. Such profile changes are beneficial for vehicles making turns and serve to reduce the potential for crashes. Profiles at pedestrian crosswalks must consider limitations on cross slope so that the crosswalk is accessible to and usable by individuals with disabilities.

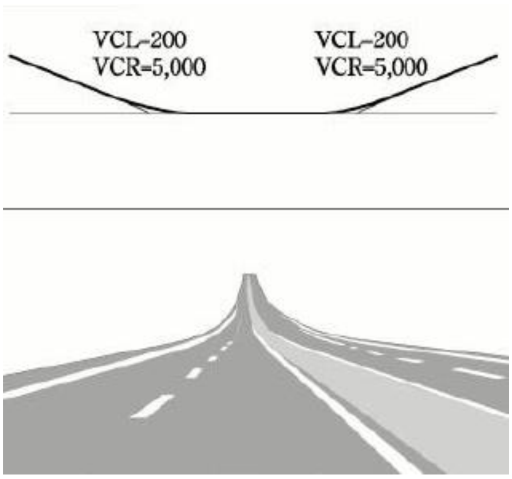
g. Sag vertical curves should be avoided in cuts unless adequate drainage can be provided.

h. Sag vertical curves at under crossings should be designed to provide vertical clearance for the largest legal vehicle that could use the undercrossing without a permit.

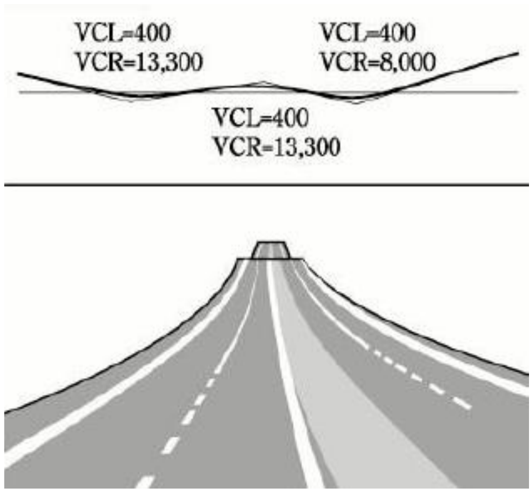
i. Adoption of vertical alignment with repeated unevenness over short sections as shown in **Figure 7.25b.**

j. Insertion of longer vertical curves than needed on sag sections as shown in **Figure 7.25c**. In **Figure 7.25c** the first vehicle moving upwards reduces speed without the driver noticing, resulting in the preceding drivers slowing down thereby causing traffic congestion.

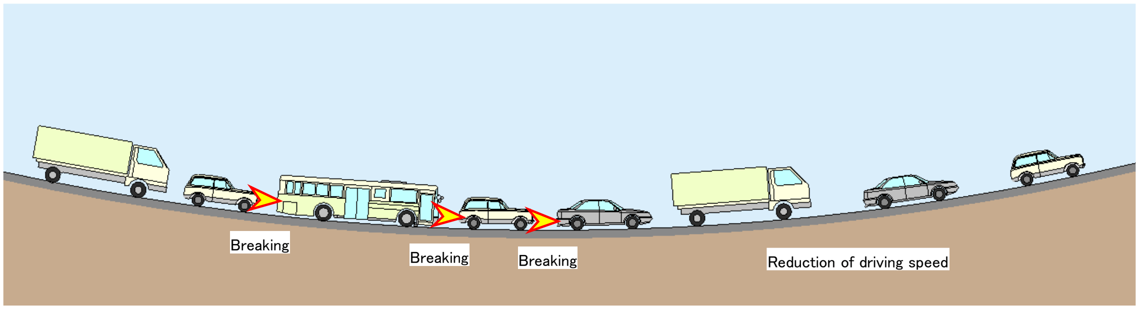
k. Sudden gradient changes should be avoided in vertical alignment design to ensure that the alignment visually appears smooth. For example, if the vertical gradient is abruptly changed around a tunnel exit, depending on the terrain, it can appear that vehicles are flying through the sky (**Figure 7.25e**). In such cases, it is necessary to design in consideration of the continuity of vertical alignment to ensure that the alignment beyond the tunnel exit can be seen from inside the tunnel.



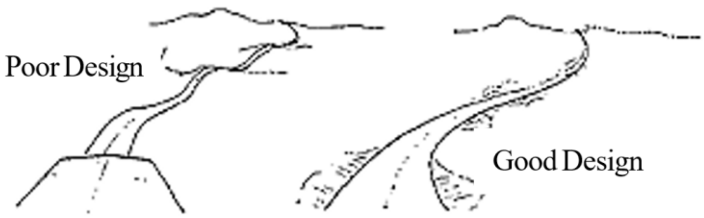
(a) Broken back curve



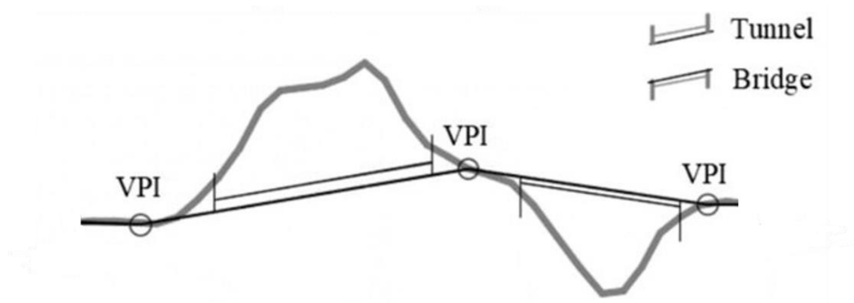
(b) Repeadeted vertrical curves



(c) Excessively long sag curve



(d) Roller coaster profile



(e) vertical gradient abruptly changed around a tunnel exit

Figure 7.25 Design principles for vertical alignment

In addition to the foregoing principles, the designer should also consider the following:

i. In level terrain, the designer's ability to efficiently satisfy the design controls can be accomplished without construction difficulty or extraordinary expense; however, as the terrain becomes more challenging, as in rolling or mountainous terrain and developed areas, significantly more complicated construction techniques must be employed to achieve compatibility between the road alignment and the surrounding ground. Introducing vertical curves to minimize the disruption to the existing environment may result in sight distance or clearance issues and may require truck climbing lanes for higher-speed facilities. The designer must balance these factors when introducing vertical curves into a roadway alignment.

ii. Where a road crosses a waterway, the profile of the road must be consistent with the design flood frequency and elevation.

iii. The roadway elevation must provide sufficient clearance and cover for construction of culverts and other components of the drainage system.

iv. When a road is located where environmental resources exist, the vertical alignment should be designed to minimize impacts.

v. Vertical alignment should be properly coordinated with the natural topography, available right-of-way, utilities, roadside development, and natural and man-made drainage patterns.

7.3.2 MAXIMUM GRADIENTS

Vehicle operations on gradients are complex and depend on a number of factors: severity and length of gradient; volume and composition of traffic; and the number of overtaking opportunities on the gradient and in its vicinity. The maximum gradient and the limit length for absolute grades for the various design speeds are shown in **Table 7.17** and **Table 7.18** respectively. The maximum design grade should be used infrequently; in most cases, grades should be less than the maximum design grade.

Table 7.17 Vertical gradient value

Design Speed (km/h)	Vertical Gradient (%)		
	Desirable maximum gradient	Absolute maximum gradient	
		Motorway/Rural Road	Urban Road
120	2	5	-
100	3	6	-
80	4	7	-
60	5	8	7
50	6	9	8
40	7	10	9
30	8	11	10
20	9	12	11

It is the designer's responsibility to select a maximum gradient appropriate to the project being designed. The values offered in **Table 7.17** are thus only intended to provide an indication of gradients appropriate to the various circumstances.

Table 7.18 Limit lengths for absolute gradients

Design speed (km/h)	Desirable Maximum Gradient (%)	Limit length of gradient	
		Absolute maximum gradient (%)	Critical length of gradient (m)
120	2	3 4 5	800 500 400
100	3	4 5 6	700 500 400
80	4	5 6 7	600 500 400
60	5	6 7 8	500 400 300
50	6	7 8 9	500 400 300
40	7	8 9 10	400 300 200
30	8	9 10 11	400 300 200
20	9	10 11 12	300 200 100

The term "critical length of grade" in **Table 7.18** is to indicate the maximum length of a designated upgrade upon which a loaded truck can operate without an unreasonable reduction in speed. A reduction 15 km/h or more could be considered to be as “unreasonable”. On grades longer than "critical,” consideration of extra lanes should be made (see **Section 7.4**).

For very low levels of traffic of only a few four-wheel drive vehicles, various references advocate a maximum traversable gradient of up to 18 %. Small commercial vehicles can usually negotiate an 18 % gradient, whilst two-wheel drive trucks can successfully manage gradients of 15-16 % except when heavily laden. However, it is pointed out that compaction with a normal 12/14 tonne roller is virtually impossible on a gradient steeper than about 12 %. It is recommended that 12% be considered the absolute maximum gradient that can be applied to any road.

When gradients of 7 % or greater are reached, consideration should be given to paving the steep sections (spot improvements) to enable sufficient traction to be achieved as well as to minimise maintenance requirements. As traffic increases, the economic disbenefits of severe gradients, measured as increased vehicle operating and travel time costs, will justify reducing the severity and/or length of a gradient or paving the steep sections. On the higher design classes of road, the lower maximum recommended gradients reflect these economics. However, an economic assessment of alternatives to long or severe gradients should be undertaken where possible.

Other factors that should be borne in mind in selecting a maximum gradient include:

i. Traffic operations, where high volumes would suggest a reduction in maximum gradient in order to maintain an accept able Level of Service.
ii. Costs, being the whole-life cost of the road and not merely its initial construction cost.
iii. Property, where relatively flat gradients in a rugged environment may result in high fills or deep cuts necessitating the acquisition of land additional to the normal road reserve width.
iv. Environmental considerations.
v. Adjacent land use in heavily developed or urban areas.

The standard gradients that have no 'Critical lengths of grade' can be climbed almost at an average running speed by cars and at half the design speed by trucks.

Trucks going uphill on non-standard gradients will attain the allowable speeds at the end of the 'critical lengths of grade'. However, cars can climb almost at the average running speeds throughout (See **Table 7.17** and **Table 7.18**).

Where design speeds are below 60km/h, calculations give very short uneconomic critical lengths of grade for the different gradients. Critical lengths of grade for such low speeds in **Table 7.19** are recommended based on studies conducted.

Table 7.19 Speed of Trucks at Bottom of Hills and Allowable Speeds at the Limit Lengths

Design speed (km/h)	120	100	80	60
Speed at bottom of hill(km/h)	80	80	80	80
Allowable speed for climbing the limit length(km/h)	60	50	40	30

Some points of design gradient are;

Where the gradient falls within a curve section, check for the combined gradient that is the resultant gradient of the cross slope and the longitudinal gradient. This gradient (G_c) is given by **Equation 7.14**:

$$G_c = \sqrt{e^2 + g^2} \qquad (7.14)$$

Where,

e: Cross slope or superelevation.
g: Longitudinal gradient.

The combined gradient is related to the design speed as recommended in **Table 7.20**. The minimum combined gradient also requires 0.4% for surface drainage.

Table 7.20 Combined Gradient

Design speed (km/h)	Maximum combined gradient (%)
120 or 100	10.0
80 or 60	10.5
50 or 40	11.5

Road sections with steep gradients must have appropriate signs to caution drivers.

On gradient sections where it is forecast that passage of large trucks and other vehicles with low hill-climbing ability will cause the traffic capacity to decline, it is desirable to provide climbing lanes. Refer to **Section 7.4**.

In design of vertical alignment, effort should be made to adopt as gentle a vertical gradient as possible by sufficiently examining the terrain and other conditions while keeping standard values in mind. Vertical gradient sections hinder travel by vehicles at the design speed, so it is not desirable to let such sections stop uniform travel speeds from being maintained. If it is possible in cost and technical terms, it is preferable to spend more money on reducing vertical gradient.

7.3.3 MINIMUM GRADIENTS

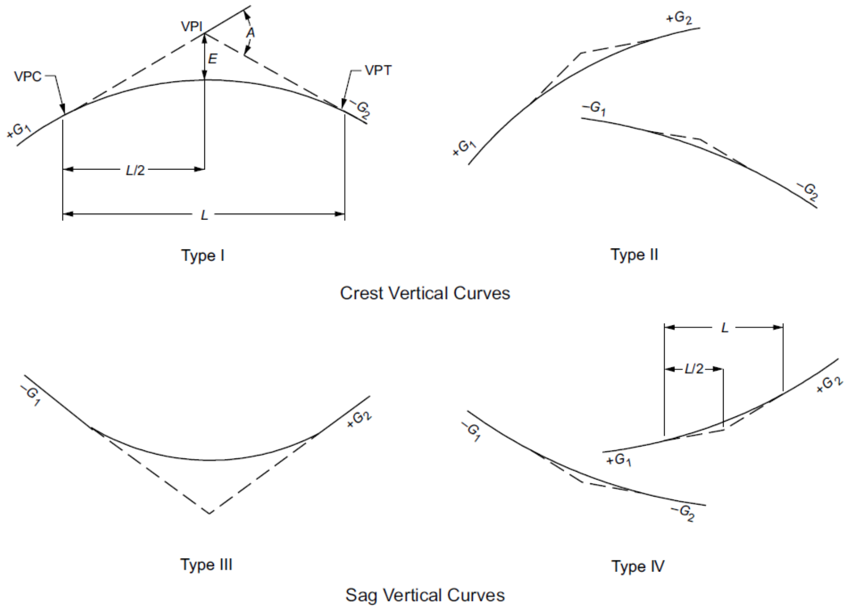
The minimum gradient is a function of road drainage. On rural roads, in cuttings and at the superelevation transitions, gradients should be adequate for side drains, i.e., minimum 0.4%. In fill and outside the superelevation transitions, unkerbed roads could have 0% gradient provided that side drains have positive gradient and that the cross slope and carriageway elevation above the surrounding ground is adequate to drain the surface laterally.

On all kerbed roads, the absolute minimum gradient shall be 0.4%. Grades flatter than 0.4% may be permitted only in exceptional circumstances (i.e., overlay/widening of existing pavement). The designer shall outline procedures that will ensure that the finished gutter profile and pavement surface are free draining and that the width of flow is within the requirements. The designer should also provide satisfactory evidence that drainage provision and the proposed construction practices are such that no ponding shall occur. As a recommendation, for longitudinal gradients below 0.4%, the cross slopes in **Table 6.4** should be increased by 0.5% (i.e., 2.5%, 3.0% 3.5% and 4.5% for cement concrete, asphalt concrete, surface dressing and gravel surfaces respectively).

Unkerbed roads may be provided as the first stage of kerbed roads. To minimise abortive work and maximise utilisation of the most expensive single road component (the pavement), it is a good practice to adopt grades as required for the kerbed roads.

7.3.4 VERTICAL CURVES

Vertical curves to effect gradual changes between tangent grades may be any one of the crest or sag types depicted in **Figure 7.26**.



G1 and G2: Tangent Grades, %

A: Algebraic Difference in Grade, %

L: Length of Vertical Curve, m

E: Vertical Offset at the VPI, m

VPI/VIP: Vertical point of intersection/ Vertical intersection point

VPC/PVC: Vertical point of curvature/ Point of vertical curvature

VPT/PVT: Vertical point of tangency/ Point of vertical tangency

Figure 7.26 Types of Vertical Curves

Vertical curves should be simple in application and should result in a design that enables the driver to see the road ahead, enhances vehicle control, is pleasing in appearance, and is adequate for drainage.

7.3.4.1 VERTICAL CURVE LENGTHS

Vertical curves provide smooth transitions between consecutive different gradients. The two major types of curves in use are the simple parabola and the circular curve. They are considered identical in their mathematical properties.

The respective formulae are provided in **Equation 7.15** and **Equation 7.16**:

$$\text{Simple parabole } y = \frac{G \cdot L}{200}\left(\frac{X}{L}\right)^2 \qquad (7.15)$$

$$\text{Circular curve } R = \frac{100L}{G} \qquad (7.16)$$

Where,

y: Vertical distance from the tangent to the curve (m).

X: Horizontal distance from the start of the vertical curve (m).
G: Algebraic difference in gradient (%).
L: Length of vertical curve (m).
R: The radius of the curve (m).

However, the cubic parabola can be used in special cases.

7.3.4.1.1 Crest curve length

Crest curve considerations are based on stopping sight distance, passing sight distance and driver comfort criteria. Whilst passing sight distance considerations give very long vertical curves, the driver comfort criteria give short vertical lengths especially for high speeds. **Figure 7.27** shows the minimum length of vertical crest curve.

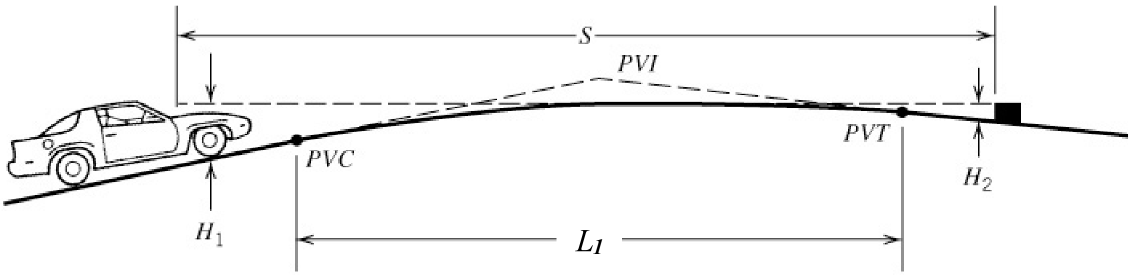


Figure 7.27 Minimum length of vertical crest curve

The rounded K values (Distance required for a 1 % change of gradient, m) for design of crest curves are based on stopping sight distance as shown in **Equation 7.19.** The minimum length of vertical crest curve based on stopping sight distance is given by **Equations 7.17**, **7.18** and **7.20**.

$$L_1 = G\left(\frac{S^2}{200\left(\sqrt{H_1}+\sqrt{H_2}\right)^2}\right) \tag{7.17}$$

$$L_1 = G\left(\frac{S^2}{398}\right) \tag{7.18}$$

$$K = \frac{S^2}{398} \tag{7.19}$$

$$L_1 = GK \tag{7.20}$$

Where,

L_1: The minimum length of crest curve based on stopping sight distance (m).
S: Stopping sight distance required (m) See **Table 7.10.**
G: Algebraic difference in gradients (%).
H_1: Driver's eye height (1.2m).
H_2: Object height (0.1m).

K.: Rounded K value.

The rounded K values based on the comfort criteria is given by **Equation 7.23.** The minimum length of vertical crest curve based on comfort criteria is given by **Equations 7.21**, **7.22** and **7.24**.

$$L_2 = G\left(\frac{V^2}{1300C}\right) \tag{7.21}$$

$$L_2 = G\left(\frac{V^2}{360}\right) \tag{7.22}$$

$$K = \frac{V^2}{360} \tag{7.23}$$

$$L_2 = GK \tag{7.24}$$

Where,

L_2: The minimum length of crest curve based on comfort criteria (m).
G: Algebraic difference in gradients (%).
V: Design speed (km/h).
C: Vertical acceleration (0.028g m/sec^2, g is 9.8 m/sec^2).
K: Rounded K values.

Table 7.21 gives the recommended K values for crest curve. Also refer to **Appendix B** for the recommended lengths of crest curves based on the recommended K values.

Table 7.21 K values for Crest Curve

Design speed (km/h)	120	100	80	60	50	40	30	20
Stopping Sight distance, m (Equation 7.19)	110.8	64.3	30.2	14.1	7.6	4.1	2.2	1
Driver comfort criteria, m/% (Equation 7.23)	40.0	27.8	17.8	10.0	7.0	4.4	2.5	1.1
Recommended K value m/%	111	64	30	14	8	4	2	1

There is a level point on a crest vertical curve, but no difficulty with drainage on highways with kerbs is typically experienced if the curve is sharp enough so that a minimum grade of 0.40 percent is reached at a point about 30m from the crest. This corresponds to K of 75m per percent change in grade, as the drainage maximum. K above this value involve flatter vertical curves. Special attention is needed in these cases to provide proper pavement drainage near the high point of crest vertical curves. It is not intended that K of 75m per percent grade be considered a design maximum, but merely a value beyond which drainage should be more carefully designed.

7.3.4.1.2 Sag curve length

It is assumed that adequate sight distance is available on sag curves during the day. Sag curve considerations are therefore based on headlight distance considerations and driver comfort criteria. **Figure 7.28** shows the minimum length of vertical sag curve.

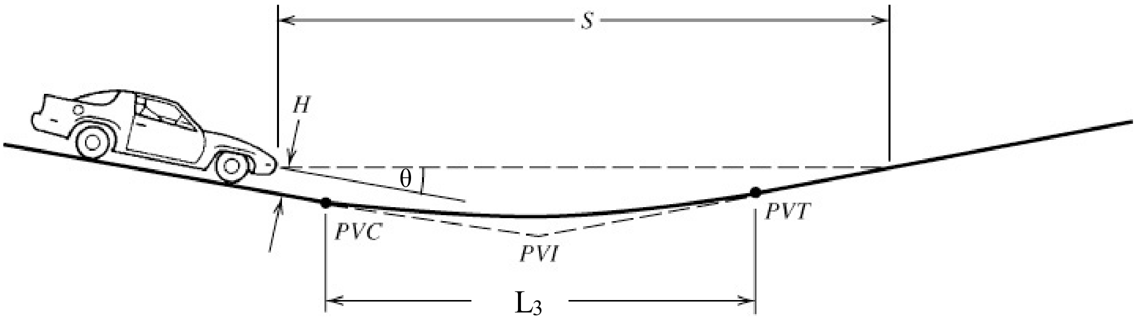


Figure 7.28 Minimum length of vertical sag curve

The rounded K values based on the headlight distance is given by **Equation 7.27.** The minimum length of vertical sag curve based on headlight consideration is given by **Equation 7.25**, **7.26** and **7.28**.

$$L_3 = G\left(\frac{S^2}{200(h_1 + S.\tan\theta)}\right) \quad (7.25)$$

$$L_3 = G\left(\frac{S^2}{120 + 3.49S}\right) \quad (7.26)$$

$$K = \frac{S^2}{120 + 3.49S} \quad (7.27)$$

$$L_3 = GK \quad (7.28)$$

Where,

L_3:	The minimum length of sag curve based on headlight consideration (m).
G:	Algebraic difference in gradients (%).
S:	Stopping sight distance required (m) See **Table 7.10**.
H:	Headlight height (0.6m).
θ:	Angle of upward divergence of light beam (1°).
K:	Rounded K values.

K-values appropriate to headlight distance should be used in rural areas and also where street lighting is not provided. Where street lighting is provided, the lower K-values associated with the comfort criterion may be adopted. **Table 7.22** gives the recommended K values for sag curve.

Also refer to **Appendix B** for the recommended lengths of sag curves based on the recommended K values.

Table 7.22 K Values for Sag Curve

Design speed (km/h)	120	100	80	60	50	40	30	20
Driver comfort criteria (Equation 7.23), m	40.0	27.8	17.8	10.0	7.0	4.4	2.5	1.1
Headlight distance (Equation 7.27), m	51.7	37.7	24.0	14.7	9.7	6.1	4	2.1
Recommended K values (where street lighting is provided), m/%	40	28	18	10	7	5	3	1
Recommended K values (where street lighting is not provided), m/%	52	38	24	15	10	6	4	2

To provide adequate drainage in sag vertical curves in kerbed pavement sections, a minimum slope of 0.4 percent should be maintained within 30 meters of the low point of the curve. This is accomplished where K is less or equal to 75m per percent. The drainage criterion differs from other criteria in that the length of sag vertical curve determined for it is a maximum, whereas the length for any other criterion is a minimum.

7.3.4.2 MINIMUM VERTICAL CURVE LENGTHS.

The overall minimum vertical curve lengths are based on minimum time of 3 seconds required to traverse crest or sag curves smoothly. Minimum lengths are shown in **Table 7.23.**

Table 7.23 Minimum vertical curve length

Design speed (km/h)	120	100	80	60	50	40	30	20
Minimum length (m)	100	85	70	50	40	35	25	20

7.3.4.3 OMISSION OF VERTICAL CURVES.

It has been established that changes in gradients must be connected by vertical curves. The vertical curves are deemed unnecessary where the changes in grades are below 0.5%.

7.3.4.4 VERTICAL CURVE RADII

The radius for circular vertical curve is given by **Equation 7.29**.

$$R = 100K \qquad (7.29)$$

Where,

$K = \frac{L}{G}$ from **Equations 7.24** and **7.28.**

The minimum vertical curve radii recommended in **Table 7.24** are obtained using the K values for crest and sag curves obtained in **Table 7.21** and **Table 7.22** respectively.

Table 7.24 Minimum vertical curve radii

Design speed (km/h)		120	100	80	60	50	40	30	20
Crest curve (m)		11,100	6,400	3,000	1,400	800	400	200	100
Sag curve (m)	Where street lighting is provided	4,000	3,000	1,800	1,000	700	500	300	100
	Where street lighting is not provided	5,200	3,800	2,400	1,500	1,000	600	400	200

7.4 CLIMBING LANE

A climbing lane is an effective means of reducing the impact of a steep gradient. A climbing lane is an auxiliary lane added outside the continuous lanes and has the effect of reducing congestion in the through lanes by removing slower vehicles from the traffic stream. It also enhances road safety by reducing the speed differential in the through lane. The requirements for climbing lanes are therefore based on road standard, speed and traffic volume. Benefits from the provision of a climbing lane accrue because faster vehicles are able to overtake more easily, resulting in shorter average journey times, reduced vehicle-operating costs, and increased safety. Benefits increase with increases in gradient, length of gradient, traffic flow, the proportion of trucks, and reductions in overtaking opportunities. The effect of a climbing lane in breaking up queues of vehicles held up by a slow-moving truck will continue for some distance along the road.

The typical layout of climbing lane is illustrated in **Figure 7.29**. They should not commence, terminate or cover sharp curves or hairpins. Intersections, particularly crossroads and important intersections, should be avoided within road sections with climbing lanes.

Figure 7.29 Typical layout of climbing lane

Opposing traffic should be separated by a solid centreline line, preferably double-sided. Increasing protection may be offered with the use of a wide centreline, longitudinal rumble strips and lane delineators. Alternatively, a median may be provided with physical separation as illustrated in **Plate 7.1**. In this case, the downhill lane will need to have adequate clear width for traffic to bypass a stranded vehicle.

Plate 7.1 Climbing Lane with a median

The following conditions are necessary to set up climbing lanes:

a. Upgrade traffic flow rate in excess of 200 veh/h.

b. The proportion of large vehicle and trailers or commercial vehicles must be high, that is more than 20%.

c. The critical length of grade must be exceeded.

d. One of the following conditions exists:

- A 15km/h or greater speed reduction is expected for a typical heavy truck.
- Level of service E or F exists on the grade.
- A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.

In addition, high crash frequencies may justify the addition of a climbing lane regardless of grade or traffic volumes.

7.4.1 STRUCTURE OF CLIMBING LANE

i. The minimum width of climbing lanes should be 3.5m as shown in **Figure 7.30.**

ii. A run-off of 100m is recommended for the entry and exit tapers.

iii. Set the superelevation of climbing lane according to the values shown in relation to the main road superelevation in **Table 7.25**.

iv. On climbing lanes, either establish no-parking controls or provide parking zones according to necessity.

Table 7.25 Superelevation of Climbing Lane

Superelevation of carriageway (%)	10	9	8	7	6	5	4	3	2
Superelevation of climbing lane (%)	5	5	4	4	4	4	4	3	2

Where restrictions, upgrade approaches, or other conditions indicate the likelihood of low speeds for approaching trucks, the added lane should be introduced near the foot of the grade. The beginning of the added lane should be preceded by a tapered section with a desirable taper length of 100 m long.

The ideal design is to extend a climbing lane to a point beyond the crest, where a typical truck could attain a speed that is within 15 km/h of the speed of the other vehicles with a desirable speed of at 60 km/h approximately at level of service D. Even this may not be practical in many instances because of the unduly long distance required for trucks to accelerate to the desired speed. For such a condition, a practical point to end the added lane is where the truck can return to the normal lane without undue hazard. In particular, this would be feasible where the sight distance becomes sufficient to permit passing with safety when there is no oncoming traffic or, preferably at least 60m beyond this point. In addition, a corresponding length of taper of at least 100 m should be provided to permit the truck to return to the normal lane.

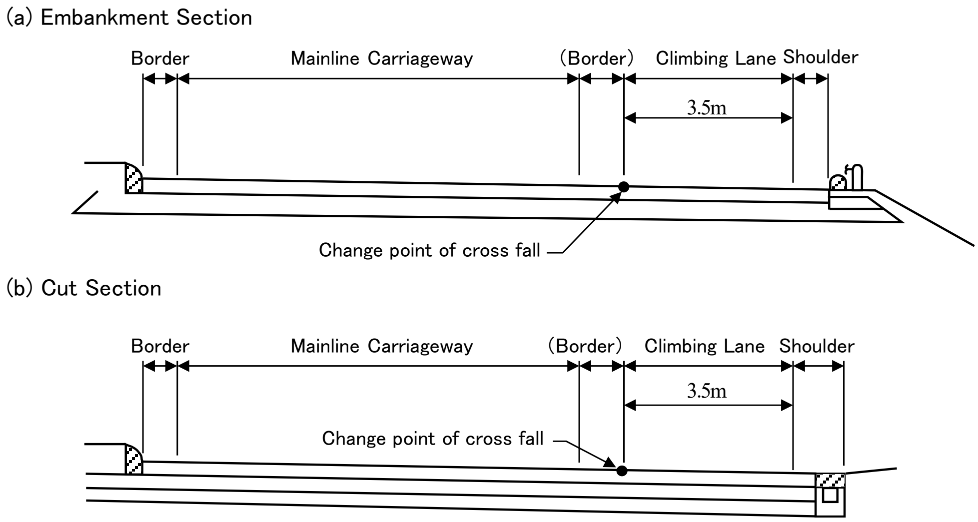


Figure 7.30 Climbing Lane Cross Section

7.5 PASSING LANES

An added lane can be provided in one or both directions of travel to improve traffic operations in sections of lower capacity to at least the same quality of service as adjacent road sections. Passing lanes can also be provided to improve overall traffic operations on two-lane highways by reducing delays caused by inadequate passing opportunities over significant lengths of highways, typically 10 to 100 km. Where passing lanes are used to improve traffic operations over a length of road, they frequently are provided systematically at regular intervals.

The location of the added lane should appear logical to the driver. The value of a passing lane is more obvious at locations where passing sight distance is restricted than on long tangents that may provide passing opportunities even without passing lanes. On the other hand, the location of a passing lane should recognize the need for adequate sight distance at both the lane addition and lane drop tapers. A minimum sight distance of 300 m on the approach to each taper is recommended. The selection of an appropriate location also needs to consider the location of intersections and high-volume accesses in order to minimize the volume of turning movements on a road section where passing is encouraged. Furthermore, other physical constraints such as bridges and culverts should be avoided if they restrict provision of a continuous shoulder.

The following is a summary of the design procedure to be followed in providing passing sections on two-lane highways:

a) Horizontal and vertical alignment should be designed to provide as much of the roadway as practical with passing sight distance (see **Table 7.12**).
b) Where the design volume approaches capacity, the effect of lack of passing opportunities in reducing the level of service should be recognized.
c) Where the critical length of grade is less than the physical length of an upgrade, consideration should be given to constructing added climbing lanes (Refer to **Section 7.4**).
d) Where the extent and frequency of passing opportunities made available by application of Criteria (a) and (c) are still too few, consideration should be given to the construction of passing-lane sections.

Passing-lane sections, which may be either three or four lanes in width, are constructed on two-lane roads to provide the desired frequency of passing zones or to eliminate interference from low-speed heavy vehicles, or both. Where a sufficient number and length of passing sections cannot be obtained in the design of horizontal and vertical alignment alone, an occasional added lane in one or both directions of travel may be introduced as shown in **Figure 7.31** to provide more passing opportunities. Such sections are particularly advantageous in rolling terrain, especially where alignment is winding or the profile includes critical lengths of grade.

A minimum length of 300 m, excluding tapers, is needed so that delayed vehicles have an opportunity to complete at least one pass in the added lane. Where such a lane is provided to reduce delays at a specific bottleneck, the needed length is controlled by the extent of the bottleneck.

A lane added to improve overall traffic operations should be long enough, over 0.5 km, to provide a substantial reduction in traffic platooning. The optimal length is usually 0.8 to 3.2 km, with longer lengths of added lane appropriate where traffic volumes are higher. Operational benefits typically result in reduced platooning 5 to 15 km downstream depending on volumes and passing opportunities. After that, normal levels of platooning will occur until the next added lane is encountered.

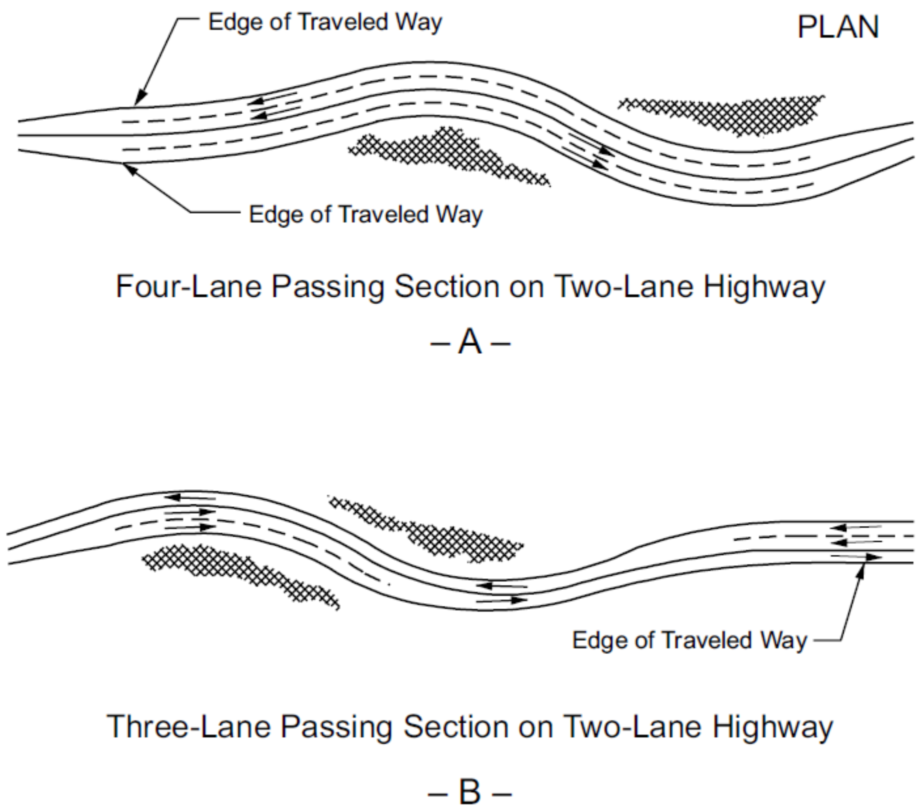


Figure 7.31 Passing Lane sections on two-lane roads

The introduction of a passing-lane section on a two-lane highway does not necessarily involve much additional grading. The width of an added lane should normally be the same as the lane widths of the two-lane highway. It is also desirable for the adjoining shoulder to be at least 1.2 m wide and, whenever practical, the shoulder width in the added section should match that of the adjoining two-lane highway. However, a full shoulder width is not as needed on a passing lane section as on a conventional two-lane highway because the vehicles likely to stop are few and there is little difficulty in passing a vehicle with only two wheels on the shoulder.

Four-lane sections introduced explicitly to improve passing opportunities need not be divided because there is no separation of opposing traffic on the two-lane portions of the highway. The use of a median, however, is beneficial and should be considered on highways carrying a total of 500 veh/h or more, particularly on highways to be ultimately converted to a four-lane divided cross section.

The transition tapers at each end of the added-lane section should be designed to encourage efficient operation and reduce crashes. The lane-drop taper length where the posted or statutory speed limit is 70 km/h or greater should be computed with **Equation 7.30**. Where the posted or statutory speed limit is less than 70 km/h, the lane-drop taper length should be computed with **Equation 7.31**. The recommended length for the lane addition taper is one-half to two-thirds of the lane-drop length.

$$L = 0.62\mathrm{WS} \tag{7.30}$$

$$L = \frac{WS^2}{155} \qquad (7.31)$$

Where;

L = Length of taper, m

W = Width, m

S = Speed, km/h

The transitions between the two- and three- or four-lane pavements should be located where the change in width is in full view of the driver. Sections of four-lane highway, particularly divided sections, longer than about 3 km may cause the driver to lose his sense of awareness that the highway is basically a two-lane facility. It is essential, therefore, that transitions from a three- or four-lane cross section back to two lanes be properly marked and identified with pavement markings and signs to alert the driver of the upcoming section of two-lane highway. An advance sign before the end of the passing lane is particularly important to inform drivers of the narrower roadway ahead.

A passing lane should be sufficiently long for a following vehicle to complete at least one passing manoeuvre. Short passing lanes, with lengths of 0.4 km or less are not very effective in reducing traffic platooning. As the length of a passing lane increases above 1.6 km, a passing lane generally provides diminishing operational benefits, and is generally appropriate only on higher volume facilities with flow rates over 700 veh/h. **Table 7.26** presents optimal design lengths for passing lanes.

Table 7.26 Optimal Passing Lane Lengths for Traffic Operational Efficiency

One-Way Flow Rate (veh/h)	Passing Lane Length (km)
100 - 200	0.8
201 - 400	0.8 - 1.2
401 - 700	1.2 - 1.6
701 - 200	1.6 - 3.2

7.6 "2+1" ROADWAYS

The "2+1" roadway concept has been found to improve operational efficiency and reduce crashes for selected two-lane highways. **Figure 7.32** is a schematic of the concept. The concept provides a continuous three-lane cross section, and the highway is striped in a manner as to provide for passing lanes in alternating directions throughout the section. This concept can be an attractive alternate to two- or four-lane roads for some highways with higher traffic volumes where continuously alternating passing lanes are needed to obtain the desired level of service.

"2+1" roads generally have speed limit in the range of 70 to 100km/h. They are not suitable for built-up areas and their peripheries as well as roads with a constrained alignment.

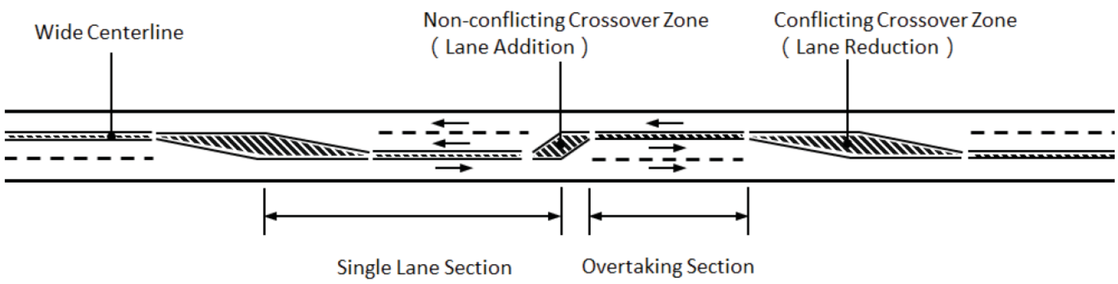


Figure 7.32 Schematic for 2+1 Roadway

The "2+1" configuration may be a suitable treatment for roadways with traffic volumes higher than can be served by isolated passing lanes, but not high enough to justify a four-lane roadway. The configuration is also potentially applicable for use at locations where environmental or fiscal constraints, or both, make provision of a four-lane facility impractical. A "2+1" road will generally operate at least two levels of service higher than a conventional two-lane highway serving the same traffic volume.

A "2+1" road should not generally be considered where current or projected flow rates exceed 1,200 veh/h in one direction of travel. A four-lane roadway generally is more efficient at such high flow rates. This concept can be used over a broad range of traffic composition to provide passing opportunities as the percentage of heavy vehicles increases.

A "2+1" road should only be used in level or rolling terrain. In mountainous terrain and on isolated steep grades, it is normally more appropriate to introduce climbing lanes on upgrades as discussed in Section 7.8. Stopping sight distance should be provided continuously along a "2+1" roadway. Decision sight distance should be considered at intersections and lane drops.

The transition tapers at each end of the added-lane section should be designed to encourage efficient operation and reduce crashes. The lane-drop taper length where the posted or statutory speed limit is 70 km/h or greater should be computed with **Equation 7.30**. Where the posted or statutory speed limit is less than 70 km/h, the lane-drop taper length should be computed from **Equation 7.31**. The recommended length for the lane addition taper is one-half to two-thirds of the lane-drop length. **Figure 7.33** and **Figure 7.34** are schematics for adjacent lane drop and lane addition tapers on a "2+1" roadway.

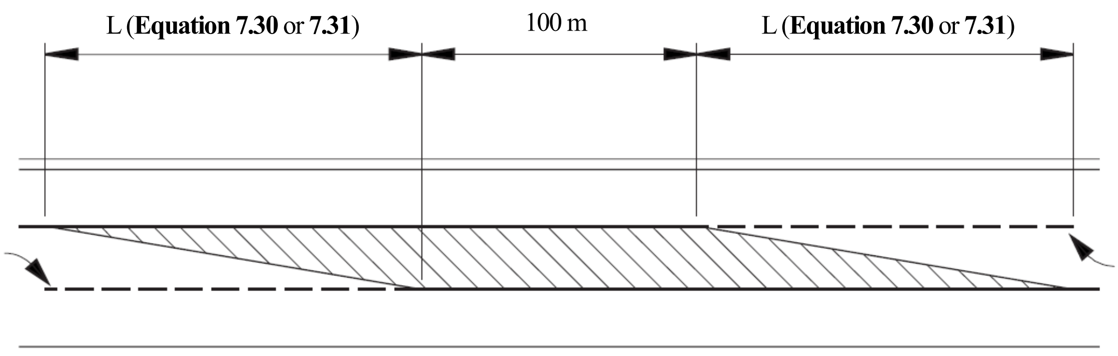


Figure 7.33 Schematic for Adjacent Lane Drop Tapers on a 2+1 Roadway

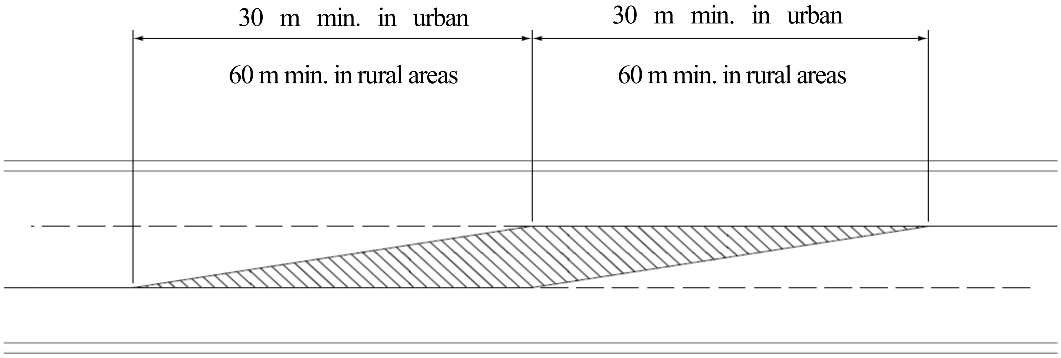


Figure 7.34 Schematic for Adjacent Lane Addition Tapers on a 2+1 Roadway

Lane and shoulder widths should be comparable to the widths determined for the volumes and speeds for two-lane roadways for the specific design class. The nominal width of traffic lanes is 3.5m, but may be reduced to 3.25m for the overtaking lanes or both lanes of the two-lane section. Excessive lane widths could lead to excessive speeds.

It is generally necessary to exclude pedestrians or slow vehicles travelling along "2+1" overtaking sections or "2+1" roads operating at high traffic speeds. Segregated parallel facilities may be considered on each side of the road.

Furthermore, at-grade intersections and crossings for pedestrians or slow vehicles should not be provided within the three-lane section. Such intersections or crossings may be provided at nonconflicting crossovers or by first changing the road back to a two-lane road. This is illustrated in **Figure 7.35**.

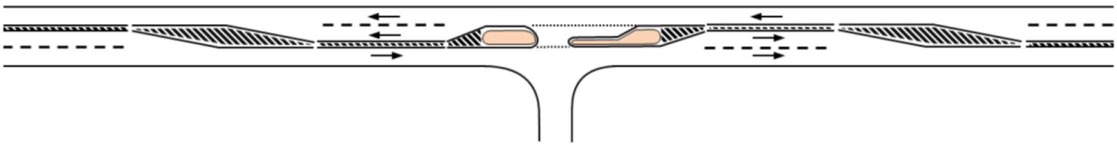


Figure 7.35 Provision of intersections on "2+1" roads

Where existing two-lane roadways with a normal crown are converted to "2+1" roadways, the location and transition of the crown is perhaps one of the more complicated design issues. A variety of practices relate to the location of the crown. Where an existing two-lane highway is restriped as a "2+1" road or widened to become a "2+1" road, the placement of the crown within the travelled way may be permitted. An existing highway may also be widened on one side only, with the result that the crown is located at a lane line. There is no indication of any difference in crashes between placing the roadway crown at a lane boundary and placing it within a lane. For newly designed 2+1 highways, the crown should be placed at a lane boundary. Superelevation should be handled no differently on a "2+1" road than on a comparable two-lane or four-lane undivided road.

While separation of the opposing traffic lanes may not be needed on every roadway, to further enhance the safety of "2+1" roads, it is desirable to provide a flush separation of 1.2 m between the opposing directions or a physical safety barrier over the centreline to separate opposing traffic. A range of safety barriers could be considered including rigid concrete barriers or wire-rope safety barriers. This is illustrated in **Plate 7.2**. If safety barriers are provided to separate opposing traffic, adequate shoulders are required on the single lane section to permit traffic passing a broken-down vehicle. Laybys should also be provided at regular intervals.

Plate 7.2 "2+1" Road with median safety barrier

7.7 PHASING OF HORIZONTAL AND VERTICAL ALIGNMENT

7.7.1 GENERAL CONSIDERATIONS

Generally, alignment design follows the undermentioned sequence.

a. Road location.
b. Design of horizontal alignment.
c. Design of vertical alignment.
d. Phasing of the horizontal and vertical alignment.

Horizontal and vertical alignment are permanent design elements for which thorough study is warranted. It is extremely difficult and costly to correct alignment deficiencies after a highway is constructed. Thus, compromises in the alignment designs should be weighed carefully because any initial savings may be more than offset by the economic loss to the public in the form of crashes and delays.

Horizontal and vertical alignment should not be designed independently. They complement each other, and poorly designed combinations can spoil the good points and aggravate the deficiencies of each. Horizontal alignment and profile are among the more important of the permanent design elements of the highway. Excellence in the design of each and of their combination enhances vehicle control, encourages uniform speed, and improves appearance, nearly always without additional cost.

Phasing of the horizontal and vertical alignment of a road implies their coordination so that the line of the road appears to a driver to flow smoothly, avoiding the creation of hazards and visual defects. It is particularly important in the design of high-speed roads on which a driver must be able to anticipate changes in both horizontal and vertical alignment well within his safe stopping distance. It becomes more important with small radius curves than with large.

Defects may arise if an alignment is mis-phased. Defects may be purely visual and do no more than present the driver with an aesthetically displeasing impression of the road. Such defects often occur on sag curves. When these defects are severe, they may create a psychological obstacle and cause some drivers to reduce speed unnecessarily. In other cases, the defects may endanger the safety of the user by concealing hazards on the road ahead. A sharp bend hidden by a crest curve is an example of this kind of defect.

7.7.2 TYPES OF MIS-PHASING AND CORRESPONDING CORRECTIVE ACTION

When the horizontal and vertical curves are adequately separated or when they are coincident, no phasing problem occurs, and no corrective action is required. Where defects occur, phasing may be achieved either by separating the curves or by adjusting their lengths so that vertical and horizontal curves begin at a common chainage and end at a common chainage. In some cases, depending on the curvature, it is sufficient if only one end of each of the curves is at a common chainage.

Cases of mis-phasing fall into four types. These are described below together with the necessary corrective action for each type.

7.7.2.1 VERTICAL CURVE OVERLAPS ONE END OF THE HORIZONTAL CURVE

If a vertical curve overlaps either the beginning or the end of a horizontal curve, a driver's perception of the change of direction at the start of the horizontal curve may be delayed because his sight distance is reduced by the vertical curve. This defect is hazardous. The position of the crest is important because the vehicles tend to increase speed on the down gradient following the highest point of the crest curve, and the danger due to an unexpected change of direction is consequently greater. If a vertical sag curve overlaps a horizontal curve, an apparent kink may be produced, as indicated in **Figure 7.36b** and **Figure 7.36c**.

The defect may be corrected in both cases by completely separating the curves. If this is uneconomic, the curves must be adjusted so that they are coincident at both ends, if the horizontal curve is of short radius, or they need be coincident at only one end, if the horizontal curve is of longer radius.

7.7.2.2 INSUFFICIENT SEPARATION BETWEEN THE CURVES

If there is insufficient separation between the ends of the horizontal and vertical curves, a false reverse curve may appear on the outside edge-line at the beginning of the horizontal curve. This is a visual defect, illustrated in **Figure 7.36**.

Corrective action consists of increasing the separation between the curves, or making the curves concurrent, as in **Figure 7.36a**.

7.7.2.3 BOTH ENDS OF THE VERTICAL CURVE LIE ON THE HORIZONTAL CURVE

If both ends of a crest curve lie on a sharp horizontal curve, the radius of the horizontal curve may appear to the driver to decrease abruptly over the length of the crest curve. If the vertical curve is a sag curve, the radius of the horizontal curve may appear to increase. An example of such a visual defect is shown in **Figure 7.36e**. The corrective action is to make both ends of the curves coincident as in **Figure 7.36a**, or to separate them.

7.7.2.4 VERTICAL CURVE OVERLAPS BOTH ENDS OF THE HORIZONTAL CURVE

If a vertical crest curve overlaps both ends of a sharp horizontal curve, a hazard may be created because a vehicle has to undergo a sudden change of direction during the passage of the vertical curve while sight distance is reduced.

The corrective action is to make both ends of the curves coincident. If the horizontal curve is less sharp, a hazard may still be created if the crest occurs off the horizontal curve. This is because the change of direction at the beginning of the horizontal curve will then occur on a downgrade (for traffic in one direction) where vehicles may be increasing speed.

The corrective action is to make the curves coincident at one end so as to bring the crest on to the horizontal curve. No action is necessary if a vertical curve that has no crest is combined with a gentle horizontal curve. If the vertical curve is a sag curve, an illusory crest or dip, depending on the “hand” of the horizontal curve will appear in the road alignment. The corrective action is to make both ends of the curves coincident or to separate them.

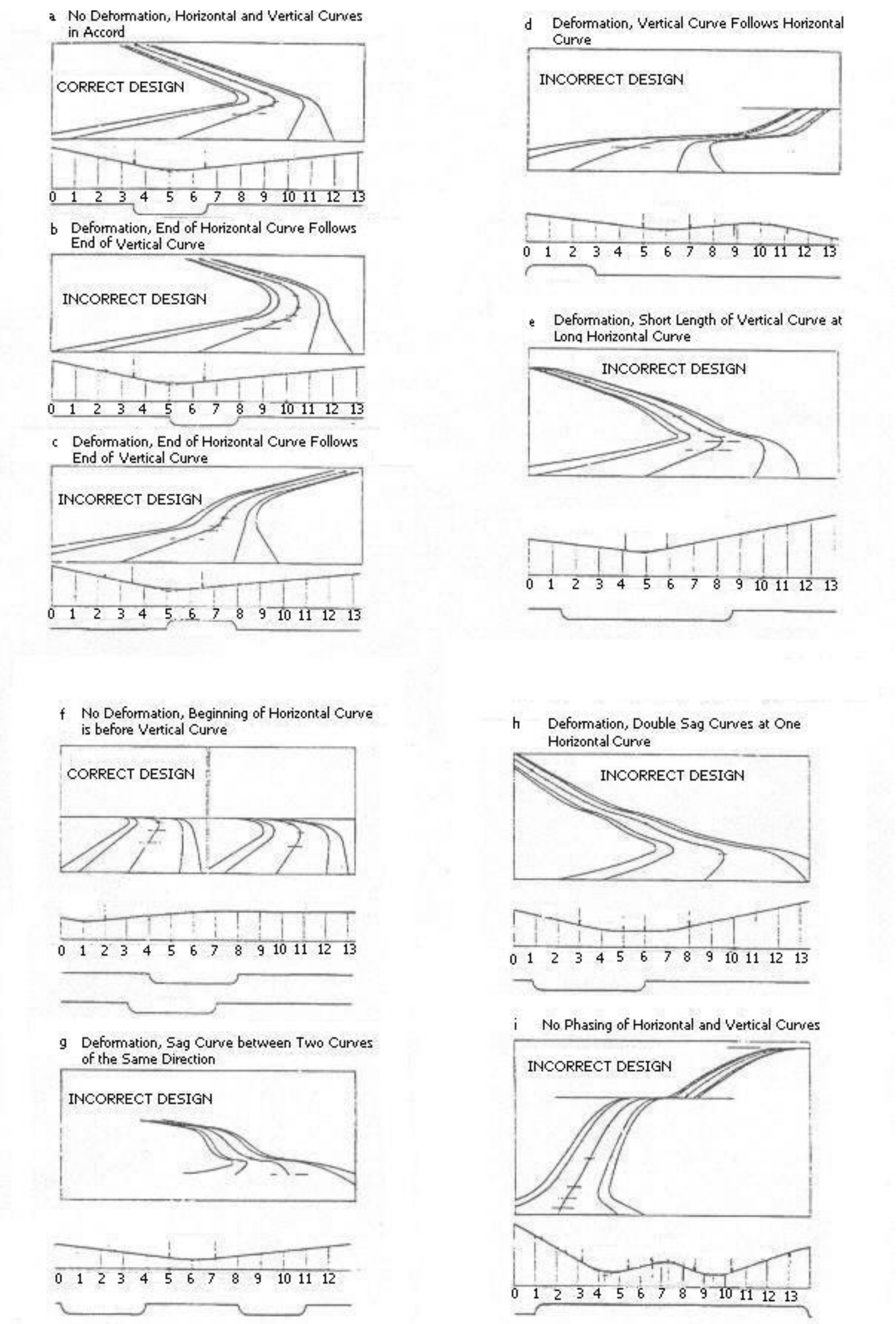


Figure 7.36 Phasing of Horizontal and Vertical Curves

7.7.2.5 OTHER MIS-PHASING

Other types of mis-phasing are also indicated in **Figure 7.36g** to **Figure 7.36i**:

i. A sag curve occurs between two horizontal curves in the same direction in **Figure 7.36g**. This illustrates the need to avoid broken back curves in design.
ii. A double sag curve occurs at one horizontal curve in **Figure 7.36h**. This illustrates the effect in this case of a broken back vertical alignment on design.
iii. **Figure 7.36i** shows a lack of phasing of horizontal and vertical curves. In this case, the vertical alignment has been allowed to be more curvilinear than the horizontal alignment.

7.7.3 BALANCING HORIZONTAL AND VERTICAL ALIGNMENTS

The following general controls should be considered in balancing horizontal and vertical alignments:

i. Curvature and grades should be in proper balance. Tangent alignment or flat curvature at the expense of steep or long grades and excessive curvature with flat grades both represent poor design. A logical design that offers the best combination of safety, capacity, ease and uniformity of operation, and pleasing appearance within the practical limits of terrain and area traversed is a compromise between these two extremes.
ii. Vertical curvature superimposed on horizontal curvature, or vice versa, generally results in a more pleasing facility, but such combinations should be analysed for their effect on traffic. Successive changes in profile not in combination with horizontal curvature may result in a series of humps visible to the driver for some distance which represents an undesirable condition.
iii. Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This condition is undesirable because the driver may not perceive the horizontal change in alignment, especially at night. The disadvantages of this arrangement are avoided if the horizontal curvature leads the vertical curvature (i.e., the horizontal curve is made longer than the vertical curve). Suitable designs can also be developed by using design values well above the appropriate minimum values for the design speed.
iv. Somewhat related to the preceding guideline, sharp horizontal curvature should not be introduced near the bottom of a steep grade approaching or near the low point of a pronounced sag vertical curve. Because the view of the road ahead is foreshortened, any horizontal curvature other than a very flat curve assumes an undesirable distorted appearance. Further, vehicle speeds, particularly for trucks, are often high at the bottom of grades, and erratic operations may result, especially at night.
v. Curvature in the horizontal plane should be accompanied by comparable length of curvature in the vertical plane.
vi. Awkward combinations of curves and tangents in both the horizontal and vertical planes should be avoided (i.e., "broken back" curves).
vii. Horizontal and vertical curvatures should be coordinated to avoid combinations that appear

awkward when viewed from a low angle.

viii. Ideally the vertices of horizontal curves (PI) and vertical curves (PVI) should coincide or be within 1/4 phase of each other.

ix. Horizontal curvature should lead vertical curvature. i.e., the horizontal curve should be longer than the vertical curve and the PVT and PC should not be at the same point.

x. The alignment designs should enhance attractive scenic views of the natural and manmade environment, such as rivers, rock formations, parks, and outstanding man-made structures.

xi. In residential areas, the alignment design should minimize nuisance factors to the neighbourhood. Generally, a depressed facility makes a highway less visible and less noisy to adjacent residents. Minor horizontal adjustments can sometimes be made to increase the buffer zone between the highway and clusters of homes.

xii. Horizontal curvature and profile should be as flat as feasible at intersections where sight distance along both roads is important and vehicles may have to slow or stop.

xiii. On divided highways, consideration of variation in the width of the median and the use of independent alignments is needed to derive the design and operational advantages of one-way roadways.

xiv. On two-lane roads, the need for safe passing sections (at frequent intervals and for an appreciable percentage of the length of the roadway) often supersedes the general desirability for combination of horizontal and vertical alignment. Passing zones with long tangent sections are needed to secure sufficient passing sight distance.

xv. Avoidance of a sharp horizontal curve at or near the low point of a pronounced sag vertical curve is important. The road ahead is foreshortened and any horizontal curve that is not flat assumes an undesirably distorted appearance. Further, vehicular speeds, particularly of trucks, often are high at the bottom of grades and erratic operation may result, especially at night.

xvi. To maintain drainage, vertical and horizontal curves should be designed so that the flat profile of a vertical curve will not be located near the flat cross slope of the superelevation transition.

7.7.4 THE ECONOMIC PENALTY DUE TO PHASING

The phasing of vertical curves restricts their movement and fitting to the ground so that the designer is prevented from obtaining the lowest cost design. Therefore, phasing is usually bought at the cost of extra earthworks and the designer must decide at what point it becomes uneconomic. He will normally accept curves that have to be phased for reasons of safety. In cases when the advantage due to phasing is aesthetic, the designer will have to balance the costs of trail alignments against their elegance.

Test results in Germany have indicated that a balance between horizontal and vertical curve radii reduces the effect of mis-phasing. **Table 7.27** shows the recommended values of the balance. In practice, however, actual phasing of the curves may be limited to high-speed roads because it is generally uneconomic.

Table 7.27 Radius of horizontal/vertical curve

Radius of horizontal curve (m)	500	700	800	900	1,000	1,100
Radius of vertical curve(m)	10,000	12,000	16,000	20,000	25,000	30,000

7.8 DESIGN STANDARDS AND GEOMETRIC CONDITIONS

Table 7.28 shows design speeds according to each road design class, and the design standards and geometric conditions corresponding to each design speed.

Table 7.28 Design Standards and Geometric Conditions

Design Class			**A**	**B**	**C**	**D**	**E**
General							
Design speed	**Flatland (figures in brackets indicate absolute values for urban roads)**	**km/h**	120 (100)	100 (80)	80 (60)	60 (40)	40 (20)
	Hilly land (figures in brackets indicate absolute values for urban roads)		100 (80)	80 (60)	60 (40)	50 (30)	40 (20)
	Mountainous area (figures in brackets indicate absolute values for urban roads)		80 (60)	60 (40)	50 (30)	40 (20)	40 (20)
Geometric Structure							
Horizontal alignment			**Design speed (km/h)**				
			120	**100**	**80**	**60**	**40**
Minimum horizontal curve	**Desirable (5% superelevation)**	**m**	1030	700	420	220	100
	Minimum (9% superelevation)		540	370	230	130	50
Maximum superelevation		**%**	Urban areas: 5% is desirable Rural areas: 9% is desirable				
Minimum curve length		**m**	200	170	140	100	70
Minimum transition curve length			67	56	44	33	22
Radius above which transition curve is unnecessary			1310	910	580	330	150
Superelevation and curve radius		**9%**	637	449	286	161	72
		8%	739	509	327	185	82
		7%	849	587	370	212	92
		6%	996	694	441	249	174
		5%	1,206	849	540	302	212
		4%	1,527	1,091	674	395	273
		3%	1,910	1,348	880	498	347
		2%	3,510	2,560	1,710	1,030	525

Design Class					A	B	C	D	E
Minimum curve radius without superelevation				**m**	7,600	5,300	3,500	2,000	850
Superelevation runoff length					1/263	1/244	1/200	1/167	1/143
Increase and decrease of lane runoff				**Urban**	1/70	1/60	1/50	1/40	1/25
				Rural	-	-	1/40	1/30	1/20
Vertical alignment									
Maximum vertical gradient	**Standard value**			**%**	2	3	4	5	7
	Absolute Grade (Critical grade length)			**% (m)**	3 (800)	4 (700)	5 (600)	6 (500)	8 (400)
					4 (500)	5 (500)	6 (500)	7 (400)	9 (300)
					5 (400)	6 (400)	7 (400)	8 (300)	10 (200)
Sight distance	**Stopping**	**Flat**		**m**	210	160	110	75	40
		Upgrade	**3%**		199	145	101	69	37
			6%		188	138	96	67	37
			9%		179	132	93	65	36
		Downgrade	**3%**		228	164	112	76	39
			6%		249	178	120	80	41
			9%		276	194	130	85	43
	Passing				780	620	500	360	210
	Decision	**Interchange Right exit**			400	350	250	200	NA
		Interchange Left exit			500	430	340	275	NA
		Lane drop			400	350	250	200	150
		Lane shift			250	200	150	100	85
		Intersections			400	350	250	200	150
Vertical Crest Curve	**K-value**				111	64	30	14	4
	Radius			**m**	11,100	6,400	3,000	1,400	400
Vertical Sag Curve	**K-value (where street lighting is provided), m/%**				40	28	18	10	5
	K-value (where street lighting is not provided), m/%				52	38	24	15	6
	Radius (Where Street lighting is provided)			**m**	4,000	3,000	1,800	1,000	500
	Radius (Where Street lighting is not provided)				5,200	3,800	2,400	1,500	600
Minimum vertical curve length					100	85	70	50	35
Note: Concerning standard values that are not specified in the Ghana Road Design Guide, adopt the AASHTO recommended values.									

7.9 REFERENCES

1. Ghana Highway Authority Road Design Guide, 1991.
2. Japanese Road Structure Ordinance, April 2021.
3. Explanation and Operation of the Road Structure Ordinance
4. Geometric Design Manual of Uganda, (2005).
5. A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.
6. Asian Highway Design Standard for Road Safety, 2017.
7. Austroads. 2015. Guide to Road Design Part 1, AGRD01-15, Austroads, Sydney, Australia. Austroads. 2019.
8. South African Geometric Design Guidelines (2003).
9. Geometric Design Manual, Federal Democratic Republic of Ethiopia, Ethiopian Roads Authority, 2013.

Volume III

Chapter 8 At-grade Intersections

Chapter 9 Grade Separated Intersections

GHANA ROAD DESIGN GUIDE 2023

CHAPTER 8

AT-GRADE INTERSECTIONS

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CHAPTER 8 AT-GRADE INTERSECTIONS

8.1 INTRODUCTION

An intersection is defined as the general area where two or more roads join or cross, including the roadway and roadside facilities for traffic movements within the area.

The intersection includes the areas needed for all modes of travel: pedestrian, bicycle, motor vehicle, and transit. Thus, the intersection includes not only the pavement area, but typically the adjacent walkways and pedestrian kerb cut ramps. The intersection encompasses all alterations (for example, turning lanes) to the otherwise typical cross-sections of the intersecting roads.

Each road radiating from an intersection and forming part of it is an intersection leg. The most common intersection at which two roads cross one another has four legs. It is recommended that an intersection have no more than four legs. **Figure 8.1** shows the basic elements of an at-grade intersection.

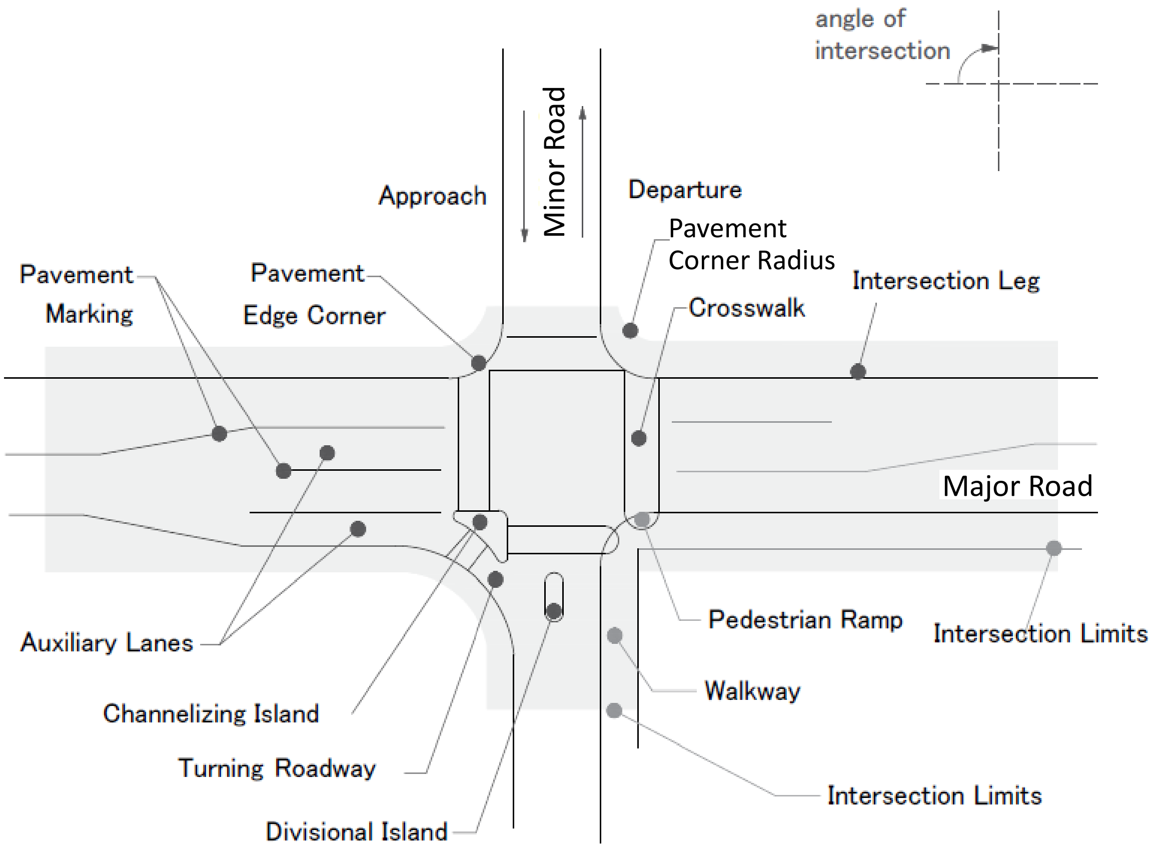


Figure 8.1 At-grade intersection elements

Study shows that a large proportion of road crashes occur at intersections. Most of the junction-accidents occur at the very lightly trafficked at-grade intersections and from a traffic-safety aspect these lightly trafficked intersections require as much attention as do those intersections where heavier conflicting traffic movements occur. Good intersection design should allow transition from one route to another or through movement on the main route and intersecting route with minimum delay and maximum safety. The need for careful design of intersections to limit inherent hazard and maintain acceptable traffic capacity is of great importance. To accomplish this, the layout and operation of the intersections should be obvious to the driver, with good visibility between conflicting movements. Furthermore, the number of intersections should be kept as low as possible consistent with traffic demands and their spacing should be as great as possible.

Intersections should be located where they can be clearly seen and easily understood by drivers on all the approach roads and where the provision of desirable, as opposed to minimum, safety standards are possible and economical to obtain. Crests, gradients and curves should be avoided. T-intersections on the outside of a curve will have much better visibility than those that are located on the inside of a curve.

Intersections should not be located where it is difficult or expensive to provide adequate visibility or driving comfort. Locations which should be avoided are for example where earthworks are heavy, near bridges, on small radius curves, on the outside of superelevated curves, on high embankments, steep grades (>3%) or on crests. Careful location as well as landscaping of the surrounding terrain and planting can be used to improve the visual guidance and visibility at intersections. This can be of great importance for the perception and comprehension of the intersection.

8.1.1 ACCESS

An access is the intersection of an unclassified road with a classified road. It is generally provided within the road reserve boundary of the classified road. Access roads (driveways) are used to connect properties etc. to the road network. Accident risk increases with the frequency of access roads, so they should, as far as possible, be discouraged on higher classes of roads. However, in certain locations, the constant daily vehicular movement or heavy peak hour flows at an access may justify its design to at-grade intersection standards. This may occur, for example, at an entrance to an industrial development or factory site.

Access points should not be located where earthworks are heavy, near bridges, on small radius curves, on the outside of superelevated curves or on steep grades where the provision of desirable, as opposed to minimum, safety standards are either expensive or impossible to obtain.

An access shall have entry and exit radii of between 6 and 15 meters depending upon the turning characteristics of the expected traffic with no left or right turning lanes, left turn merging lane or traffic islands. The minimum width shall be 4m. The layout and location of the access must satisfy the visibility requirement for "stop” conditions given in **Figure 8.2**. A drainage channel shall be placed as required. The approach to the main road along the access road should be level with the surface of the main road for the last 5-10 meters.

Sufficient storage length must be provided for a vehicle to stand clear of the carriageway when stopped. Where the entrance has a gate, the set back from the edge of the carriageway to the gate will vary with the type of vehicle likely to use the access.

Vertical alignment elements are also important in access design and should allow vehicles to be operated efficiently as they enter or exit the access. Profiles should be designed to minimize the possibility of a vehicle dragging or hanging up on the access. Where an access crosses a walkway/shared path/cycle path, the access profile shall match the path to emphasise that the path has the right of way. In addition, profiles should allow for adequate drainage and they should minimize the potential for ponding of water at the interface between the access and the walkway, as well as between the access and the intersecting roadway. The recommended maximum gradient is 15% for small cars and medium vehicles, and 8% for large vehicles and trailers. If the proposed access has a grade greater than the recommended grades, then consideration should be given to relocating the access to achieve a lower gradient.

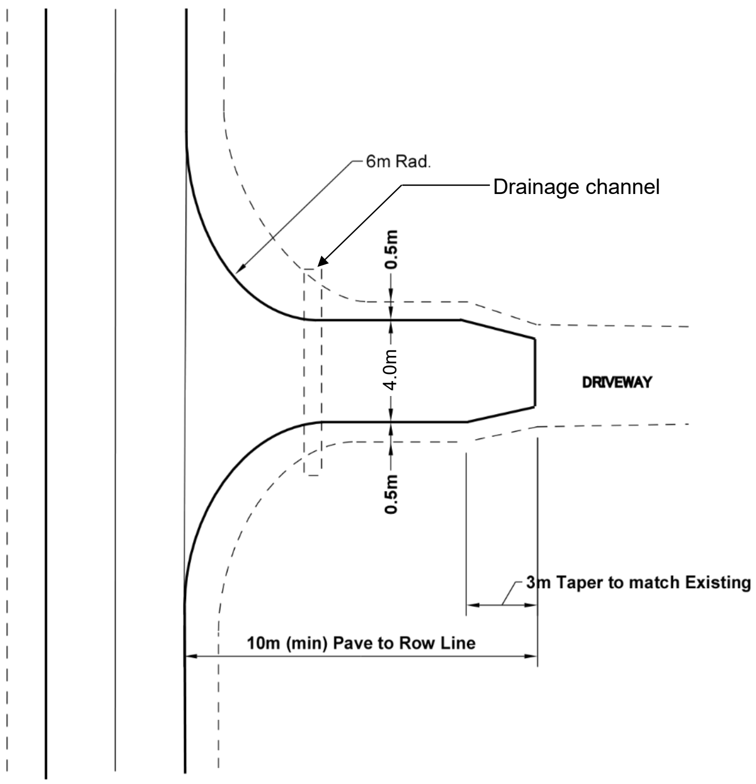


Figure 8.2 Typical Access

An access shall be located:

i. to minimise the impact on road safety and efficiency to the through movement of traffic, pedestrians and cyclists

ii. to minimise damage to road verge vegetation and

iii. as far as practical from intersections to minimise points of conflict and confusion.

Accesses near roundabouts shall be located as far as practical from the roundabout and shall not be located:

i. Within the functional area of the roundabout

ii. Within the roundabout

Accesses shall not be located:

i. Opposite the terminating road of a T-junction or

ii. Opposite median openings for legal U-turn purposes or

iii. In or opposite auxiliary lanes or

iv. Within traffic lane diverge or merge zones associated with auxiliary lanes, acceleration lanes or lane drops associated with traffic signals or roundabouts.

As a general rule, median openings solely to provide access to private property shall not be permitted.

8.2 INTERSECTION TYPES AND CONFIGURATION

8.2.1 TYPE OF INTERSECTIONS

Different at-grade intersection types will be appropriate under different circumstances depending on traffic flows, speeds, and site limitations. Intersections can be categorized into four major types, as illustrated in **Figure 8.3**.

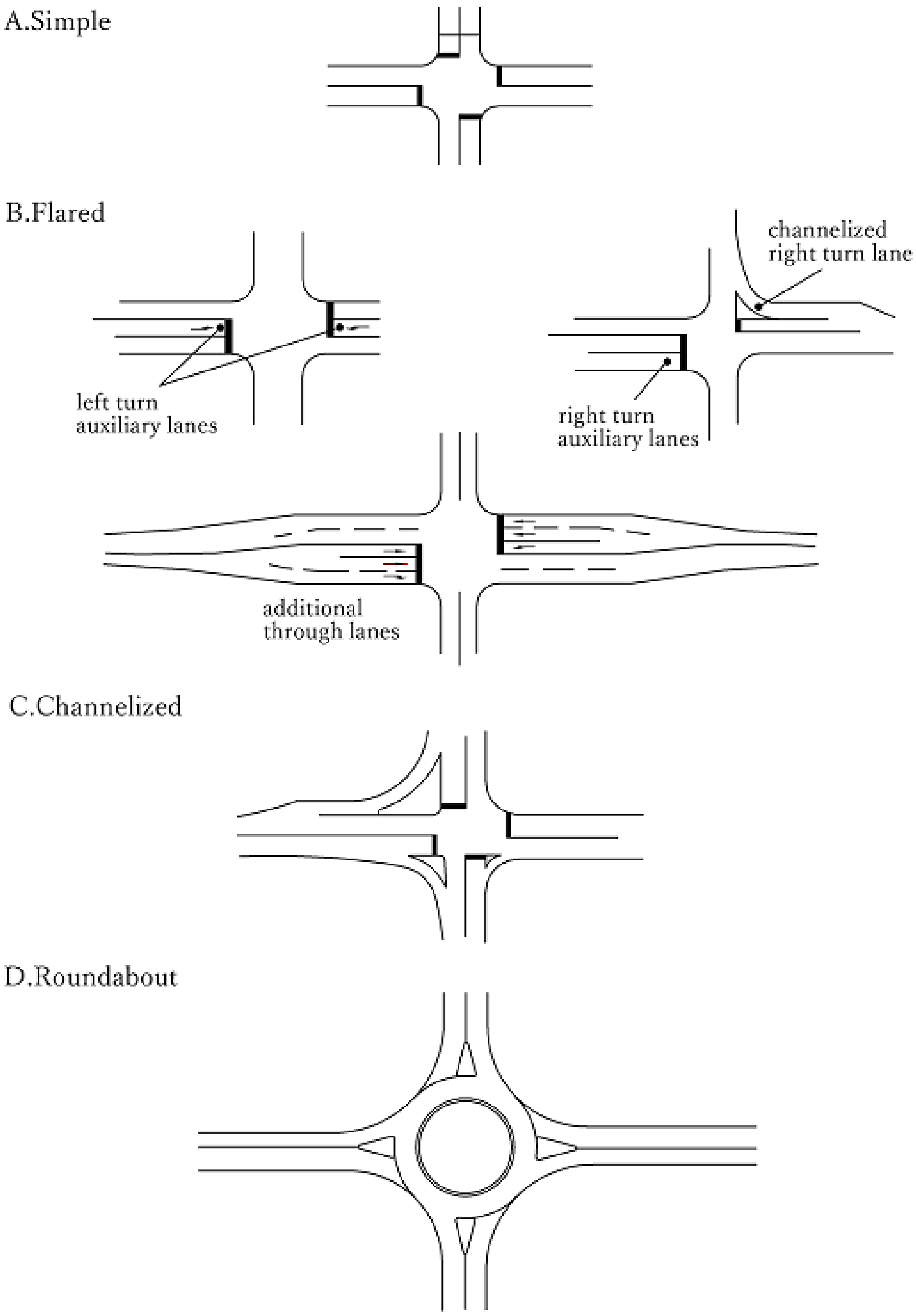


Figure 8.3 Intersection Types

8.2.1.1 SIMPLE INTERSECTIONS

Simple intersections maintain the road's typical cross section and number of lanes throughout the intersection, on both the major and minor roads. Simple intersections are best suited to locations where auxiliary (turning) lanes are not needed to achieve the desired level of service or are infeasible due to nearby constraints. Generally, simple intersections provide the minimum crossing distances for pedestrians and are common in low-volume locations.

8.2.1.2 FLARED INTERSECTIONS

Flared intersections expand the cross section of the road (main, cross or both). The flaring is often done to accommodate a left-turn lane, so that left turning bicycles and motor vehicles are removed from the through-traffic stream to increase capacity at high-volume locations, and safety on higher speed roads. Right-turn lanes, less frequently used than left-turn lanes, are usually a response to large volumes of right turns.

Intersections may be flared to accommodate an additional through lane as well. This approach is effective in increasing capacity at isolated rural or suburban settings in which lengthy widening beyond the intersection is: not needed to achieve the desired level-of-service; not feasible due to nearby constraints; or, not desirable within the context of the project.

Intersection approaches can be flared slightly, not enough for additional approach lanes but simply to ease the vehicle turning movement approaching or departing the intersection. This type of flaring has benefits to bicycle and motor vehicular flow since higher speed turning movements at the intersection are possible and encroachment by larger turning vehicles into other vehicle paths is reduced. However, adding flare to an intersection increases the pedestrian crossing distance and time.

8.2.1.3 CHANNELIZED INTERSECTIONS

Channelized intersections use pavement markings or raised islands to designate the intended vehicle paths. The most frequent use is for right turns, particularly when accompanied by an auxiliary right-turn lane. At skewed intersections, channelization islands are often used to delineate right turns, even in the absence of auxiliary right turn lanes. At intersections located on a curve, divisional islands can help direct drivers to and through the intersection. At large intersections, short median islands can be used effectively for pedestrian refuge.

Channelization islands are also used in support of left turn lanes, forming the ends of the taper approaching the turn bay, and often the narrow divisional island extending to the intersection. At “T”-type intersections, a channelization island can guide oncoming traffic to the right of the left-turn lane.

Channelized intersections are usually large and, therefore, require long pedestrian crosswalks.

However, the channelization islands can effectively reduce the crosswalk distance in which pedestrians are exposed to moving motor vehicles. The design of channelized intersections needs to ensure that the needs of pedestrians are considered, including pedestrian kerb cut ramps or “cut-throughs” that allow wheelchair users the same safe harbour as other pedestrians on channelization islands.

8.2.1.4 ROUNDABOUTS

The roundabout is a channelized intersection with one-way traffic flow circulating in an anti-clockwise direction around a central island. All traffic (through as well as turning) enters this one-way flow. Although usually circular in shape, the central island of a roundabout can be oval or irregularly shaped.

Roundabouts can be appropriate design alternative to both stop controlled and signal-controlled intersections, as they have fewer conflict points than traditional intersections (8 versus 32, respectively). At intersections of two-lane roads, roundabouts can usually function with a single circulating lane, making it possible to fit them into most settings.

Roundabouts differ from "rotaries" in the following respects:

Size – Single Lane roundabouts have an outside diameter between 24 and 42m, whereas rotaries are typically much larger with diameters as large as 200m.

Speed – The small diameter of roundabouts limits circulating vehicle speeds to 20 to 40 km/h, whereas, circulating speeds at rotaries is typically 50 to 65km/h.

Capacity – The slower circulating speeds at roundabouts allow entering vehicles to accept smaller gaps in the circulating traffic flow, meaning more gaps are available, increasing the volume of traffic processed. At rotaries, vehicles need larger gaps in the circulating traffic flow reducing the volume of traffic processed.

Safety – The slower speeds at roundabouts not only reduce the severity of crashes, but minimizes the total number of all crashes, whereas rotaries typically see high numbers of crashes with a greater severity.

Roundabouts are also considered as traffic-calming devices in some locations since all traffic is slowed to the design speed of the one-way circulating roadway. This is in contrast with application of two way stop control, where the major street is not slowed by the intersection, or all-way stop control where all traffic is required to stop. Roundabouts can also be considered for retrofit of existing rotaries; however, in cases with very high traffic volumes, traffic signal control may be more suitable.

8.2.2 TYPICAL INTERSECTION CONFIGURATIONS

Most intersections have three or four legs, but multi-leg intersections (five and even six-leg intersections) are not unusual. Examples of intersection configurations frequently encountered by the designer are shown in **Figure 8.4.** Ideally, roads in three-leg and four-leg intersections cross at right angles or nearly so. However, skewed approaches are a regular feature of intersection design. When skew angles are less than 60 degrees, the designer should evaluate intersection modifications to reduce the skew.

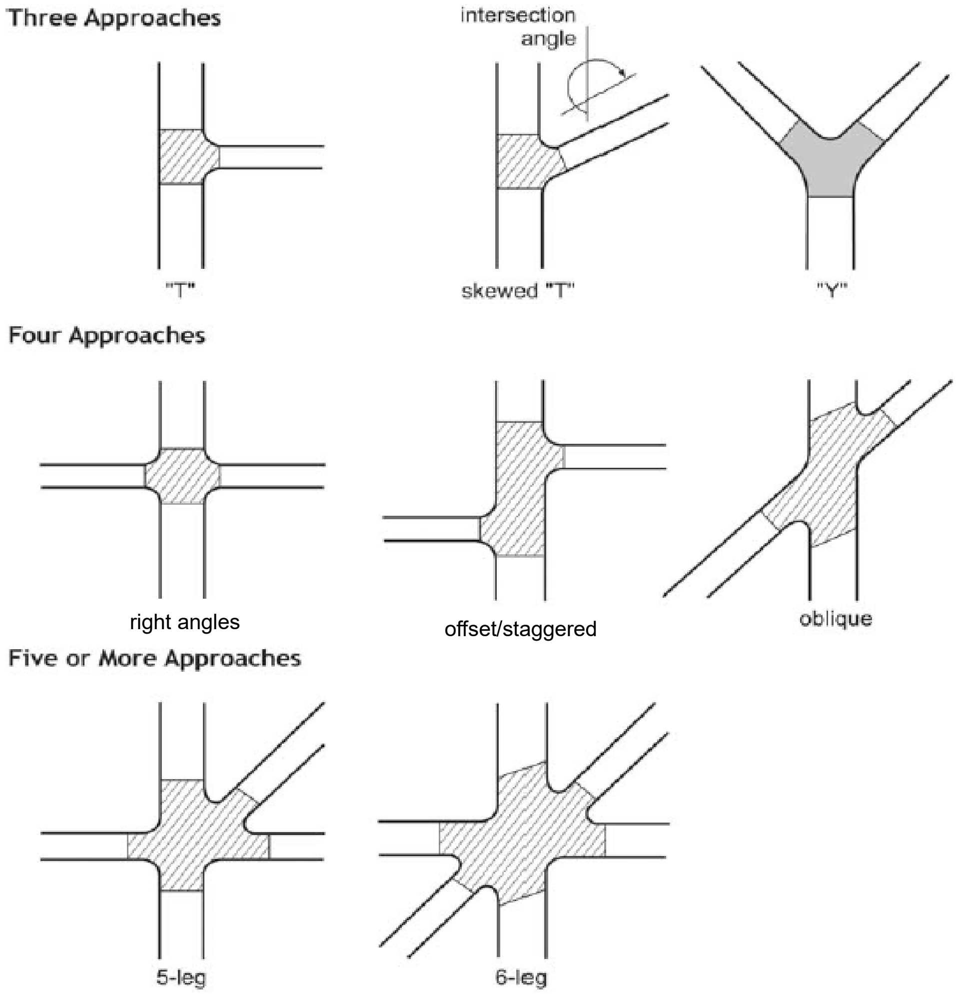


Figure 8.4 Intersecting road configuration and nomenclature

8.2.3 INTERSECTION MANOEUVRES

Three basic movements or manoeuvres occur at intersections, namely merging, diverging, and crossing. These manoeuvres are illustrated in **Figure 8.5**.

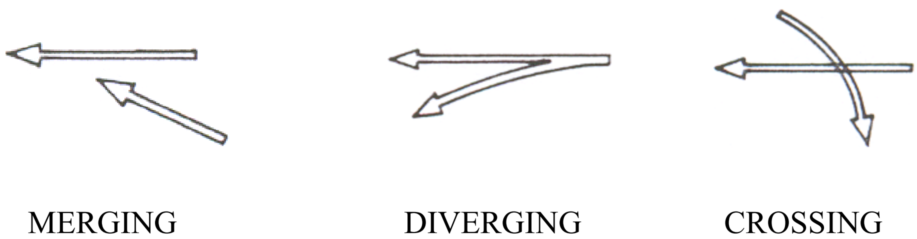


Figure 8.5 Intersection manoeuvres

8.2.4 SELECTION OF INTERSECTION TYPE

Generally, the selection of intersection type should be made from a socioeconomic point of view where the following factors are considered:

i. Cost of construction
ii. Type of area
iii. Land use and land availability
iv. Functional classes of the intersecting roads
v. Approach speeds
vi. Proportion of traffic on each approach and
vii. Volumes to be accommodated

A worldwide review of intersection design practice reported that, "typically the cheapest intersection type providing the required level of service is chosen". This cost is usually the sum of the design, construction and right-of-way costs.

However, for some cases the selection can be based on experiences from other similar intersections. Thus, it is not always necessary to make a socioeconomic calculation considering all possible types of intersections. The traffic safety aspect is suggested to be the primary criteria. Thus, the safety should first be checked to meet the requirements. Other effects should then be checked to be acceptable.

Figure 8.6 is a model for selecting an intersection type. The model is divided into three steps with a number of selection criteria. The criteria are based on road and traffic conditions concerning type of road, location and traffic, standard requirements concerning safety, speed and delays and experiences of safety and capacity performances of different intersection types.

The model is based on the following assumptions concerning different types of at-grade intersections:

i. The traffic volumes may be too high to be operated by an at-grade intersection and for certain road e.g., national motorways at-grade intersections are not accepted.
ii. Priority intersections are safe and give sufficient capacity for certain traffic volumes and speed limits.
iii. If a priority intersection is not sufficient for safety and capacity the main road traffic must also be controlled. This might not be accepted.
iv. Depending on location, traffic conditions and speed limits different types of priority or control intersection should be selected.

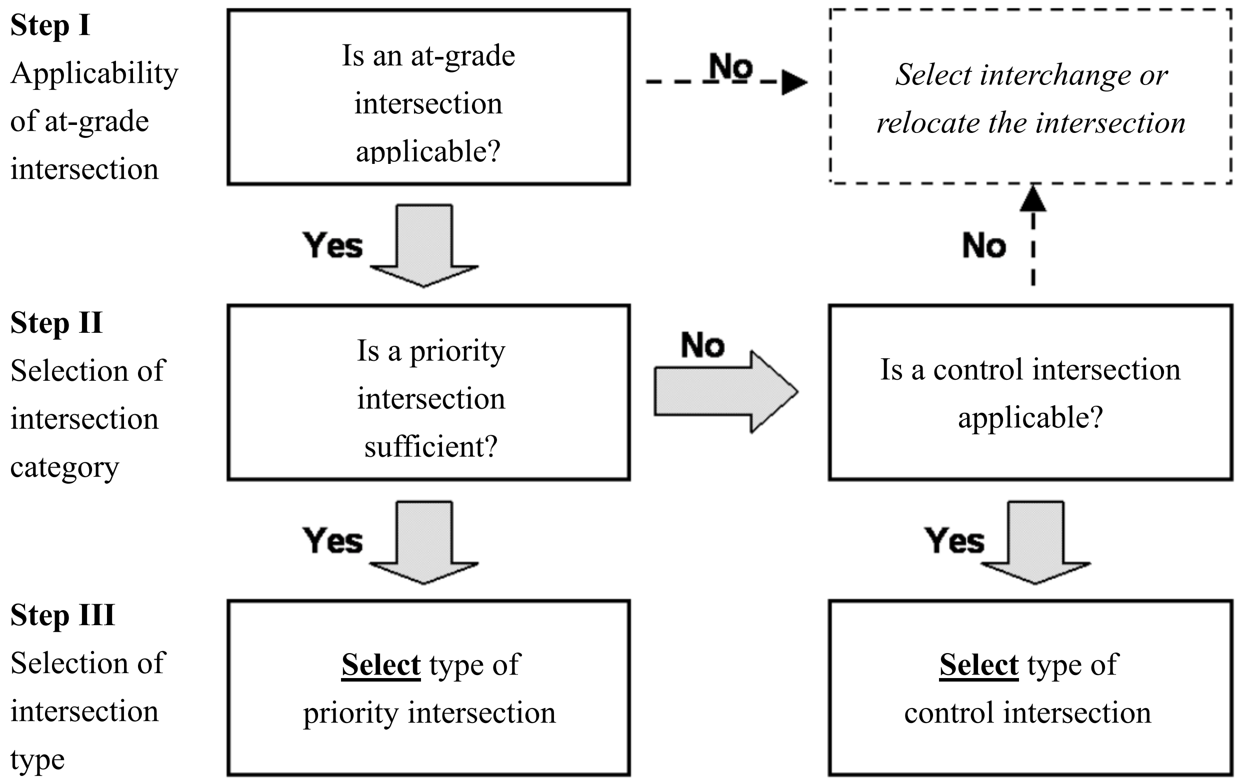
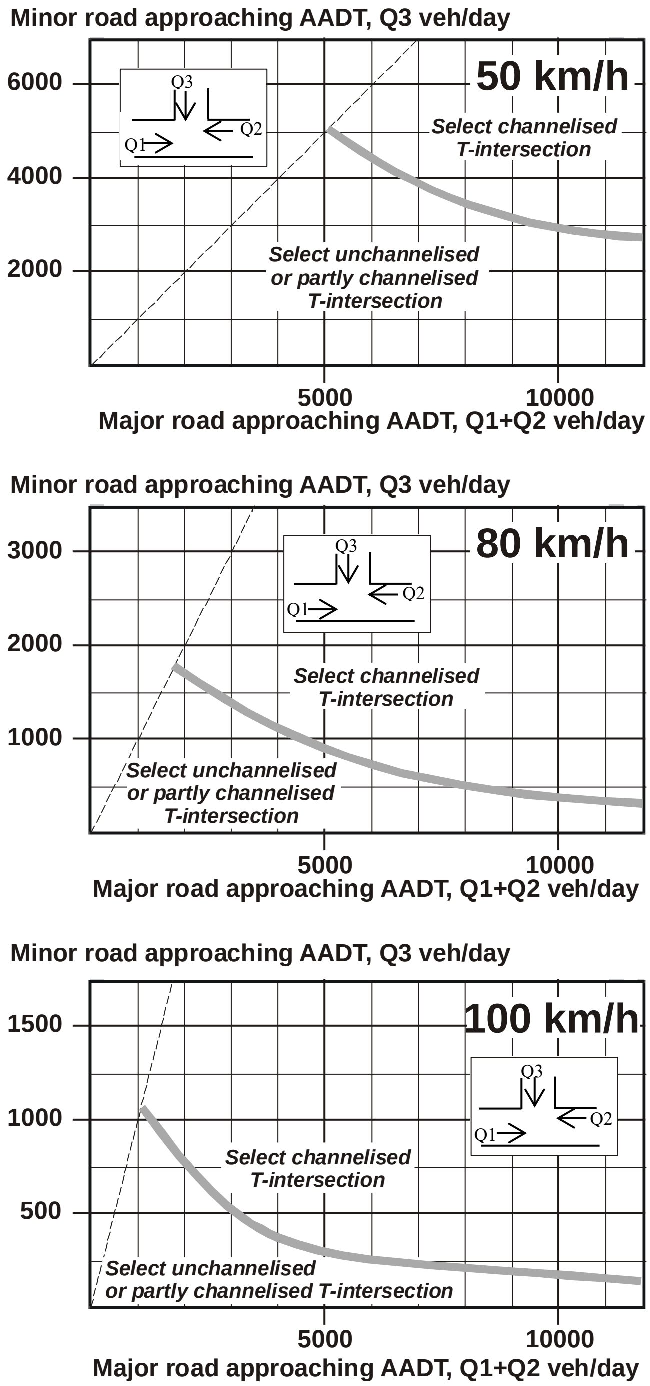


Figure 8.6 Intersection selection model

8.2.4.1 PRIORITY INTERSECTIONS

The selection of priority intersection type should mainly be based on safety. The selection can be made by using diagrams with the relationships between the safety levels and the average annual daily approaching traffic volumes (AADT in veh/day) based on accident statistics. The diagrams shown in **Figure 8.7** are for T-intersections on 2-lane roads with 50, 80 and 100 km/h speed limit. Crossroads should be avoided. The number of right turners should obviously also impact the decision.



Source: Uganda geometric design manual

Figure 8.7 Selection of priority intersection type as to safety

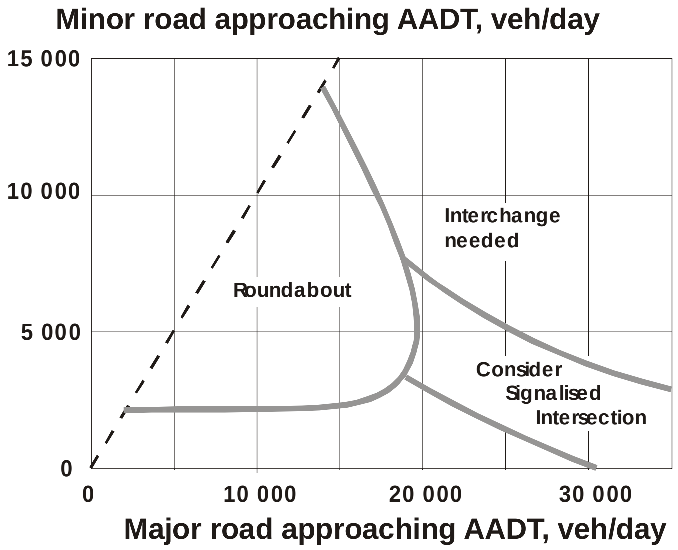
Partly channelized T-intersection should normally be used if needed to facilitate pedestrian crossings and also if the minor road island is needed to improve the visibility of the intersection.

8.2.4.2 CONTROL INTERSECTIONS

Roundabouts are suitable for almost all situations, provided there is enough space. Roundabouts have been found to be safer than signalised intersections and are suitable for both low and medium traffic flows. At very high traffic volumes they tend to become blocked due to drivers failing to obey the priority rules. Well-designed roundabouts slow traffic down, which can be useful at the entry to a built-up area, or where there is a significant change in road standard, such as the change from a dual carriageway to a single carriageway.

Traffic signals are the favoured option in the larger urban areas. Co-ordinated networks of signals (Area Traffic Control) can bring major improvements in traffic flow and a significant reduction in delays and stoppages. However, they must be demand-responsive, in order to get the maximum capacity from each intersection.

For some traffic distributions (for example high traffic volumes on the major road), the total delay can be shorter in a signalised intersection than in a roundabout. The diagram in **Figure 8.8** shows the traffic conditions for which signalised intersections are most suited based on best practise.



Source: Uganda geometric design manual

Figure 8.8 Selection of control intersection type

If a signalised intersection is considered due to planning conditions or traffic volumes, a capacity analysis and economic analysis should be made. This should include road construction and maintenance costs, accident costs, travel time costs, vehicle operating costs and environmental costs.

8.3 DESIGN OBJECTIVES

The main objective of intersection design is to facilitate the convenience, ease, and comfort of people traversing the intersection while enhancing the efficient movement of passenger cars, buses, trucks, bicycles, and pedestrians. Intersection design should be fitted closely to the natural transitional paths and operating characteristics of its users.

The goal of any intersection design, regardless of type or location, should be to implement the following principles:

i. Reduce vehicle speeds through the intersection, as appropriate.
ii. Provide the appropriate number of lanes and lane assignment to achieve adequate capacity, lane volume, and lane continuity.
iii. Provide channelization that operates smoothly, is intuitive to drivers, and results in vehicles naturally using the intended lanes.
iv. Provide adequate accommodation for the design vehicles.
v. Meet the needs of pedestrians and bicyclists.
vi. Provide appropriate sight distance and visibility.

Each element described above influences the operational efficiency and potential for crashes at intersections. When developing a design, the appropriate balance of operational performance for various modes, safety, and cost considerations should be sought throughout the design process. Favouring one component of the design may negatively affect another.

The design of each intersection should achieve an appropriate balance among the competing needs of pedestrians, bicyclists, motor vehicles, and transit with respect to safety, operational efficiency, convenience, ease, and comfort.

The four basic elements considered in intersection design are provided in **Table 8.1**.

Table 8.1 Four elements in intersection design

Element	Factors
Human Factors	Driving habits Ability of drivers to make decisions Driver expectancy Decision and reaction time Conformance to natural paths of movement Pedestrian use and habits Bicycle traffic use and habits
Traffic Considerations	Classification of each intersecting roadway Design and actual capacities Design-hour turning movements Size and operating characteristics of vehicle Variety of movements (diverging, merging, weaving, and crossing) Vehicle speeds Transit involvement Crash history Bicycle movements Pedestrian movements
Physical Elements	Character and use of abutting property Vertical alignments at the intersection Sight distance Angle of the intersection Conflict area Speed-change lanes/auxiliary lanes Geometric design features Traffic control devices Lighting equipment Roadside design features Environmental factors Cross walks Accesses Access management treatments
Economic Factors	Cost of improvements Effects of controlling or limiting rights-of-way on abutting residential or commercial properties Where channelization restricts or prohibits vehicular movements Energy consumption

8.4 DESIGN CONSIDERATIONS FOR INTERSECTION USER GROUPS

Intersection designers should utilize performance measures and apply engineering judgment to balance the needs of all roadway users and transportation modes in the design of each intersection. The size and design of physical elements such as roadway width, lane width, and corner radii are selected according to the volume and priority given to each of the intersection user groups. For an intersection in the urban core context, design priority may be given to design for pedestrians, bicyclists, passenger vehicles, and buses with basic accommodation given to trucks, except that additional accommodation to trucks may be provided on designated truck routes. Intersections in the suburban or rural contexts near industrial and commercial areas may be designed for automobiles and trucks with basic accommodation for pedestrians, bicyclists, and transit. In the other contexts, an appropriate balance should be found for all transportation modes that use a given facility. Design considerations for users include:

i. **Automobiles and Other Motor Vehicles Other Than Trucks** - Key elements affecting intersection performance for motor vehicles are:
   a. the type of traffic control;
   b. the vehicular capacity of the intersection, determined primarily from the number of lanes and traffic control;
   c. the ability and capacity to make turning movements;
   d. the visibility of approaching and crossing pedestrians and bicyclists; and
   e. the speed and visibility of approaching and crossing motor vehicles.

ii. **Bicyclists** - Key elements affecting intersection performance for bicycles are:
   a. the degree to which roadway surface is shared or used exclusively by bicyclists;
   b. the relationship between turning and through movements for motor vehicles and bicycles;
   c. traffic control for bicyclists;
   d. the differential in speed between motor vehicles and bicycles; and
   e. conflicts with pedestrian movements.

iii. **Pedestrians** - Key elements affecting intersection performance for pedestrians are:
   a. the amount of right-of-way provided for pedestrians including both walkway and crosswalk width;
   b. the crossing distance and resulting duration of exposure to motor vehicle and bicycle traffic;
   c. the volume of conflicting traffic;
   d. the speed and visibility of approaching traffic;
   e. turning speeds;

f. permissive right-turn-on-red;
g. permissive left-turn movements;
h. crosswalk lighting; and
i. accessibility for persons with disabilities.

iv. **Transit -** Transit operations on roadways usually involve the operation of buses, which share the same key characteristics as vehicles previously described. In addition, transit operations may sometimes involve a transit stop in the intersection area, thereby creating potential conflicts with pedestrian, bicycle, and motor vehicle flow. Transit stops should be physically connected to pedestrian facilities to serve arriving and departing transit patrons. Additionally, where light-rail, trolley, or other transit is present, their unique physical and operating features should be taken into account.

v. **Trucks** - Trucks share many of the same key characteristics as other motor vehicles described above. In addition, trucks may be three to four times the length of other motor vehicles, may be much slower starting than most motor vehicles, and may need much larger turning radii than most motor vehicles. Therefore, the presence and frequency of trucks affects the capacity of the intersection, the width of the driving surface needed for turning movements, and the radius of turning movements.

Design of intersection elements for one group of users often has consequences for other users. For example, an intersection designed to accommodate trucks with no encroachment into adjacent lanes needs large corner radii, wide turning roadways, and results in greater distances for pedestrians to cross.

Automobile drivers can often negotiate intersection turns at speeds that are too fast to adequately detect and stop for pedestrians crossing the roadway. The turning roadways are sometimes wide enough for automobiles to overtake or pass one another within the turning roadway, and results in pedestrian exposure equivalent to crossing two lanes. Conversely, an intersection designed to accommodate pedestrians with minimum exposure to other traffic often involves encroachment on adjacent lanes by turning trucks both on the intersection approach and departure roadways.

8.5 PLANNING AND DESIGN OF AT-GRADE INTERSECTIONS

In the planning and design of at-grade intersections, attention must be paid to traffic volume, vehicle type, speed, traffic distribution over the road network, and future changes in traffic volume. However, when improving existing at-grade intersections, it is also essential to scrutinize and examine records of accidents, which clearly show defects (problems that need to be improved) in the existing at-grade intersections. In situations where accident records are not available, traffic conflict studies should be carried out as a surrogate for accident records.

Moreover, because at-grade intersections are nodal points of road traffic in the road network and they reflect land use in the local area, the role of road traffic intersections within the road network should be considered. It is important to also seek balance with other related at-grade intersections and road sections of uninterrupted flow and other conditions.

Additionally, in the planning and design of at-grade intersections, functions must be secured to enable non-motorized road users (bicycles, pedestrian, wheelchair users etc.) to pass and stop safely and smoothly. Particularly on intersections that have large numbers of pedestrians, it is necessary to provide pedestrian priority spaces with stopping and staying functions on walkways and give ample consideration to spatial functions for forming landscape, etc.

In principle, the structural design of intersections is conducted according to the road design hourly traffic volume. However, in cases where traffic volume at the time of construction is much smaller than the design hourly traffic volume, it is possible to conduct initial phased construction upon assuming the estimated traffic volume around 5~10 years after the start of use to be the design hourly traffic volume at the intersection.

In this case, it is necessary to consider the execution procedure, securing of the right-of-way, redoing of works and so on, in the construction from the subsequent phases to the final phase.

8.5.1 PROCEDURE FOR PLANNING AND DESIGN

Figure 8.9 shows the general procedure for the planning and design of at-grade intersection from the preparatory stage through the preliminary design stage to the detail design stage.

The following points, however, need careful attention:

i. In the preliminary design stage, though the preliminary geometric design (Step 6) and the deliberation of traffic handling capability (step 7) are independent steps, they are in fact closely interrelated.
ii. In the flow chart, the actual work from step 6 to 10 has to be treated as a whole.
iii. At the preliminary design step 6, values of various design components such as grade, profile and cross-section have to be decided and with these, the channel configurations, pedestrian crossings, stopping lines and major road markings have to be drawn.
iv. The detail design step 12 involves the preparation of drawings showing the added lane and taper lengths, grade, profile, and cross-sectional structures, turning radii of right and left turning traffic, channelization, median, cornering of walkways, drainage provisions, road markings, signals, traffic signs and traffic safety devices.

The fundamental data necessary for planning and desing of at-grade intersection is shown in **Table 8.2**.

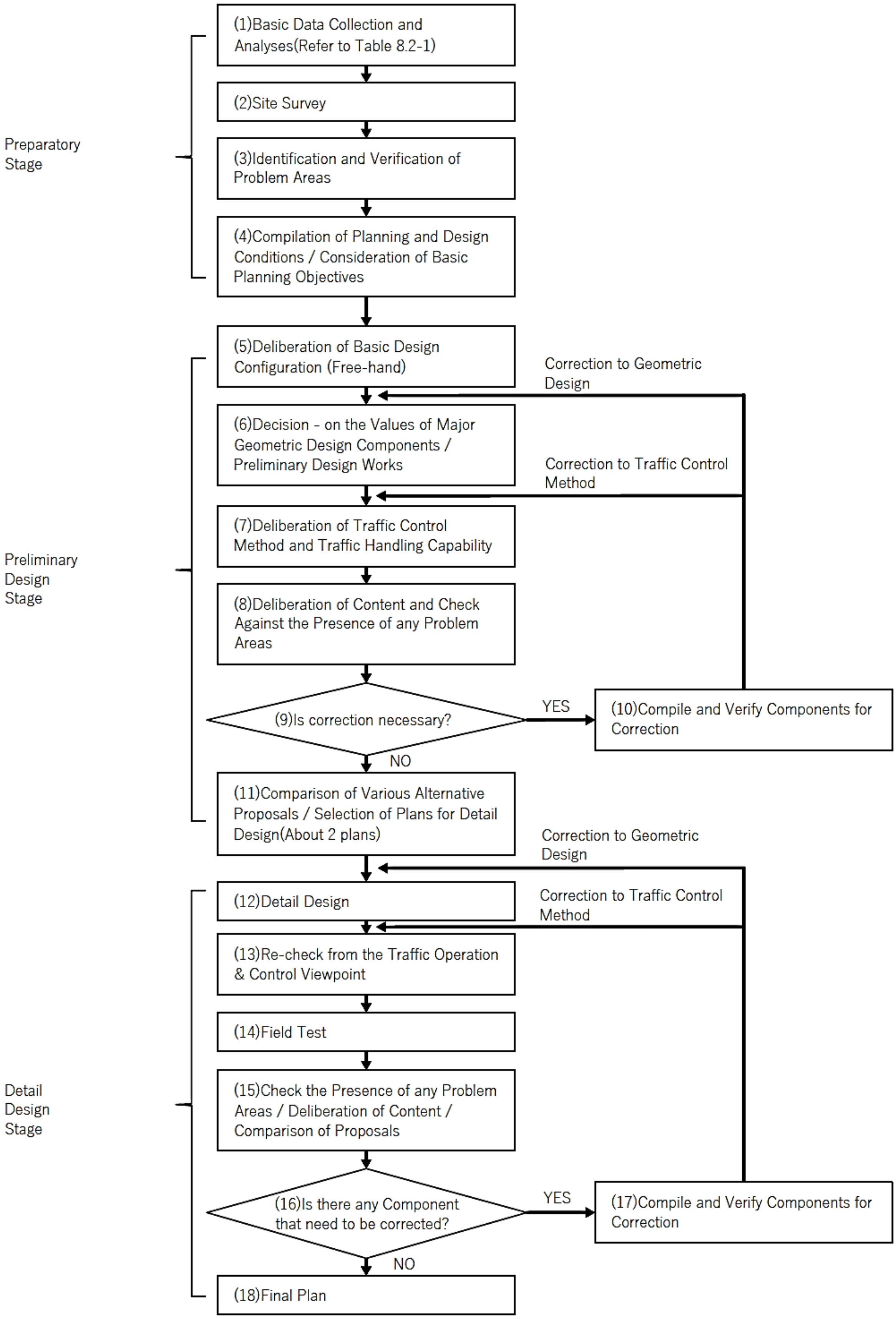


Figure 8.9 Basic Procedure for the Planning and Design of At-grade Intersection.

Table 8.2 Fundamental Data necessary for Planning and Design of At-grade Intersection.

Condition	Data category	Remarks
Traffic	Hourly traffic volume by direction and vehicular type.	Morning and evening peak (2-3 hours) volume, or if necessary 12 hour or 24-hour traffic volume. Vehicles are to be classified into small, medium, large, trailers, and others. Other forms of classification depend on the requirements.
	Vehicle, pedestrian, and bicycle traffic volume.	For cases with particularly high volume the peak period and accident data for the last 3 years by each crossing location and conflict point are required.
	Traffic control situation.	Include adjacent intersections and the nearby minor roads.
	Traffic signal control method.	Include the adjacent intersections.
Road	Road network characteristics	Data to include the surrounding areas and minor roads. Urban planning road network map on topographic map at a scale of 1/2,500 to 1/5,000.
	Land features and structures	The land features of the surrounding areas, land use and building conditions along the road.
	Road conditions	Number of approaches and their intersecting angles, road structure (grade, profile, and cross-section), road markings on a topographic map at a scale of 1/250 to 1/500, photographs.

8.5.2 GEOMETRIC STRUCTURE AND TRAFFIC CONTROL

8.5.2.1 HARMONIZATION OF GEOMETRIC STRUCTURE AND TRAFFIC CONTROL

Ample consideration must be taken to ensure that the design of an at-grade intersection is consistent with the traffic signals and various traffic regulations. The safety and smooth flow of traffic in at-grade intersections is largely determined by the type of traffic control, for example, traffic signals, various regulations, etc.

Moreover, the geometric structure of an at-grade intersection also varies according to the types of traffic control. Accordingly, when planning and designing at-grade intersections, it is necessary to examine the traffic control method and adopt a geometric structure that corresponds to that.

Conversely, if traffic control is conducted without paying attention to geometric structure, there is a risk that safety and smooth traffic flow will be compromised. In other words, because geometric structure and traffic control at an at-grade intersection are mutually unexclusive it is not possible to deal with one only.

Therefore, when designing at-grade intersections, regardless of whether a new intersection is being constructed or an existing intersection is being improved, it is always necessary to match geometric structure with traffic control.

8.5.2.2 TRAFFIC CONTROL

The basic principles of traffic control, which provides the basis for intersection design, are as follows:

i. On road class B1 and C1, at-grade intersections should be permitted to the extent that does not hinder main line traffic, hence the basic principle is not to conduct signal control.
ii. Implementing stop control in respect to the main line runs counter to driving customs, disturbs traffic and risks causing accidents in many cases. Therefore, as a rule, stop control should not be implemented in respect to the main line on roads with a design speed of 60km/h or more.
iii. The stop control threshold traffic volume differs greatly depending on the road width, volume of right-turning and left-turning traffic and other conditions. Generally speaking, if the total volume of mutually intersecting traffic is approximately 1,000 vehicles/hour or less, it is possible to process by implementing stop control on the side that has less traffic volume, however, as traffic volume increases, waiting times becomes longer, drivers become more irritated, and accidents are induced.
iv. Therefore, concerning high-specification roads, as a safety consideration, it is necessary to design the stop control threshold traffic volume smaller than the capacity.
v. Traffic control devices (signals, STOP, or YIELD signs and pavement markings) often control the entry of vehicles into the intersection. Traffic control devices may also be required at intersections of important private accesses with public roads. Examples of important accesses include alleys serving multiple homes, commercial alleys accessing parking, and commercial accesses.

8.5.2.3 TRAFFIC CONTROL MEASURES

Potentially conflicting flows (vehicle-to-vehicle or vehicle-to-nonvehicle) are an inherent feature of intersections. At most intersections therefore, traffic control measures are necessary to assign the right of way. Types of intersection traffic control include:

i. Where sufficient visibility is provided in low volume situations, some intersections operate effectively without formalized traffic control. In these cases, normal right of way rules applies.
ii. Yield control, with traffic controlled by "YIELD" signs (sometimes accompanied by pavement markings) on the minor road approaches. Major road traffic is not controlled.
iii. All-way yield control on roundabouts.
iv. Two-Way Stop Control (TWSC), with traffic controlled by "STOP" sign or beacons on the minor road approaches. Major road traffic is not controlled. The term "two-way stop control" can also be applied to "T" intersections, even though there may be only one approach under stop control. STOP control should not be used for speed reduction.
v. All-Way Stop Control (AWSC), with traffic on all approaches controlled by STOP signs or STOP beacons. All-way stop control can also be a temporary control at intersections for which traffic signals are warranted but not yet installed.
vi. Traffic signals, controlling traffic on all approaches.
vii. Flashing warning beacons on some or all approaches.

Generally, the preferred type of traffic control correlates most closely with safety concerns and volume of motor vehicles, bicycles, and pedestrians. For intersections with lower volumes, STOP or YIELD control on the cross (minor) road is the most frequently used form of vehicular traffic control.

8.5.3 GEOMETRIC STRUCTURE AND TRAFFIC SAFETY

In response to traffic accidents, countermeasures are often taken according to conditions of occurrence after roads have been commissioned, however, it is sometimes less costly in terms of accident countermeasures, maintenance and repairs, accident handling and so on to design the road structure and road alignment, etc. to ensure that traffic accidents do not occur from the start, even if this may entail slightly higher construction costs. Accordingly, in the planning and design of at-grade intersections, ample consideration should be given to safety.

The main types of traffic accidents that occur at intersections are: accidents that involve pedestrians using pedestrian crossings, rear-end collisions, head-on collisions, accidents when turning right or left, and other accidents involving two or more vehicles. Factors that lead to accident can be categorized into three (3):

i. Human factors (drivers, pedestrians, etc.)
ii. Road and Environment deficiencies (adverse road design, skidding, etc.)
iii. Vehicle defect (defective tyres, breaks etc.)

Accident prevention measures should be taken from the above-mentioned factors. In other to prevent accidents caused by road and environment deficiencies, road geometric structure, traffic safety facilities， operation of traffic control, traffic regulations etc. must be considered in the design of an at- grade intersection.

Table 8.3 shows the relationship between the main risk factors at intersections and measures for securing safety in road geometric structure.

Table 8.3 Intersection Risk Factors and Measures to Secure Safety through Road Geometric Structure

Main risk factors	Safety measures based on road geometric structure
Visibility issues due to inappropriate intersection shape.	i. Avoid acute angle intersections and multi-branch. ii. Avoid irregular shaped intersections (offset intersections, etc.). iii. Avoid placing at-grade intersections on bends or at points of vertical alignment sags or crests.
Mismatching between road geometric structure and traffic characteristics	i. Securing of road geometric structure that conforms with design speed on road sections of uninterrupted flow (securing of lane width. ii. Adoption of the same design speed as on road sections of uninterrupted flow).
Poor visibility inside and around intersections	i. Securing of visibility inside and around intersections (planning of road alignment and walkways corner cut-offs, piers, pedestrian overpasses, etc. in consideration of visibility). ii. Arrangement of planting, road sign-post, bill boards, utility poles etc. in consideration of mutual visibility between pedestrians and vehicles.
Inappropriate channelization	i. Separation of right-turning and left-turning traffic through provision of auxiliary lanes. ii. Appropriate design according to specifications and characteristics of crossing roads, installation of vehicle selection channelizing islands. iii. Avoidance of unnecessarily large corner angle radii and channel widths.
Insufficient functions of pedestrian spaces and bicycle spaces	i. Securing of gathering spaces for pedestrians and bicycles. ii. Installation of refuge islands for pedestrians (two-stage road crossing). iii. Appropriate walkways, lowering of kerbstones.

Out of the points in **Table 8.3**, the following are especially important for securing traffic safety and need to be given particular attention when implementing planning and design.

i. Securing of visibility inside and around intersections.

   Secure visibility inside and around intersections to ensure that vehicles approaching and entering intersections can easily confirm the road conditions ahead (existence of intersection, signals, etc.) and traffic conditions (existence of other vehicles, behaviour of vehicles such as stopping, accelerating, decelerating, turning, etc., existence of pedestrians, bicycles, etc.).

ii. Separation of right-turning and left-turning traffic through provision of auxiliary lanes.

   Provide auxiliary lanes to appropriately separate through traffic from right-turning and left-turning traffic and thereby prevent rear-end collisions arising when right-turning and left-turning vehicles decelerate or stop.

iii. Securing of gathering spaces for pedestrians and bicycles.

   Secure ample gathering spaces that consider the local characteristics and volume of pedestrian and bicycle traffic, etc. in the design of walkways and corner cut-offs at intersections to ensure that pedestrians and cyclists can safely gather and pass at intersections.

8.5.4 DESIGN VEHICLE, TURNING PATHS AND DESIGN SPEED

8.5.4.1 DESIGN VEHICLE AND TURNING PATHS

The design vehicle is the largest type of vehicle typically expected to be accommodated on the road. At intersections, be it at-grade or grade-separated, the most important attribute of design vehicles is their turning radius, which in turn influences the pavement corner radius and therefore the size of the intersection. Lane width, another feature related to the design vehicle, has some impact on intersection design, but less than turning radius. The design vehicle may also affect the choice of traffic control device and the need for auxiliary lanes.

The design vehicle for intersections is the larger of the design vehicles selected for the intersecting roads. For example, at the intersection of a minor arterial and a local road, the appropriate design vehicle for the intersection is that required by the minor arterial (i.e., “larger” road).

The turning path of a vehicle is the smallest circular turn that it can make. Decisions about design vehicle and turning path must be made upon comprehensively judging the character, functions, local characteristics, roadside conditions, motorised and non-motorised traffic on the road. Selection of the turning paths of vehicles in the planning and design of intersection mainly entails deciding which parts of the carriageway to use when turning right and left. For example, small vehicles can turn right from the furthest-right-turn lane and enter the furthest-right of the crossroad, however, in the case of a semi-trailer coupled vehicle, the combination of design vehicle and turning path is set so that a right turn cannot be made unless the entire right side of the carriageway on the intersection outflow side is used.

Selection of the turning path greatly impacts intersection safety and capacity. As a rule, this is designed to ensure that vehicles can make right and left turns without encroaching on other lanes as much as possible. Refer to **Section 5.9** on design vehicle and turning paths.

8.5.4.2 DESIGN SPEED

In consideration of safety and smooth flow through at-grade intersections, as a rule, the design speed of straight-moving vehicles going through intersections should be the same as the design speeds on the sections of uninterrupted flow of the roads concerned.

However, when the order of preference between a major road and minor road is obvious, sometimes the design speed for the intersection approach road on the side of the minor road is made lower than that for the section of uninterrupted flow. Generally, rather than having vehicles intersect at high design speed with a small crossing angle, it is more advisable to reduce design speed at the approach part of the minor road, insert a curve into the approach part and bring the crossing angle closer to a right angle.

Moreover, for at-grade intersections, it is almost always necessary to provide auxiliary width components such as turning lanes and median strips, however, the intersection design speed is sometimes reduced to enable these components.

In other words, depending on the traffic and roadside conditions, rather than maintaining high design speed at the expense of auxiliary width components, there are cases where greater merits can be realized by providing the necessary components even if it means reducing the design speed. Accordingly, it is also possible to lower the design speed by 10-20km/h in unavoidable cases such as these.

However, since such a measure cannot be communicated to road users, there is a risk that it will inadvertently lead road users towards danger. Therefore, as was mentioned earlier, to ensure safe and smooth flow through at-grade intersections, as a rule, the design speed of through traffic at an intersection should be the same as the design speeds on the sections of uninterrupted flow of the roads concerned, while simply lowering design speed should be avoided.

When there is no choice but to adopt design speed through at-grade intersections and approaches that is lower than on road sections of uninterrupted flow, if the speed disparity is too large, there is a risk that safety problems will arise at the runoff parts. Therefore, this difference in design speed should be limited to no greater than 20km/h. Furthermore, when designing the connection between an intersection approach and the runoff of a road section of uninterrupted flow (runoff where width changes, shifting of main lane, sight distance with eased curve section, etc.), it is necessary to take ample care to ensure that drivers naturally decelerate.

Moreover, when it is necessary to obtain the safe distance for accelerating or decelerating around intersections in consideration of speed changes, **Equation 8.1** is used. **Table 8.4** provides acceleration and deceleration when considering speed change.

Table 8.4 Acceleration and deceleration when considering speed change

Region	Type of Road	Acceleration (α) (m/s^2)	Deceleration(α) (m/s^2)
Urban	Main Road	0.5	-3.0
	Minor Road		
Rural	Main Road	1.0	-2.5
	Minor Road	1.5	-3.0

Acceleration is reduced by 0.1% for every 1% uphill gradient.

No change on deceleration gradient section.

$$L = \frac{1}{2 \times 3.6^2 \alpha} (V^2 \text{-} V_0^2) \tag{8.1}$$

Where:

L: Travel distance (m)

α: Acceleration /deceleration (m/s^2)

V: Terminal speed (km/h)

V_0: Initial speed (km/h)

8.5.5 LAYOUT OF AT-GRADE INTERSECTIONS

The elements that determine the basic form of an at-grade intersection, for example, intersection legs, intersection angles, shape, interval, etc., are determined in the planning stage. This basic form has a decisive impact on the safety, traffic flow and capacity of the intersection.

Since it is difficult to fundamentally correct planning defects in the design or improvement stage, ample consideration must be given to the following principles to avoid reducing the traffic processing capacity and prevent the causes of traffic accidents.

8.5.5.1 INTERSECTION LEGS AND ANGLES

Intersection legs are those segments of the roadway connecting to the intersection. The leg used by traffic approaching the intersection is the approach leg, and that used by traffic leaving is the departure leg as depicted in **Figure 8.1**.

The angle of intersection is formed by the intersecting roads' centrelines. Where the angle of intersection departs significantly (more than approximately 200) from right angles (900), the intersection is referred to as a skewed intersection.

8.5.5.2 INTERSECTION LEGS

It is not advisable to plan five or more legs for at-grade intersections even if the intersection legs are minor roads except in special cases such as station plazas for e.g., multi-modal transport terminal. Under difficult circumstances or other related situations, where this cannot be observed efforts should be made to realign the minor roads into the major roads before the intersection.

New roads must not be planned to cross with existing at-grade intersections. This is not desirable, even if an existing at-grade intersection involves minor roads. However, exceptions are permitted in cases of turning a T-shaped intersection into a cross-shaped intersection.

In cases where there is no choice but to plan a new road into an existing at-grade intersection due to other factors, it is necessary to simultaneously replace or reorganize the existing roads.

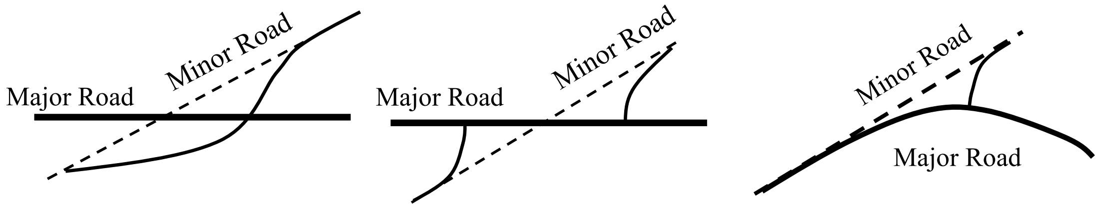
8.5.5.3 INTERSECTION ANGLES

Intersecting traffic flows must be planned to intersect at right angles or close to right angles. In cases of at-grade intersections at right angles or close to right angles, the distance across intersecting carriageways is short and the area of the intersecting part is also small. This is also desirable in terms of visibility. Therefore, the desirable minimum crossing angle should be made 75° or more. However, in case of difficulty, an absolute minimum crossing angle of 60° is permitted.

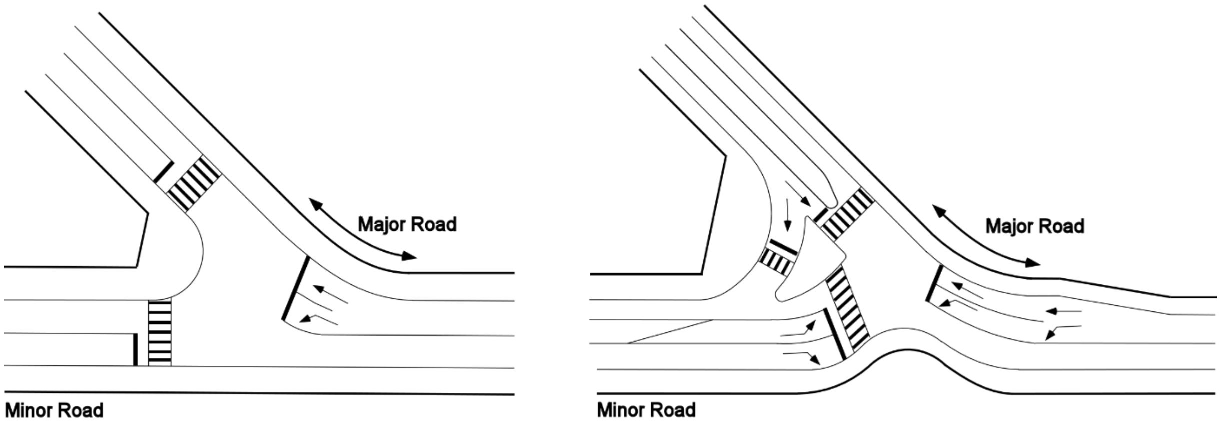
A small intersecting angle creates large area of intersection, poor visibility and longer crossing length. Improvement of intersection angle is primarily done targeting traffic on the minor road.

In cases where minor roads that handle local traffic intersect with a major road and are subject to temporary stop control, it is necessary to modify the approach geometry so that the crossing angle is at a right angle or close to a right angle. The same applies to traffic turning left out of a major road into a minor road.

Improvements for intersection angles are illustrated in **Figure 8.10**.



(a) (b) (c)



Before improvement After improvement

(d)

Figure 8.10 Improvements for intersection angles

8.5.6 SHAPE OF INTERSECTION

Irregular intersections, offset intersections, and dogleg intersections, where mainstream traffic turns right or left, should be avoided. The alignment should be such that the mainstream traffic goes through the intersection in a straight line; moreover, there should not be two or more confluences on the side of the mainstream traffic. The intersection angles of the crossroads should be close to right angle.

8.5.6.1 IRREGULAR INTERSECTION

In cases where the mainstream traffic becomes right-turning and left-turning traffic, as is shown in **Figure 8.11**, it is desirable in terms of traffic processing to improve alignment in the direction of the mainstream traffic. If two or more legs intersect on either side of the mainstream traffic, as is shown in **Figure 8.11**, roads on the side of the minor road should be integrated.

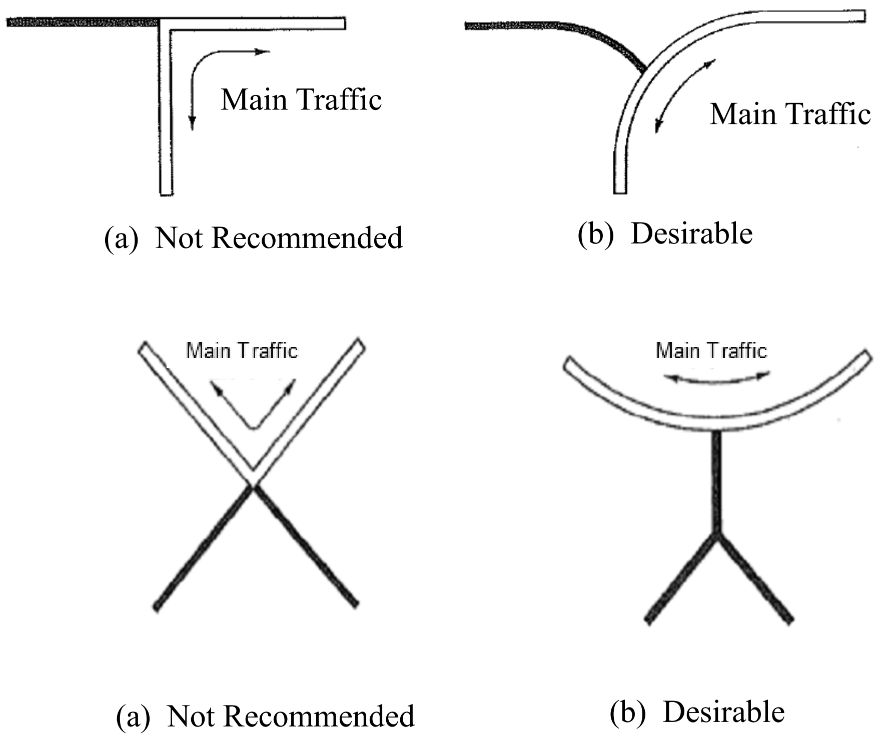


Figure 8.11 Improvement of Cross-Intersection

8.5.6.2 DEFORMED INTERSECTION

An offset intersection can be regarded as two T-shaped intersections in very close proximity to each other. At such at-grade intersections, the area of the intersection becomes very large and the respective traffic flows also become complicated.

Moreover, in many cases, since there is crossing with crossing pedestrian traffic, attention is needed regarding the placement of signal lights, and this isn't desirable in terms of safety and processing capacity. It is desirable to modify the shape of an offset intersection in the manner shown in **Figure 8.12.**

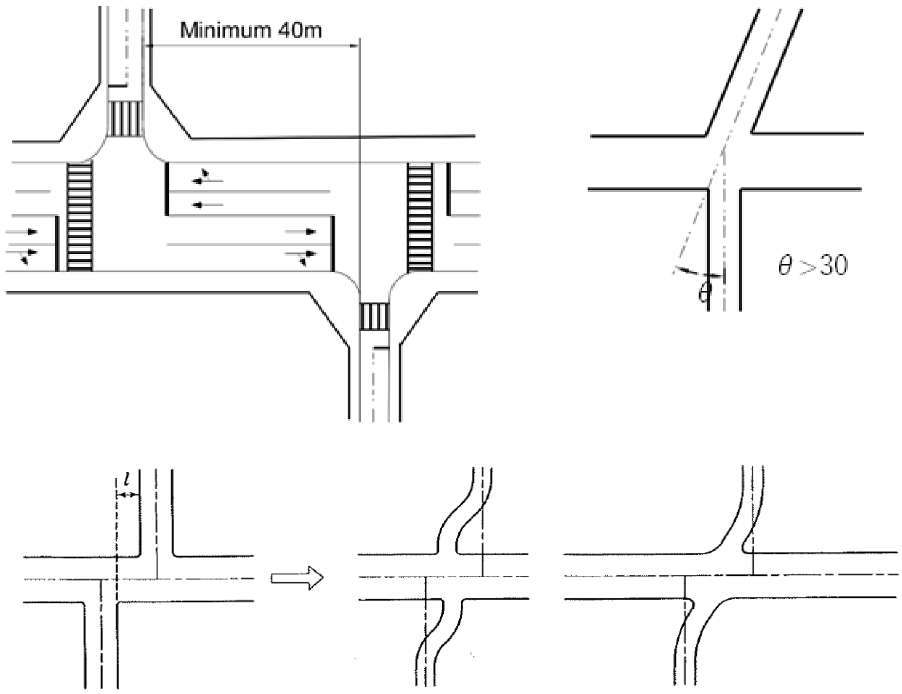


Figure 8.12 Improvement of Offset Intersection

In cases where there is no choice but to adopt an offset intersection, as a rule, traffic regulations permitting only right-turning should be conducted, or a median strip should be established to physically regulate the left-turning of mainstream traffic and left road crossing. In such cases, an opening should be placed in the median strip at an appropriate position and the demand for left turning can be satisfied by enabling change of direction as shown in **Figure 8.13**.

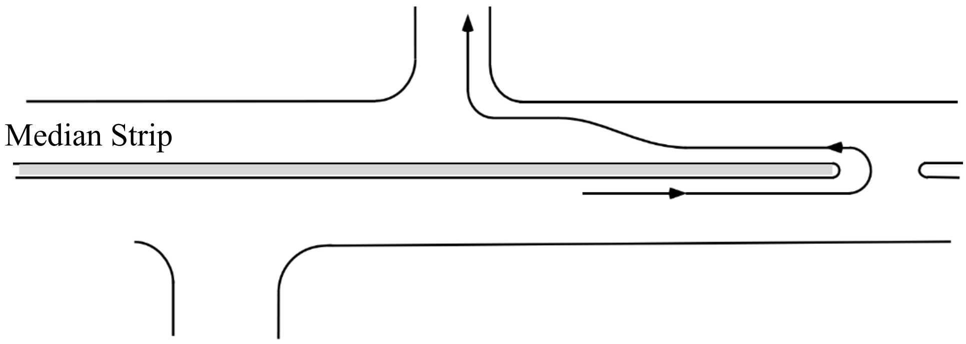


Figure 8.13 Regulation based on Median Strip

8.5.7 WIDENING OF MINOR ROAD

Examination of the number of lanes (provision of auxiliary lanes) often tends to focus only on the side of the major road, however, since the green signal time of signals on the minor road accounts for a high share of the signal cycle, it is possible to shorten the green signal time in this direction by increasing the incoming number of lanes on the minor road as shown in **Figure**

8.14.

In cases where the minor road comprises two lanes or less and it is difficult to widen the major road due to site issues with roadside buildings and site land, it is effective to widen the minor road. This is because, the increased traffic capacity of the major road made possible by the shortened green time on the minor road will extend to all lanes of the major road. This will lead to a far greater increase in processing capacity than that produced by provision of an extra lane and proving effective in the clearance of the bottlenecks.

Moreover, widening the minor road would improve the unbalanced shape of the intersection resulting from the wide major road and narrow minor road, and could also prevent the clearance time on the minor road becoming unreasonably long. In this case, it should be noted that the green time on the minor road must not be made less than 15 seconds. However, this could increase depending on the number and needed time of pedestrians crossing the major road on the same green time.

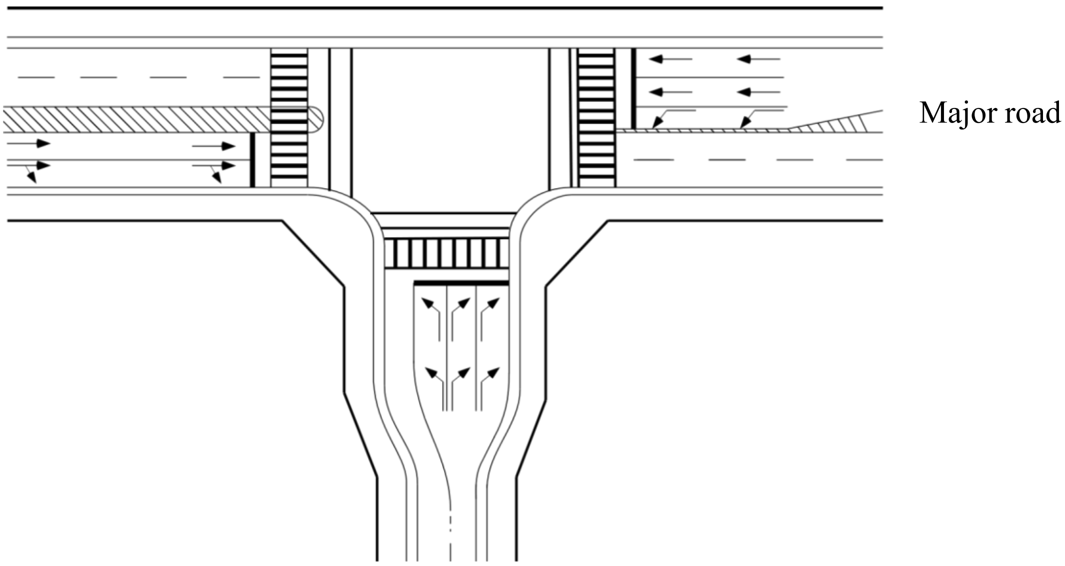


Figure 8.14 Widening of minor road

8.5.8 SPACING OF INTERSECTIONS

Ideally, the spacing of intersections should be as far apart as possible. The minimum distance between consecutive intersections shall preferably be equal to (10 x VD) meters, where VD is the major road design speed in km/h.

Where it is impossible to provide this minimum spacing, then the design shall incorporate either, or both, of the following:

i. A distance between minor road centrelines equal to the passing sight distance appropriate for the intersection design speed plus half the length of the widened major road sections at each intersection, or

ii. A grouping of minor road intersections into pairs to form staggered T-junctions and a distance between pairs as in (i) above.

The three (3) main factors that dictate the minimum spacing of intersections are:

i. Restriction by weaving length
ii. **Restriction by queue length at signal control**
iii. Restriction by length of storage lane

8.5.8.1 RESTRICTION BY WEAVING LENGTH

The spacing of intersections controlled by weaving length is illustrated in **Figure 8.15.** If the volume of weaving traffic is small, there will be few traffic conflict problems. However, if one of the streams of weaving traffic is the mainstream traffic, problems often arise in terms of both safety and processing capacity.

The space between two at-grade intersection assuming a maximum volume of weaving traffic while maintaining a reasonable level of safety is given by **Equation 8.2**.

$$L = V \times N \times 2 \tag{8.2}$$

Where,

L: Weaving length (m)
V: Design Speed (km/h)
N: Number of Lanes (in one direction)

In cases where the weaving traffic volume is not very large, it is often permissible to adopt a smaller spacing than the value obtained by **Equation 8.2.**

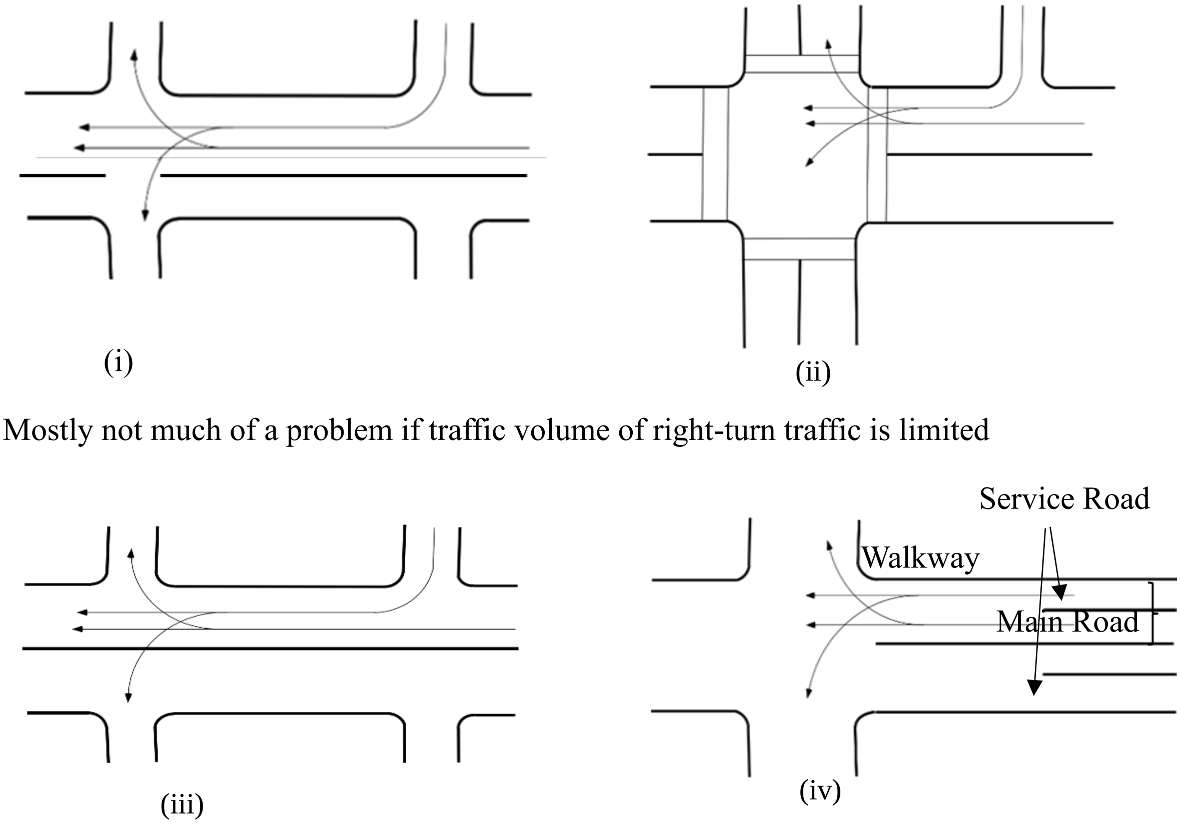


Figure 8.15 Example of Restriction by Weaving Length

8.5.8.2 RESTRICTION BY QUEUE LENGTH AT SIGNAL CONTROL

It is necessary to secure sufficient spacing between intersections to ensure that vehicles stopped due to signal control will not tailback into and congest the next intersection. Since it is safe to assume that the signals of two adjacent intersections are almost simultaneously controlled, there are not many cases where the intersection interval is limited by the queue length of mainstream traffic. However, there are cases where intersection spacing is limited by the queue length of minor stream traffic turning right or left into the mainstream flow.

8.5.8.3 RESTRICTION BY LENGTH OF STORAGE LANE

Figure 8.16 is an example of a set intersection controlled by the length of the storage lane. The minimum intersection spacing is determined by the design left-turning traffic volume per cycle. In this case, it is possible to determine the required intersection spacing for each case by following the left-turn lane design method, however, it is not possible to prescribe a uniform minimum spacing.

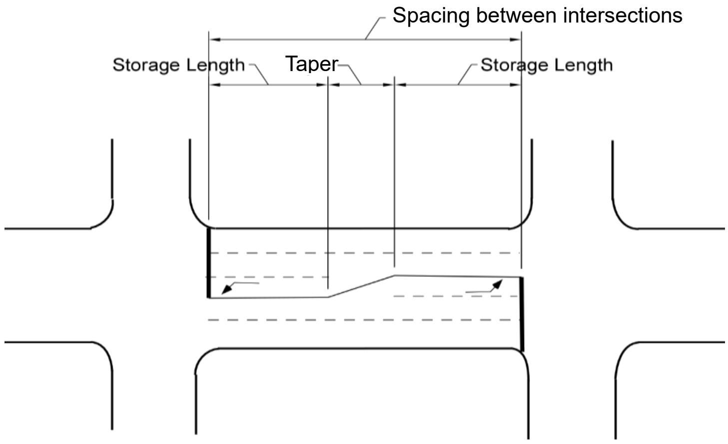


Figure 8.16 Restriction by length of storage lane

8.6 VISIBILITY

8.6.1 SIGHT DISTANCE

Two (2) factors are important in the provision of adequate sight distance to ensure safe and smooth passage of vehicles through intersections.

i. The driver should be able to recognize at a safe distance the presence of the intersection and its control facilities in advance.
ii. **There should be adequate visibility within the intersection itself.**

8.6.1.1 SIGHT DISTANCE FOR BOTH SIGNAL AND NON-SIGNAL CONTROLLED INTERSECTIONS

In order for vehicles to pass safely and smoothly, it is necessary to ensure that the sight distance is adequate for the road. The same principle applies to at-grade intersections, where the sight distance of such road must be secured. In addition, at at-grade intersection, the driver has more information to pay attention to and more actions to take than at a single section of the road, such as deciding whether to go straight or turn right or left at the intersections, or to stop following the car in front.

Therefore, it is necessary to be able to confirm the existence of intersections, traffic signals, stopped vehicles, pedestrians and others from a reasonable distance before the intersection. When a driver passes through an intersection, the driver's behaviour at the intersection is governed by the traffic control method of the intersection. The sight distance at signal controlled and stop sign-controlled intersections should not be less than the values given in **Table 8.5**.

8.6.1.1.1 INTERSECTIONS CONTROLLED BY SIGNAL

In the case of signal control, which is the most common method of traffic control in at-grade intersections, obviously the signals must be clearly visible supplemented by signal traffic signs, however, the issue concerns the minimum distance at which the signals can be seen. This minimum sight distance is deemed to be the sum of the distance that a vehicle travels between the driver seeing the signal and putting on the brakes (judgment distance) and the distance that the vehicle travels between putting on the brakes and stopping before the stop line without feeling any discomfort.

The time between seeing the signal and putting on the brakes (total reaction time) comprises the time required to judge whether or not to apply brakes and the time it takes between deciding to brake and reacting. There is not much study data concerning this total reaction time, however, AASHTO estimates the selective reaction time to be approximately 10 seconds.

Here, urban roads and rural roads are considered separately. In cities, since there are numerous intersections and drivers realize there are signals when they drive, the total reaction time is deemed to be shorter; therefore, a time of 6 seconds is assumed for urban areas and 10 seconds is assumed for rural areas. Moreover, the deceleration rate deemed sufficient not to impart discomfort is 1.96 m/s^2.

Based on the above, the travel distance between sighting a signal and stopping (minimum sight distance) is given by **Equation 8.3**.

$$S = \frac{Vt}{3.6} + \frac{1}{2\alpha}\left(\frac{V}{3.6}\right)^2 \tag{8.3}$$

Where,

S:	minimum sight distance (m)
t:	judgement + reaction time (6s for urban and 10s for rural)
α:	deceleration rate, 0.2g (1.96m/s^2)
V:	design speed (km/h)

8.6.1.1.2 INTERSECTIONS CONTROLLED BY STOP SIGN

In cases where an intersection does not have signal control, it is desirable in terms of traffic processing to clarify the major road and the minor road and install a stop sign on the minor road just before the intersection. As is also the case in intersections with signal control, it must be possible to sight this stop sign from a distance that allows drivers to brake and stop just before the intersection without feeling any discomfort. However, in this case, since no time is needed to make judgment, it is safe to assume that drivers will start braking immediately after confirming the sign.

The time taken between confirmation and braking (reaction time) differs according to the driver, however, AASHTO (2018) gives 2 seconds. The deceleration rate deemed sufficient not to impart discomfort is 0.2g, and the sight distance corresponding to each design speed has been calculated by inserting the values of t = 2s, α ＝1.96 m/s2 (= 0.2g) into **Equation 8.3**. If it is not possible to secure this minimum sight distance, it is necessary to augment the stop sign with a warning sign.

Meanwhile, concerning the major road, it is necessary to secure a sight distance that considers vehicles or pedestrians exiting from the minor road. In this case, it is possible to use the sight distance that is prescribed for road sections of uninterrupted flow. **Table 8.5** shows the minimum sight distance of at-grade intersection.

Table 8.5 Minimum sight distance of at-grade intersections

Design speed (km/h)	Minimum Sight Distance (m)		
	Signal control		Stop Sign Control
	Rural	Urban	Rural and Urban
120	620	480	350
100	480	370	250
80	350	260	170
60	240	170	105
50	190	130	80
40	140	100	55
30	100	70	35
20	60	40	20

8.6.2 VISIBILITY INSIDE INTERSECTION

To ensure that vehicles can safely and easily pass-through intersections, it is necessary for the driver to accurately comprehend the existence and behaviour of other vehicles and pedestrians inside the intersection.

i. In securing visibility inside intersections, it is necessary to pay attention to the following points.
ii. Since there are cases where planting and roadside trees can obstruct visibility around intersections, it is necessary to consider tree species, height, protruding branches, position and spacing. Moreover, to ensure that drivers can see toddlers, children, wheelchair users, etc., plants and trees on median strips, planting zones, etc. close to channelizing islands and intersections should not be higher than 60cm.
iii. When planning bridges, pedestrian overpasses and other structures near at-grade intersections with side roads and crossroads that connect with grade-separated intersections, the positions of bridge piers, etc. must be decided upon fully considering the visibility of oncoming vehicles, motor bikes, tricycles, bicycles, pedestrians, signal lights, signs, etc. from the viewpoint of drivers.
iv. As a rule, when vehicles on a minor road stop at an intersection with stop sign control,

sufficient visibility to see the major road traffic from around the stop line must be secured to ensure that the vehicles can safely cross or merge with the major road.

8.6.3 HORIZONTAL AND VERTICAL ALIGNMENT

Road alignment in the vicinity of at-grade intersection must ensure capacity, safety, and reliability of stop and start operations. The road alignment during planning and design must conform with standards in the uninterrupted section described in **Chapter 7**.

Other points for consideration are as follows:

i. An at-grade intersection should avoid a curve section, cut section of road and bridge approaches.
ii. The gradients at the approaches of an at-grade intersection should be as gentle as possible. Up to 2.5% is desirable.
iii. An at-grade intersection should not be located at the bottom of a sag or apex of a crest of a vertical curve.

If there is no choice but to position an intersection in places as indicated above, planning and design must be implemented to ensure the minimum values given in **Table 8.6** and **Table 8.7** are adopted. Moreover, for at-grade intersections with approach lane(s) close to the minimum values, it is desirable to install warning signs or signals to give notice of the at-grade intersections.

8.6.3.1 HORIZONTAL ALIGNMENT

The curve radius of the centreline should be greater than or equal to the values given in **Table 8.6,** depending on the control method and design speed of the intersection.

Table 8.6 Minimum curve radius of at-grade intersections

Design speed (km/h)	Minimum Curve Radius (m)		
	Signal & Stop Sign Control		Stop Sign Control
	Major Road		Minor Road
	Desirable	Absolute	
120	710	570	280
100	160	380	190
80	280	230	110
60	150	120	60
50	100	80	40
40	60	50	30
30	30	-	15
20	15	-	15

When an at-grade intersection is planned in a curve section, from the viewpoint of ensuring the safe and smooth passage of vehicles, it is necessary to apply superelevation to the road according to the curve radius and design speed. However, when an at-grade intersection is planned in a curve section with a large superelevation, problems arise concerning the safety of stopping and turning vehicles, height disparity with roadside areas making it difficult to access the major road. In cases of signal-controlled intersections, problems arise on the crossing road in terms of the safety and comfort of straight-moving vehicles. It is not appropriate to plan at-grade intersections with these problems, and such cases must be avoided in principle. On the other hand, changing the superelevation within the intersection to solve the above issues will break the balance and affect the safety of the vehicular traffic. Moreover, at stop sign controlled intersections, from the viewpoint of ensuring the safe and smooth passage of vehicles, the specified superelevation should be applied to the major road.

8.6.3.2 VERTICAL ALIGNMENT

The vertical gradient at approaches to at-grade intersections should be gentle and not more than 2.5% for as long as possible as permitted by the roadside conditions to ensure the safe and smooth passage of vehicles. The minimum length of the gradient is the product of the number of incoming vehicles per lane per cycle (per minute at a stop-controlled intersection) and the average interval between vehicles. Even if this length cannot be attained due to the terrain and other constraints, values not less than those shown in **Table 8.7** should be adopted.

Table 8.7 Minimum section length at approaches to at-grade intersections

Design Class	Minimum section length (m)
A1 & A2	40
B1	40
B2	15
C1	35
C2	15
D1	15
D2	10
E	6

Modification of the vertical gradient to a gentle slope at approaches to at-grade intersections would not only contribute to improving visibility at intersections, but also improve the efficiency, safety of stop and start operations as well as preventing the decline of traffic capacity.

8.7 CROSS SECTION COMPOSITION AROUND AT-GRADE INTERSECTIONS

8.7.1 LANE WIDTH AND NUMBER OF LANES

8.7.1.1 LANE WIDTH

Generally, the lane widths are decided from the viewpoint of traffic safety, capacity, and comfort. The recommended lane widths are shown in **Table 8.8.**

Table 8.8 Lane width of carriageway at intersections

<table>
<tr><th>Classification</th><th>Straight through lane width (m)</th><th>Added lane width (m)</th></tr>
<tr><td>Rural</td><td>Desirable 3.50 or 3.65
Standard 3.00
Minimum 2.75</td><td rowspan="2">Desirable 3.25
Standard 3.00
Minimum 2.75</td></tr>
<tr><td>Urban</td><td>Desirable 3.25
Standard 3.00
Minimum 2.75</td></tr>
</table>

On existing roads, it is often very difficult to secure the necessary additional land due to roadside constraints, and when such constraints exist, it is sometimes difficult to provide auxiliary lanes with the above widths, even if the stopping zone, median strip and other cross section composition elements are downsized as much as possible.

Therefore, on urban existing at-grade intersections, rather than applying uniform widths, it is allowed to adopt a flexible approach to application according to the road conditions, traffic conditions and other conditions surrounding the intersection concerned.

8.7.1.1.1 WIDTH OF THROUGH LANE WHERE AUXILIARY LANE IS PROVIDED

It is desirable to make the width of lanes at approaches to at-grade intersections the same width as uninterrupted sections. However, when adding auxiliary lanes or through lanes to at-grade intersections in built-up areas, difficulties often arise in securing land due to the existence of buildings or other reasons around the road. Therefore, around intersections, it is permitted to reduce the width of through lanes by 0.25m upon assuming that the design speed can be reduced by 20km/h. This is because, around intersections, it is desirable to provide auxiliary lanes, even if this means downsizing the through lanes, planting zones, stopping lanes, median strips and others.

8.7.1.1.2 WIDTH OF AUXILIARY LANE

The standard width of auxiliary lanes is 3.0m, however, on urban left-turn lanes, it is permitted to reduce the width to 2.75m in cases where there is little mixing of large vehicles and the standard width cannot be provided even after reducing the cross-sectional elements such as through lanes, stopping lanes, median strips and others.

8.7.1.1.3 POINTS TO CONSIDER REGARDING CROSS SECTION COMPOSITION ELEMENTS AROUND AT-GRADE INTERSECTIONS

When setting cross section composition elements around intersections, it is necessary to consider the following principles concerning safety and comfort.

i. In the vicinity of intersections, vehicles should be discouraged from stopping as much as possible, for example, by not having a parking zone or enough shoulder width to stop/park.

ii. On intersections where the crossing distance for pedestrians is long, since there are cases where pedestrians cannot cross the entire road during a single green phase, sufficient median strip width or refuge island should be provided to enable pedestrians to standby at the midway point.

iii. Since pedestrians tend to gather around at-grade intersections and spaces are required for the stopping and staying of pedestrians, endeavour not to reduce the width of walkways as much as possible.

iv. Bicycle lanes should not be reduced as much as possible around at-grade intersections due to difficulty in securing land for the provision of additional lanes.

v. Where the minor road is under STOP or YIELD control, the crown of the major road is typically carried through the intersection. Meeting this major road cross-section can result in minor road grades near the intersection that are steeper than that which would occur with the major street crown removed. At intersections where the major road retains the crown through the intersection, the minor road crown is gradually reduced, typically starting at the beginning of the approach grade, and completed slightly outside the intersection.

vi. At intersections with signal control, it is customary to remove the crown from both the major and minor roads. This removal of the crown is advisable for the comfort and safety of motor vehicle drivers and bicyclists proceeding, on either road, at the design speed through a green signal indication. At intersections with all-way STOP control, it may be desirable to remove the crown from both intersecting roads, to emphasize that all approaches are equal in terms of their traffic control.

8.7.1.2 SECURING OF LEFT-TURN LANE WIDTH IN A CONSTRAINT AREA

The separation of left-turning vehicles plays an important role in handling traffic through an intersection. In cases where sufficient width cannot be secured for a left-turn lane due to various constraints on an existing road, a width of 1.7m which corresponds to a left-turn lane can be provided. It is sufficient to simply provide a bulge of 1.7m or more without having any indication of a boundary with the through lane. A typical example of bulge of left-turn lane is shown in **Figure 8.17**.

For the length of the bulge ($L=L_t+L_d$) refer to **Section 8.8.1**.

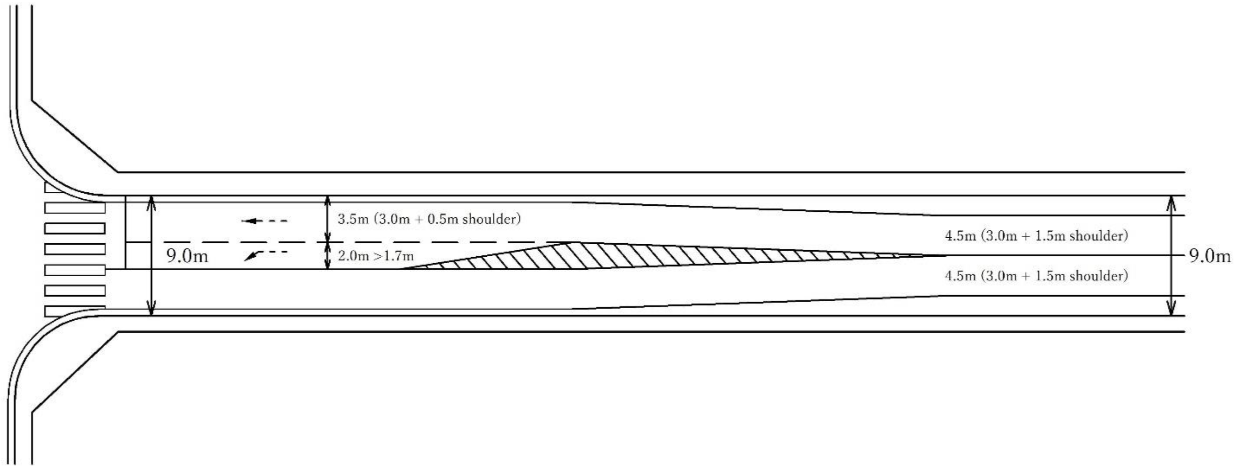


Figure 8.17 Example of Bulge of Left-Turn Lane

8.7.1.3 NUMBER OF LANES

It is necessary to appropriately determine the number of lanes at an intersection based on the traffic capacity of the intersection. The number of lanes at the departure must be the same or more than the number of through lanes at the approach (the number remaining after subtracting right-turn lanes and left-turn lanes from the total number of lanes at the approach). In this regard any excess through lane at the approach can be used as turning lanes to ensure lane balancing.

The departure lanes must be in line with the through lanes on the approach. In cases where through lanes are shifted at the approach to allow for the establishment of a left-turn lane as shown in **Figure 8.18**, lanes at the departure should be an extension of the through lanes from the approach.

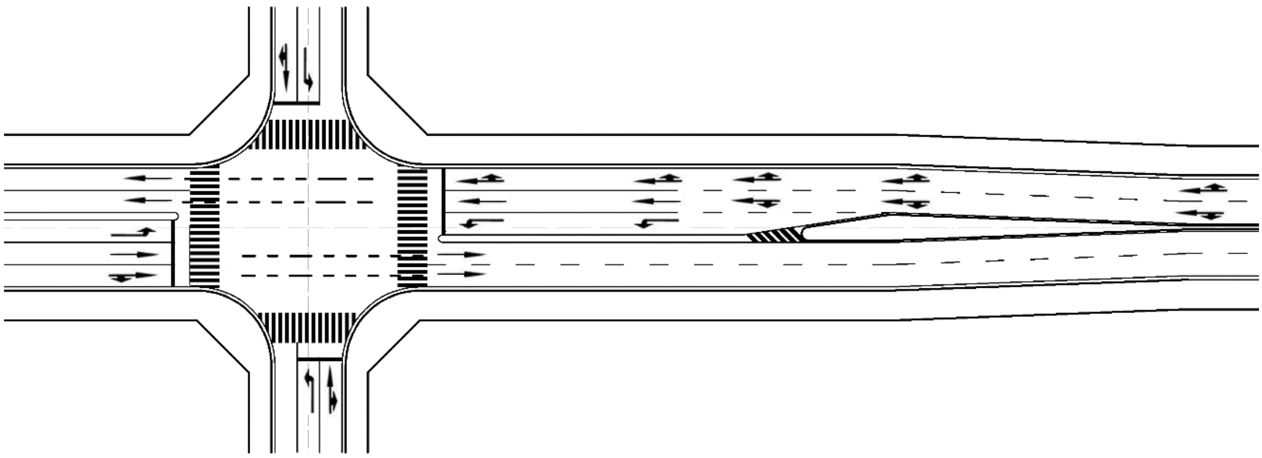


Figure 8.18 Exit Straight Lanes in Line with Shifted Approach Lane

8.7.2 LANE SHIFT RUN-OFF

The through lane may be shifted to create a left turn lane by creating a lane shift run-off (L_t).

The length of the lane shift run-off is obtained from the design speed of the road, location of the at-grade intersection (urban or a rural area), and the horizontal alignment.

In the case of a curve section as shown in **Figure 8.19**, conditions vary depending on the curve radius, however, since it is not desirable to construct a reverse curve (S-curve), it is generally easier to make a runoff in a curve than on straight sections.

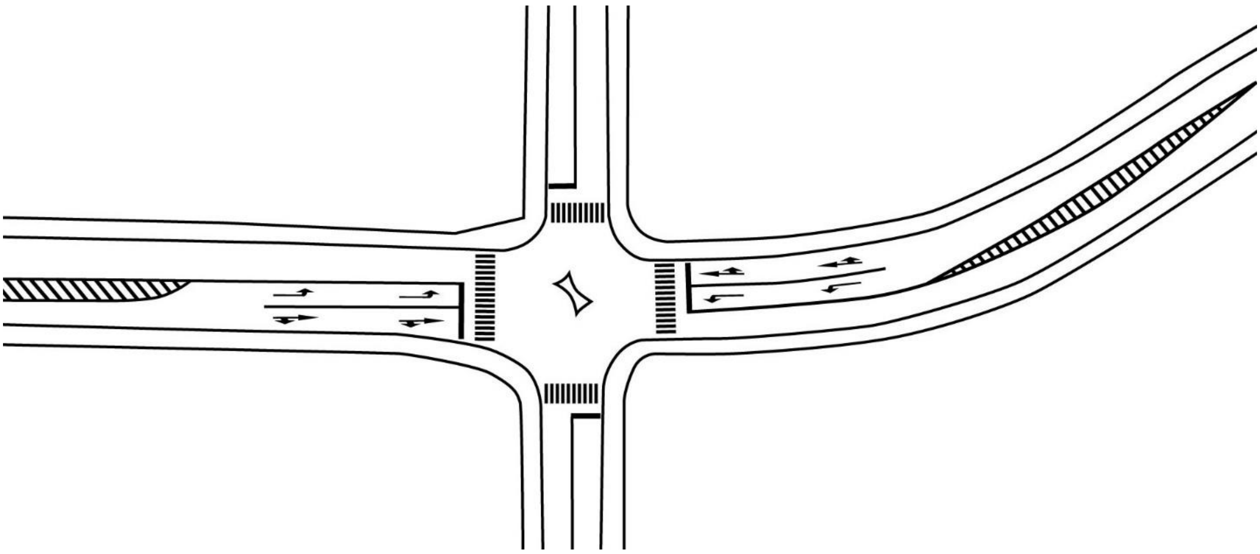


Figure 8.19 Runoff in a curve section

When shifting the through lane on a straight section, the length of the lane shift run-off (L_t) is obtained by adopting the greater of the equation and the minimum value as given in

Table 8.9. The lengths of lane shift runoff and left turn lane are shown in **Figure 8.20**.

Table 8.9 Length of main lane shift section (L_t)

Design speed (km/h)	Rural		Urban	
	Equation	Minimum value (m)	Equation	Minimum value (m)
120	V×△W/2	-	V×△W/2	-
100		-		-
80		85		-
60		60	V×△W/3	40
50	V×△W/3	40		35
40		35		30
30		30		25
20		25		20

Where,

V: Design speed(km/h)

ΔW: Maximum shift in the transverse direction (m)

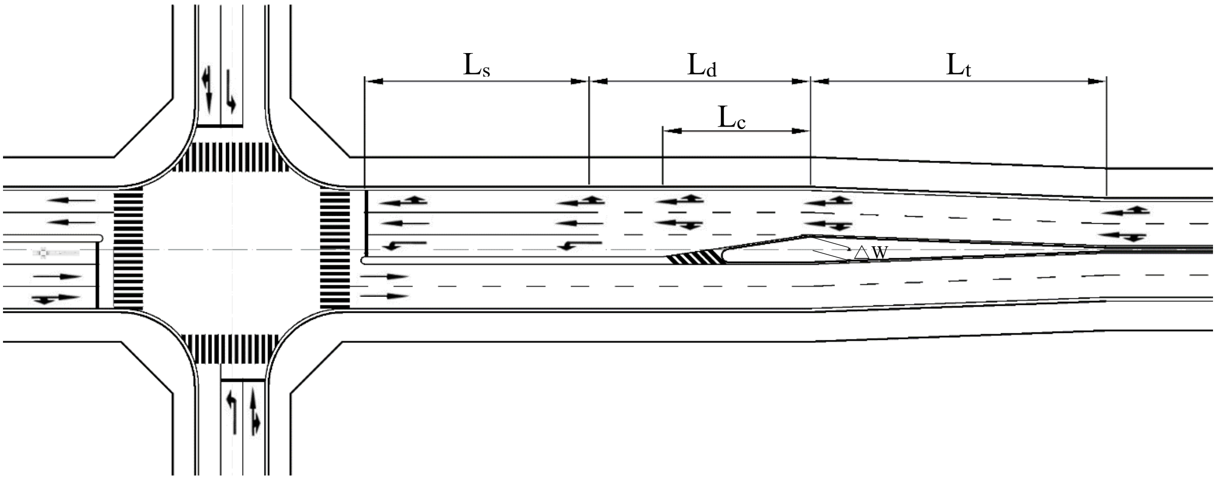


Figure 8.20 Lengths of lane shift runoff and left turn lane

For definition of L_S, L_d and L_C refer to **Section 8.8**.

8.8 AUXILIARY LANES

Auxiliary lane is the portion of the road adjoining the travelled way for speed change, turning, weaving, truck climbing, manoeuvring of entering and leaving traffic, and other purposes supplementary to through-traffic movement.

8.8.1 LEFT-TURN LANE

Left-turn lanes remove stopped or slow-moving left-turning vehicles from the stream of through traffic, eliminating the primary cause of rear-end crashes at intersections. The safety benefits of left-turn lanes increase with the design speed of the road, as they greatly reduce both the incidence and severity of rear-end collisions. Left-turn lanes also improve capacity by freeing the travel lanes for through traffic only. The safety and capacity benefits of left-turn lanes apply to all vehicular traffic, motorized as well as non-motorized. However, left-turn lanes add to the pedestrian crossing distance and pedestrian crossing time. The additional street width needed for left-turn lanes may require land taking or removal of on-street parking.

8.8.1.1 ESTABLISHMENT OF LEFT-TURN LANE

Left turn lanes may be provided at all at-grade intersections because it is effective in preventing accidents and maintaining intersection capacity. Left turn lanes comprising deceleration and storage sections, shall be provided under any of the following conditions:

i. On dual carriageway roads.

ii. When the mainline design speed is 100 km/h or greater and the AADT on the major road in design year 10 is greater than 2000 p.c.u.

iii. When the AADT of the left turning traffic in design year 10 is greater than 800 p.c.u.

iv. **Where intersections are sited on left-hand bends and perception of the intersections for major road traffic would be greatly improved by its inclusion.**

v. On four or more lane undivided highways.

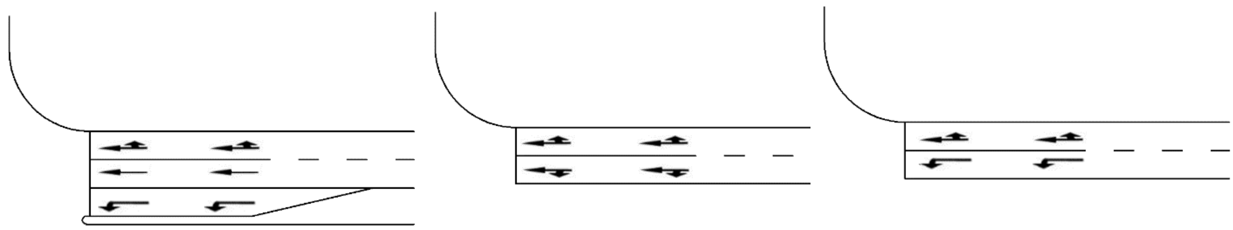
However, left turn lanes may be omitted in the following cases:

i. Where left turning is not permitted

ii. Where there is ample processing capacity on the intersection at the peak traffic hours on both the main and minor road.

iii. Where the road is a 2-lane road with design speed of 40km/h or less and the design traffic volume is less than 200 vehicles/hour, and the ratio of left-turning traffic is less than 20% of the approach design traffic volume.

In special cases where the left-turning traffic becomes the mainstream traffic, left-turn lane(s) must be established separately from the through lane(s) as shown in **Figure 8.21(a)**, while lane(s) on the uninterrupted section (for example, one out of two lanes) must not be used as left-turn lanes as shown in **Figure 8.21(b) and (c)**. Therefore, left-turn lanes cannot be jointly used as through lanes. However, on rural roads, it is desirable to establish left-turn lanes for safety purposes, for example, to prevent rear-end collision accidents by separating the through and left-turning traffics.



(a) Desirable (b) Undesirable (c) Undesirable

Figure 8.21 Desirable and undesirable left turn lane

Road line markings as shown **Figure 8.22** are necessary as a traffic management tools to guide vehicles at intersections.

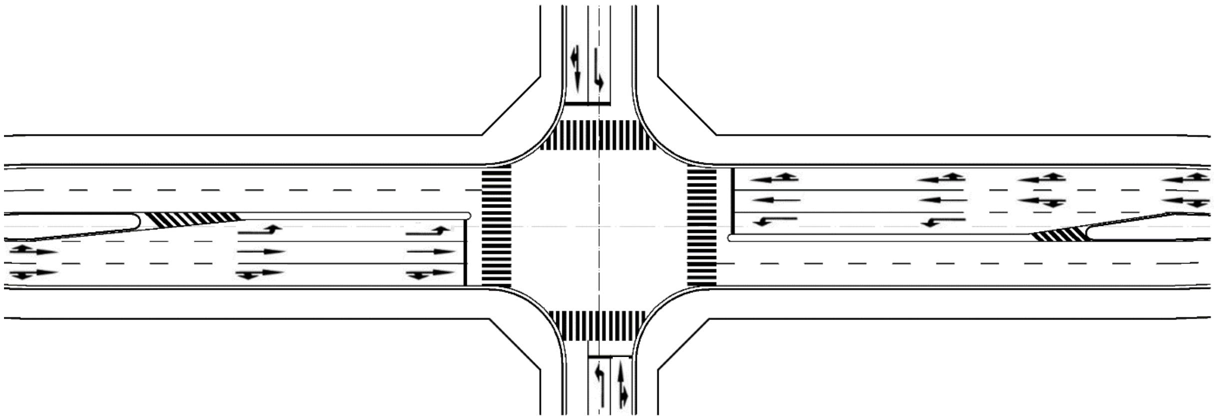


Figure 8.22 Left turn lane marking

8.8.1.2 LENGTH OF LEFT-TURN LANE

The length of the left-turn lane is determined based on the design speed and number of left turning vehicles. The length (L) of a left-turn lane consists of length for deceleration (L_d) and storage length (L_s) as shown in **Figure 8.23. Equation 8.4** is used in computing the length of the left turn lane.

$$L = L_d + L_s \tag{8.4}$$

Where:

L: Length of Left-turn Lane (m)

L_d: Deceleration length (m)

L_s: Storage length(m)

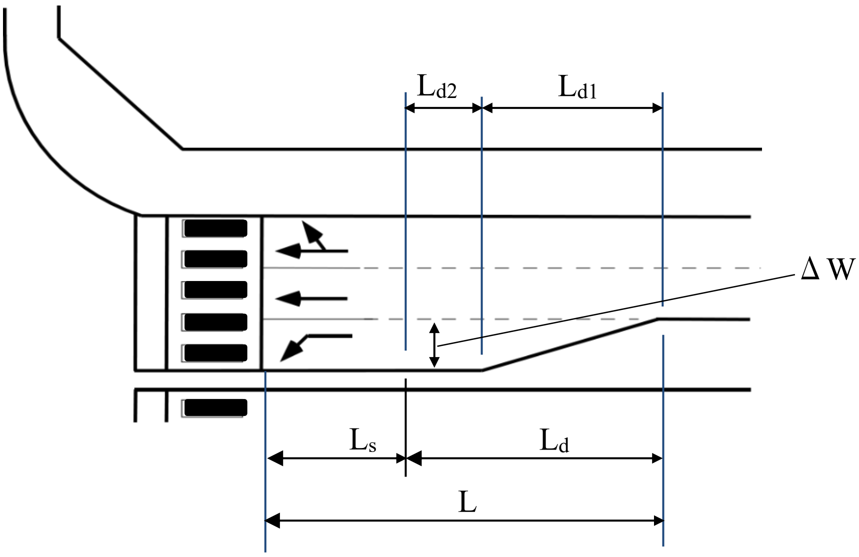


Figure 8.23 Length of Left-turn Lane

The deceleration length (Ld) is the section for deceleration and transitioning of vehicles from the through lane into the left-turn lane. The deceleration length (L_d) is composed of taper length (L_{d1}) and parallel length (L_{d2}). The minimum length for deceleration (L_d) at an at-grade intersection is as indicated in **Table 8.10.**

Table 8.10 Minimum length for deceleration (L_d)

Design speed (km/h)	Rural			Urban		
	Taper length (m) L_{d1}	Parallel length (m) L_{d2}	Deceleration length (m) L_d	Taper length (m) L_{d1}	Parallel length (m) L_{d2}	Deceleration length (m) L_d
120	100	180	280	70	90	160
100	90	100	190	60	30	90
80	70	50	120	50	20	60
60	60	10	70	35	-	35
50	50	-	50	30	-	30
40	50	-	50	25	-	25
30	20	-	20	20	-	20
20	20	-	20	10	-	10

In a situation where the taper width is greater than 3.5m, the taper length (L_{d1}) for transitioning from the through lane to the left-turn lane is given by **Equation 8.5**.

$$L_{d1} = \frac{V \times \Delta W}{6} \quad (8.5)$$

Where,

L_{d1}: Taper length for transitioning into the left-turn lane (m).

V : Design speed (km/h)

ΔW: Maximum shift in the transverse direction (m) or auxiliary lane width.

The storage length for left turn vehicles with signal control is calculated using **Equation 8.6**:

$$L_S = \lambda\gamma \times N \times S \quad (8.6)$$

Where:

L_s: storage length (m)

$\lambda\gamma$: Left-turn lane length coefficient

N: Average number of left-turning vehicles per cycle (vehicles)

S: Average of the sum of headway and vehicles length (m)

Values of left-turn lane length coefficient are shown in **Table 8.11**. Moreover, in cases where the average number of left-turning vehicles is between two known values in **Table 8.11,** the left-turn lane length coefficient is calculated using interpolation.

Table 8.11 Left-turn Lane Length Coefficient (λγ)

Average number of left-turning vehicles(veh/cycle)	2 or less	3	5	8	10 or more
Left-turn lane length coefficient (λy)	2.2	2.0	1.8	1.6	1.5

On at-grade intersections that have no signal control, L_s is obtained from **Equation 8.7** upon considering fluctuations in traffic volume:

$$L_S = 2 \times \mathrm{M} \times S \tag{8.7}$$

Where:

L_s: storage length (m)

M: Average number of left-turning vehicles per minute (vehicles)

S: Average of the sum of headway and vehicles length (m)

S is a weighted average and takes account of the proportion of small and large vehicles using 5m for small vehicles and 12m for large vehicles. If the large-size vehicles ratio is unknown, S may be set at 7m.

The minimum length (L_S) for storing left-turning vehicles should not be less than the value obtained through **Equation (8.6)** and **(8.7)** above, and it is desirable to secure a length not less than 30m.

The minimum length of the left turn lane on roads with strict travel conditions is therefore given as the sum of L_d and L_S.

Conversely, in urban areas that are subjected to strict conditions, it is often the case that the above minimum length of the left-turn lane cannot be secured. If the length of the left-turn lane is shortened, the frequency and extent of obstruction to through traffic caused by the left-turning stored vehicles will increase. However, under fluctuating traffic conditions, even if the length of a left-turn lane is shorter than the values calculated above, there is still something of an effect.

Therefore, even if values are below the results of calculation, the largest length of the left-turn lane that is permitted by the conditions at hand should be adopted. When reducing the length of a left-turn lane, the reduction should be done on the taper length first, while the storage length (L_s) is secured as much as possible. Refer to **Appendix F** for further information on design of left-turn lanes.

8.8.1.3 LEFT-TURN CHANNEL MARKING

Inside intersections, left-turn channel should be appropriately indicated by road line markings to guide the left-turning traffic as shown in **Figure 8.24.**

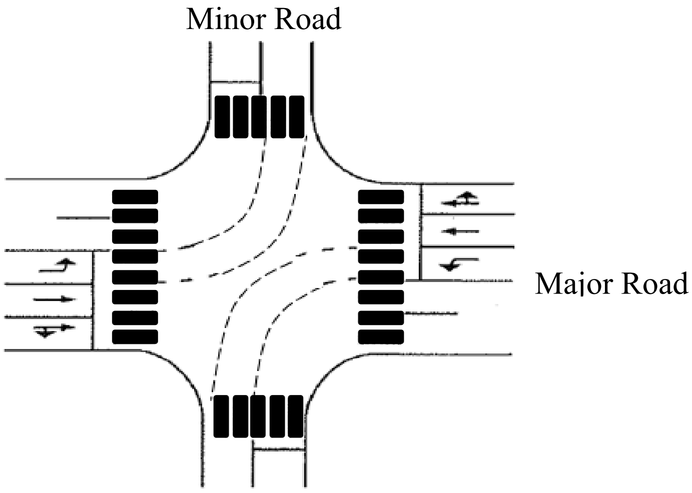


Figure 8.24 Left-turn channel marking

8.8.1.4 LEFT-TURN LANE AT A NEW AT-GRADE INTERSECTION

In planning and designing a new intersection, whether in urban or in rural areas, the forecast of the left-turning traffic volume is based on the following:

i. Land use of surrounding areas and crossroads
ii. Sources of traffic generation

However, it is not easy to accurately predict future traffic volume due to the fluctuations of the above two factors. After a new intersection goes into service, it is necessary to monitor the general traffic conditions and make corrections in the planning and design.

The adequacy of the length of the left turn lane is a major check point during the monitoring stage, hence enough space should be secured to make improvements to the length of the left-turn lane as and when necessary.

On a dual carriageway with plans for future expansion, often two (2) lanes on each side goes into provisional service. It is appropriate at this stage to survey the left turning traffic volume and conduct other data gathering and analysis geared to optimizing the planning and design of at-grade intersections.

8.8.1.5 LEFT-TURN WITH MULTIPLE LANES

When the left-turning traffic volume is heavy, it is advisable to plan a left-turn with 2 or more lanes. In such a case, the lane length is obtained by dividing the required storage length (L_s) calculated for a single left-turn lane by the number of left-turn lanes.

When there are multiple left-turn lanes, it is necessary to pay attention to separating the left-turning traffic from the opposing through traffic and establishing a facility to separate the traffic streams (e.g., median strip).

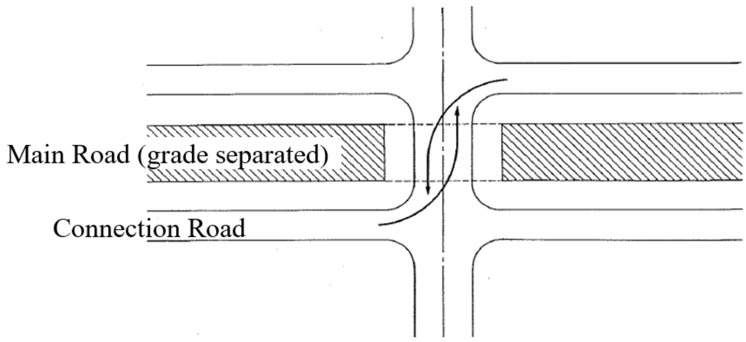
At a signalized intersection, it is desirable to design a traffic phase to ensure the separation of the left-turning and the through traffic. Also, the number of lanes on the departure side must not be less than the number of left-turn lanes on the approach side.

8.8.1.6 LEFT-TURN LANE AT GRADE SEPARATED INTERSECTION

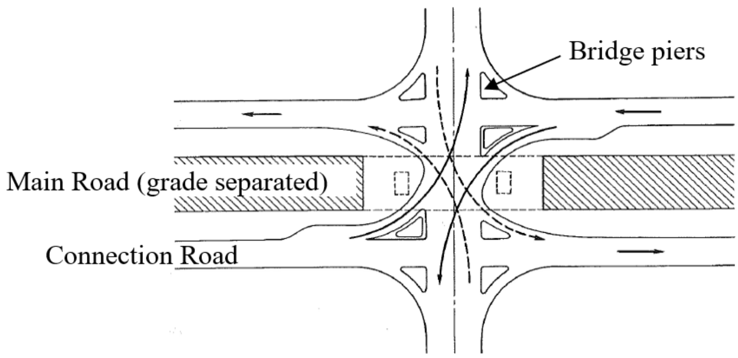
When a layout as shown in **Figure 8.25(a)** is adopted for an intersection with a wide median strip under a grade-separated intersection, problems arise in terms of safety and the capacity of the intersection. At such an intersection, there is crossing of the left-turning traffic from the connecting road, resulting in increased risk of vehicular collision and longer clearance time.

On the contrary as shown in **Figure 8.25 (b)**, it is possible to enhance safety and capacity by separating the left-turn lanes before the intersection (separation of through and left-turning traffic, rearrangement of vehicular traffic and manoeuvres, shortening of clearance time).

Adopting **Figure 8.25 (b)** increases the span between the bridge piers and the length of the intersection approach sections. It is therefore necessary to select the appropriate type upon examining the traffic characteristics, adjoining land use and cost implications.



(a) Method for handling left-turning traffic on the outside



(b) Method for handling left-turning traffic on the inside

Figure 8.25 Left-turn lanes under a Grade-separated Intersection with wide median

8.8.2 RIGHT-TURN LANES

The right-turn lanes and right-turn channels are provided under any of the following conditions:

i. Intersection angle less than or equal to 60°

ii. High volume of right turning traffic

iii. Running speed of right-turning traffic is high

iv. High volume of pedestrians at the right turning exit

v. Other cases (where required)

8.8.2.1 LENGTH OF RIGHT-TURN LANE

The length of the right-turn lane is determined based on the design speed and number of stored vehicles. As in the case of left-turn lanes, right-turn lanes must be established independently of the mainstream lane (through lane).

As shown in **Figure 8.26,** the length of the right-turn lane (L) consists of the length necessary for deceleration (L_d) and storage length (L_S), both of which are determined according to the same procedure described for the length of the left-turn lane. Right-turn channels can be used independently or at the exit of right-turn lanes and deceleration lanes. The calculation of length of right-turn is the same as for left-turn lanes already discussed.

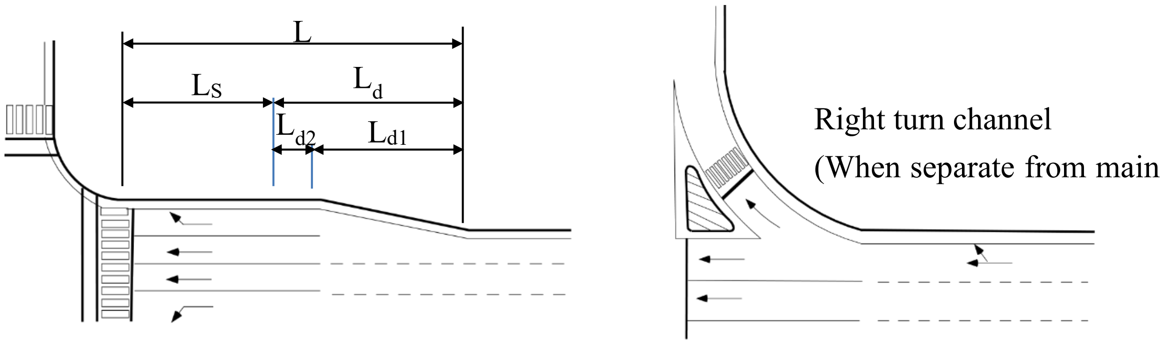


Figure 8.26 Right-turn lanes

8.8.3 SPEED CHANGE LANE

Speed change lane is a separate lane for the purpose of enabling a vehicle entering or leaving a roadway to increase or decrease its speed to a rate at which it can safely merge with or diverge from through traffic. The length of a speed change lane (L) consists of the parallel length (L_p) and taper length (L_T) for deceleration or acceleration as shown in **Figure 8.27.**

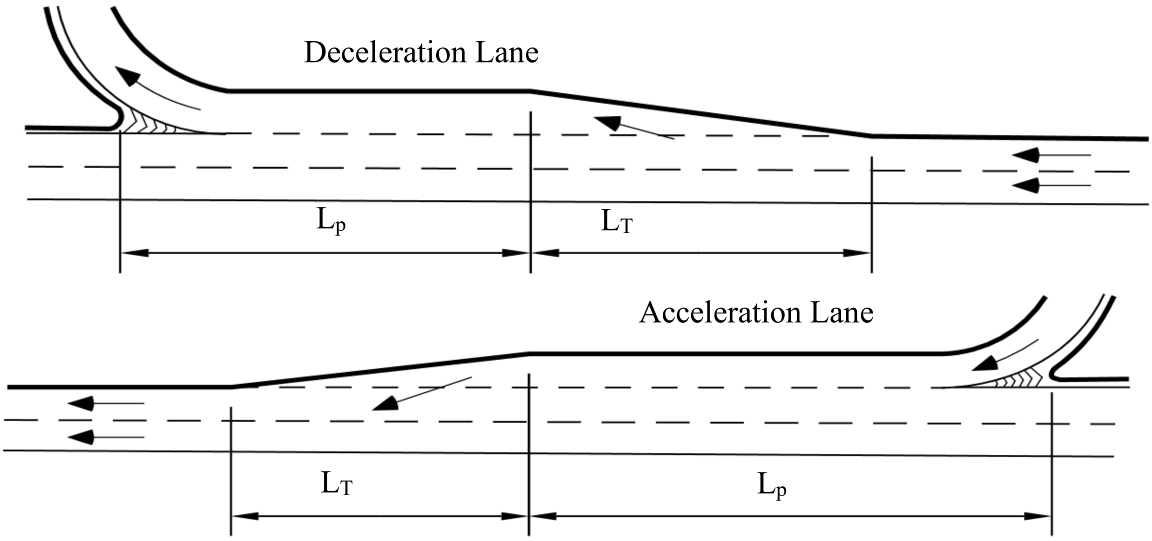


Figure 8.27 Speed Change Lane

A deceleration lane should be provided in the following cases:

i. Where traffic decelerates and diverges from a class A & B road.
ii. Where traffic decelerates and diverges from a road that has heavy traffic volume subject to partial access control.
iii. Other cases that are deemed necessary.

An acceleration lane should be provided in the following cases:

i. Where traffic accelerates and merges with a class A & B road.
ii. Where traffic accelerates and merges with a road that has heavy traffic volume subject to partial access control.
iii. Other cases that are deemed necessary.

The length of the speed change lane differs according to the character of the road, the difference in design speed between the main lane and speed change lane. Desirable minimum values of the parallel length (L_p) and Taper Length (L_T) are provided in **Table 8.12 and Table 8.13**.

In a situation where the taper width is greater than 3.5m, the taper length ($L_T = L_{d1}$) is given by **Equation 8.5**.

Table 8.12 Length of Deceleration Lane (L)

Design speed (km/h)	Rural			Urban		
	Parallel length, L_P (m)	Taper length, L_T (m)	Length of Deceleration Lane, L (m)	Parallel length, L_P (m)	Taper length, L_T (m)	Length of Deceleration Lane, L (m)
120	190	100	290	110	70	180
100	110	90	200	80	60	140
80	80	70	150	50	45	95
60	80	60	140	30	20	50
50	60	50	110	20	20	40
40	50	40	90	20	15	35
30	30	20	50	20	10	30
20	20	20	40	10	10	20

Table 8.13 Length of Acceleration Lane (L)

Design speed (km/h)	Rural			Urban		
	Parallel length, L_P (m)	Taper length, L_T (m)	Length of Acceleration Lane, L (m)	Parallel length, L_P (m)	Taper length, L_T (m)	Length of Acceleration Lane, L (m)
120	370	100	470	250	70	320
100	280	90	370	190	60	250
80	140	70	210	90	50	140
60	100	60	160	65	35	100
50	40	50	90	40	30	70
40	30	50	80	25	25	50
30	20	20	40	20	10	30
20	10	20	30	10	10	20

The lengths prescribed here are primarily intended for speed change lanes provided for at-grade intersections.

The speed change lane should be visible to drivers from an appreciable distance. It is therefore necessary to pay attention to the vertical and horizontal alignments of the mainline around the speed change lane. Also, to ensure smooth traffic handling around speed change lanes, parking lane/layby should not be positioned near or in a speed change lane.

8.9 CHANNEL, ISLANDS AND CORNER CUT - OFF

Channelization is the separation or regulation of conflicting traffic movements into definite paths of travel by islands or pavement marking to facilitate the orderly movements of both motorised and non-motorised traffic. Proper channelization increases capacity and provides positive guidance to motorists. Improper channelization has the opposite effect and may be worse than none.

A traffic island is a solid or painted object in a road that channels traffic. It can also be a narrow strip of island between roads that intersect at an acute angle. If the island uses road markings only, without raised kerbs or other physical obstructions, it is called a painted island or ghost island.

A corner cut-off is an area of land where two sides of a corner of road meet to create an open space to secure refuge space of pedestrians or sight distance into the intersection. In the corner cut-off, it is not allowed to construct facilities since it is part of the road space.

8.9.1 CHANNEL

In designing a channel, some factors form a basis in selecting a suitable curve radius, width of road, and the diverging or merging angle. These factors include the following:

i. **anticipated speed of vehicles**
ii. traffic volume
iii. traffic regulation method
iv. the presence of other road users (pedestrians, cyclists etc.)
v. Design vehicle
vi. Intersection angle

If the intersection gap and intervals between channels are too wide, traffic flows will be disturbed when there is heavy traffic volume. Therefore, it is necessary to adopt appropriate channel widths, avoiding the temptation to needlessly add channels or make them too wide just because there is a lot of space. Reduction in the size of intersection will:

i. Enhance safety and comfort
ii. Shorten crossing times and distances for pedestrian
iii. Reduce loss time leading to increased capacity of the intersection.

Therefore, reducing the size of intersections and guide traffic flows orderly, it is advisable to concentrate on channels as much as possible.

8.9.1.1 CURVE RADIUS OF CHANNEL

For right-turn channels in areas where there are no land constraints, the curve radius is determined based on the design speed of the mainline. In cases where the channel is independent of the mainline and becomes a right-turn road, superelevation shall be placed. The criteria for the superelevation shall be in accordance with the provisions for ramps in **Section 9.4.5.4**. However, if a pedestrian crossing is to be provided on a right-turn road, careful considerations must be given to the value of the superelevation, visibility, safety, and comfort of pedestrians.

Left-turning traffic will have to make a temporary stop and go through curves at very slow speed. When the turning angle is close to 90°, the left turn channels can be designed to have an outer radius of 15-30m. Since the inside curve will start at approximately 5m in front, the tangent length should be extended by 5m from the inside to the outside as shown in **Figure 8.28**.

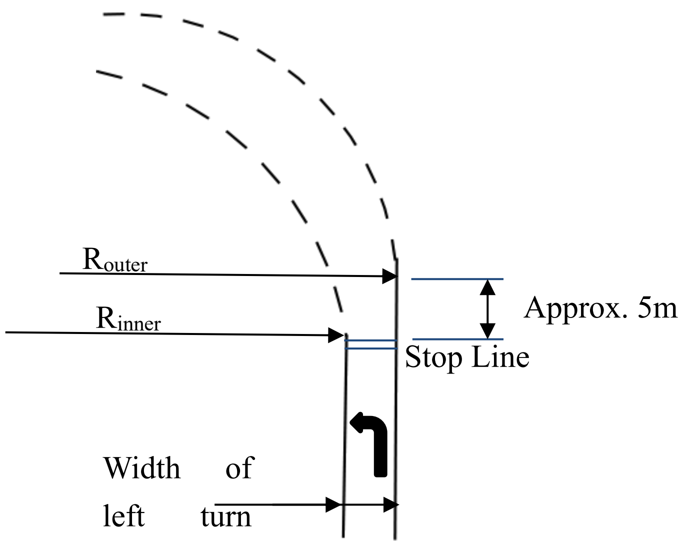


Figure 8.28 Design of Left-turn Channel

8.9.1.2 CHANNEL WIDTH

The width at the middle of the channel is usually wider than at the entry and exit to be able to contain the swept path of vehicles. The recommended widths based on design vehicle and curve radius are shown in **Table 8.14**.

Table 8.14 Channel outer radius and width

Outer Radius of Channel (m)		Width (m)	
More than	**Less than**	**Trailer**	**Other vehicles**
13	14	8.5	5.5
14	15	8.0	
15	16	7.5	5.0
16	17	7.0	
17	19	6.5	
19	21	6.0	4.5
21	25	5.5	
25	30	5.0	4.0
30	40	4.5	
40	60	4.0	3.5
60		3.5	

In cases where a channel is separated by a traffic island, borderline with the same pavement type as the carriageway are added on both sides (refer to **Section 6.4.4** for border line width). These borderlines can also serve as the necessary width for shoulders, street gutters and channel setbacks. The widening runoff should be introduced on the inner side. In such cases, a clothoid curve or a simple curve is used as the runoff curve. If a simple curve is adopted, a radius equivalent to 3-4 times the inner radius of the channel should be used.

8.9.1.3 CHANNEL DESIGN METHOD

The following steps outline the procedure for channel design as shown in **Figure 8.29.**

i. Determine the direction XYX' and DCD' of the approach and exit sections based on the lane consistence. {**Figure 8.29(a)**}
ii. Determine and draw the radius Ro of the outer edge EE' from the angle of intersection θ, condition of the area, traffic, and the design vehicle. {**Figure 8.29(b)**}
iii. Depending on the value of the outer circle and the design vehicle, the width of the channel (w) is determined from **Table 8.14.** Draw a circle (Ri) offset inwards from the outer circle (Ro) by this width (w). Ro and Ri become concentric circles. Determine the runoff circle Rr (=nRi), which is n times the radius Ri. In general, n should be between 3 to 4. {**Figure 8.29(c)}**
iv. Draw a circle around the centre of the circle (Or) of R and Ri with a radius of Rr - Ri. Let the point of intersection of this circle and a line parallel to DC at a distance of runoff circle (Rr) be O. Also, let the point of intersection at other side be O'. {**Figure**

8.29(d)}

v. Draw a perpendicular line from O to DC and O' to CD'. The intersection points A and A' will be the start and the end point respectively of the runoff circle. {**Figure 8.29(e)**}

vi. Draw an extension line of O and Or, and O' and Or to Ri. The intersection points B and B' becomes the tangent of Rr and Ri. {**Figure 8.29(f)}**

vii. Line connecting points A, B, B' and A' will be the inner curve of turn channel. {**Figure 8.29(g)**}

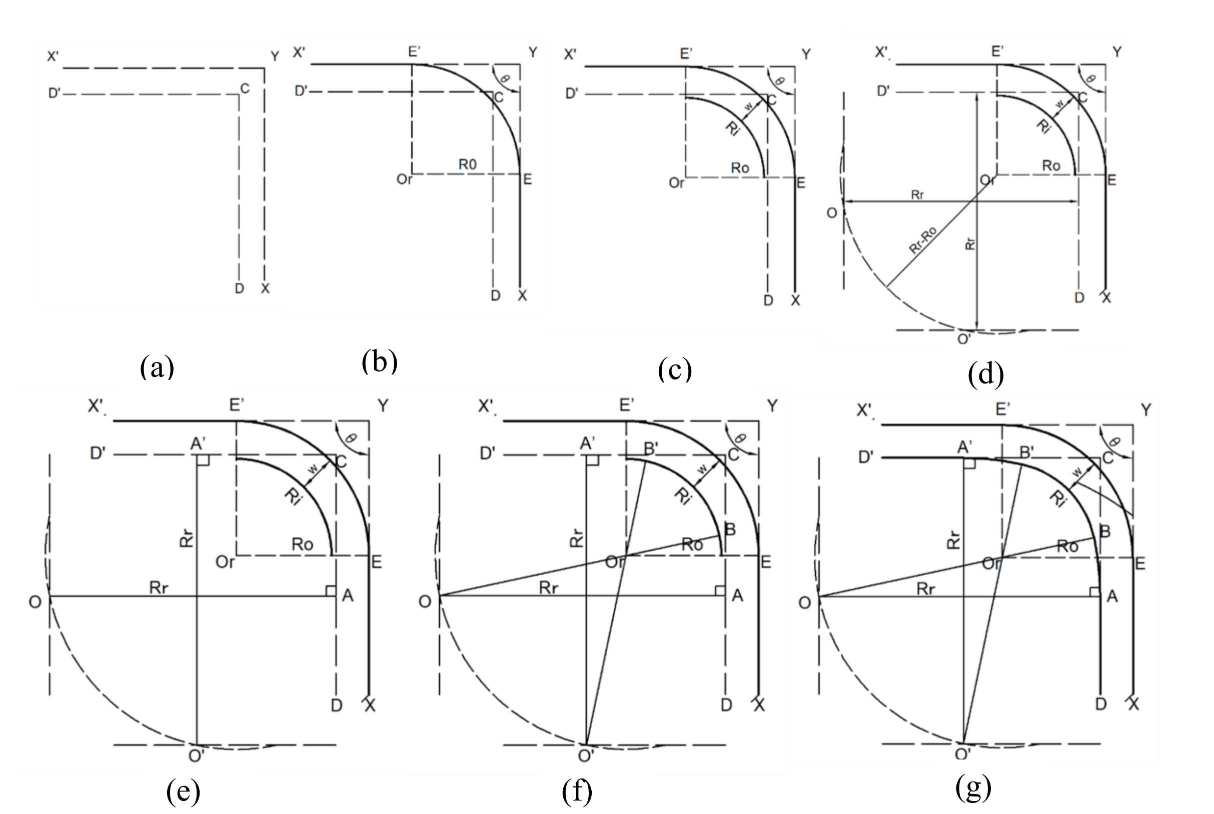


Figure 8.29 Left-turn channel design

8.9.1.4 WIDE CHANNELS

Normally channel width designed to contain the swept paths of trailer is wide enough to permit entry of two or more small cars. To eliminate traffic ambiguity and risk of collisions, the channel must set up a ghost island to restrict multiple entry of small vehicles as shown in **Figure 8.30.**

Except in cases where right-turning vehicles comprise the main traffic flow and multi-lane channels are purposely established, it is necessary to regulate traffic flow by leaving sufficient width for ordinary passenger vehicles to pass in the middle of the channel and placing chevron markings on both sides as shown in **Figure 8.30.**

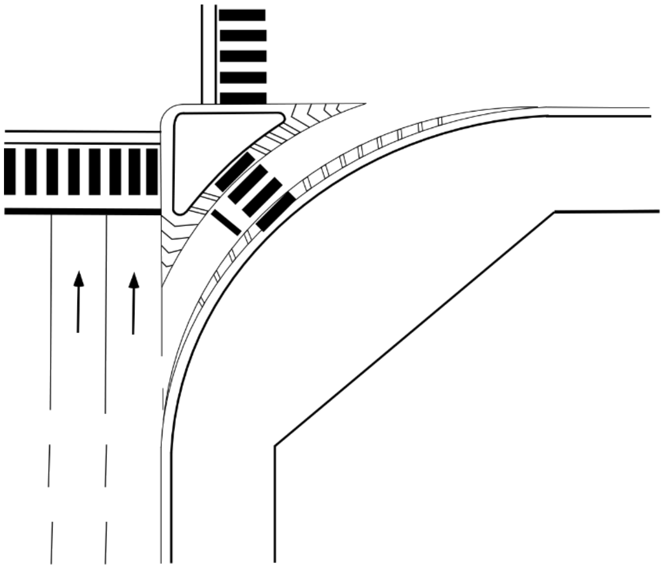


Figure 8.30 Wide Channel

8.9.2 ISLANDS

An island is a defined area between traffic lanes used for control of vehicle movement. Islands also provide an area for pedestrian and cyclist refuge and traffic control devices. Within an intersection, a median or an outer separation is also called an island. This definition makes evident that an island is no single physical type. It may range from an area delineated by a raised kerb to a pavement area marked out by paint or thermoplastic markings. Kerbed islands are safe and mostly preferred. However, marking can be used to create ghost island for the purpose. The latter is cheaper and can provide an additional space for emergency manoeuvres. It can be used where there is space limitation.

The kerb line should be a combination of curve and straight, with a standard height of 12 to 15 cm above the road surface. Higher than that, the driver feels confined, which is undesirable. If possible, grass should be planted over all areas except where people are crossing, to create a contrast with the road surface as shown in **Plate 8.1.**

Plate 8.1 Examples of traffic island

It is also important to indicate the approach ends by means of road line markings. Where traffic entering an intersection is directed into definite path by islands, this design feature is termed a channelized intersection.

Islands are provided for one or more of the following purposes:

i. Separation of conflicts
ii. Control of angle of conflict
iii. Reduction in excessive pavement areas
iv. Regulation of traffic and indication of proper use of intersection
v. Arrangements to favour a predominant turning movement
vi. Protection of pedestrians and cyclists
vii. Protection and storage of turning and crossing vehicles
viii. Location of traffic control devices

In the following cases, even if there is no median strip on the uninterrupted section, it is desirable to provide an island at the approach to an intersection;

i. where roads having design speed of 60km/h or more intersect with each other.
ii. where lots of pedestrians cross the roads and the crossing length is long.

Islands serve three primary functions:

i. Channelization - to control and direct traffic movement, usually turning.
ii. Division - to divide opposing or same direction traffic streams, usually through movements.
iii. Refuge - to provide refuge for pedestrians and cyclist and space for installation of traffic furniture.

Most islands combine two or all of these functions. Islands for channelised right turn lanes typically serve all three functions.

8.9.2.1 ISLAND SIZE

Poor visibility can render small islands dangerous. Islands must therefore be large enough to draw the attention of drivers. The minimum recommended dimensions of islands are indicated in **Table 8.15.** The minimum radius of an island nose should be 0.5m.

Installing many small islands complicates traffic flow and confuses drivers. It is therefore desirable to adopt a small number of large-size islands as much as possible. A small island is not only annoying for the driver, but also dangerous because of the possibility of collisions in a rainy night. If an island is absolutely necessary but cannot be built to the prescribed size due to width or other reasons, it may be replaced by a road marking.

Table 8.15 Minimum dimensions of island

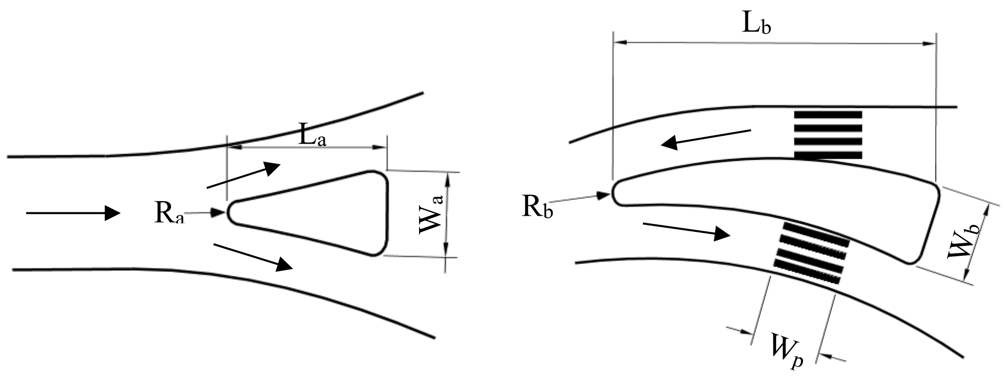
	Diverging Type			**With Pedestrian Crossing**				**With Facility**		**Non-Tapered**
	W_a (m)	**L_a (m)**	**R_a (m)**	**W_b (m)**	**L_b (m)**	**R_b (m)**	**Area (m^2)**	**W_c (m)**	**L_c (m)**	**W_d (m)**
Urban	1.0	3.0	0.5	1.5	(W_p + 1.0)	0.5	5.0	(D + 1.0)	5.0	1.0
Rural	1.5	5.0	0.5	2.0	(W_p+ 1.0)	0.5	7.0	(D + 1.5)	5.0	1.5

Where:

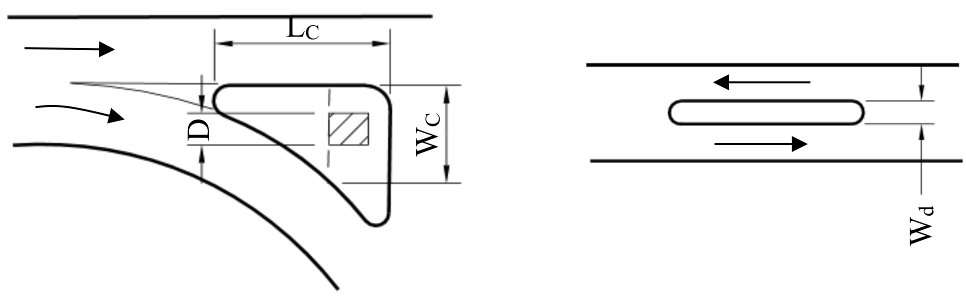
D: Width of facility

Wa, La, Ra, Wb, Lb, Rb. W_p, W_c, L_c, W_d: Given in **Figure 8.31**

W_p: Width of pedestrian crossing



(a) Diverging type (b) With pedestrian crossing



(c) Where facilities are established (d) For non-tapered separation

Figure 8.31 Types of islands

8.9.2.2 ISLAND SHAPE

The shape of an island takes into consideration the lateral clearance to through or turning traffic. As shown in **Figure 8.32**, a nose offset (O_1, O_2) and setback (S_1, S_2, S_3) should be adopted. These values vary according to the design speed, size of the traffic island, whether the road is in an urban area or rural area, and the classification of the road. Recommended values are shown in **Table 8.16** and **Table 8.17**.

Nose offset runoff should be applied for the entire island on both the main line side and channel side. Moreover, in cases where the island is very large, runoff should be applied as 1/10-1/20 on the main line side and 1/5-1/10 on the channel side.

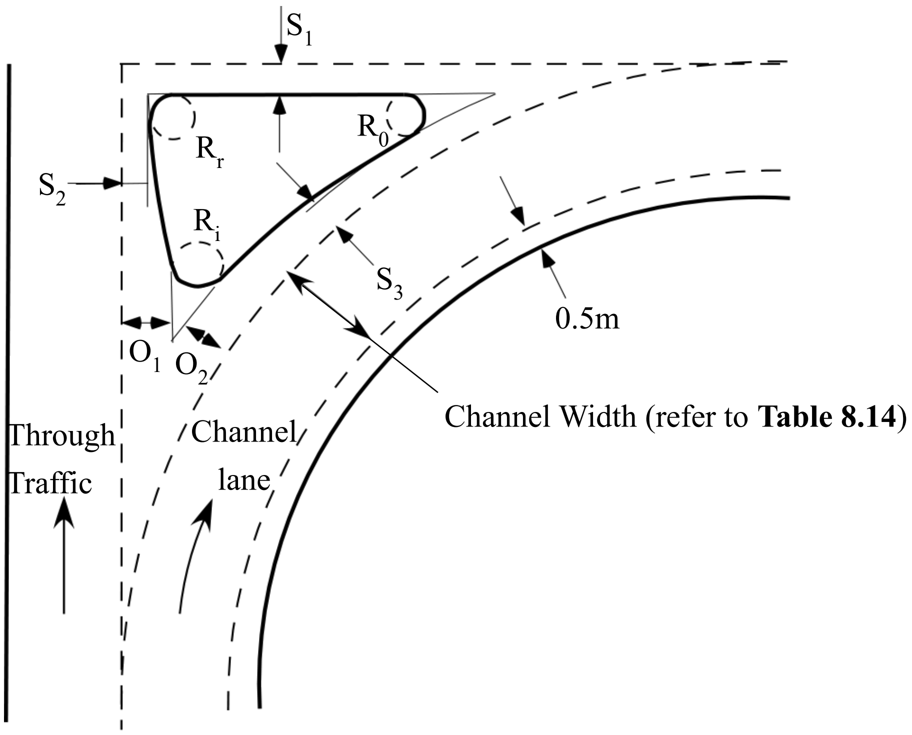


Figure 8.32 Setback and Nose Offset

Table 8.16 Size of Setback and Nose Offset

Design Speed (km/h)	S_1, S_2 (m)	S_3 (m)	O_1 (m)	O_2 (m)
80	1.00	0.50	1.50	1.00
60	0.75	0.50	1.00	0.75
50 or less	0.50	0.50	0.50	0.50

Table 8.17 Radius of nose

R_i (m)	R_0 (m)	R_r (m)
0.50 to 1.00	0.50	0.50 to 1.50

8.9.2.3 NOSE MARKINGS OF ISLANDS

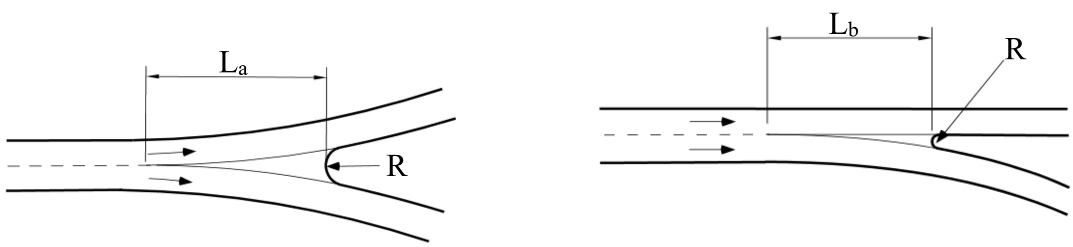
Nose markings at the end of islands are important for road safety. As shown in **Figure 8.33,** the length of the nose marking is calculated based on the design speed (V) and the nose radius (R) using **Equation 8.8** for separating legs and **Equation 8.9** for shifting leg.

$$L_a = \frac{1}{3}VR \tag{8.8}$$

$$L_b = \frac{2}{3}VR \tag{8.9}$$

Where:

L_a, L_b:	Required taper length of road marking (m)
V:	Design Speed (km/h)
R:	Nose radius (m)



(a) Separating to both sides from the centre (b) Shifting to one side

Figure 8.33 Required taper length of nose marking before island

If the shifting leg enters a minor road **Equation 8.8** is to be used.

8.9.2.4 ESTABLISHMENT OF ISLAND IN AN INTERSECTION WITH LONG CROSSING DISTANCES

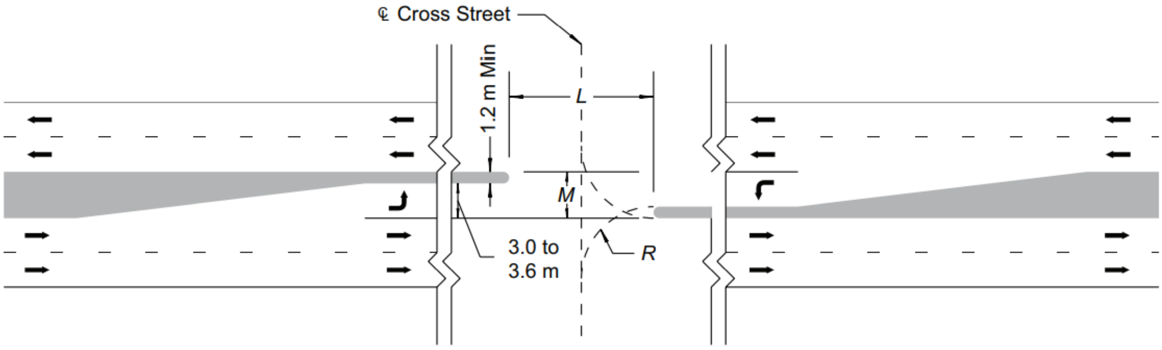
At intersections where pedestrians are to cross long distances, for the sake of pedestrians who cannot cross the entire road during a green phase, establishment of an island should be considered as a refuge in the middle of the carriageway if space is available.

It is necessary to consider the safety of pedestrians by installing guardrails to prevent conflict between pedestrians and vehicles. Moreover, since long crossing distances does not only diminish the intersection capacity but are also not desirable from the viewpoint of pedestrian safety, the basic principle is to shorten crossing distances.

8.9.3 MEDIAN OPENING

8.9.3.1 GENERAL DESIGN CONSIDERATIONS

At at-grade intersection, the median width, the location and length of the opening and the design of the median end are developed in combination to fit the character and volume of through and turning traffic. **Figure 8.34** illustrates the appropriate dimensions for the median width and the length of median opening.



Where:

L= length of median opening (m)

M = median width (4.2m min)

R= control radius conspicuous line marking (m)

Figure 8.34 Median strip left-turn design

Median openings should reflect road or block spacing and the access classification of the carriageway. In some situations, median openings should be eliminated or made directional. Median openings for the exclusive use of pedestrians and bicyclists or emergency vehicles may

be appropriate in some situations.

Spacing of openings should be consistent with access management classifications or criteria. Where the traffic pattern at an intersection shows that nearly all traffic travels through on the divided road and the volume is well below capacity, a median opening of the simplest and least costly design may be sufficient.

The design of a median opening and median ends should be based on traffic volumes, urban/ rural area characteristics, and type of turning vehicles. Crossing and turning traffic should operate in conjunction with the through traffic on the divided carriageway. Design should be based on the volume and composition of all movements occurring simultaneously during the design hours. The design of a median opening becomes a matter of considering what traffic is to be accommodated, choosing the design vehicle to use for layout controls for each cross and turning movement, investigating whether larger vehicles can turn without undue encroachment on adjacent lanes, and finally checking the intersection for capacity. If the capacity is exceeded by the traffic demand, the design should be expanded, possibly by widening or otherwise adjusting widths for certain movements.

Urban/rural characteristics may influence the median width selected. Intersections with narrow medians in urban areas have been found to operate with lower crash frequencies than wider medians, while unsignalized intersections with wider medians in rural areas have been found to operate with lower crash frequencies than narrower medians. Research results indicate that medians on arterial road in urban areas should generally be no wider than needed to accommodate the selected left-turn treatments at intersections for vehicles turning onto and off the roadway. By contrast, medians on arterials in rural areas may be as wide as appropriate as long as the major roadways in both directions of travel are visible to drivers on the minor-road approaches. Traffic control devices such as yield signs, stop signs, or traffic signals may be needed to regulate the various movements effectively and improve the effectiveness of operations.

8.9.3.2 CONTROL RADII FOR MINIMUM TURNING PATHS

An important factor in designing median openings is the path of each design vehicle making a minimum left turn at 15 to 25 km/h.

The paths of design vehicles making right turns are given in **Section 5.9**. Designers should refer to turning templates as well as turning path software compatible with CAD systems to evaluate the effects of the turning radii of various design vehicles on a specific median opening design.

The customary intersection on a divided roadway does not have a continuous physical edge of travelled way delineating the left-turn path. Instead, the driver has guides at the beginning and at the end of the left-turn operation:

i. the centreline of an undivided crossroad or the median edge of a divided crossroad and,
ii. the curved median end.

These areas may be sufficiently large to result in erratic manoeuvring by small vehicles, which may interfere with other traffic. To reduce the effective size of the intersection for most motorists, consideration should be given to providing an edge marking corresponding to the desired turning path for passenger cars, while providing sufficient paved area to accommodate the turning path of an occasional large vehicle. Minimizing the median opening length has shown to reduce crashes for divided highways in rural areas; as such, the minimum turning radius for the design vehicle should be used for design purposes.

Encroachments for various design vehicles turning from the divided carriageway occur beyond the two-lane (projected) crossroad edge of travelled way. With wide crossroads this encroachment is within the median opening, but with two-lane crossroads the encroachment may be beyond the median end, particularly with wide medians having a minimum length of opening. As the left turn is completed, the encroachment may be beyond the edge of travelled way for right turns located diagonally opposite the beginning of the left-turn movement off the divided roadway. With wide crossroads this encroachment does not extend beyond the right-turn edge of travelled way, but with two-lane crossroads and narrow medians it may extend beyond. By swinging over a short distance on the divided roadway before beginning the turn, most drivers could pass through these openings and remain on the paved areas. Although this procedure is used extensively, it should be discouraged by using a more expansive design where practical.

Encroachment distances can be lessened by the drivers anticipating the turn and swinging right before turning left, if space is available. This space depends on the median width, the length of opening as governed by the number of lanes on the crossroad, and other limitations such as triangular islands for channelizing right-turn movements. Minimum median openings based on a control radius of 12 m are not well suited for lengths of opening for two-lane crossroads with rural highways because trucks cannot turn left without difficult manoeuvring and encroachment on median ends or outer shoulders, or both, depending on the median width. It may be suitable for wide crossroad travelled ways, but for these cases it is advantageous to use a control radius greater than 12 m, which enables all vehicles to turn at a little greater speed and enables trucks to manoeuvre and turn with less encroachment. Use of a squared or truncated bullet nose design in conjunction with the 16.8m or 13.2m minimum length of opening is beneficial in these minimum situations. Provision of longer tapers not only avoids this somewhat awkward-looking design but also provides for other important objectives as well. This topic is discussed further in **Section 8.9.3.5** on “Design Considerations for Higher Speed Left Turns.”

8.9.3.3 SHAPE OF MEDIAN END

One form of a median end at an opening is a semicircle, which is a simple design that is satisfactory for narrow medians. However, several disadvantages of semi-circular ends for medians greater than 3.0 m in width are widely recognized, and other more desirable shapes are generally used. An alternate median end design that fits the paths of design vehicles is a bullet nose. The bullet nose is formed by two symmetrical portions of control radius arcs and an assumed small radius (e.g., 0.6 m is used, to round the nose). The bullet nose design closely fits the path of the inner rear wheel and results in less intersection pavement and a shorter length of opening than the semi-circular end. These advantages are operational in that the driver of the left-turning vehicle is channelized for a greater portion of the path and has a better guide for the manoeuvre. The elongated median is better positioned to serve as a refuge for pedestrians crossing the divided roadway, if it is of sufficient width.

For medians 1.2 m wide, there is little or no difference between the two forms of median end. For a median width of 3.0 m or more, the bullet nose is superior to the semi-circular end and preferably should be used in design. On successively wider medians, the bullet nose end results in shorter lengths of openings. The ends for medians 2.4 m wide or wider may also take the shape of squared or flattened bullet ends, the flat end being parallel to the crossroad centreline. This shape retains the advantages over semi-circular median ends regardless of the median width because of the channelizing control. The bullet nose curves are such as to position the left-turning vehicles to turn to or from the crossroad centreline, whereas the semi-circular end tends to direct the left-turning movement onto the opposing traffic lane of the crossroad due to vehicle off-tracking. The need for pedestrian refuge within the median should be considered in the selection of a final design.

8.9.3.4 EFFECT OF SKEW

A control radius for design vehicles as the basis for minimum design of median openings results in lengths of openings that increase with the skew angle of the intersection. Although the bullet nose end remains preferable, the skew introduces other variations in the shape of the median end. Several alternate designs that depend on the skew angle, median width, and control radius may be considered.

Semi-circular ends result in very long openings and minor channelizing control for vehicles making a left turn with less than 90 degrees in the turning angle.

A symmetrical bullet nose with curved sides determined by the control radius and point of tangency has little channelizing control for vehicles turning left less than 90 degrees from the divided roadway. An asymmetrical bullet nose has the most positive control and less paved area than the other types of median ends.

In general, median openings longer than 25 m should be avoided, regardless of the skew. Achieving this may call for special channelization, left-turn lanes, or adjustment to reduce the crossroad skew, all of which result in less than the maximum median-opening length.

Preferably, each skewed crossing should be studied separately with turning templates or turning path software compatible with CAD systems to permit the designer to make comparisons and choose the preferred layout. The need for pedestrian refuge within the median should be considered when selecting a final design.

8.9.3.5 DESIGN CONSIDERATIONS FOR HIGHER SPEED LEFT TURNS

Median openings that enable vehicles to turn on minimum paths and at 15 to 25 km/h are adequate for intersections in urban areas and also in rural areas where most major-road traffic for the most part proceeds straight through the intersection and does not make a left-turn manoeuvre. In rural areas, where through-traffic volumes and speeds are high and left-turning movements are frequent, undue interference with through traffic should be avoided by providing median openings that permit turns without encroachment on adjacent lanes. This arrangement would enable turns to be made at speeds greater than the minimum vehicle paths allow and provide space for vehicle protection while turning or stopping. The general pattern for minimum design can be used with larger dimensions.

A variety of median-opening arrangements may be considered that depend on the control dimensions (width of median and width of crossroad or street, or other), the size of the vehicle to be used as a design control, and the need to provide pedestrian refuge within the median.

Median openings having above-minimum control radii and bullet nose median ends are shown in **Table 8.17.** The radii of 30, 50, and 70 m represent minimum radii for turning speeds of 30, 40 and 50 km/h, respectively. The design controls are the three radii R, R_1, and R_2. Radius R is the control radius for the sharpest portion of the turn, R_1 defines the turnoff curve at the median edge, and R_2 is the radius of the tip. When a sufficiently large R_1 is used, vehicles leaving the major road can turn at an acceptable speed and a sizable area inside the inner edge of through-traffic lane between points 1 and 2 may be available for speed change and protection from turning vehicles. Radius R_1 may vary from 25 to 120 m or more. Dimension B is the offset of the beginning of R_1 for the passenger cars to the target lane line of the crossroad. The radii shown in Figure 8.35 will vary depending on the maximum superelevation rate selected. In this case, the ease of turning probably is more significant than the turning speeds because the vehicle will need to slow down to about 15 to 25 km/h at the sharp part of the turn or may need to stop at the crossroad. Radius R_2 can vary considerably, but is pleasing in proportion and appearance when it is about one-fifth of the median width. Radius R is tangent to the crossroad centreline (or edge of crossroad median). Radii R and R_1 comprise the two-centred curve between the terminals of the left turn. For simplicity, the PC is established at point 2. Radius R cannot be smaller than the minimum control radius for the design vehicle, or these vehicles will be unable to turn to or from the intended lane even at low speed. To avoid a large opening, R should be held to a reasonable minimum (e.g., 15 m), as used in **Figure 8.35**.

The length of median opening is governed by the radii. For medians wider than 9 m coupled with a crossroad of four or more lanes, the control radius R generally will need to be greater than 15 m or the median opening will be too short. A rounded value can be chosen for the length of opening (e.g., 15 or 18 m) and that dimension can be used to locate the centre for R. Then R becomes a check dimension to verify the workability of the layout. **Table 8.17** shows the resultant lengths of median openings over a range of median widths for three assumed values of R_1 and for R assumed to be 15 m. Dimension "B" is included as a general design control and for comparison with other above-minimum designs. The median end designs in Figure 8.35 do not positively provide protection areas within the limits of the median width. A design using R_1=30 m or more provides space for at least a single passenger vehicle to pause in an area clear of both the through-traffic lanes and the crossroad lanes; such radii may provide enough protection space for larger design vehicles. At skewed intersections, above-minimum designs with bullet nose median ends can be applied directly. Where the skew is 10 degrees or more, adjustments in R and R_2 from the values shown are needed to provide the appropriate length of opening.

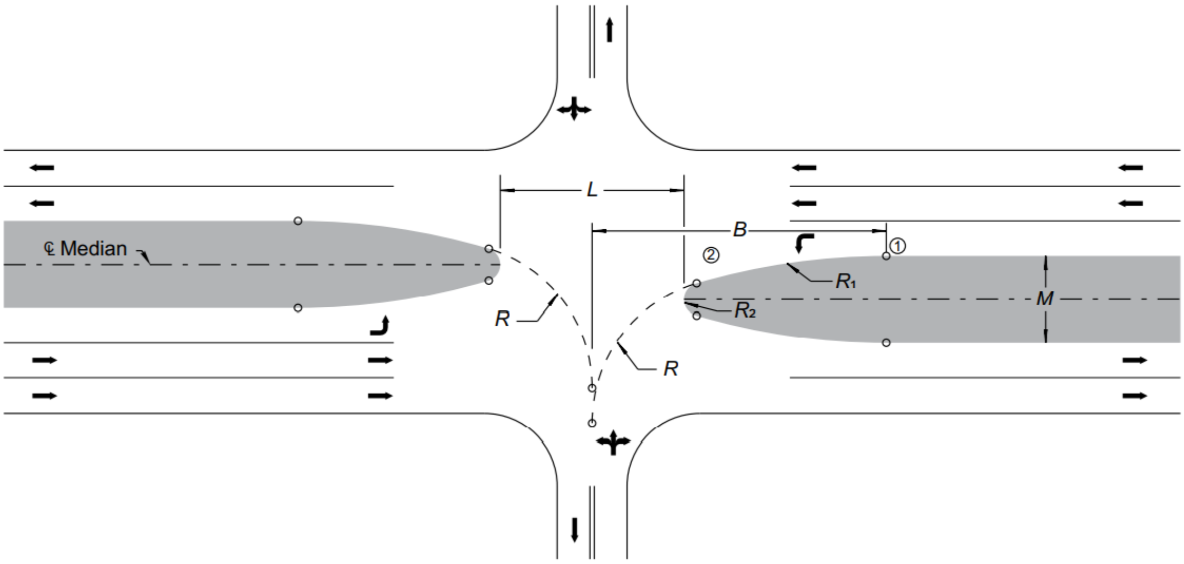


Figure 8.35 Typical bullet-nose ends

Table 8.17 Above-Minimum design of median opening (Typical bullet-nose ends)

Width of Median M (m)	R_1=30 m		R_1=50 m		R_1=70 m	
	L	B	L	B	L	B
6.0	18.0	20.2	20.2	24.4	21.3	27.6
9.0	15.1	21.4	17.7	26.5	19.0	30.4
12.0	12.8	22.4	15.6	28.3	17.1	32.7
15.0	-	-	13.8	29.9	15.4	34.7
18.0	-	-	-	-	13.8	36.7
21.0	-	-	-	-	12.4	38.4

8.9.3.6 LOCATION AND DESIGN OF U-TURN MEDIAN OPENINGS

Median openings designed to accommodate vehicles making U-turns only are needed on some divided roadways in addition to openings provided for cross and left-turning movements. Separate U-turn median openings may be appropriate at the following locations:

i. Locations beyond intersections to accommodate minor turning movements not otherwise provided in the intersection or interchange area. The major intersection area is kept free for the important turning movements, in some cases obviating expensive ramps or additional structures.

ii. Locations just ahead of an intersection to accommodate U-turn movements that would interfere with through and other turning movements at the intersection. Where a fairly wide median on the approach roadway has few openings, U-turns are needed for motorists to reach roadside areas. Advance separate openings to accommodate them outside the intersection proper will reduce interference.

iii. Locations occurring in conjunction with minor crossroads where traffic is not permitted to cross the major roadway but instead is required to turn right, enter the through-traffic stream, weave to the left, U-turn, and then return. On high-speed or high-volume highways, the difficulty of weaving and the long lengths involved usually make this design pattern undesirable unless the volumes intercepted are light and the median is of adequate width. This condition may occur where there is a crossroad with high-volume traffic, a shopping area, or other traffic generator that needs a median opening nearby and additional median openings would not be practical.

iv. Locations occurring where regularly spaced openings facilitate maintenance operations, policing, repair service of stalled vehicles, or other roadway-related activities. Openings for this purpose may be needed on controlled-access highways and on divided highways through undeveloped areas. Locations occurring on roadways without control of access where median openings at optimum spacing are provided to serve existing frontage developments and at the same time minimize pressure for future median openings. A preferred spacing at 0.40 to 0.80 km is suitable in most instances. Fixed spacing is not necessary, nor is it fitting in all cases because of variations in terrain and local service needs.

Sight distance is needed at median openings; therefore, median opening locations downstream of relatively sharp horizontal and vertical curves should be avoided, where practical. For a satisfactory design for U-turn manoeuvres, the width of the roadway, including the median, should be sufficient to permit the design vehicle to turn from an auxiliary left-turn lane in the median into the lane next to the outside shoulder or outside kerb and gutter on the roadway of the opposing traffic lanes.

Where U-turn openings are proposed for access to the opposite side of a multilane divided road, they should be located 15 to 30 m in advance of the next downstream left-turn lane. For U-turn openings designed specifically for the purpose of eliminating left-turn movement at a major intersection, they should be located downstream of the intersection. In urban areas, they should be located midblock between adjacent crossroad intersections. This type of U-turn opening should be designed with a median left-turn lane for storage. In a rural area, U-turn openings should be between 300 and 450 m apart. Additionally, a U-turn opening can be provided upstream of the major intersection to remove traffic wishing to make a U-turn from that intersection.

Medians of 5.5 m, and 15.6 m or wider are needed to permit small and large vehicle traffic, respectively, to turn from the inner lane (next to the median) on one roadway to the outer lane of a two-lane opposing roadway. Also, a median left-turn lane is highly desirable in advance of the U-turn opening to eliminate stopping on the through lanes. This scheme would increase the median width by approximately 3.6 m.

There are two types of median crossover intersections: bidirectional (sometimes called conventional) and directional (see **Figure 8.36**). A bidirectional crossover allows vehicles to make a U-turn from either direction of travel, which creates additional points of conflict as compared to the directional crossover. Further, as turning volumes increase, an interlocking of travel paths can occur in a bidirectional crossover, which could limit sight distance and result in unpredictable driver behaviour. Several studies have shown that directional crossovers experience fewer crashes than bidirectional crossovers for signalized corridors and that directional crossovers provide better operational performance.

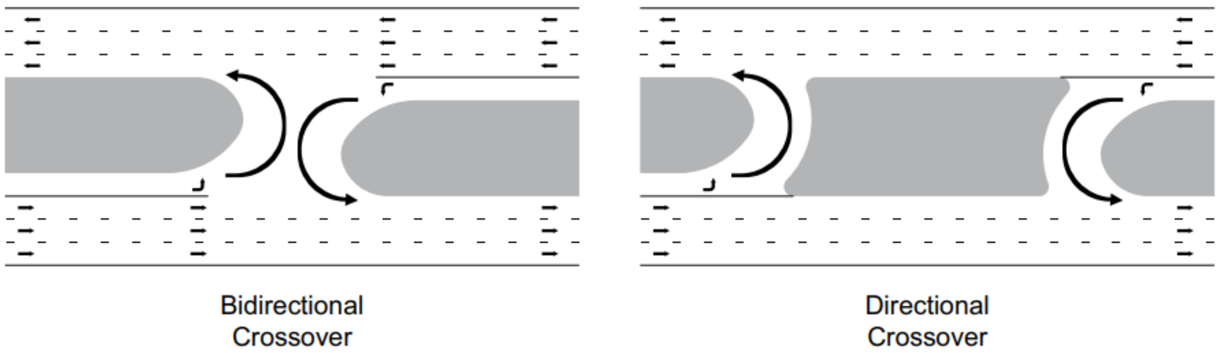


Figure 8.36 Bidirectional (conventional) and Directional Openings

If the median is wide enough to permit storage of vehicles, the use of a centreline and stop bar in the median storage area can communicate to drivers how the median should be negotiated and provide a sense of storage area. This can reduce undesirable manoeuvres such as side-by-side queuing and lane encroachment. Further, it can communicate that it is allowable to make crossing and turning manoeuvres in stages at this intersection.

Special U-turn designs, called loons, should be considered where right-of-way is restricted. In conditions where the U-turn crossover is unsignalized, sufficient gaps may be present in the traffic stream due to natural gaps in the traffic stream on lower volume roadways or through the presence of an upstream signal. When establishing the clearance intervals for a signalized crossover, it is essential to provide additional time to account for the extra travel distance needed for drivers to navigate the loon. Median widths of 2.4 to 12.5 m may be used for U-turn openings to permit small or large vehicles to turn from the inner lane in one direction onto the loon. This special U-turn feature can be incorporated into the design of an urban area roadway section by constructing a short segment of shoulder area along the outside edge of the travelled way across from the U-turn opening as shown in **Figure 8.37**. The outside kerb and gutter section would then be carried behind the shoulder area and the shoulder would be designed as a pavement. Through the use of loons, agencies can improve operations for U-turn manoeuvres to a level similar to those on divided roadways with wider medians without the high cost of widening the median or the opposing roadway, which could require acquiring right-of way continuously along the entire corridor.

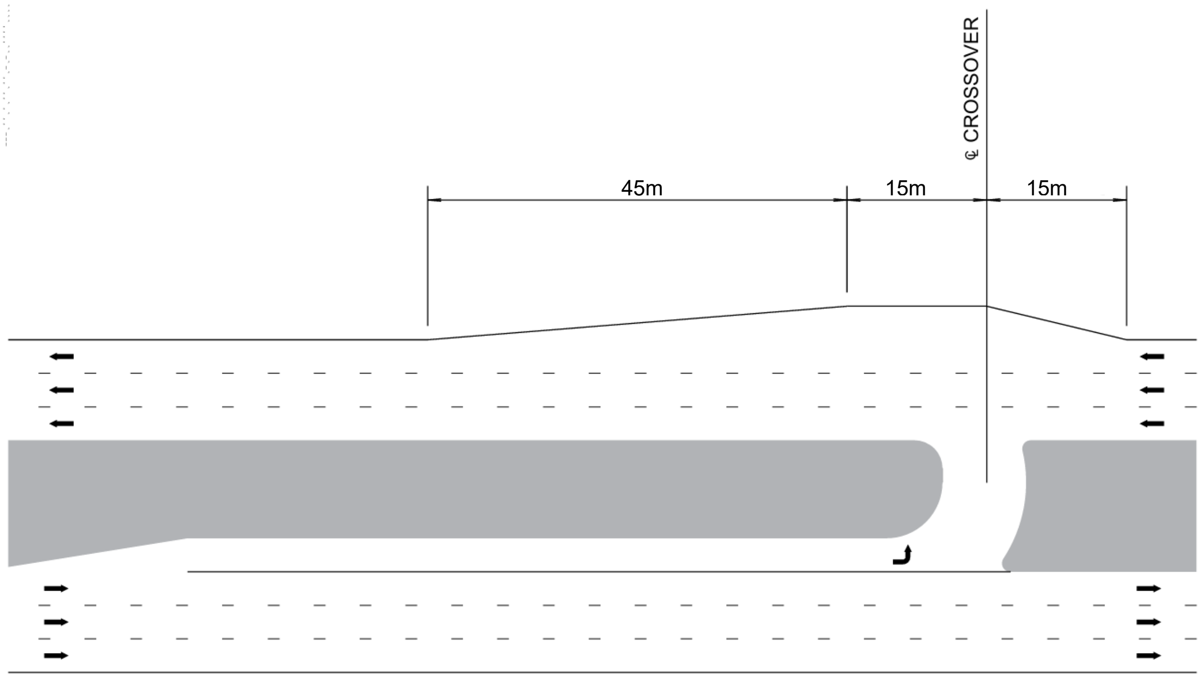


Figure 8.37 Typical Loon Design of facilitate U-Turn traffic on Arterials with Restricted Median Widths

Normally, U-turns should not be permitted from the through lanes. However, where medians have adequate width to shield a vehicle stored in the median opening, through volumes are low and left-turn/U-turns are infrequent, this type of design may be permissible. Minimum widths of median to accommodate U-turns by different design vehicles turning from the lane adjacent to the median are given in **Table 8.18.** These dimensions are for a four-lane divided facility. If the U-turn is made from a median left-turn/U-turn lane, the width needed is the separator width; the total median width needed would include an additional 3.6 m for a single median turn lane. At major intersections, both left turns and U-turns can be made around the kerbed nose at the end of a left-turn lane. Where dual left-turn lanes are needed and the turning volume of large vehicle is high, left turns and U-turns may be permitted from the inside lane and left turns only may be allowed from the outside turn lane. However, when the turning volume of large vehicles is low, a dual lane crossover manoeuvre may be permitted allowing both lanes to make a U-turn movement as shown in **Figure 8.38**. Under this condition, the minimum width of the median opening is 11m, which does not accommodate a large vehicle turning adjacent to another vehicle.

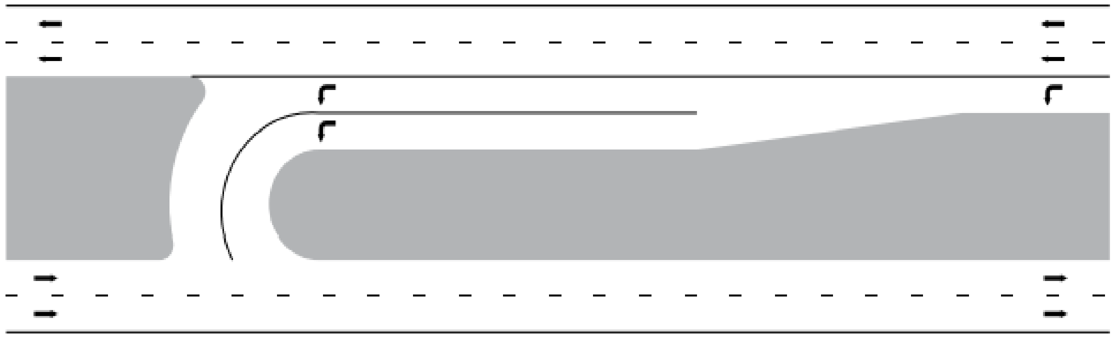


Figure 8.38 Dual U-turn Directional Crossover Design

Table 8.18 Minimum Width (m) of Median for Design Vehicle for U-turns

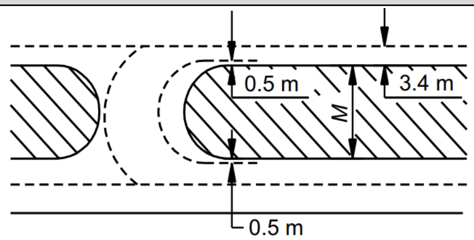
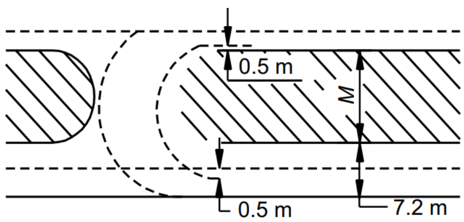
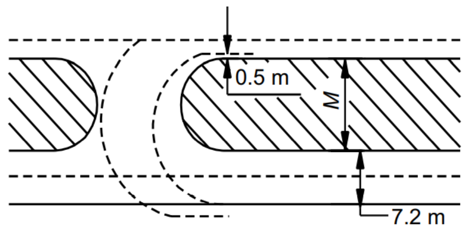
Type of Manoeuvre		**S-5**	**M-6**	**L-9**	**L-12**	**T-17**	**T-21**
		Length of Design Vehicle (m)					
		4.7	**6.0**	**9.0**	**12.0**	**16.5**	**21**
		Minimum Width of median, M (m) for design vehicle					
Inner Lane to Inner Lane		9.0	9.0	19.0	19.0	19.0	21.0
Inner Lane to Outer Lane		5.0	5.0	15.0	16.0	16.0	17.0
Inner Lane to Shoulder		2.0	2.0	12.0	12.0	13.0	14.0

Figure 8.39 illustrates special U-turn designs with narrow medians. In **Figure 8.39A**, the U-turning vehicle swings right from the outer lane, loops around to the left, stops clear of the divided roadway until a suitable gap in the traffic stream develops, and then makes a normal left turn onto the divided roadway. In **Figure 8.39**B, the U-turning vehicle begins on the inner lane of the divided roadway, crosses the through-traffic lanes, loops around to the left, and then merges with the traffic. Both scenarios result in pedestrians along the major roadway crossing

two additional roadways, one operating under free-flow conditions. To deter vehicles from stopping on through lanes, a left-turn lane with proper storage capacity should be provided to accommodate turning vehicles.

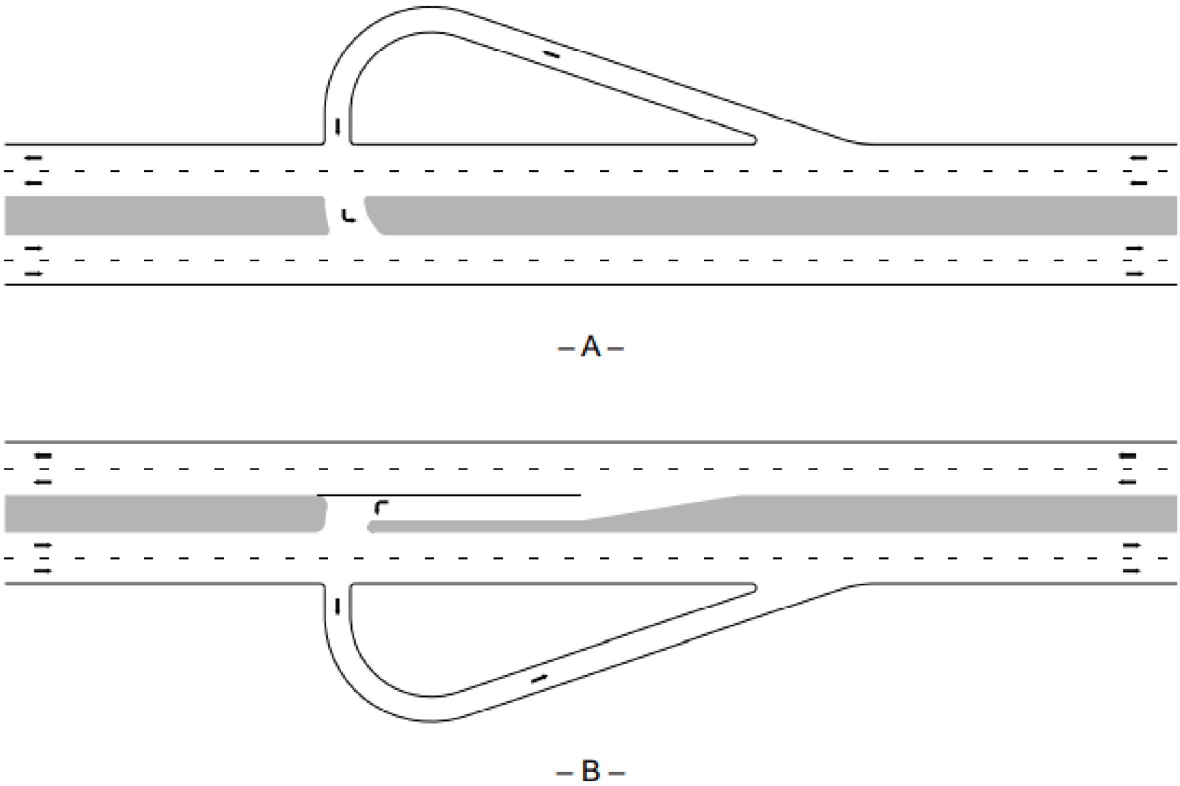


Figure 8.39 Special indirect U-turn Roadways with narrow Medians

8.9.4 INTERSECTION TURNING PATH AND CORNER CUT-OFF

8.9.4.1 TURNING PATH

Turning paths of design vehicles form the basis of the turning widths required at intersections. All intersection layouts must be checked to ensure that they can accommodate the turning path envelope (swept path) for the design vehicle plus necessary clearances. The swept path is the dynamic envelope traversed by the outer extremities of the vehicle. Vehicle swept paths can be checked by using a turning path template or a computer program.

The turning path through an intersection differs according to the design vehicle, road classification and whether it is a signal-controlled intersection or otherwise. The geometric structure of the intersection differs according to the turning path. Therefore, it is necessary to envisage and consider the turning path when designing an intersection. **Table 8.19** gives the recommended turning paths on left/right turn at intersections. **Figure 8.40** shows examples of turning paths.

Table 8.19 Turning Path at Intersections on Left/Right turn

(a) Rural

Design Class	Stop Sign Controlled			Signal Controlled
	Approach	Exit		Approach
		Main	Minor	
B1	T4*	T4*	-	T4
C1	L4	L4	L3	L4
D1	L4	L3	L3	L4
E	L1	L1	L1	L1

* Indicates that if the design vehicle for the main road is different from that of the minor road, the design vehicle for the minor road is used.

(b) Urban

Design Class	Stop Sign Controlled			Signal Controlled
	Approach	Exit		Approach
		Main	Minor	
B2	-	T4	-	-
C2	L4	L3	L2	L4
D2	L4	L2	L2	L4
E	L1	L1	L1	L1

* Indicates that if the design vehicle for the main road is different from that of the minor road, the design vehicle for the minor road is used.

Where:

T: Trailer

L: Large Vehicle

The numbers after the T and L describes the following turning paths.

1. Use the full width of the carriageway
2. Use the right-hand side of the carriageway from the centre. Do not use the oncoming lane.
3. Use the left-turn lane or left-most lane (when turning left) or right-most lane (when turning right) and one other lane abutting it. However, the oncoming lane shall not be used.

4. Only use the right-turn lane or right-most lane (when turning right) or left-most lane (when turning left).

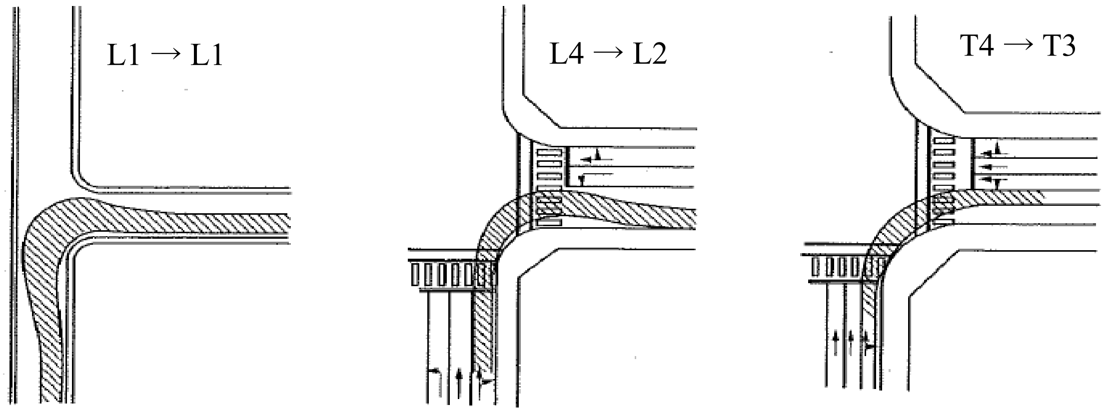


Figure 8.40 Turning Path at Intersection

The designer should be flexible in selecting the turning path at the intersection based on the location, land use of the area and the road network. For example, in areas that have a lot of trailers such as industrial areas, even if the road has a road class D or E, the design vehicle should be a trailer.

On the other hand, in residential areas, where it is possible for large vehicles to connect from other roads, left and right turning should not be considered that much. In this situation the designer should consider selecting the appropriate design vehicle or chose a lower Ti or Li in **Table 8.19**.

Moreover, where turning path T1 or L1 (use the full width of the carriageway) at the exit side is selected, consideration should be made to set back the stop line to the position that the turning could be possible. Refer to **Appendix F** for further information.

8.9.4.2 CORNER CUT-OFF

A "Corner Cut-Off" as shown in **Figure 8.41** and **Figure 8.42** is a triangular parcel of land at the corner of a property located at the intersection of two roadways (quadrant).

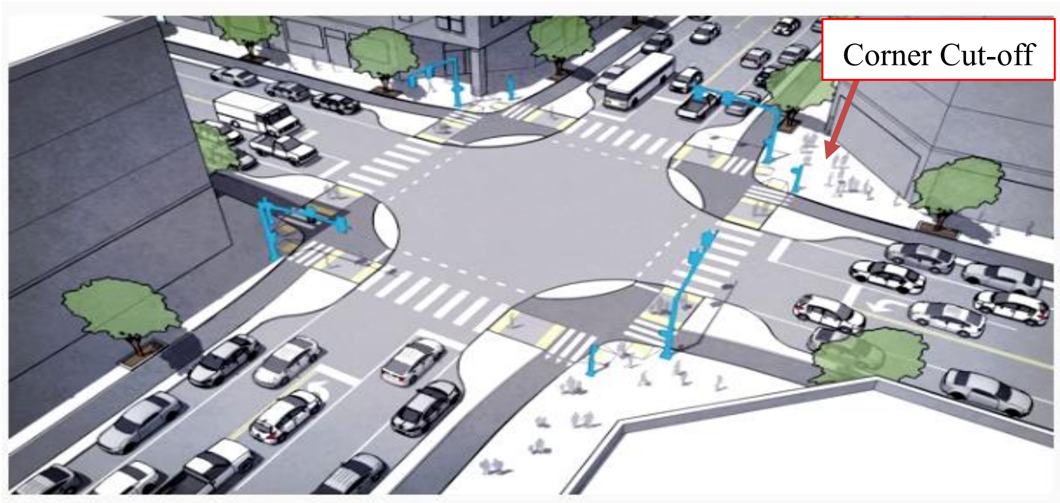


Figure 8.41 Image of corner cut-off

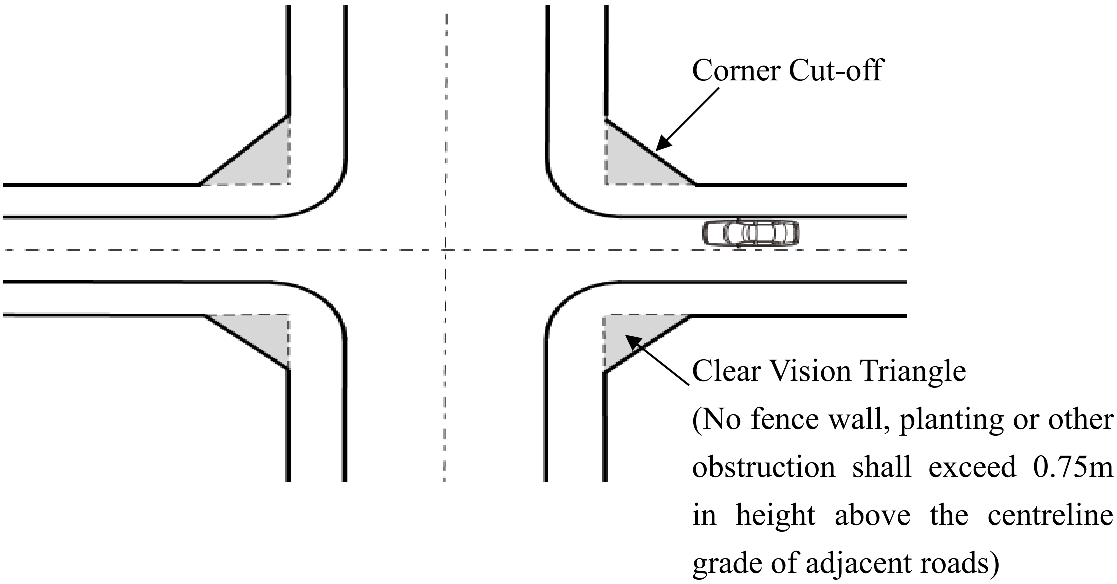


Figure 8.42 Image of clear vision triangle at corner cut-off

At an at-grade intersection, corner cut-offs create a clear vision triangle that is necessary to ensure safe and smooth traffic flow for vehicles, pedestrians, and cyclists as well as to provide a comfortable waiting area to cross the road without retention caused by signal or traffic flow (non- signal).

The following factors are used to determine the size of the corner cut-off:

i. Sufficient sight line or visibility splay
ii. Radius of corner kerb
iii. Effective walkway width
iv. Road greening and landscaping

Particularly in urban areas, where there is a considerable number of pedestrians and cyclist, it is necessary to give due consideration to the creation of safe and comfortable walking and cycling space, as well as road space to ensure smooth traffic flow of vehicles to minimise conflict at the intersection.

The length of the corner cut-off is determined by the underlisted factors:

i. Intersection angle of the roads
ii. Walkway width
iii. Waiting space for pedestrian and cyclist
iv. Sight line or visibility splay
v. Space for greening and landscaping

It is advisable to design each corner cut-off of at-grade intersections independently based on the forgoing factors. However, in urban areas, where roads of various classes form the network and makes the city a block, it is not practical to calculate the corner cut-offs for each of the many at-grade intersections, so as a reference cut-off length values in **Table 8.20** are used.

Table 8.20 Cut-off length in urban area

Design Class	B2	C2	D2	E
	Cut-off Length (m)			
B2	12	10	5	3
C2	-	10	5	3
D2	-	-	5	3
E	-	-	-	3

These values were calculated based on the following considerations:

i. **The design vehicle and turning path was referred to in Table 8.19(b) in the case of signal-controlled intersection.**
ii. **If the exit side is a two-lane road with a narrow width, the corner cut-off length was computed by following the Ti or Li in Table 8.19(b) on the exit side.**
iii. For C2, D2 and E roads mostly within residential areas, which are rarely used by large vehicles such as trucks, the corner cut-off lengths were given assuming a vehicle of 10m length, 2.5m width, 1.5m front overhang, 5.0m distance between axles and 9.0m minimum turning radius. However, even in this case, consideration was made to ensure that large vehicles can physically pass through.
iv. The turning path of design class D2 and E roads are designed to have a stop and turn on the approach lane and a smooth turning path on the exit lane.
v. All traffic except design class E is designed to approach from the rightmost lane,

however design class E may use the whole width of the carriageway.

vi. For the consistency of the cross-section of the road, the shoulder width is made 0.5m, walkway and bicycle lane width are made 4.5m for intersecting B2/C2 roads, no walkway for intersecting E roads, and 3.5m for all other road combinations. In case of walkway provided for E road, subject to reviewing traffic characteristics and pedestrian traffic flow, value on D2 or C2 roads should be applied.

These considerations are for general cases, and if special circumstances as given bellow should be taken into account, it should be considered separately in accordance with the general approach described above.

i. High volume of right/left turning traffic
ii. Design vehicle other than a large vehicle
iii. Wide walkway/bicycle lane
iv. Wide parking/stopping zone
v. Road crossing angles varies considerably from 90^0

Even if the corner cut-off is not needed from the characteristic of the traffic flow, to ensure the safety and comfort of pedestrians and cyclists, it is advisable to provide adequate traffic space with a clear sight line. To form a good urban scenery, the corner cut-off length should be as large as conditions permit. In rural areas where settlements are formed, values in **Table 8.20** could also be used as a guide.

At intersections with high volume of pedestrians, consideration should be given to the extended waiting function of the walkway (e.g., providing a pedestrian square). It is also desirable to secure comfortable “gathering” spaces that give priority to pedestrians through design and planting of greenery as shown in **Figure 8.43**.

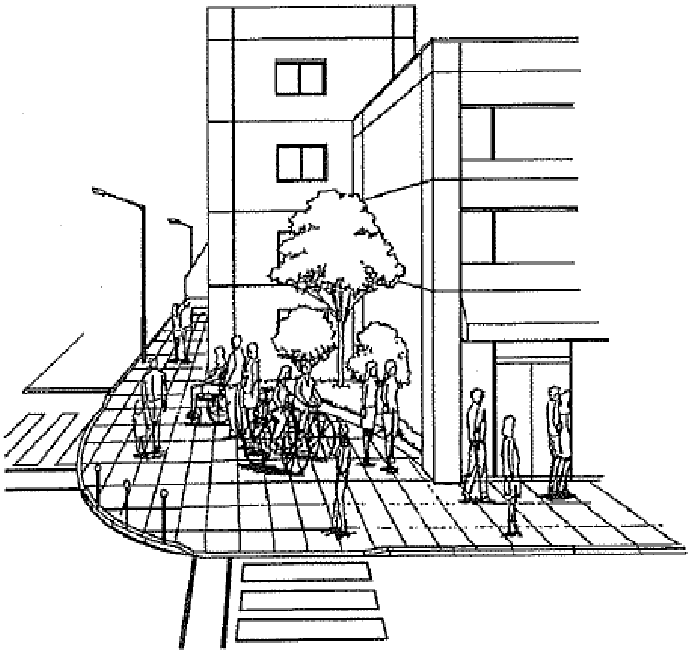


Figure 8.43 Example of storage function at an Intersection

8.9.4.2.1 PROCEDURE FOR THE DESIGN OF CORNER CUT-OFF AT AT-GRADE INTERSECTIONS

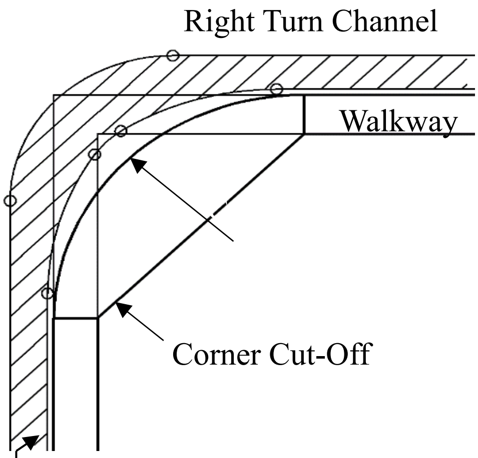
The procedure for the design of corner cut-off at at-grade intersections are as follows:

i. Consideration should be given to intersection angle of approach lane, width of walkway and bicycle lane, design vehicle and turning path.

ii. Pay attention to the provision of waiting space for the pedestrian, cyclist and wheelchair user, sight line and space for greenery.

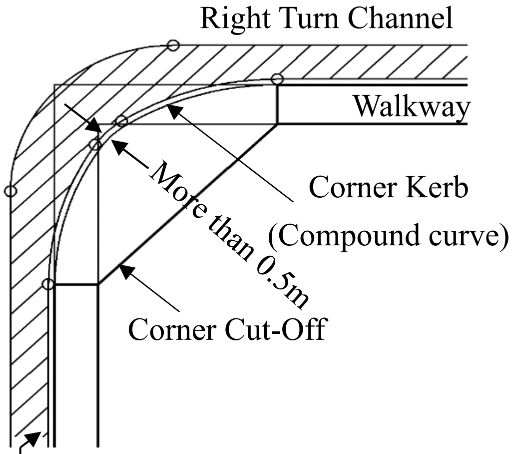
iii. In general, a corner cut-off is determined by connecting the edges of the walkways in a straight line around the starting and ending point of the corner kerb taking pedestrian traffic flow into account.

iv. As a rule, even if there is land constraint, absolute widths for the pedestrian, bicycle and wheelchair should be secured.



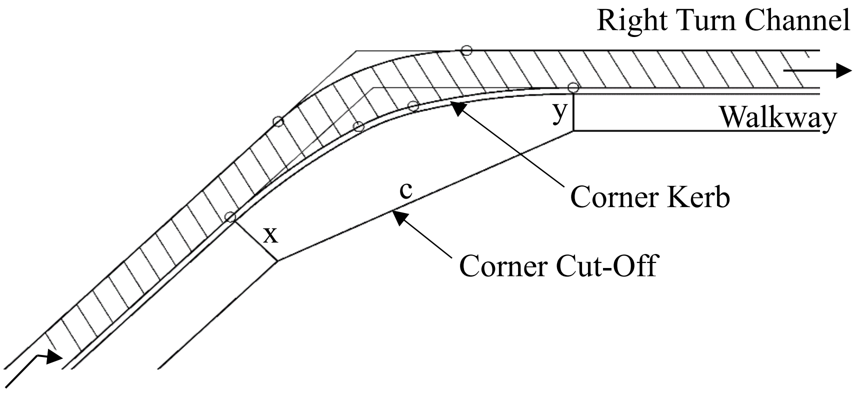
A gap between the right-turn channel and the walkway will be made, however, in case of no interference to the right of way, the layout will be simple compared with designing the corner kerb next to the right turn channel as given in (b)

Case (a): Corner kerb with Simple curve



For the case of designing a corner kerb parallel to the right turn channel, the curve will be a compound curve and relatively become difficult to layout at site.

Case (b): Corner kerb with Compound



If there is no land constraint, a line (c) connecting the ends of two lines (x, y) which is perpendicular to the start and end point of the corner kerb becomes a corner cut-off.

Case (c): Corner Kerb with obtuse angle approach lane or walkway with varying width

Figure 8.44 Design of corner cut-off

8.9.5 KERB RAMPS

According to the "Persons with Disability Act, 2006 Act 715", facilities for pedestrian use should be readily accessible to, and usable by individuals with disabilities.

When designing a project that includes kerbs and adjacent walkways, proper attention should be given to the needs of persons with disabilities, such as those with mobility or visual impairment. Kerb ramps are necessary to provide access between the walkway and the road at pedestrian crossings as shown in **Plate 8.2.** Detectable warnings (yellow in **Plate 8.2**) are needed where the kerb has been removed to alert pedestrians with visual impairment that they have arrived at the road/walkway interface.

Design details of kerb ramps will vary in relation to the following factors:

i. Walkway width
ii. Walkway location with respect to the kerb
iii. Height and width of kerb cross section
iv. Design turning radius and length of curve along the kerb face
v. Angle of road intersections
vi. Planned or existing location of sign and signal control devices
vii. Stormwater inlets and public service utilities
viii. Potential sight obstructions
ix. Road width
x. Border width

The location of the kerb ramp should be carefully coordinated with respect to the pedestrian crossing lines. The bottom of the kerb ramp should be situated within the parallel boundaries of the crossing markings and should be perpendicular to the face of the kerb, or bottom grade break, without warping in the walkway or kerb ramp. If the sides of the kerb ramp are not the same length, it will be difficult to provide a cross slope that is accessible to and usable by individuals with physical challenges and avoid warping.

Kerb ramps for the physically challenged are not limited to intersections and marked crosswalks. Kerb ramps should also be provided at other appropriate or designated points of pedestrian concentration, such as loading islands and midblock pedestrian crossings. Because non-intersection pedestrian crossings are generally unexpected by the motorist, warning signs should be installed, and parking should be prohibited to provide adequate visibility.

Plate 8.2 Example of kerb ramp

8.10 PEDESTRIAN AND BICYCLE CROSSINGS

8.10.1 PEDESTRIAN CROSSING

When pedestrians encounter an intersection, there is a major interruption in pedestrian flow. The walkway should provide sufficient storage area for those waiting to cross as well as an area for pedestrian cross traffic to pass. Once pedestrians are given the walk indication, the crosswalk width and length become important. Crosswalks should be wide enough to accommodate the pedestrian flow in both directions within the duration of the pedestrian signal phase. The wider the road, the longer it takes a pedestrian to cross, and proportionately less green signal time will be available for the primary road movements. Additionally, the longer the pedestrian crossing time, the longer the exposure to potential pedestrian–vehicular conflicts.

If the intersection is not signal controlled or if stop signs do not control the through motor vehicular traffic, pedestrians need to wait for suitable gaps in the traffic to cross. The wider the road, the longer the gap that is needed to provide sufficient pedestrian crossing times. Under urban area conditions, pedestrian crossing times may be reduced by using narrower lanes or by providing median refuge areas and two-stage crossings. However, the potential for vehicle–pedestrian collisions and reasonable roadway and intersection capacity needs should be considered when reducing crossing times.

8.10.1.1 PRINCIPLES OF PLANNING PEDESTRIAN CROSSING

The principles of planning pedestrian crossings at intersections are given as follows:

i. Conform to the desired lines of pedestrians.

   The provision of pedestrian crossings that force pedestrians to make unnatural detours is undesirable for road safety because it induces pedestrians to cross outside of the crossing (Jaywalk).

ii. Establish pedestrian crossings at right angles to the carriageway.

iii. Establishing pedestrian crossings at right angles will make the crossing distance as short as possible, and thereby shorten the carriageway crossing time for pedestrians and contribute to pedestrian's safety. Also, the green time allotted to pedestrians in the signal phase will be reduced, and consequently improve the intersection capacity.

iv. Locate pedestrian crossings close to the intersection.

Pedestrian crossings and stop lines determine the external outline of at-grade intersections, it is therefore desirable to provide them close to the intersection area as shown in **Figure 8.45**. Expanding the intersection area means that vehicles take longer time to pass through the intersection and accordingly increase the clearance time and reduce the traffic capacity of the signal phase. Moreover, in cases with large inter-crossing zone as shown in **Plate 8.3**, undesirable situation in terms of traffic processing arises. For example, "residual passing vehicles" that enter an intersection on a yellow light and "pedestrians rushing to cross" are more likely to collide.

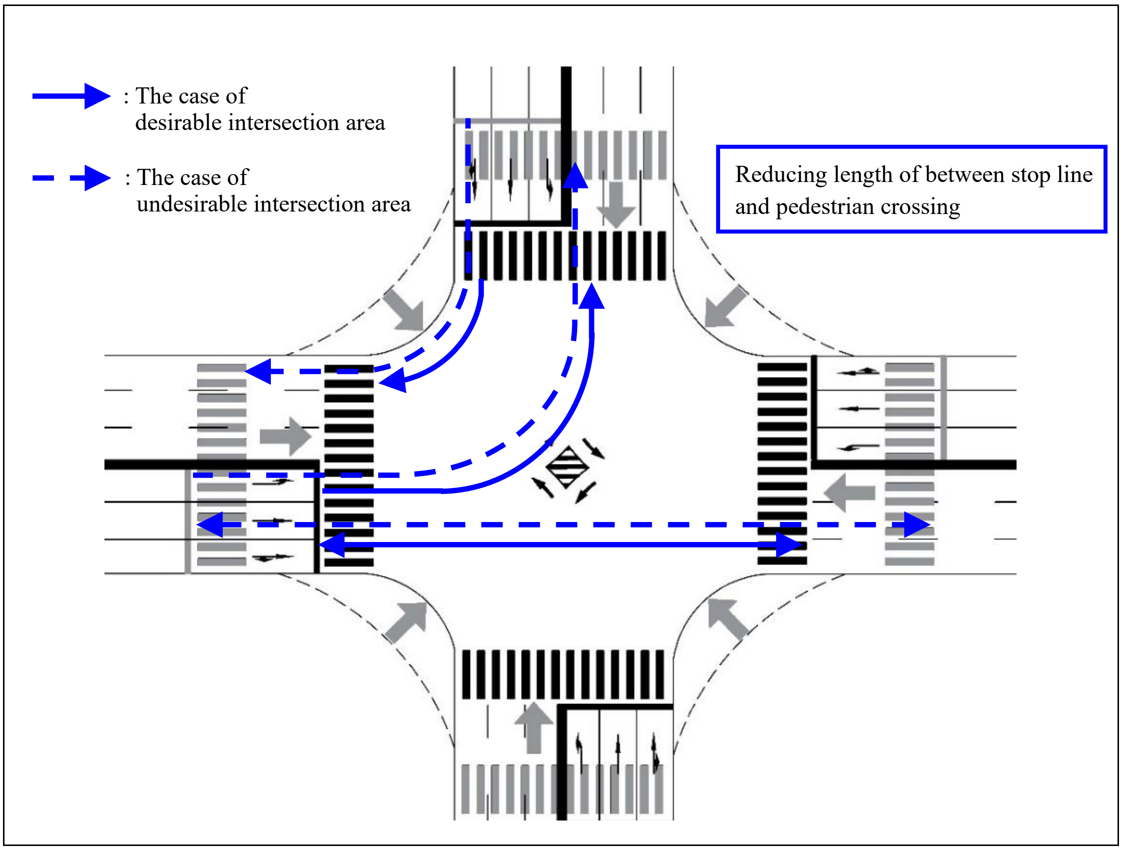


Figure 8.45 Pedestrian crossings at intersection area

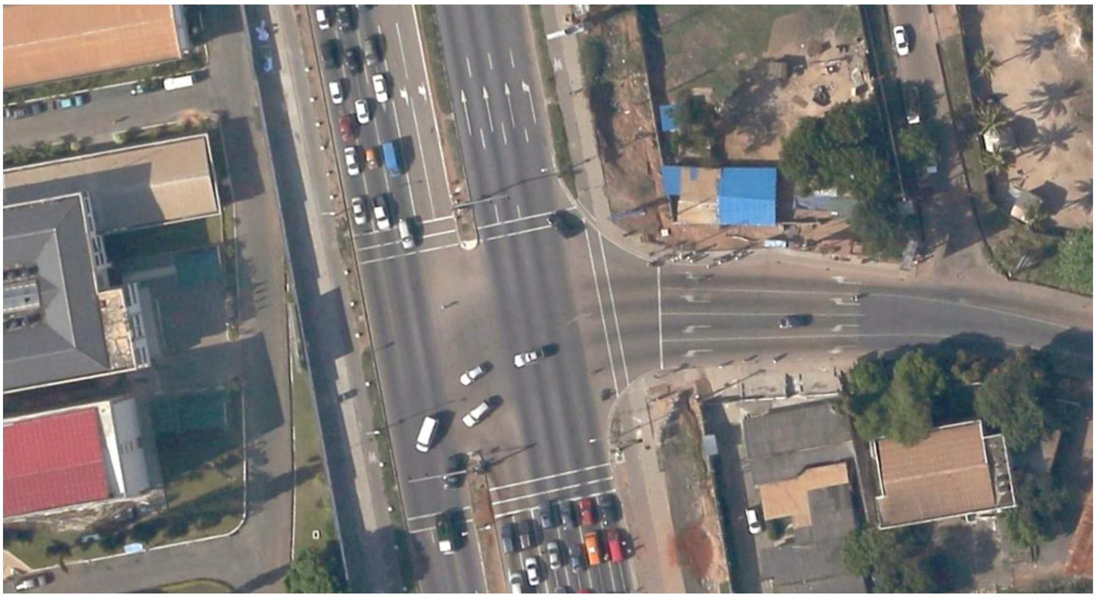


Plate 8.3 Intersection with wide pedestrian crossing zone

v. **Visibility at pedestrian crossings.**

Location of pedestrian crossings should be visible to all drivers on the approach legs.

vi. **The length of the pedestrian crossing**.

The length of pedestrian crossings should not be greater than 15m. For pedestrian crossing distance greater than 15m, consideration should be given to the establishment of a refuge island in the middle of the road for a two-stage crossing.

vii. Width of the pedestrian crossing.

Minimum width of a pedestrian crossing should be 4m at major road intersections and 3m at minor road intersections. The width of a pedestrian crossing should be set according to actual condition of the at-grade intersection, volume of crossing pedestrians and the allotted time on the signal phase for crossing pedestrians. However, regardless of the volume of crossing pedestrians, a minimum width of 4m and 3m should be adopted for all major and minor road intersections respectively. The width may be increased in units of 1m based on the situation.

viii. Location of pedestrian crossing in the intersection.

Pedestrian crossing should be located on the tangent section beyond the corner radius.

8.10.1.2 PROVISION OF PEDESTRIAN CROSSING

Provision of pedestrian crossing should be an extension of the walkway. However, at the location where the walkway and pedestrian crossing intersect, usually road facilities such as guard rails are provided, and the walkway and carriageway are not directly connected. Pedestrian crossing must be set back from the extension of the walkway by at least one (1) meter.

On at-grade intersections between two arterial roads where right-turning vehicles and crossing pedestrians are likely to collide, it is desirable to set back the pedestrian crossing between 3-4m from the extension of the walkway as shown in **Figure 8.46**. This is to ensure that the right-turning vehicles waiting for pedestrians to cross do not obstruct through going vehicles coming from behind.

This is also necessary to secure space for pedestrians waiting to cross the road encroaching unto the carriageway. This space can be used for the installation of pedestrian/traffic signals, road signs, street lights, drainage catch pits and other facilities (**Plate 8.4**) that need to be installed on the walkway at the corner of an at-grade intersection. Moreover, such spaces have psychological benefits in terms of enhancing the sense of security and safety of pedestrians.

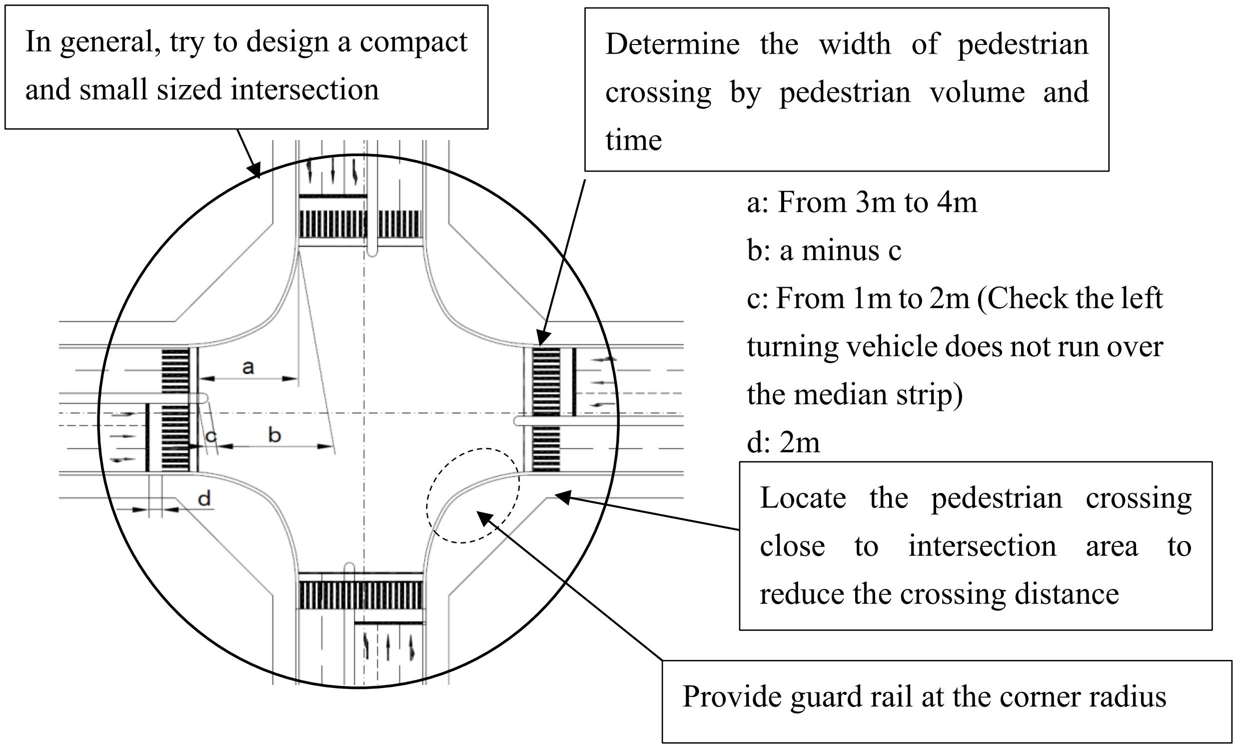


Figure 8.46 Key points in the design of right-angled intersection

Plate 8.4 Example of facilities at at-grade intersection

On roads with a median strip, the inner curve of the left turning channel is used to determine the position of the nose. It is recommended that the pedestrian crossing be located 1 to 2m back from the nose of the medium strip.

Figure 8.47 is an example of the design of the turning path and pedestrian crossing for a Y-Shaped intersection.

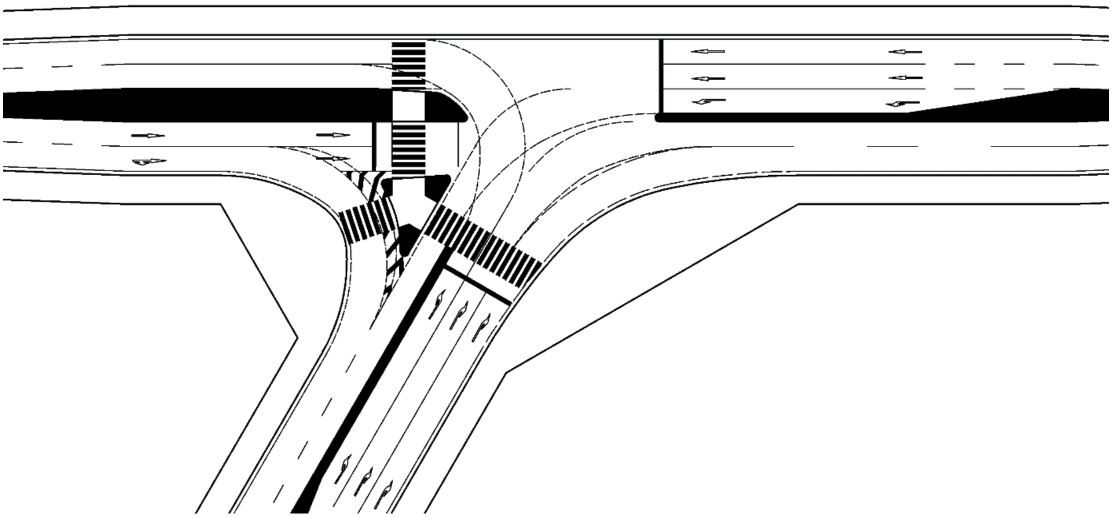


Figure 8.47 Example of Design of Y shape intersection

8.10.2 BICYCLE CROSSING

As a means of separating bicycle from vehicular traffic, in situations where bicycle lane (or pedestrian / bicycle lane) is established separately from the carriageway, a bicycle crossing should be provided for crossing the carriageway at an at-grade intersection as shown in **Plate 8.5**.

The bicycle crossing has the same functions as a pedestrian crossing in respect to bicycle traffic, and the basic principle regarding the installation method is the same as for pedestrian crossings. The standard width of a bicycle crossing should be 1.5m.

Plate 8.5 Bicycle crossing

8.10.3 LOCATION OF THE STOP LINE

The stop line is an indication informing drivers that they must stop without allowing any part of their vehicle cross the line. Stop lines are always established at the approach section of the signal-controlled intersection, before pedestrian crossings, and the approach section of minor roads on non-signal controlled intersections. Inappropriate location of stop lines leads to a lower compliance rate and a higher risk of traffic accidents. Therefore, when conducting design, the location of the stop line should be determined based on a full understanding of traffic operation of the intersection. Important principles to consider in establishing stop lines are as follows:

i. The stop line should be established at right angles to the carriageway centreline.
ii. If there is a pedestrian crossing, establish at least one (1) m setback from the crossing.
iii. It should be established in the following locations;
    a. that allows vehicles running on the crossroad side to be seen with ample sight distance.
    b. where there will be no impedance to right and left turning vehicles on the crossroad side.
iv. To prevent accidents of motorcycles in the intersection, a 2-stage stop line can be installed as shown in **Figure 8.48**. The interval between the 2 stop lines should be around 2-5m to provide enough space for motorcycles.

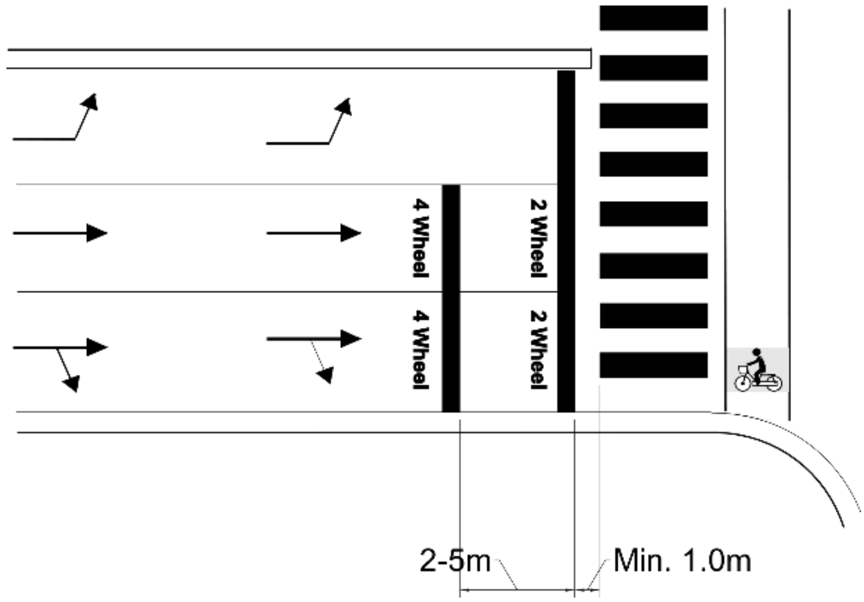


Figure 8.48 Example of a Two-stage Stop Line

8.10.3.1 CONSIDERATIONS ON NARROW ROADS

On narrow roads, the stop line will be set back a few meters so as not to interfere with vehicles turning right or left from the major roads. In this case, the sight distance to see the vehicles on the main road becomes a problem and the following countermeasures could be considered.

i. Secure sufficient corner cut distance.
ii. Where roadside conditions make it impossible to provide adequate corner cuts, it should be changed to signal controlled intersection.
iii. Where the traffic volume on a narrow road is very low and corner cutting is difficult, road reflectors should be installed to make the main road visible.

In application of these countermeasures, the conditions of the intersection should be carefully confirmed.

8.10.4 SIGHT TRIANGLE

Specified areas along intersection approach legs and across their included corners should be clear of obstructions that might block a driver’s view of potentially conflicting vehicles. These specified areas are known as clear sight triangles. The dimensions of the legs of the sight triangles depend on the design speeds of the intersecting roadways and the type of traffic control used at the intersection.

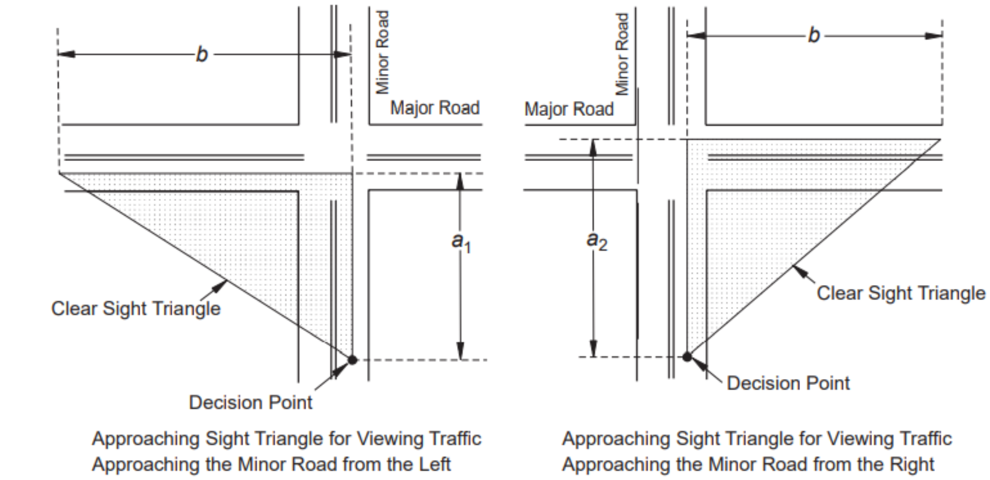
Each quadrant of an intersection should contain a clear sight triangle free of obstructions that may block a driver's view of potentially conflicting vehicles on the opposing approaches. Two different forms of sight triangle are required.

i. Approach Sight Triangle

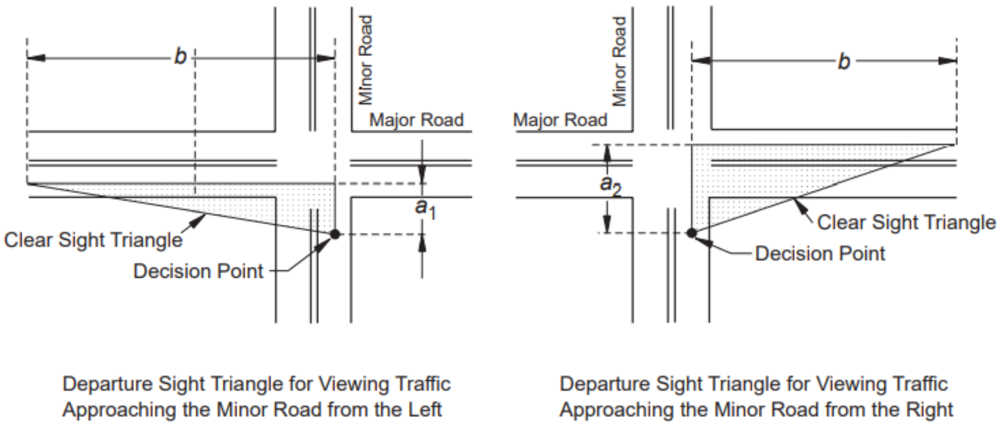
ii. Departure Sight Triangle

In the first instance, reference is to approach sight triangles. The approach triangle will have sides with sufficient lengths on both intersecting roadways such that drivers can see any potentially conflicting vehicle in sufficient time to slow, or to stop if need be, before entering the intersection. For the departure sight triangle, the line of sight described by the hypotenuse of the sight triangle should be such that a vehicle just coming into view on the major road will, at the design speed of this road, have a travel time to the intersection corresponding to the gap acceptable to the driver of the vehicle on the minor road. Both forms of sight triangle are required in each quadrant of the intersection. The line of sight assumes a driver eye height of 1.2m and an object height of 1.3m. The approach and departure sight triangles are illustrated in **Figure 8.49**. The areas shown shaded should be kept clear of vegetation or any other obstacle to a clear line of sight. To this end, the road reserve is normally splayed to ensure that the entire extent of the sight triangle is under the control of the road authority. Furthermore, the profiles of the intersecting roads should be designed to provide the required sight distance. Where one or other of the approaches is in cut, the affected sight triangles may have to be "day lighted", i.e. the natural material occurring within the sight triangles may have to be excavated to ensure intervisibility between the relevant approaches.

Sight distance values are based on the ability of the driver of a small vehicle to see an approaching small vehicle. It is also necessary to check whether the sight distance is adequate for trucks (large vehicle and trailer). Because their rate of acceleration is lower than that of small vehicle and as the distance that the truck has to travel to clear the intersection is longer, the gap acceptable to a truck driver is considerably greater than that required by the driver of a small vehicle. For design purposes, the eye height of truck drivers is taken as 1.8m for checking the availability of sight distance for trucks.



(a) Approach Sight Triangles (Uncontrolled or Yield controlled)



(b) Departure (exit) sight triangles

Figure 8.49 At-grade intersection sight triangles

8.10.4.1 INTERSECTION CONTROL

The recommended dimensions of the sight triangles vary with the type of traffic control used at an intersection because different types of control impose different legal constraints on drivers and, therefore, result in different driver behaviour. Sight distance policies for intersections with the following types of traffic control are presented below:

i. Case A—Intersections with no control

ii. Case B—Intersections with stop control on the minor road

   Case B1 —Left turn from the minor road

   Case B2 —Right turn from the minor road

   Case B3 —Crossing manoeuvre from the minor road

iii. Case C—Intersections with yield control on the minor road

Case C1 —Crossing manoeuvre from the minor road

Case C2 —Left or right turn from the minor road

iv. Case D—Intersections with traffic signal control

v. Case E—Intersections with all-way stop control

vi. Case F—Left turns from the major road

8.10.4.1.1 INTERSECTIONS WITH NO CONTROL (CASE A)

Uncontrolled intersections are not used in conjunction with the main road network, but are common in rural networks and access roads to rural settlements. In these cases, drivers must be able to see potentially conflicting vehicles on intersecting approaches in sufficient time to stop safely before reaching the intersection. Ideally, sight triangles with legs equal to stopping sight distance should be provided on all the approaches to uncontrolled intersections.

If sight triangles of this size cannot be provided, the lengths of the legs on each approach can be determined from a model that is analogous to the stopping sight distance model, with slightly different assumptions.

Field observations indicate that vehicles approaching uncontrolled intersections typically slow down to approximately 50 per cent of their normal running speed. This occurs even when no potentially conflicting vehicles are present, typically at deceleration rates of up to 1.5m/s.

Braking at greater deceleration rates, which can approach those assumed in the calculation of stopping sight distances, begins up to 2.5s after a vehicle on the intersecting approach comes into view. Thus, approaching vehicles may be travelling at less than their normal running speed during all or part of the perception-reaction time and can brake to a stop from a speed less than the normal running speed. **Table 8.21** shows the distance travelled by an approaching vehicle during perception, reaction and braking time as a function of the design speed of the roadway on which the intersection approach is located. These distances should be used as the legs of the sight triangles shown in **Figure 8.49**.

Where the gradient of an intersection approach exceeds 3%, the leg of the clear sight triangle along that approach should be adjusted by multiplying the sight distance in **Table 8.21** by an adjustment factor in **Table 8.22**. If these sight distances cannot be provided, advisory speed signing to reduce speeds or installing Stop signs on one or more approaches should be investigated. Uncontrolled intersections do not normally require departure sight triangles because they typically have very low traffic volumes. If a motorist finds it necessary to stop at an uncontrolled intersection because of the presence of a conflicting vehicle, it is unlikely that another potentially conflicting vehicle will be encountered as the first vehicle departs the intersection.

Table 8.21 Recommended sight distance for intersections with no traffic control (Case A)

Design speed (km/h)	Sight distance (m)
120	165
100	120
80	80
60	50
50	40
40	30
30	25
20	20

Table 8.22 Adjustment Factors for Intersection Sight Distance Based on Approach Grade

Approach Grade (%)	Design Speed (km/h)							
	20	30	40	50	60	80	100	120
−6	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
−5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.2
−4	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1
−3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9

8.10.4.1.2 INTERSECTIONS WITH STOP CONTROL ON THE MINOR ROAD (CASE B)

Departure sight triangles for intersections with Stop control on the minor road should be considered for three situations:

i. Right turns from the minor road (Case B1);
ii. Left turns from the minor road (Case B2); and
iii. Crossing the major road from the minor road approach (Case B3).

Approach sight triangles, as shown in **Figure 8.49 (a),** need not be provided at Stop-controlled intersections because all minor-road vehicles should stop before entering or crossing the major road.

Vehicles turning right from the minor road have to cross the stream of traffic approaching from the right and then merge with the stream approaching from the left. Left-turning vehicles need only merge with the stream approaching from the right. As the merging manoeuvre requires that turning vehicles should be able to accelerate approximately to the speed of the stream with which they are merging, it necessitates a gap longer than that for the crossing manoeuvre.

8.10.4.1.3 LEFT TURN FROM THE MINOR ROAD (CASE B1)

A departure sight triangle for traffic approaching either from the left or the right, as shown in **Figure 8.49 (b)**, should be provided for left turns from the minor road onto the major road for all Stop-controlled approaches.

Field observations of the gaps accepted by the drivers of vehicles turning to the left onto the major road have shown that the values in **Table 8.23** provide sufficient time for the minor-road vehicle to accelerate from a stop and merge with the opposing stream without undue interference. These observations also revealed that major-road drivers would reduce their speed to some extent to accommodate vehicles entering from the minor road. Where the gap acceptance values in **Table 8.23** are used to determine the length of the leg of the departure sight triangle along the major road, most major-road drivers need not reduce speed to less than 70% of their initial speed.

Table 8.23 applies to small vehicles. However, for minor-road approaches from which substantial volumes of large vehicles and trailers enter the major road, the values for large vehicle or trailer should be applied. **Table 8.23** includes adjustments to the acceptable gaps for the number of lanes on the major road and for the approach gradient of the minor road. The adjustment for the gradient of the minor-road approach need be made only if the rear wheels of the design vehicle would be on an upgrade steeper than 3% when the vehicle is at the stop line of the minor-road approach.

The length of the sight triangle along the major road (distance "b" in **Figure 8.49**) is the product of the design speed of the major road in m/s and the critical gap in seconds as listed in **Table 8.23**.

If the sight distances along the major road based on **Table 8.23** (including the appropriate adjustments) cannot be provided, consideration should be given to the installation of speed limit warning signs on the major-road approaches.

Dimension "a_1/a_2" in **Figure 8.49 (b)** depends on the context within which the intersection is being designed. In urban areas, drivers tend to stop their vehicles immediately behind the Stop line, which may be located virtually in line with the edge of the major road. A small vehicle driver would, therefore, be located about 2.4m away from the Stop line. In rural areas, vehicles usually stop at the edge of the shoulder of the major road. In the case of a 3m wide shoulder the driver would thus be approximately 5.4m away from the edge of the travelled way.

Table 8.23 Time Gap for Case B1, Left Turn from Stop

Design Vehicle	Time gap (s) at design speed of major road
Small vehicle (S-5) Medium vehicle (S-6)	7.5
Large vehicle (L-9 & L-12)	9.5
Trailer (T-17 & T-21)	11.5

Note:
Adjustment for multilane roads: For left turns onto two-way roads with more than two lanes, add 0.5s for small vehicle/medium vehicle or 0.7s for large vehicle for each additional lane, from the left, in excess of one, to be crossed by the turning vehicle. Median widths should be converted to an equivalent number of lanes in applying the 0.5 and 0.7s criteria presented above; for example, a 5.5m median is equivalent to one and a half lanes and would require an additional 0.75s for a small vehicle to cross and an additional 1.05 s for a large vehicle to cross.
Adjustment for approach grades on the minor road: If the approach grade is an upgrade that exceeds 3%, add 0.2 s for each percent grade by which the approach grade exceeds zero percent.

Where the major road is a dual carriageway, two departure sight triangles have to be considered: a sight triangle to the right, as for the crossing movement (Case B3) and one using the acceptable gap as listed in **Table 8.23** for vehicles approaching from the left. This presupposes that the width of the median is sufficient to provide a refuge for the vehicle turning from the minor road. If the median width is inadequate, the adjustment in **Table 8.23** for multilane major roads should be applied with the median being counted as an additional lane.

The departure sight triangle should be checked for various possible design vehicles because the width of the median may be adequate for one vehicle type and not for another so that two different situations have to be evaluated.

8.10.4.1.4 RIGHT TURN FROM THE MINOR ROAD (CASE B2)

A departure sight triangle for traffic approaching from the left, as shown in **Figure 8.49**, should be provided for right turns from the minor road. The lengths of the legs of the departure sight triangle for right turns should generally be the same as those for the left turns used in Case B1.

Specifically, the length of the leg of the departure sight triangle (dimension "b") along the major road should be based on the travel times in **Table 8.24**, including appropriate adjustment factors.

Table 8.24 Time Gap for Case B2—Right Turn from Stop

Design Vehicle	Time gap (s) at design speed of major road
Small vehicle (S-5) Medium vehicle (S-6)	6.5
Large vehicle (L-9 & L-12)	8.5
Trailer (T-17 & T-21)	10.5

Note: Time gaps are for a stopped vehicle to turn right onto or to cross a two-lane roadway with no median and with minor-road approach grades of 3 percent or less. The table values should be adjusted as follows:
For minor-road approach grades—If the approach grade is an upgrade that exceeds 3%,add 0.1s for each percent grade by which the approach grade exceeds zero percent.

Dimensions "a_1 & a_2" depends on the context of the design and can vary from 2.4m to 5.4m. Where sight distances along the major road based on the travel times from **Table 8.23** cannot be provided, it should be kept in mind that field observations indicate that, in making right turns, drivers generally accept gaps that are slightly shorter than those accepted in making left turns. The travel times in **Table 8.23** can be decreased by 1.0 to 1.5s for right turn manoeuvres, where necessary, without undue interference with major-road traffic. When the recommended sight distance for a left-turn manoeuvre cannot be provided, even with a reduction of 1.0 to 1.5s, consideration should be given to the installation of speed limit warning signs on the major road approaches.

8.10.4.1.5 CROSSING MANOEUVRE FROM THE MINOR ROAD (CASE B3)

In most cases it can be assumed that the departure sight triangles for left and right turns onto the major road, as described for Cases B1 and B2, will also provide more than adequate sight distance for minor-road vehicles crossing the major road. However, it is advisable to check the availability of sight distance for crossing manoeuvres:

i. Where right and/or left turns are not permitted from a particular approach and crossing is the only legal manoeuvre.
ii. Where the crossing vehicle has to cross four or more lanes; or
iii. Where substantial volumes of large vehicles/ trailer cross the roadway and where there are steep gradients on the departure roadway on the far side of the intersection that might slow the vehicle while its rear is still in the intersection.

Figure 8.25 presents travel times and appropriate adjustment factors that can be used to determine the length of the leg of the sight triangle along the major road to accommodate crossing manoeuvres.

At divided roadway intersections, depending on the width of the median and the length of the design vehicle, sight distance may be needed for crossing both roadways of the divided roadway or for crossing the near lanes only and stopping in the median before proceeding or cross the major road without stopping. The sight distances needed by drivers on Yield-controlled approaches exceed those for Stop-controlled approaches because of the longer travel time of the vehicle on the minor road.

For four-legged intersections with Yield control on the minor road, two separate sets of approach sight triangles as shown in **Figure 8.49 (a)** should be provided.

Table 8.25 Time Gap for Case B3, Crossing Manoeuvre from the Minor Road

Design Vehicle	Time gap (s) at design speed of major road
Small vehicle (S-5) Medium vehicle (S-6)	6.5
Large vehicle (L-9 & L-12)	8.5
Trailer (T-17 & T-21)	10.5

Note: Time gaps are for a stopped vehicle to cross a two-lane highway with no median and with minor- road approach grades of 3% or less. The table values should be adjusted as follows:

For multilane roadways or medians—For crossing manoeuvres that cross roadways with more than two lanes, including turn lanes, add 0.5s for small/medium vehicles or 0.7s for large vehicle/trailer for each additional lane, from the left, in excess of two, to be crossed by the turning vehicle. Median widths should be converted to equivalent lanes; for example, a 5.5m median would be equal to one and a half lanes and would need an additional time gap of 0.75s for small/medium vehicles and 1.05s for large vehicle and trailer.

For minor-road approach grades—If the approach grade is an upgrade that exceeds 3%, add 0.2s for each percent grade by which the approach grade exceeds zero percent.

8.10.4.1.6 INTERSECTIONS WITH YIELD CONTROL ON THE MINOR ROAD (CASE C)

Vehicles entering a major road at a Yield-controlled intersection may, because of the presence of opposing vehicles on the major road, be required to stop. Departure sight triangles as described for Stop control must therefore be provided for the Yield condition. However, if no conflicting vehicles are present, drivers approaching Yield signs are permitted to enter sight triangles to accommodate right and left turns onto the major road and the other for crossing movements. Both sets of sight triangles should be checked for potential sight obstructions.

A. CROSSING MANOEUVRES (CASE C1)

The lengths of the leg of the approach sight triangle along the minor road to accommodate the crossing manoeuvre from a Yield-controlled approach (distance "a_1" in **Figure 8.49(a)** are given in **Table 8.26**. The distances in **Table 8.26** are based on the same assumptions as those for Case A control except that, based on field observations, minor-road vehicles that do not stop are assumed to decelerate to 60% of the minor-road design speed rather than to 50%. The distances and times in **Table 8.26** should be adjusted for the gradient of the minor road approach, using the factors in **Table 8.22**.

The length of the leg of the approach sight triangle along the major road to accommodate the crossing manoeuvre (distance "b" in **Figure 8.49(a)**) should be calculated using **Equations 8.10** and **8.11**:

$$t_c = t_a + \frac{w + L_a}{0.167V_{minor}} \tag{8.10}$$

$$b = 0.278V_{major}t_c \tag{8.11}$$

Where,

t_g = travel time to reach and clear the major road (s)

b = length of leg of sight triangle along the major road (m)

t_a = travel time to reach the major road from the decision point for a vehicle that does not stop (s) (use appropriate value for the minor-road design speed from **Table 8.26** adjusted for approach grade, where appropriate)

w = width of intersection to be crossed (m)

L_a = length of design vehicle (m)

V_{minor} = design speed of minor road (km/h)

V_{major} = design speed of major road (km/h)

These equations provide sufficient travel time for the major road vehicle, during which the minor-road vehicle can:

a. Travel from the decision point to the intersection, while decelerating at the rate of 1.5m/s² to 60 % of the minor-road design speed; and then
b. Cross and clear the intersection at the same speed.

Field observations did not provide a clear indication of the size of the gap acceptable to the river of a vehicle located at the decision point on the minor road. If the required gap is longer than that indicated by the above equations, the driver would, in all probability, bring the vehicle to a stop and then select a gap on the basis of Case B. If the acceptable gap is shorter than that indicated by the above equations, the sight distance provided would, at least, provide a margin of safety.

If the major road is a divided roadway with a median wide enough to store the design vehicle for the crossing manoeuvre, then only crossing of the near lanes need be considered and a departure sight triangle for accelerating from a stopped position in the median should be provided, based on Case B1.

Table 8.26 Case C1—Crossing Manoeuvres from Yield-Controlled Approaches, Length of Minor Road Leg and Travel Times

Design Speed (km/h)	Minor-Road Approach		Travel Time (t_g) (s)	
	Length of Leg[a] (m)	Travel Time T_a [a,b] (s)	Calculated Value	Design Value[c,d]
120	180	7.0	7.7	7.7
100	135	135	7.1	7.1
80	100	5.5	6.5	6.5
60	65	4.8	6.1	6.5
50	55	4.4	6.0	6.5
40	40	4.0	6.0	6.5
30	30	3.6	6.2	6.5
20	20	3.2	7.1	7.1

a. For minor-road approach grades that exceed 3%, multiply the distance or the time in this table by the appropriate adjustment factor from **Table 8.22**.

b. Travel time applies to a vehicle that slows before crossing the intersection but does not stop.

c. The value of t_g should equal or exceed the appropriate time gap for crossing the major road from a stop-controlled approach.

d. Values shown are for a passenger car crossing a two-lane roadway with no median and with minor-road approach grades of 3% or less.

The design values for the time gap (t_g) shown in **Table 8.26** incorporate these crossing times for two-lane roadways and are used to develop the length of the leg of the sight triangle along the major road in **Table 8.27**.

Table 8.27 Length of Sight Triangle Leg along Major Road—Case C1, Crossing Manoeuvre at Yield-Controlled Intersections

Major Road Design Speed (km/h)	Minor-Road Design Speed (km/h)			
	120	100	80-30	20
	Design Values (m)			
120	260	240	220	240
100	215	200	185	200
80	175	160	145	160
60	130	120	110	120
50	110	100	95	100
40	90	80	75	80
30	65	60	55	60
20	45	40	40	40

Note -Values in the table are for small vehicles and are based on the unadjusted distances and times in **Table 8.26**. The distances and times in **Table 8.26** need to be adjusted using the factors in **Table 8.22**.

B. **LEFT AND RIGHT-TURN MANOEUVRES (CASE C2)**

To accommodate left and right turns without stopping (distance "a_1" in **Figure 8.49 (a)**), the length of the leg of the approach sight triangle along the minor road should be 25m. This distance is based on the assumption that drivers making right or left turns without stopping will slow to a turning speed of 15 km/h. The length of the leg of the approach sight triangle along the major road (distance "b" in **Figure 8.49 (b)**) is similar to that of the major-road leg of the departure sight triangle for Stop-controlled intersections in Cases B1 and B2.

For a Yield-controlled intersection, the travel times in **Table 8.23** should be increased by 0.5s. The appropriate lengths of the sight triangle leg are shown in **Table 8.28** for small vehicles. The minor-road vehicle requires 3.5s to travel from the decision point to the intersection. These 3.5s represent additional travel time that is needed at a Yield-controlled intersection (Case C). However, the acceleration time after entering the major road is 3.0s less for a Yield sign than for a Stop sign because the turning vehicle accelerates from 15km/h rather than from a stop. The net 0.5s increase in travel time for a vehicle turning from a Yield-controlled approach is the difference between the 3.5s increase in travel time on approach and the 3.0s reduction in travel time on departure explained above.

Since approach sight triangles for turning manoeuvres at Yield-controlled are larger than the departure sight triangles used at Stop-controlled intersections, no specific check of departure sight triangles at Yield-controlled intersections should be necessary.

Table 8.28 Design Intersection Sight Distance—Case C2, Left or Right Turn at Yield-Controlled Intersections

Design Speed (km/h)	Length of Leg (m)
120	270
100	225
80	180
60	135
50	115
40	90
30	70
20	45

Note: Intersection sight distance shown is for a small vehicle making a right or left turn without stopping onto a two-lane road.

8.10.4.1.7 INTERSECTIONS WITH TRAFFIC SIGNAL CONTROL (CASE D)

In general, approach or departure sight triangles are not needed for signalised intersections.

Indeed, signalisation may be an appropriate accident countermeasure for higher volume intersections with restricted sight distance and a history of sight-distance related accidents.

However, traffic signals may fail from time to time. Furthermore, traffic signals at an intersection are sometimes placed on two-way flashing operation under off-peak or nighttime conditions. To allow for either of these eventualities, the appropriate departure sight triangles for Case B, both to the left and to the right, should be provided for the minor-road approaches.

8.10.4.1.8 INTERSECTIONS WITH ALL-WAY STOP CONTROL (CASE E)

At intersections with all-way Stop control, the first stopped vehicle on each approach would be visible to the drivers of the first stopped vehicles on each of the other approaches. It is thus not necessary to provide sight distance triangles at intersections with All-way Stop control. All-way Stop control may be an option to consider where the sight distance for other types of control cannot be achieved. This is particularly the case if signals are not warranted.

8.10.4.1.9 LEFT TURNS FROM A MAJOR ROAD (CASE F)

Left-turning drivers need sufficient sight distance to enable them to decide when it is safe to turn left across the lane(s) used by opposing traffic. Sight distance design should be based on a left turn by a stopped vehicle, since a vehicle that turns left without stopping would need less sight distance. The sight distance along the major road to accommodate left turns is the distance that would be traversed at the design speed of the major road in the travel time for the appropriate design vehicle given in **Table 8.29**. This table also contains appropriate adjustment factors for the number of major-road lanes to be crossed by the turning vehicle.

If stopping sight distance has been provided continuously along the major road and, if sight distance for Case B (Stop control) or Case C (Yield control) has been provided for each minor-road approach, sight distance should generally be adequate for left turns from the major road. However, at three-leg intersections or accesses located on or near horizontal or vertical curves on the major road, the availability of adequate sight distance for left turns from the opposing traffic are possible, there should be sufficient sight distance to accommodate these manoeuvres. In the case of dual carriageways, the presence of sight obstructions in the median should also be checked.

At four-legged intersections, opposing vehicles turning right can block a driver's view of oncoming traffic. If right-turn lanes are provided, offsetting them to the right, to be directly opposite one other, will provide right-turning drivers with a better view of oncoming traffic.

Table 8.29 Time Gap for Case F, Left Turns from the Major Road

Design Vehicle	Time gap (s) at design speed of major road
Small vehicle (S-5) Medium vehicle (S-6)	5.5
Large vehicle (L-9 & L-12)	6.5
Trailer (T-17 & T-21)	7.5

Note: Time gaps are for a stopped vehicle turning left from a two-lane roadway with no median

For multilane and/or divided roadways—For left turns on two-way roadways across more than one opposing lane, including turn lanes, add 0.5s for small/medium vehicles or 0.7s for large vehicles and trailers for each additional lane to be crossed in the left-turn manoeuvre in excess of one lane. Where the left-turning vehicle must pass through a median, the median width should be converted to an equivalent number of lanes; for example, a 5.5m median would be equivalent to one and a half lanes and crossing through the median would require an additional 0.75s for small/medium vehicles and 1.05 s for large vehicles and trailers. The table also contains appropriate adjustment factors for the number of major-road lanes to be crossed by the turning vehicle. The unadjusted time gap in **Table 8.29** for passenger cars was used to develop the sight distances in **Table 8.30**.

Table 8.30 Intersection Sight Distance—Case F, Left Turn from the Major Road

Design Speed (km/h)	Length of Leg (m)
120	185
100	155
80	125
60	95
50	80
40	65
30	50
20	35

Note: Intersection sight distance shown is for a small vehicle making a left turn from an undivided roadway. For other conditions and design vehicles, the time gap should be adjusted, and the sight distance recalculated.

8.10.4.2 EFFECT OF SKEW

Where two roadways intersect at an angle less than 75^0 or greater than 105^0, and where conversion to a roundabout or realignment to increase the angle of intersection is not justified, some of the factors for determination of intersection sight distance may need adjustment.

Each of the clear sight triangles described above are applicable to oblique-angle intersections.

As shown in **Figure 8.50**, the legs of the sight triangle will lie along the intersection approaches and each sight triangle will be larger or smaller than the corresponding sight triangle would be at a right-angle intersection. The area within each sight triangle should be clear of potential sight obstructions as described previously.

At an oblique-angle intersection, the length of the travel paths for some turning and crossing manoeuvres will be increased. The actual path length for a turning or crossing manoeuvre can be computed by dividing the total widths of the lanes (plus the median width, where appropriate) to be crossed by the sine of the intersection angle. If the actual path length exceeds the total widths of the lanes to be crossed by 3.6m or more, then an appropriate number of additional lanes should be considered in applying the adjustment for the number of lanes to be crossed shown in **Table 8.23** for Case B1, **Table 8.24** for Case B2, and **Figure 8.25** for Case B3.

For Case C1, the w term in the equation for the major-road leg of the sight triangle to accommodate the crossing manoeuvre should also be divided by the sine of the intersection angle to obtain the actual path length. In the obtuse-angle quadrant of an oblique-angle intersection, the angle between the approach leg and the sight line is often so small that drivers can look across the full sight triangle with only a small head movement.

However, in the acute-angle quadrant, drivers often need to turn their heads considerably to see across the entire clear sight triangle. For this reason, it is recommended that the sight distance criteria for Case A not be applied to oblique-angle intersections and that sight distances at least equal to those for Case B should be provided, whenever practical.

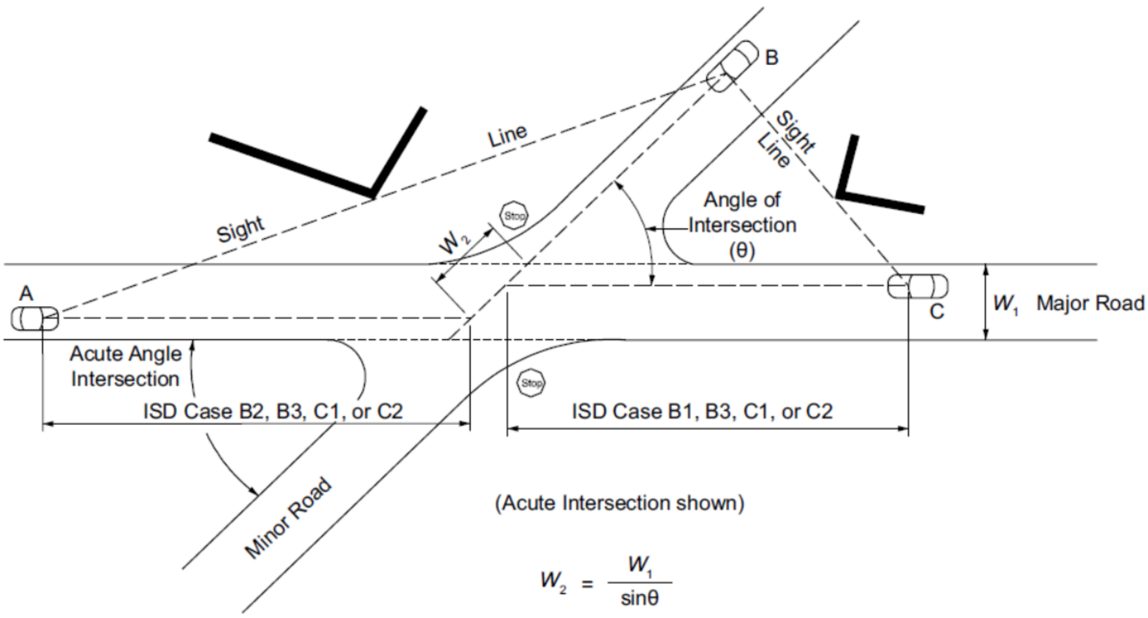


Figure 8.50 Sight Triangles at Skewed Intersections

8.11 ROUNDABOUT

8.11.1 OVERVIEW

A roundabout is an intersection with a central island around which traffic must travel anti-clockwise and in which entering traffic must yield to circulating traffic. Roundabouts ensures that safe and smooth traffic flow is secured under certain conditions of traffic volume. Roundabouts provides high capacity and minimum delay. They also have a good safety record largely because traffic speeds are low, and the number of potential traffic conflict is reduced to just 8 for a four-legged roundabout as compared to 32 at a crossroad.

The following should be considered when introducing a roundabout at an at-grade intersection:

i. Clarification of the expected function of the roundabout
ii. Feasibility should be confirmed from the viewpoints of traffic volume and geometric structure
iii. The benefits in terms of safety, smooth traffic flow, ease of maintenance and cost compared with other intersection types.

8.11.2 DEFINITION OF ROUNDABOUT COMPONENT ELEMENTS

The main elements and dimensions that make up a roundabout are shown in **Figure 8.51** and **Figure 8.52** respectively.

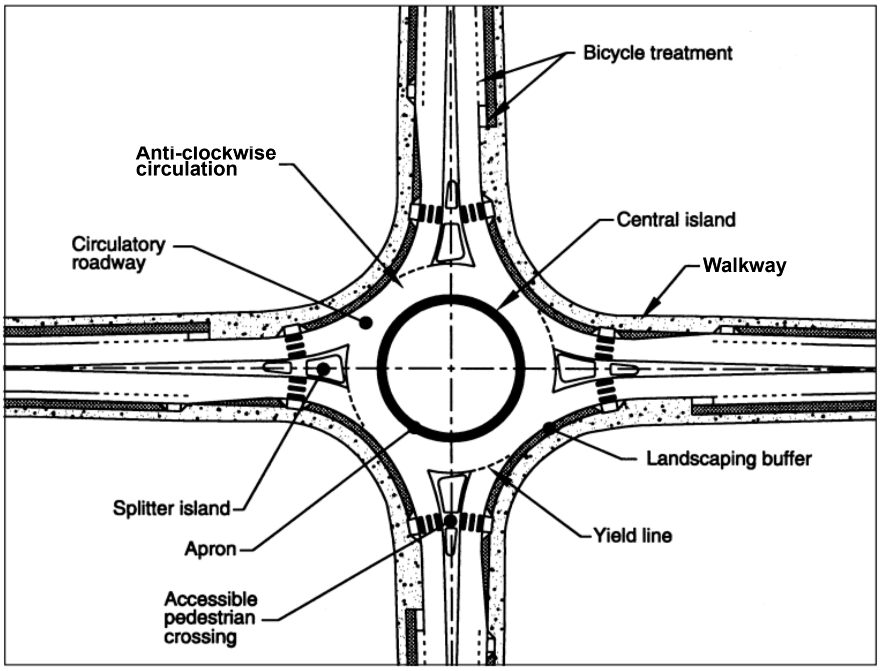


Figure 8.51 Basic Geometric Element of Roundabout

a. **Circulatory roadway**: Is the curve path used by vehicles to travel in a central anti-clockwise fashion around the central island.
b. **Central island:** Is the raised area in the centre of a roundabout around which traffic

circulate. This island is established in the middle of the roundabout to secure the safe and smooth passage of vehicles on the circular road.

c. **Apron:** Also known as truck apron, refers to a part that is provided for passage of larger vehicles and ordinary vehicles that have difficulty passing using only the circular road.

a. **Splitter Island**: A splitter island is a raised or painted area on an approach used to separate entering from exiting traffic, deflect and slow entering traffic, and provide storage space for pedestrians crossing the road in two stages.

b. **Yield Line:** A yield line is a pavement marking used to mark the point of entry from an approach into the circulatory roadway and is generally marked along the inscribed circle. Entering vehicles must yield to any circulating traffic coming from the left before crossing this line into the circulatory roadway.

c. **Accessible pedestrian crossings:** Accessible pedestrian crossings should be provided at all roundabouts. The crossing location is set back from the yield line, and the splitter island is cut to allow pedestrians, wheelchairs, strollers, and bicycles to pass through.

d. **Bicycle treatments:** Bicycle treatments at roundabouts provide bicyclists the option of traveling through the roundabout either as a vehicle or as a pedestrian, depending on the bicyclist's level of comfort.

e. **Landscaping buffer**: Landscaping buffers are provided at most roundabouts to separate vehicular and pedestrian traffic and to encourage pedestrians to cross only at the designated crossing locations. Landscaping buffers can also significantly improve the aesthetics of the intersection.

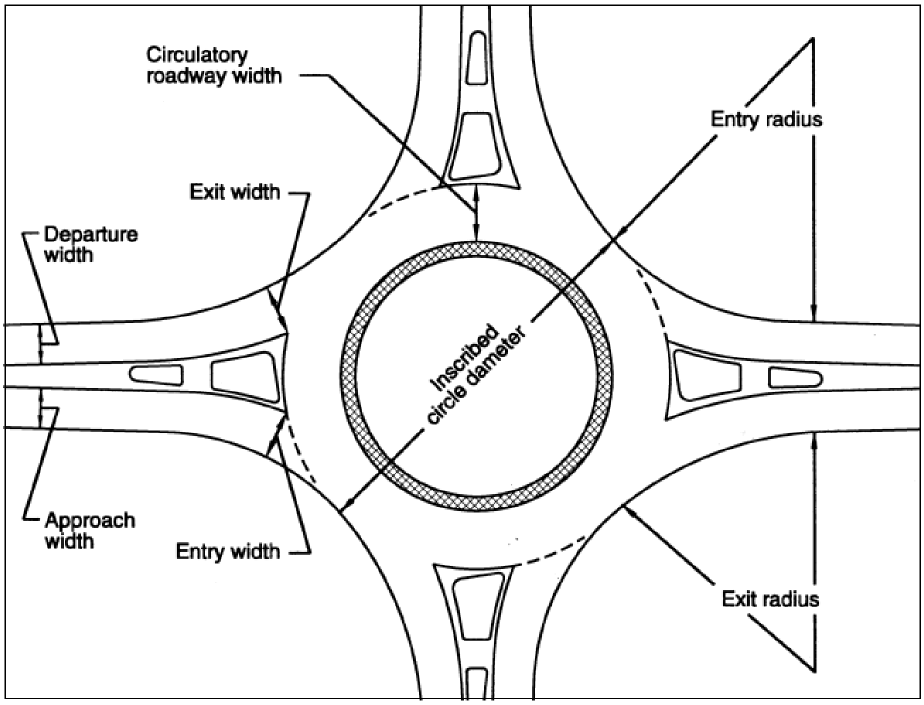


Figure 8.52 Key roundabout dimensions

f. **Inscribed circle diameter**: The inscribed circle diameter is the basic parameter used to define the size of a roundabout. It is measured between the outer edges of the circulatory roadway.

g. **Circulatory roadway width**: The circulatory roadway width defines the roadway width for vehicle circulation around the central island. It is measured as the width between the outer edge of this roadway and the central island. It does not include the width of any mountable apron, which is defined to be part of the central island.

h. **Approach width**: The approach width is the width of the roadway used by approaching traffic upstream of any changes in width associated with the roundabout. The approach width is typically not more than half of the total width of the roadway.

i. **Departure width**: The departure width is the width of the roadway used by departing traffic downstream of any changes in width associated with the roundabout. The departure width is typically less than or equal to half of the total width of the roadway.

j. **Entry width**: The entry width defines the width of the entry where it meets the inscribed circle. It is measured perpendicularly from the right edge of the entry to the intersection point of the left edge line and the inscribed circle.

k. **Exit width**: The exit width defines the width of the exit where it meets the inscribed circle. It is measured perpendicularly from the right edge of the exit to the intersection point of the left edge line and the inscribed circle.

l. **Entry radius**: The entry radius is the minimum radius of curvature of the outside kerb at the entry.

m. **Exit radius**: The exit radius is the minimum radius of curvature of the outside kerb at the exit.

8.11.3 ROUNDABOUT CATEGORIES

For the purpose of this guide, roundabouts have been categorized according to size and environment to facilitate discussion of specific performance or design issues. There are six basic categories based on environment, number of lanes, and size.

i. Mini roundabouts
ii. Urban compact roundabouts
iii. Urban single-lane roundabouts
iv. Urban double-lane roundabouts
v. Rural single-lane roundabouts
vi. Rural double-lane roundabouts

Multilane roundabouts with more than two approach lanes are possible, but they are not covered explicitly by this guide, although many of the design principles contained in this guide would still apply. For example, the guide provides guidance on the design of flaring approaches from one to two lanes. Although not explicitly discussed, this guidance could be extended to the design of larger roundabout entries. Note that separate categories have not been explicitly identified for suburban environments.

Suburban settings may combine higher approach speeds common in rural areas with multimodal activity that is more similar to urban settings. Therefore, they should generally be designed as urban roundabouts, but with the highspeed approach treatments recommended for rural roundabouts. In most cases, designers should anticipate the needs of pedestrians, bicyclists, and large vehicles.

Whenever a raised splitter island is provided, there should also be an at-grade pedestrian refuge. In this case, the pedestrian crossing facilitates two separate moves: kerb-to-island and island-to-kerb. The exit crossing will typically require more vigilance from the pedestrian and motorist than the entry crossing. Further, it is recommended that all urban crosswalks be marked. Under all urban design categories, special attention should be given to assist pedestrian users who are visually impaired or blind, through design elements.

For example, these users typically attempt to maintain their approach alignment to continue across a street in the crosswalk, since the crosswalk is often a direct extension of the walkway. A roundabout requires deviation from that alignment, and attention needs to be given to providing appropriate informational cues to pedestrians regarding the location of the walkway and the crosswalk, even at mini roundabouts. For example, appropriate landscaping is one method of providing some information. Another is to align the crosswalk ramps perpendicular to the pedestrian's line of travel through the pedestrian refuge.

8.11.3.1 COMPARISON OF ROUNDABOUT CATEGORIES

Table 8.31 summarizes and compares some fundamental design and operational elements for each of the six roundabout categories developed for this guide. The following sections provide a qualitative discussion of each category.

Table 8.31 Basic design characteristics for each of the six roundabout categories

Design Element	Mini-Roundabout	Urban Compact	Urban Single-Lane	Urban Double-Lane	Rural Single-Lane	Rural Double-Lane
Recommended maximum entry design speed (km/h)	25	25	35	40	40	50
Maximum number of entering lanes per approach	1	1	1	2	1	2
Typical inscribed circle diameter[1] (m)	13 - 25	25 - 30	30 - 40	45 - 55	35 - 40	55 - 60
Splitter island treatment	Raised, if possible, crosswalk cut if raised	Raised, with crosswalk cut	Raised, with crosswalk cut	Raised, with crosswalk cut	Raised and extended, with crosswalk cut	Raised and extended, with crosswalk cut
Typical daily service volumes on 4-leg roundabout (veh/day)	10,000	15,000	20,000	≥20,000	20,000	≥20,000

1. Assumes 90º entries and no more than four legs.

8.11.3.2 MINI-ROUNDABOUTS

Mini-roundabouts are small roundabouts used in low-speed urban environments, with average operating speeds of 60km/h or less. **Figure 8.53** provides an example of a typical mini roundabout. They can be useful in low-speed urban environments in cases where conventional roundabout design is precluded by right-of-way constraints. In retrofit applications, mini-roundabouts are relatively inexpensive because they typically require minimal additional pavement at the intersecting roads-for example, minor widening at the corner kerbs. They are mostly recommended when there is insufficient right-of-way for an urban compact roundabout. Because they are small, mini-roundabouts are perceived as pedestrian-friendly with short crossing distances and very low vehicle speeds on approaches and exits.

The mini-roundabout is designed to accommodate passenger cars without requiring them to drive over the central island. To maintain its perceived compactness and low speed characteristics, the yield lines are positioned just outside of the swept path of the largest expected vehicle. However, the central island is mountable, and larger vehicles may cross over the central island, but not to the left of it. Speed control around the mountable central island should be provided in the design by requiring horizontal deflection. Capacity for this type of roundabout is expected to be similar to that of the compact urban roundabout.

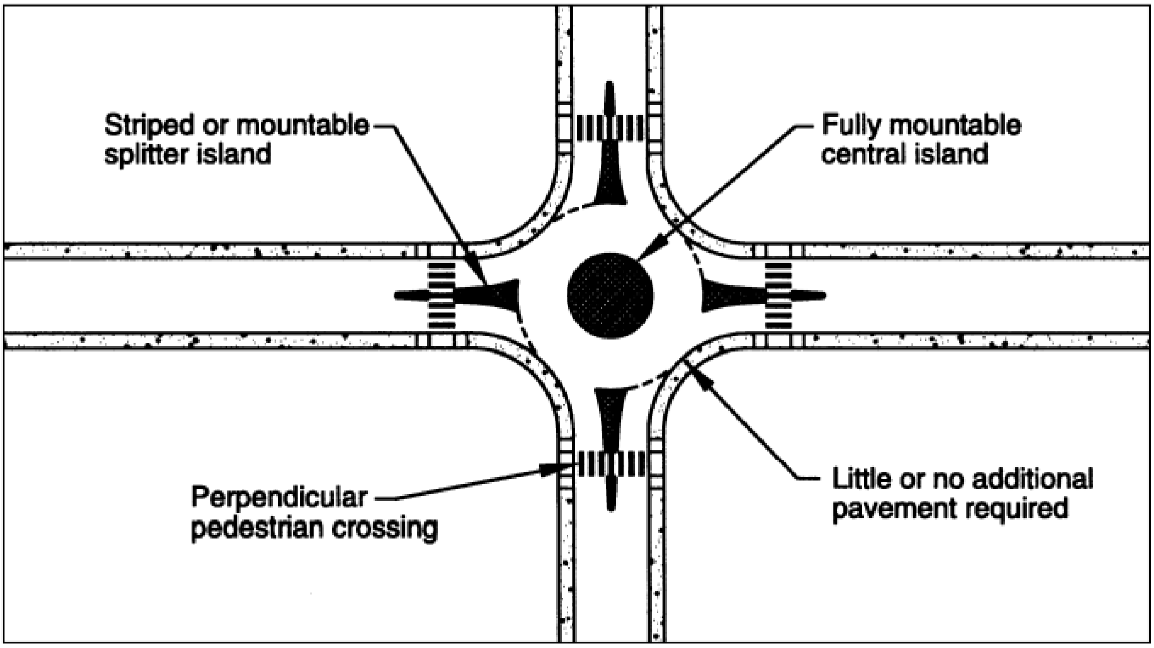


Figure 8.53 Typical mini-roundabout

8.11.3.3 URBAN COMPACT ROUNDABOUTS

Like mini-roundabouts, urban compact roundabouts are intended to be pedestrian and bicyclist-friendly because their perpendicular approach legs require very low vehicle speeds to make a distinct right turn into and out of the circulatory roadway. All legs have single-lane entries.

However, the urban compact treatment meets all the design requirements of effective roundabouts.

The principal objective of this design is to enable pedestrians to have safe and effective use of the intersection. Capacity should not be a critical issue for this type of roundabout to be considered. The geometric design includes raised splitter islands that incorporate at-grade pedestrian storage areas, and a non-mountable central island. There is usually an apron surrounding the non-mountable part of the compact central island to accommodate large vehicles. **Figure 8.54** provides an example of a typical urban compact roundabout.

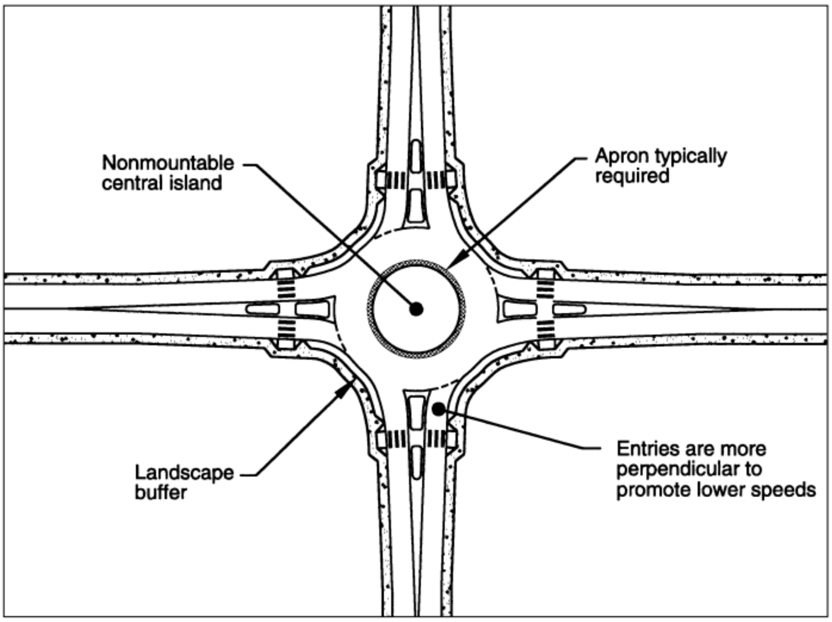


Figure 8.54 Typical urban compact roundabout

8.11.3.4 URBAN SINGLE-LANE ROUNDABOUTS

This type of roundabout is characterized as having a single lane entry at all legs and one circulatory lane. **Figure 8.55** provides an example of a typical urban single-lane roundabout. They are distinguished from urban compact roundabouts by their larger inscribed circle diameters and more tangential entries and exits, resulting in higher capacities. Their design allows slightly higher speeds at the entry, on the circulatory roadway, and at the exit. Notwithstanding the larger inscribed circle diameters than compact roundabouts, the speed ranges recommended in this guide are somewhat lower than those used in other countries, in order to enhance safety for bicycles and pedestrians. The roundabout design is focused on achieving consistent entering and circulating vehicle speeds. The geometric design includes raised splitter islands, a non-mountable central island, and preferably, no apron.

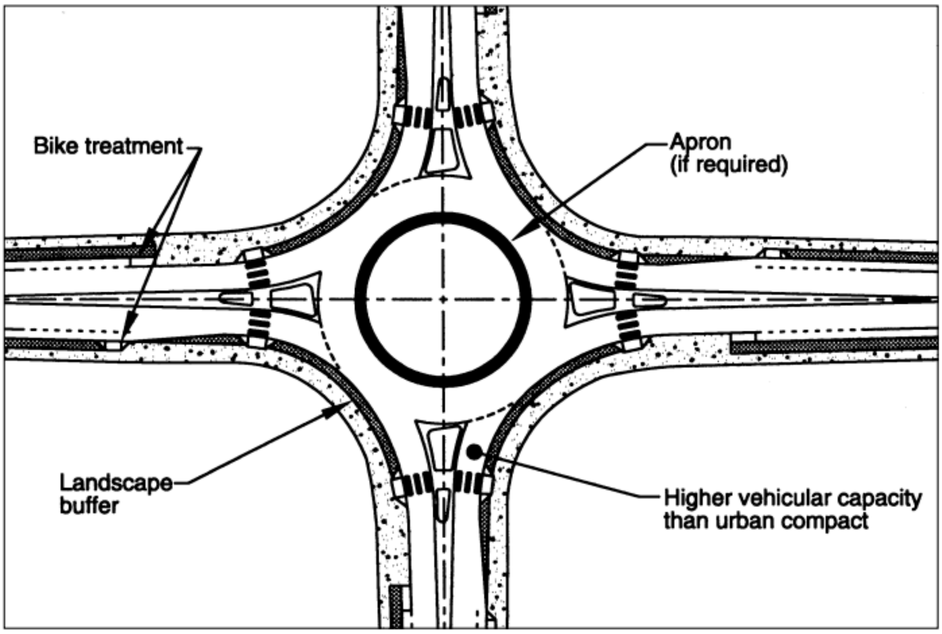


Figure 8.55 Typical urban single-lane roundabout

8.11.3.5 URBAN DOUBLE-LANE ROUNDABOUTS

Urban double-lane roundabouts include all roundabouts in urban areas that have at least one entry with two lanes. They include roundabouts with entries on one or more approaches that flare from one to two lanes. These require wider circulatory roadways to accommodate more than one vehicle traveling side by side. **Figure 8.56** provides an example of a typical urban multilane roundabout.

The speeds at the entry, on the circulatory roadway, and at the exit are similar to those for the urban single-lane roundabouts. Again, it is important that the vehicular speeds be consistent throughout the roundabout. The geometric design will include raised splitter islands, no truck apron, a non-mountable central island, and appropriate horizontal deflection.

Alternate routes may be provided for bicyclists who choose to bypass the roundabout. Bicycle and pedestrian pathways must be clearly delineated with walkway construction and landscaping to direct users to the appropriate crossing locations and alignment.

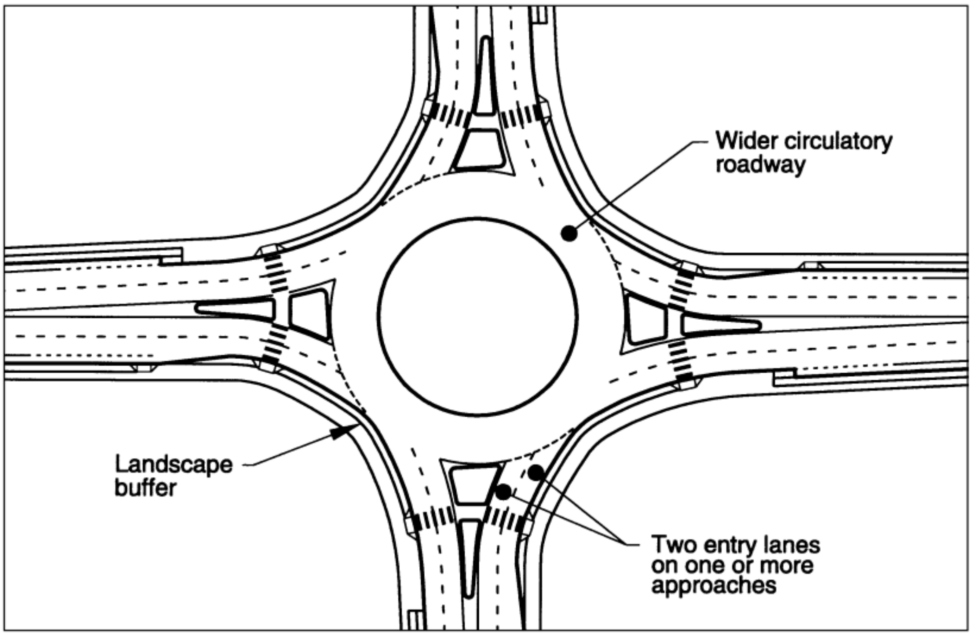


Figure 8.56 Typical urban double-lane roundabout

8.11.3.6 RURAL SINGLE-LANE ROUNDABOUTS

Rural single-lane roundabouts generally have high average approach speeds in the range of 80 to 100 km/h. They require supplementary geometric and traffic control device treatments on approaches to encourage drivers to slow to an appropriate speed before entering the roundabout.

Rural roundabouts may have larger diameters than urban roundabouts to allow slightly higher speeds at the entries, on the circulatory roadway, and at the exits. This is possible if few pedestrians are expected at these intersections, currently and in future. There is preferably no apron because their larger diameters should accommodate larger vehicles.

Supplemental geometric design elements include extended and raised splitter islands, a non-mountable central island, and adequate horizontal deflection. **Figure 8.57** provides an example of a typical rural single-lane roundabout. Rural roundabouts that may one day become part of an urbanized area should be designed as urban roundabouts, with slower speeds and pedestrian treatments. However, in the interim, they should be designed with supplementary approach and entry features to achieve safe speed reduction.

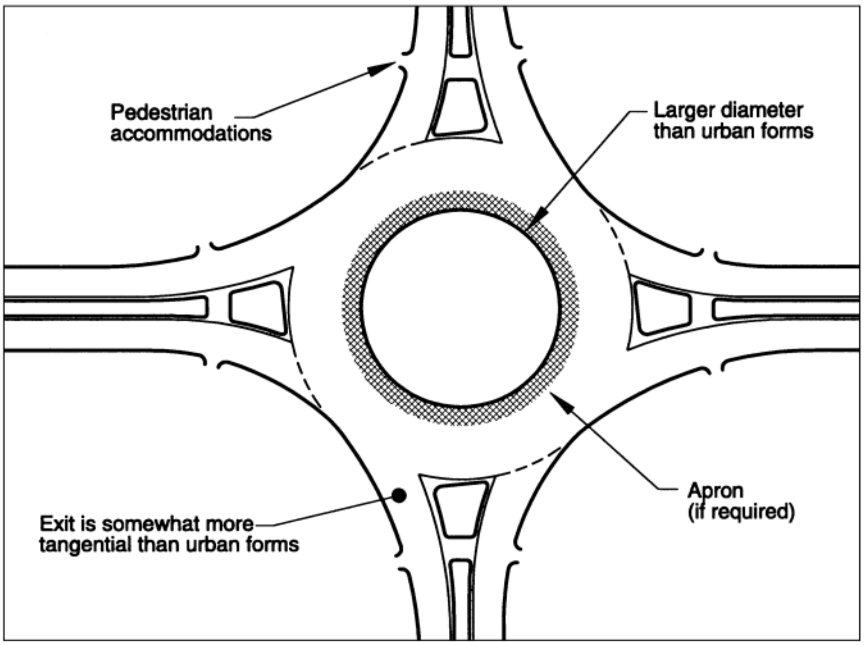


Figure 8.57 Typical rural single-lane roundabout.

8.11.3.7 RURAL DOUBLE-LANE ROUNDABOUTS

Rural double-lane roundabouts have speed characteristics similar to rural single lane roundabouts with average approach speeds in the range of 80 to 100 km/h. They differ in having two entry lanes, or entries flared from one to two lanes, on one or more approaches. Consequently, many of the characteristics and design features of rural double-lane roundabouts mirror those of their urban counterparts.

The main design differences are designs with higher entry speeds and larger diameters and recommended supplementary approach treatments. **Figure 8.58** provides an example of a typical rural double lane roundabout. Rural roundabouts that may one day become part of an urbanized area should be designed for slower speeds, with design details that fully accommodate pedestrians and bicyclists. However, in the interim they should be designed with approach and entry features to achieve safe speed reduction.

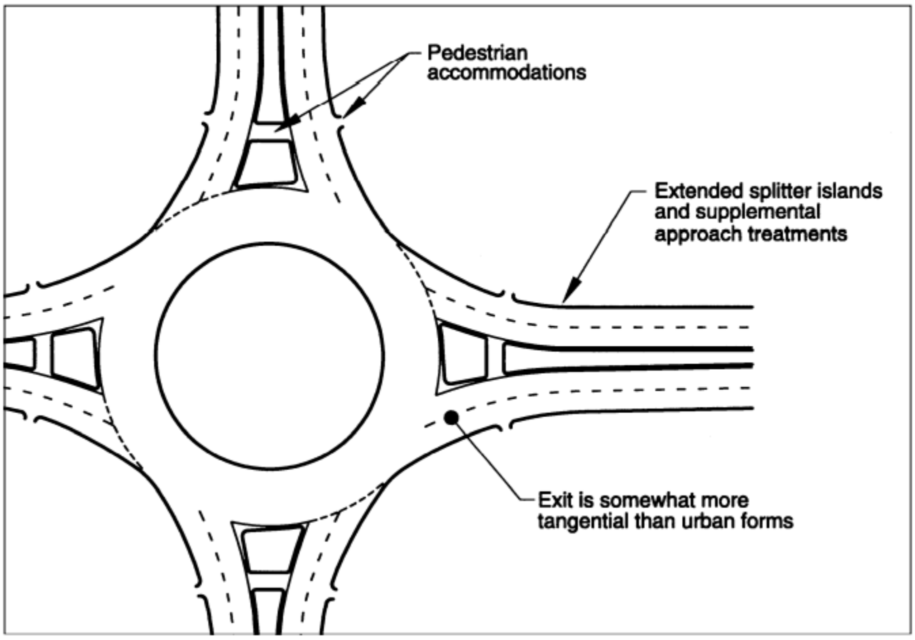


Figure 8.58 Typical rural double-lane roundabout

8.11.4 PLANNING FOR ROUNDABOUT

Planning for roundabouts begins with specifying a preliminary configuration. The configuration is specified in terms of the minimum number of lanes required on each approach and, thus, which roundabout category is the most appropriate basis for design: urban or rural, single-lane or double-lane roundabout. Given sufficient space, roundabouts can be designed to accommodate high traffic volumes. There are many additional levels of detail required in the design and analysis of a high-capacity, multi-lane roundabout that are beyond the scope of a planning level procedure. Therefore, this section focuses on the more common questions that can be answered using reasonable assumptions and approximations.

Feasibility analysis requires an approximation of some of the design parameters and operational characteristics. Some changes in these approximations may be necessary as the design evolves.

8.11.4.1 PLANNING CONSIDERATIONS

Figure 8.59 outlines many of the considerations that may need to be investigated prior to deciding whether to implement a roundabout at an intersection. Note that this is not meant to be all encompassing, nor is it intended to reflect minimum requirements. Rather, it is intended to provide a general framework for the steps typically necessary to determine feasibility.

Consider the Context

- Is this the first roundabout in a community or are roundabouts already well established?
- Are there regional policy constraints that must be addressed?
- Are there site-specific and/or community impact reasons why a roundabout of any size would not be a good choice?
- What are the site constraints?
- What is the potential for future growth within the vicinity?
- What is the current or desired environment for non-motorized modes?

Clarify the Objectives

- Clarifying the objective for considering a roundabout at the beginning of the process may help to better guide the selection of an appropriate treatment and the need for additional information.
- Is the improvement needed from an operational or safety perspective? Both?
- Is the improvement desired to control vehicle speeds?
- Is the improvement intended purely for aesthetic reasons?

Determine a preliminary lane configuration and roundabout category based on capacity requirements.

Section 8.11.4.3 will be useful for making a basic decision on the required number of lanes.

Determine the space requirements

How big does it need to be and is there enough right-of-way to build it? This is a potential rejection point in some locations due to potential cost or the additional administrative complications caused by right-of-way acquisition. Refer to for the right-of-way widths required to accommodate the inscribed circle diameter.

Compare to other alternatives

Make appropriate comparisons with alternative intersection treatments

Assess Other Impacts

Are there other impacts that may occur from the roundabout, such as:

- Utilities;
- Existing buildings/structures;
- Business access; and
- Sensitive environmental areas.

Assess Other Opportunities

Does the roundabout offer any opportunities to improve existing conditions, such as:

- Improve access management;
- Stimulate redevelopment;
- Improve safety; and
- Improve oddly shaped intersection or
- other poor geometric condition.

Is a roundabout feasible and/or the preferred alternative worthy of advancing for additional analysis and design?

Figure 8.59 Planning Framework

8.11.4.2 CONSIDERATIONS OF CONTEXT

8.11.4.2.1 DECISION ENVIRONMENTS

There are three somewhat different policy environments in which a decision may be made to construct a roundabout at a specific location. While the same basic analysis tools and concepts apply to all the environments, the relative importance of the various aspects and observations may differ, as may prior constraints that are imposed at higher policy levels.

A new roadway system -Fewer constraints are generally imposed if the location under consideration is not a part of an existing roadway system. Right-of-way is usually easier to acquire or commit. Other intersection forms also offer viable alternatives to roundabouts. There are generally no field observations of site-specific problems that must be addressed. This situation is more likely to be faced by developers than by public agencies.

The first roundabout in an area - The first roundabout in any geographic area requires an implementing agency to perform due diligence on roundabouts regarding their operational and design aspects, community impacts, user needs, and public acceptability. On the other hand, a successfully implemented roundabout, especially one that solves a perceived problem, could be an important factor in gaining support for future roundabouts at locations that could take advantage of the potential benefits that roundabouts may offer. Some important considerations for this decision environment include:

i. Effort should be directed toward gaining community and institutional support for the selection of a site for the first roundabout in an area. Public acceptance for roundabouts, like any new roadway facility, require agency staff to understand the potential issues and communicate these effectively with the impacted community.
ii. An extensive justification effort may be necessary to gain the required support.
iii. A cautious and conservative approach may be appropriate; careful consideration should be given to conditions that suggest that the benefits of a roundabout might not be fully realized. Collecting data on current users of the facility can provide important insights regarding potential issues and design needs.
iv. A single-lane roundabout in the near-term is more easily understood by most drivers and therefore may have a higher probability of acceptance by the motoring public.
v. The choice of design and analysis procedures could set a precedent for future roundabout implementation; therefore, the full range of design and analysis alternatives should be explored in consultation with other operating agencies in the region.
vi. After the roundabout is constructed, evaluating its operation and the public response could provide documentation to support future installations.

Retrofit to an existing intersection in an area where roundabouts have already gained acceptance - This environment is one in which a solution to a site-specific problem is being sought. Because drivers are familiar with roundabout operation, a less intensive process may suffice. Double-lane roundabouts could be considered, and the regional design and evaluation procedures should have already been agreed upon. The basic objectives of the selection process in this case are to demonstrate the community impacts and that a roundabout will function properly during the peak period within the capacity limits imposed by the space available; and to decide whether one is the preferred alternative.

It is a common desire to avoid intersection designs that require additional right-of-way, because of the effort and expense involved in right-of-way acquisition. Important questions to be addressed in the planning phase are therefore:

i. Will a minimally configured roundabout (i.e., single-lane entrances and circulatory roadway) provide adequate capacity and performance for all users, or will additional lanes be required on some legs or at some future time?
ii. Can the roundabout be constructed within the existing right-of-way, or will it be necessary to acquire additional space beyond the property lines?
iii. Can a single-lane roundabout be upgraded in the future to accommodate growth?

If not, a roundabout alternative may require that more rigorous analysis and design be conducted before a decision is made.

8.11.4.2.2 SITE-SPECIFIC CONDITIONS

Some conditions may preclude a roundabout at a specific location. Certain site-related factors may significantly influence the design and require a more detailed investigation of some aspects of the design or operation. A number of these factors (many of which are valid for any intersection type) are listed below:

i. Physical or geometric complications that make it impossible or uneconomical to construct a roundabout. These could include right-of-way limitations, utility conflicts, drainage problems, etc.
ii. Proximity of generators of significant traffic that might have difficulty negotiating the roundabout, such as high volumes of oversized trucks.
iii. Proximity of other traffic control devices that would require pre-emption, such as railroad tracks, drawbridges, etc.
iv. Proximity of bottlenecks that would routinely back up traffic into the roundabout, such as over-capacity signals, expressway/motorway entrance ramps, etc. The successful operation of a roundabout depends on unimpeded flow on the circulatory roadway. If traffic on the circulatory roadway comes to a halt, momentary intersection gridlock can occur. In comparison, other control types may continue to serve some movements under these circumstances.
v. Problems of grades or unfavourable topography that may limit visibility or complicate construction.

vi. Intersections of a major arterial and a minor arterial or local road where an unacceptable delay to the major road could be created. Roundabouts delay and deflect all traffic entering the intersection and could introduce excessive delay or speed inconsistencies to flow on the major arterial.

vii. Heavy pedestrian or bicycle movements in conflict with high traffic volumes. These conflicts pose a problem for all types of traffic control.

viii. Intersections located on arterial roads within a coordinated signal network. In these situations, the level of service on the arterial might be better with a signalized intersection incorporated into the system.

8.11.4.2.3 POTENTIAL APPLICATIONS

Roundabouts serve as one potential tool within the toolbox of intersection control options and should be considered in a wide array of possible applications. There are numerous reasons for selecting a roundabout as a preferred alternative, with each reason carrying its own considerations and trade-offs. **Table 8.32** provides a cursory overview of several example locations or situations where roundabouts are often considered. It also highlights situations where trade-offs may exist or certain aspects of the overall roundabout design may require further investigation to determine the feasibility of a roundabout and whether it is the preferred alternative.

Table 8.32 Potential applications of roundabout

Application	Benefits	Considerations
New residential subdivision	i. Calming effect on traffic promotes lower travel speeds ii. Aesthetic benefits (community enhancement/gateway treatment) iii. Single-lane roundabout often iv. appropriate given relatively low traffic volumes within neighbourhoods v. Pedestrian and bicycle friendly	i. Design vehicle emergency/fire, garbage, large moving trucks) ii. Right-of-way needs iii. Driveway access to corner properties iv. Landscaping v. Illumination
Urban centres	i. Promotes lower vehicular speeds and can reduce delay and emissions ii. Enhances pedestrian safety iii. Provides for aesthetic treatments (monuments, landscaping, etc.) iv. Low maintenance (no signals, detector loops) v. Complementary to access management programs	i. Design vehicle ii. Right-of-way needs iii. Accessibility for pedestrians who are blind or have low vision iv. Emergency vehicle access/parking v. Roadway system operations (e.g., interaction with adjacent signals) vi. Sight distance

Application	Benefits	Considerations
Suburban municipalities and small towns	i. May improve operations and decrease delay compared to two-way stop-control (TWSC) or signalized control ii. May provide a safer alternative to signalized control for locations where TWSC fails but minor street volumes remain relatively low iii. May address an existing safety deficiency iv. Lower speeds v. Lower maintenance costs	i. Design vehicle ii. Pedestrian, bicycle, and transit access iii. Central island maintenance iv. Intersection visibility under high speed conditions
Rural settings and small communities	i. May improve operations and decrease delay compared to TWSC or signalized control ii. May provide safer alternative to signalized control for locations where TWSC fails but minor street volumes remain relatively low iii. May address an existing safety deficiency iv. Lower speeds v. Lower maintenance costs	i. Design vehicle ii. Pedestrian, bicycle and transit access iii. Central island maintenance iv. Intersection visibility under high speed conditions v. Illumination
Schools	i. Lower vehicle speeds in and around intersection i. Improved pedestrian and vehicle safety ii. Landscaping and gateway treatment	i. Design vehicle (school bus, emergency vehicles) ii. Right-of-way iii. User education and outreach iv. If crossing guards are used, the distance between crosswalks may require two crossing guards instead of one.
Interchanges	i. Lower vehicle speeds and reduced speed differential through interchange area ii. Narrower bridge cross section—reduced cost iii. Landscaping and gateway treatments	i. Design vehicle (trucks, emergency vehicles) ii. Right-of-way iii. Signing and wayfinding iv. Driver familiarity
Gateway and traffic calming treatments	i. Central island provides ample space for aesthetic treatments ii. Minimal impact to traffic operations iii. Increases landscaping opportunities	i. Design vehicle (trucks, emergency vehicles) ii. Right-of-way

Application	Benefits	Considerations
Commercial developments	i. Introduce geometric delay to slow drivers ii. Improve safety of both vehicular and non-automobile users iii. Landscaping opportunities can enhance local neighbourhoods iv. Where a series of roundabouts is used, the roundabouts allow for easy U-turn movements, so minor commercial driveways can easily be restricted to right-in, right-out, improving safety between intersections as well.	i. Design vehicle (emergency vehicles, ii. moving trucks) iii. Right-of-way iv. Access to adjacent properties into or near the roundabout
Unusual geometry	i. Effectively manage traffic flows in situations with unique geometric conditions ii. Reduced delay compared to signalized scenarios	i. Design vehicle (trucks, emergency vehicles) ii. Right-of-way iii. Entry path deflection and alignment

8.11.4.3 NUMBER OF ENTRY LANES

A basic question that needs to be answered is how many entry lanes a roundabout would require to serve the traffic demand. The capacity of a roundabout is clearly a critical parameter and one that should be checked at the outset of any feasibility study.

First, traffic volumes are generally represented for planning purposes in terms of Average Daily Traffic (ADT), or Average Annual Daily Traffic (AADT). Traffic operational analyses must be carried out at the design hour level. This requires an assumption of a K factor and a D factor to indicate, respectively, the proportion of the AADT assigned to the design hour, and the proportion of the two-way traffic that is assigned to the peak direction. All the planning-level procedures offered are based on reasonably typical assumed values for K of 0.1 and D of 0.58.

There are two site-specific parameters that must be taken into account in all computations. The first is the proportion of traffic on the major road. For roundabout planning purposes, this value is assumed to lie between 0.5 and 0.67. All analyses assume a four-leg intersection. The proportion of left turns must also be considered, since left turns affect all traffic control modes adversely. Right turns are assumed to be 10 percent in all cases. Right turns are included in approach volumes and require capacity but are not included in the circulating volumes downstream because they exit before the next entrance.

The AADT that can be accommodated is conservatively estimated as a function of the proportion of left turns, for crossroad volume proportions of 50 percent and 67 percent. For acceptable roundabout operation, many sources advise that the volume-to-capacity ratio on any leg of a roundabout does not exceed 0.85. This assumption was used in deriving the AADT maximum service volume relationship.

8.11.4.3.1 SINGLE- AND DOUBLE-LANE ROUNDABOUTS

The resulting maximum service volumes are presented in **Figure 8.60** for a range of left turns from 0 to 40 percent of the total volume. This range exceeds the normal expectation for left turn proportions. This procedure is offered as a simple, conservative method for estimating roundabout lane requirements. If the 24-hour volumes fall below the volumes indicated in **Figure 8.60**, a roundabout should have no operational problems at any time of the day. It is suggested that a reasonable approximation of lane requirements for a three-leg roundabout may be obtained using 75 percent of the service volumes shown on **Figure 8.60**.

If the volumes exceed the threshold suggested in **Figure 8.60**, a single-lane or double-lane roundabout may still function quite well, but a closer look at the actual turning movement volumes during the design hour is required.

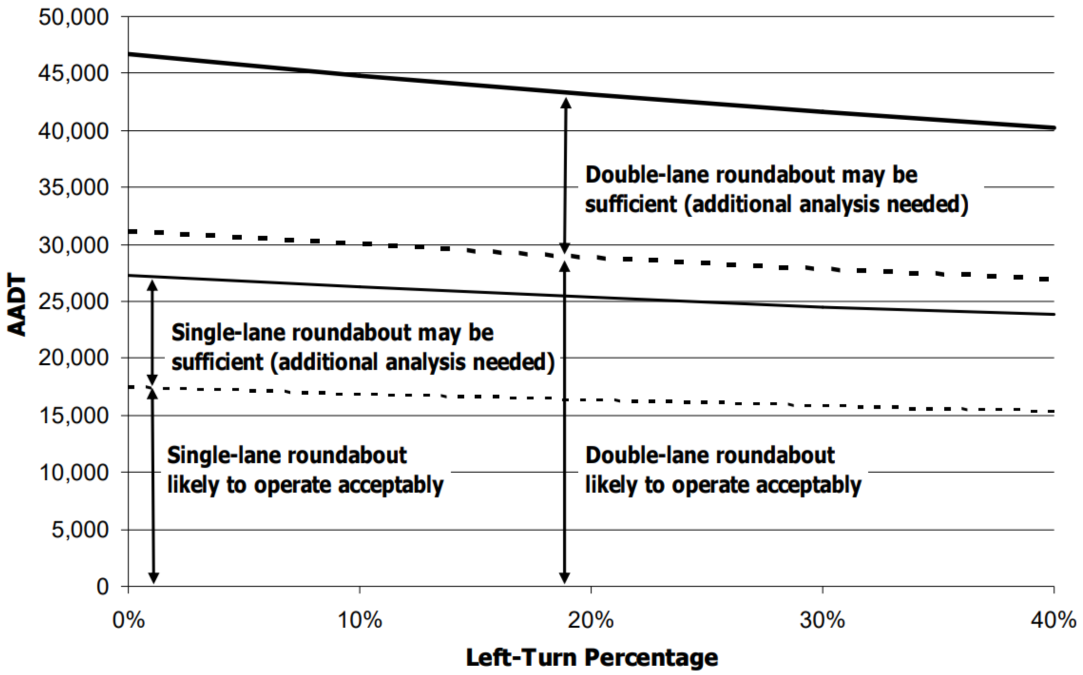


Figure 8.60 Planning-Level Daily Intersection Volumes

8.11.4.3.2 MINI-ROUNDABOUTS

Mini-roundabouts are distinguished from traditional roundabouts primarily by their smaller size and more compact geometry. They are typically designed for negotiation speeds of 25 km/h. Inscribed circle diameters generally vary from 13m to 25m. Mini-roundabouts are usually implemented with safety in mind, as opposed to capacity. Peak-period capacity is seldom an issue, and most mini-roundabouts operate on residential or collector roads at demand levels well below their capacity. It is important, however, to be able to assess the capacity of any proposed intersection design to ensure that the intersection would function properly if constructed.

At very small roundabouts, it is reasonable to assume that each quadrant of the circulatory roadway can accommodate only one vehicle at a time. In other words, a vehicle may not enter the circulatory roadway unless the quadrant on both sides of the approach is empty. Given a set of demand volumes for each of the 12 standard movements at a four-leg roundabout, it is possible to simulate the roundabout to estimate the maximum service volumes and delay for each approach. By making assumptions about the proportion of left turns and the proportion of cross road traffic, a general estimate of the total entry maximum service volumes of the roundabout can be made, and is provided in **Figure 8.61**. AADT maximum service volumes are represented based on an assumed K value of 0.10. Note that these volumes range from slightly more than 12,000 to slightly less than 16,000 vehicles per day. The maximum throughput is achieved with an equal proportion of vehicles on the major and minor roads, and with low proportions of left turns.

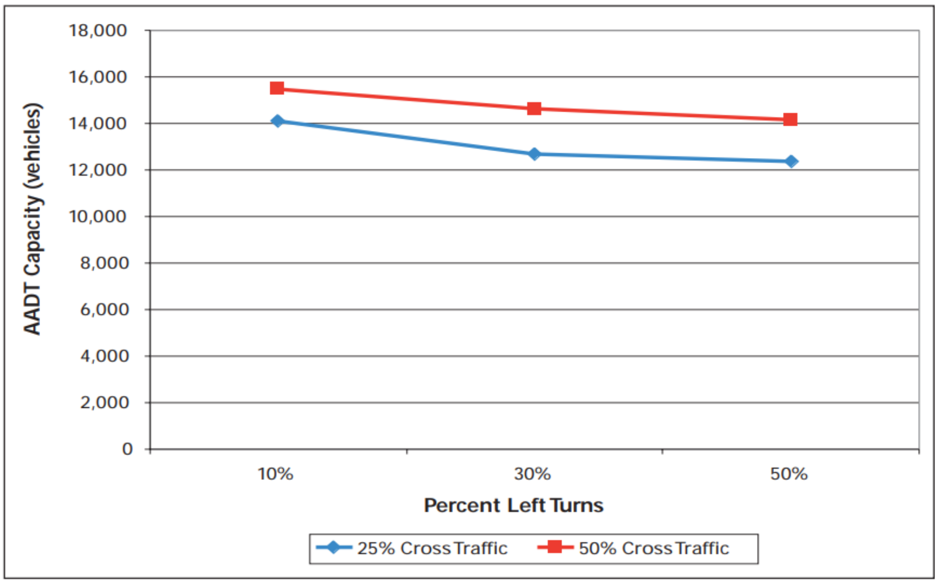


Figure 8.61 Planning-level maximum daily service volumes for mini-roundabouts

8.11.4.4 SELECTION CATEGORIES

There are many locations at which a roundabout could be selected as the preferred traffic control mode. There are several reasons why this is so, and each reason creates a separate selection category. Each selection category, in turn, requires different information to demonstrate the desirability of a roundabout. The principal selection categories will be discussed in this section, along with their information requirements.

It is reasonable that the decision to install a roundabout should require approximately the same level of effort as the alternative control mode. In other words, if a roundabout is proposed as an alternative to a traffic signal, then the analysis effort should be approximately the same as that required for a signal. If the alternative is stop sign control, then the requirements could be relaxed.

The following situations present an opportunity to demonstrate the desirability of installing a roundabout at a specific location.

8.11.4.4.1 COMMUNITY ENHANCEMENT

Roundabouts have been proposed as a part of a community enhancement project and not as a solution to capacity problems. Such projects are often located in commercial and civic districts, as a gateway treatment to convey a change of environment and to encourage traffic to slow down. Traffic volumes are typically well below the thresholds shown in **Figure 8.60** otherwise, one of the more operationally oriented selection categories would normally be more appropriate.

Roundabouts proposed for community enhancement require minimal analysis as a traffic control device. The main focus of the planning procedure should be to demonstrate that they would not introduce traffic problems that do not exist currently. Particular attention should be given to any complications that would imply either operational or safety problems. The urban compact category may be the most appropriate roundabout for such applications. **Plate 8.6** provides an example of a roundabout installed primarily for community enhancement.

Plate 8.6 Example of community enhancement roundabout

8.11.4.4.2 TRAFFIC CALMING

The decision to install a roundabout for traffic calming purposes should be supported by a demonstrated need for traffic calming along the intersecting roadways. Most of the roundabouts in this category will be located on local roads. Examples of conditions that might suggest a need for traffic calming include:

i. Documented observations of speeding, high traffic volumes, or careless driving activities.
ii. Inadequate space for roadside activities, or a need to provide slower, safer conditions for non-automobile users.
iii. New construction (road opening, traffic signal, new road, etc.) which would potentially increase the volumes of "cut-through" traffic.

Capacity should be an issue when roundabouts are installed for traffic calming purposes only because traffic volumes on local roads will usually be well below the level that would create congestion. If this is not the case, another primary selection category would probably be more suitable. The urban mini-roundabout or urban compact roundabout are most appropriate for traffic calming purposes. **Plate 8.7** provides an example of roundabouts installed primarily for traffic calming.

Plate 8.7 Example of traffic calming roundabout

8.11.4.4.3 SAFETY IMPROVEMENT

The decision to install a roundabout as a safety improvement should be based on a demonstrated safety problem of the type susceptible to correction by a roundabout. A review of crash reports and the type of accidents occurring is essential. Examples of safety problems include:

i. High rates of crashes involving conflicts that would tend to be resolved by a roundabout (right angle, head-on, left/through, U-turns, etc.)
ii. High crash severity that could be reduced by the slower speeds associated with roundabouts.
iii. Site visibility problems that reduce the effectiveness of stop sign control (in this case, landscaping of the roundabout needs to be carefully considered).
iv. Inadequate separation of movements, especially on single-lane approaches.

8.11.4.4.4 OPERATIONAL IMPROVEMENT

A roundabout may be considered as a logical choice if its estimated performance is better than alternative control modes, usually either stop or signal control. To simplify the selection process, the following assumptions are proposed for a planning-level comparison of control modes:

i. A roundabout will always provide a higher capacity and lower delays than AWSC operating with the same traffic volumes and right-of-way limitations.
ii. A roundabout is unlikely to offer better performance in terms of lower overall delays than TWSC at intersections with minor movements (including crossroad entry and major road left turns) that are not experiencing, nor predicted to experience,

operational problems under TWSC.

iii. A single-lane roundabout may be assumed to operate within its capacity at any intersection that does not exceed the peak-hour volume warrant for signals.

iv. A roundabout that operates within its capacity will generally produce lower delays than a signalized intersection operating with the same traffic volumes and right-of-way limitations.

8.11.4.4.5 SPECIAL SITUATIONS

It is important that the selection process does not discourage the construction of a roundabout at any location where a roundabout would be a logical choice. Some flexibility must be built into the process by recognizing that the selection categories above are not all-inclusive. There may still be other situations that suggest that a roundabout would be a sensible control choice. Many of these situations are associated with unusual alignment or geometry where other solutions are intractable.

8.11.4.5 SPACE REQUIREMENTS

Roundabouts that are designed to accommodate vehicles larger than passenger cars or small trucks typically require more space than conventional intersections. However, this may be more than offset by the space saved compared with turning lane requirements at alternative intersection forms. The key indicator of the required space is the inscribed circle diameter. A detailed design is required to determine the space requirements at a specific site, especially if more than one lane is needed to accommodate the entering and circulating traffic. This is, however, another case in which the use of assumptions and approximations can produce preliminary values that are adequate for planning purposes.

One important question is whether or not the proposed roundabout will fit within the existing property lines, or whether additional right-of-way will be required. Four examples have been created to demonstrate the spatial effects of comparable intersection types, and the assumptions are summarized in **Table 8.33**. Note that there are many combinations of turning volumes that would affect the actual lane configurations and design storage lengths. Therefore, these examples should not be used out of context.

Table 8.33 Assumptions for spatial comparison of roundabouts and comparable conventional intersections.

Category	Roundabout Type		Conventional Intersection	
	Main Road Approach Lanes	Minor Road Approach Lanes	Main Road Approach Lanes	Minor Road Approach Lanes
Urban compact	1	1	1	1
Urban single-lane	1	1	1 + LT pocket	1
Urban double-lane	2	1	2 + LT pocket	1 + LT pocket
Urban double-lane with flaring	1 flared to 2	1	2 + LT pocket	1 + LT pocket

Note, LT = Left turn

Roundabouts typically require more area at the junction than conventional intersections. However, as capacity needs increase the size of the roundabout and comparable conventional (signalized) intersection, the increase in space requirements are increasingly offset by a reduction in space requirements on the approaches. This is because the widening or flaring required for a roundabout can be accomplished in a shorter distance than is typically required to develop left turn lanes and transition tapers at conventional intersections.

Flared roundabouts offer the most potential for reducing spatial requirements on the approaches as compared to conventional intersections. This effect of providing capacity at the intersections while reducing lane requirements between intersections is known as “wide nodes and narrow roads” as indicated in **Figure 8.62** and **Plate 8.8**.

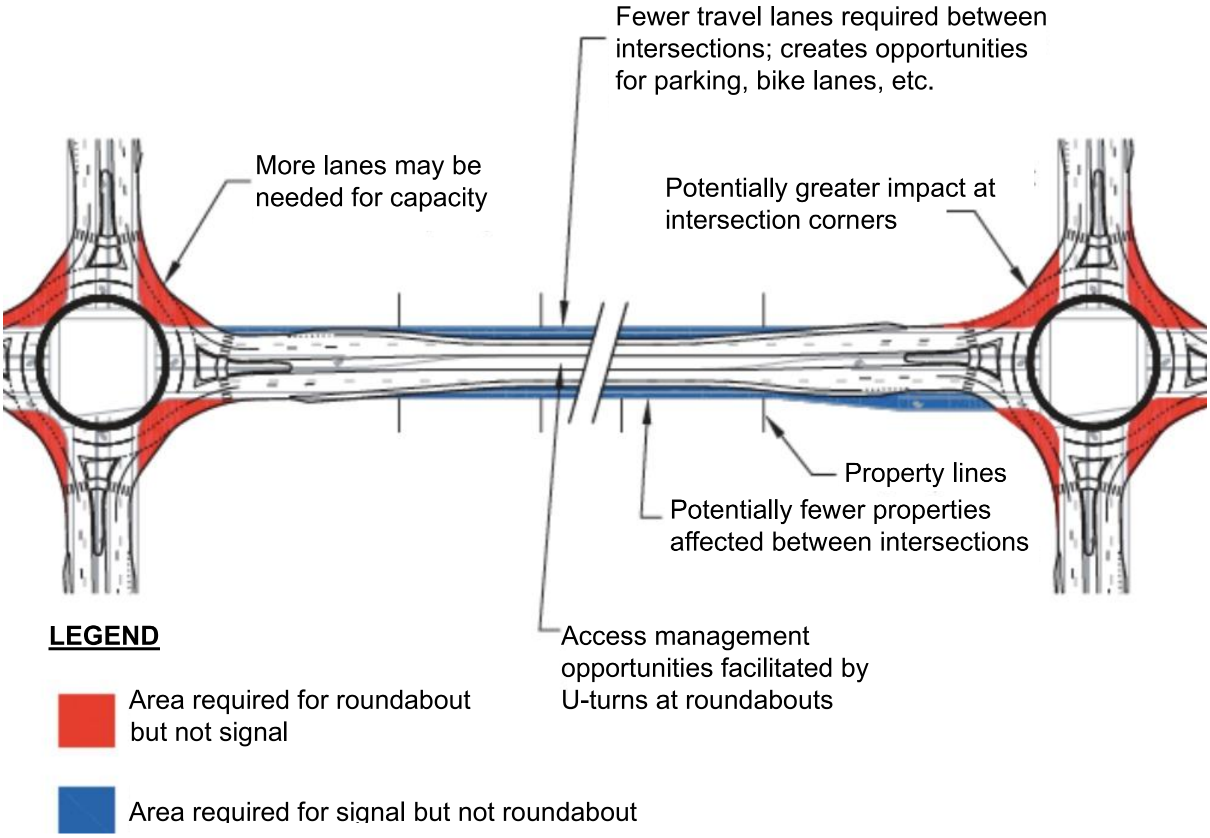


Figure 8.62 Wide nodes and narrow roads

Plate 8.8 Example of wide nodes, narrow roads concept

8.11.5 OPERATIONAL ANALYSIS

This section presents methods for analysing the operation of an existing or planned roundabout. The methods allow a transportation analyst to assess the operational performance of a facility, given information about the usage of the facility and its geometric design elements. An operational analysis produces two kinds of estimates:

i. the capacity of a facility (i.e., the ability of the facility to accommodate various streams of users) and
ii. the level of performance, often using one or more measures of effectiveness, such as delay and queues.

The Highway Capacity Manual (HCM) defines the capacity of a facility as "the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions." While capacity is a specific measure that can be defined and estimated, level of service (LOS) is a qualitative measure that "characterizes operational conditions within a traffic stream and their perception by motorists and passengers." To quantify LOS, the HCM defines specific measures of effectiveness for each highway facility type. Control delay is the measure of effectiveness that is used to define level of service at intersections as perceived by users. In addition to control delay, all intersections cause some drivers to also incur geometric delay when making turns. A systems analysis of a roadway network may include geometric delay because of the slower vehicle paths required for turning through intersections.

8.11.5.1 PRINCIPLES

The operational performance of roundabouts is relatively simple, although the techniques used to model performance can be quite complex. A few features are common to the modeling techniques employed by all analysis tools:

i. Drivers must yield the right-of-way to circulating vehicles and accept gaps in the circulating traffic stream. Therefore, the operational performance of a roundabout is directly influenced by traffic patterns and gap acceptance characteristics.
ii. As with other types of intersections, the operational performance of a roundabout is directly influenced by its geometry. The extent to which this influence is affected in the aggregate (e.g., number of lanes) or by design details (e.g., diameter) is discussed in more detail in this section.

The following sections discuss these principles in more detail.

8.11.5.1.1 EFFECT OF TRAFFIC FLOW AND DRIVER BEHAVIOR

The capacity of a roundabout entry decreases as the conflicting flow increases. In general, the primary conflicting flow is the circulating flow that passes directly in front of the subject entry. When the conflicting flow approaches zero, the maximum entry flow is given by 3,600 seconds per hour divided by the follow-up headway, which is analogous to the saturation flow rate for a movement receiving a green indication at a signalized intersection. This defines the intercept of the
capacity model.

A variety of real-world conditions occur that can affect the accuracy of a given modelling technique. The analyst is cautioned to consider these effects and determine whether they are significant for the type of analysis being performed. For example, the level of accuracy needed for a rough planning-level sizing of a roundabout is considerably less than that needed to determine the likelihood of queue spillback between intersections. Some of these conditions include the following:

i. Effect of exiting vehicles. While the circulating flow directly conflicts with the entry flow, the exiting flow may also affect a driver's decision on when to enter the roundabout. This phenomenon is similar to the effect of the right-turning stream approaching from the left side of a two-way stop-controlled intersection. Until these drivers complete their exit manoeuvre or right turn, there may be some uncertainty in the mind of the driver at the yield or stop line about the intentions of the exiting or turning vehicle.

ii. Changes in effective priority. When both the entering and conflicting flow volumes are high, limited priority (where circulating traffic adjusts its headways to allow entering vehicles to enter), priority reversal (where entering traffic forces circulating traffic to yield), and other behaviours may occur, and a simplified gap-acceptance model may not give reliable results.

iii. Capacity constraint. When an approach operates over capacity during the analysis period, a condition known as capacity constraint may occur. During this condition, the actual circulating flow downstream of the constrained entry will be less than the demand. The reduction in actual circulating flow may therefore increase the capacity of the affected downstream entries.

iv. Origin–destination patterns. Origin–destination patterns may have an influence on the capacity of a given entry.

8.11.5.1.2 EFFECT OF GEOMETRY

Geometry plays a significant role in the operational performance of a roundabout in a number of key ways:

i. It affects the speed of vehicles through the intersection, thus influencing their travel time by virtue of geometry alone (geometric delay).

ii. It dictates the number of lanes over which entering and circulating vehicles travel. The widths of the approach roadway and entry determine the number of vehicle streams that may form side-by-side at the yield line and govern the rate at which vehicles may enter the circulating roadway.

iii. It can affect the degree to which flow in a given lane is facilitated or constrained. For example, the angle at which a vehicle enters affects the speed of that vehicle, with entries that are more perpendicular requiring slower speeds and thus longer headways. Likewise, the geometry of multilane entries may influence the degree to which drivers are comfortable entering next to one another.

iv. It may affect the driver's perception of how to navigate the roundabout and their corresponding lane choice approaching the entry. Improper lane alignment can increase friction between adjacent lanes and thus reduce capacity. Imbalanced lane flows on an entry can increase the delay and queuing on an entry despite the entry operating below its theoretical capacity.

Thus, the geometric elements of a roundabout, together with the volume of traffic desiring to use a roundabout at a given time, may determine the efficiency with which a roundabout operates. These elements form the core of commonly used models.

8.11.5.2 DATA COLLECTION AND ANALYSIS

8.11.5.2.1 FIELD DATA COLLECTION

Operational analysis of roundabouts requires the collection or projection of peak period turning-movement volumes. For existing conventional intersections, these can be determined using standard techniques. For existing roundabouts, turning movements can be collected using a variety of techniques:

i. Live recording of turning-movement patterns using field observers. This is only feasible under low-volume conditions where the entire roundabout is visible from one location.

ii. Video recording of the entire intersection, followed by manual extraction of turning movements from the video. This technique is feasible under any volume condition but usually requires all of the turning movements to be visible from one location. Multiple video locations can be used, but they must be carefully synchronized for successful data extraction.

iii. Field observers at each of the exits, manually recording vehicles approaching the exit.

iv. Link counters placed across each entry, each exit, and the circulatory roadway in front of each splitter island, plus manual counting of right-turn movements.

v. Origin–destination survey techniques. This is generally more effective when multiple intersections are being studied simultaneously.

Operational performance of a roundabout can also be measured directly in the field using a variety of techniques:

i. Control delay can be estimated by measuring the average time it takes vehicles to travel between a control point upstream of the maximum queue in a lane and a point immediately downstream of the entry. The control delay is the difference between this measured travel time and the travel time needed by an unconstrained vehicle (one that did not queue or need to yield at entry).

ii. Geometric delay can be estimated by comparing the travel time of an unconstrained vehicle passing through a roundabout to that needed by an unconstrained vehicle that does not pass through the geometric features of the roundabout (either measured before construction or estimated). Geometric delay is of particular importance when comparing travel times along a corridor.

Note that field measurement of performance measures may require large sample sizes due to the inherent large variability in delay measures.

8.11.5.2.2 DETERMINING ROUNDABOUT FLOW RATES

The manual technique presented in this guide requires the calculation of entering, circulating, and exiting flow rates for each roundabout leg. Although the following sections present a numerical methodology for a four-leg roundabout, this methodology can be extended to any number of legs.

The circulating flow rate opposing a given entry is defined as the flow conflicting with the entry flow of that leg. The movements that contribute to the northbound circulating flow rate are illustrated in **Figure 8.63**. In this exhibit, $V_{c,NB}$ is the circulating flow rate in front of the northbound entry, and the contributing movements are the eastbound through (EBT), eastbound left-turn (EBL), eastbound U-turn (EBU), southbound left-turn (SBL), southbound U-turn (SBU), and westbound U-turn (WBU) movements.

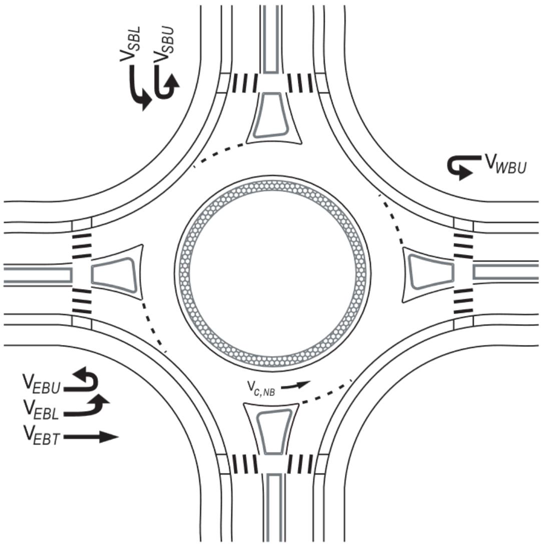


Figure 8.63 Calculation of Circulating Flow

The exiting flow rate for a given leg is used primarily in the calculation of conflicting flow for right-turn bypass lanes and in determining queuing at exit side crosswalks. The exiting flow calculation for the southbound exit is illustrated in **Figure 8.64**. If a bypass lane is present on the immediate upstream entry, the right-turning flow using the bypass lane is deducted from the exiting flow. In this exhibit, $V_{ex,SB}$ is the southbound exiting flow rate, and the contributing movements are the eastbound right-turn (EBR), southbound through (SBT), westbound left (WBL), and northbound U-turn (NBU) movements.

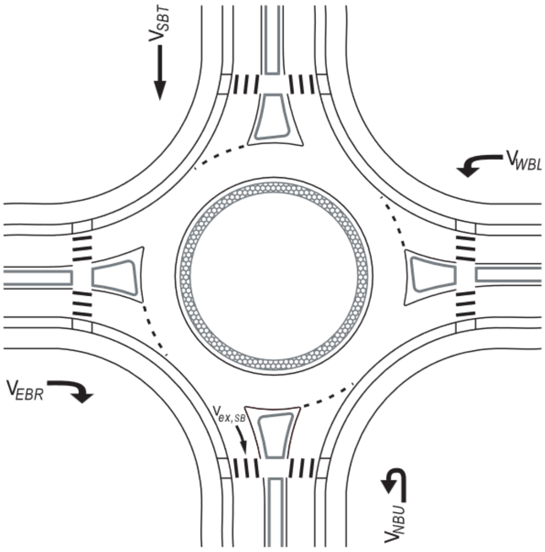
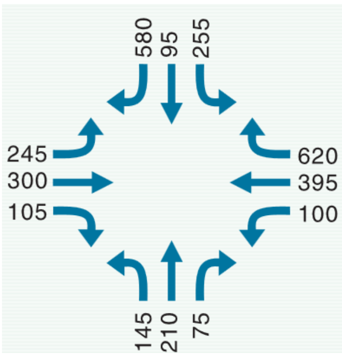


Figure 8.64 Calculation of Exiting Flow

Sample Calculation On Conversion Of Turning Movement Volumes To Roundabout Volumes

Turning-Movement Data

- Percent heavy vehicles for all movements = 2%
- Peak Hour Factor (PHF) = 0.97



Step 1: Convert Movement Demand Volumes to Flow Rates

Each turning-movement volume given in the problem is converted to a demand flow rate by dividing by the peak-hour factor. As an example, the northbound left volume is converted to a flow rate in passenger cars per hour as follows:

$$v_{NBL} = \frac{V_{NBL}}{PHF} = \frac{145}{0.97} = 149pcu/h$$

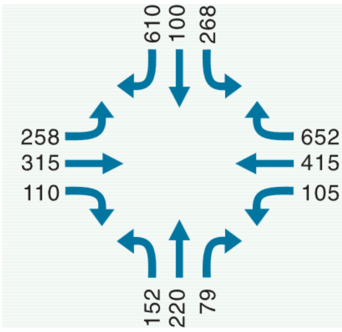
Step 2: Adjust Flow Rates for Heavy Vehicles

The flow rate for each movement may be adjusted to account for vehicle stream characteristics as follows (northbound left turn illustrated):

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.02(2 - 1)} = 0.980$$

$$v_{NBL,pce} = \frac{V_{NBL}}{f_{HV}} = \frac{149}{0.980} = 152pcu/h$$

The resulting adjusted flow rates for all movements accounting for Steps 1 and 2 are therefore computed as follows:



Step 3: Determine Entry Flow Rates by Lane

The entry flow rate is calculated by summing up the movement flow rates that enter the roundabout. For single-lane roundabouts, all approach volumes are summed together. Additional lane-use calculations are required for multilane roundabouts. The entry flow rates are calculated as follows for the south leg (northbound entry):

$$v_{e,NB,pce} = v_{NB,pce} + v_{NBL,pce} + v_{NBT,pce} + v_{NBR,pce}$$

$$v_{e,NB,pce} = 0 + 152 + 220 + 79 = 451pcu/h$$

Step 4: Determine Circulating Flow Rates

The circulating flow is calculated for each leg. The circulating volumes are the sum of all volumes that will conflict with entering vehicles on the subject approach. For the south leg (northbound entry), the circulating flow is calculated as follows:

$$v_{c,NB,pce} = v_{WB,pce} + v_{SBL,pce} + v_{SBU,pce} + v_{EBT,pce} + v_{EBL,pce} + v_{EBU,pce}$$

$$v_{c,NB,pce} = 0 + 268 + 0 + 315 + 258 + 0 = 841pcu/h$$

Step 5: Determine Exiting Flow Rates

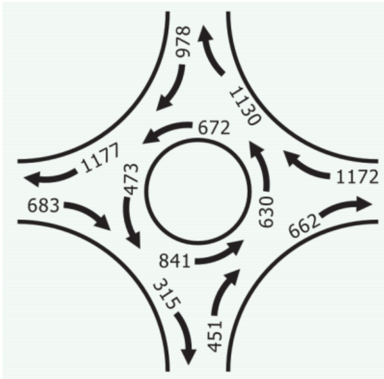
The exiting flow is calculated for each leg by summing all flow that will be exiting the roundabout on a particular leg. For the south leg (northbound entry), the exiting volume is calculated as follows:

$$v_{ex,pce,NB} = v_{NBU,pce} + v_{WBL,pce} + v_{SBT,pce} + v_{EBR,e,pce}$$

$$v_{ex,pce,NB} = 0 + 105 + 100 + 110 = 315pcu/h$$

Result

The following figure illustrates the final volumes converted into roundabout entering, exiting, and circulating flow rates.



8.11.5.2.3 ANALYSIS TECHNIQUES

A variety of methodologies are available to analyse the performance of roundabouts. All are approximations, and the responsibility is with the analyst to use the appropriate tool for conducting the analysis. The decision on the type of operational analysis method to employ should be based on a number of factors:

i. What data are available?
ii. Can the method satisfy the output requirements?

Table 8.34 presents a summary, rather than an exhaustive list, of common applications of operational analysis tools, along with the outcome typically desired and the types of input data usually available. Note that the outcome desired is distinct from the output of the analysis tool. For example, the lane configuration is commonly determined through an iterative process of assigning lane configurations as inputs to the analysis tool and then assessing the acceptability of the resultant performance measures.

Table 8.34 Selection of Analysis Tool

Application	Typical Outcome Desired	Input Data Available	Potential Analysis Tool
Planning-level sizing	Number of lanes	Traffic volumes	HCM, deterministic software
Preliminary design of roundabouts with up to two lanes	Detailed lane configuration	Traffic volumes, geometry	HCM, deterministic software
Preliminary design of roundabouts with three lanes and/or with short lanes/flared designs	Detailed lane configuration	Traffic volumes, geometry	Deterministic software
Analysis of pedestrian treatments	Vehicular delay, vehicular queuing, pedestrian delay	Vehicular traffic and pedestrian volumes, crosswalk design	HCM, deterministic software, simulation
System analysis	Travel time, delays and queues between intersections	Traffic volumes, geometry	HCM, simulation
Public involvement	Animation of no-build conditions and proposed alternatives	Traffic volumes, geometry	Simulation

8.11.6 GEOMETRIC DESIGN

Designing the geometry of a roundabout involves choosing between trade-offs of safety and capacity. Roundabouts operate most safely when their geometry forces traffic to enter and circulate at slow speeds. Horizontal curvature and narrow pavement widths are used to produce this reduced-speed environment. Conversely, the capacity of roundabouts is negatively affected by these low-speed design elements. As the widths and radii of entry and circulatory roadways are reduced, so also the capacity of the roundabout is reduced. Furthermore, many of the geometric parameters are governed by the manoeuvring requirements of the largest vehicles expected to travel through the intersection. Thus, designing a roundabout is a process of determining the optimal balance between safety provisions, operational performance, and large vehicle accommodation. While the basic form and features of roundabouts are uniform regardless of their location, many of the design techniques and parameters are different, depending on the speed environment and desired capacity at individual sites. In rural environments where approach speeds are high and bicycle and pedestrian use may be minimal, the design objectives are significantly different from roundabouts in urban environments where bicycle and pedestrian safety are a primary concern. Additionally, many of the design techniques are substantially different for single-lane roundabouts than for roundabouts with multiple entry lanes.

The goal of any roundabout design, regardless of category or location, should be to achieve these principles:

i. Provide slow entry speeds and consistent speeds through the roundabout by using deflection.
ii. Provide the appropriate number of lanes and lane assignment to achieve adequate capacity, lane volume, and lane continuity.
iii. Provide smooth channelization that is intuitive to drivers and results in vehicles naturally using the intended lanes.
iv. Provide adequate accommodation for the design vehicles.
v. Design to meet the needs of pedestrians and cyclists; and
vi. Provide appropriate sight distance and visibility.

8.11.6.1 DESIGN PROCESS

The process of designing roundabouts, more so than other forms of intersections, requires a considerable amount of iteration among geometric layout, operational analysis, and safety evaluation. Minor adjustments in geometry can result in significant changes in the safety and/or operational performance. Thus, the designer often needs to revise and refine the initial layout attempt to enhance its capacity and safety. It is rare to produce an optimal geometric design on the first attempt. **Figure 8.65** provides a graphical flowchart for the process of designing and evaluating a roundabout.

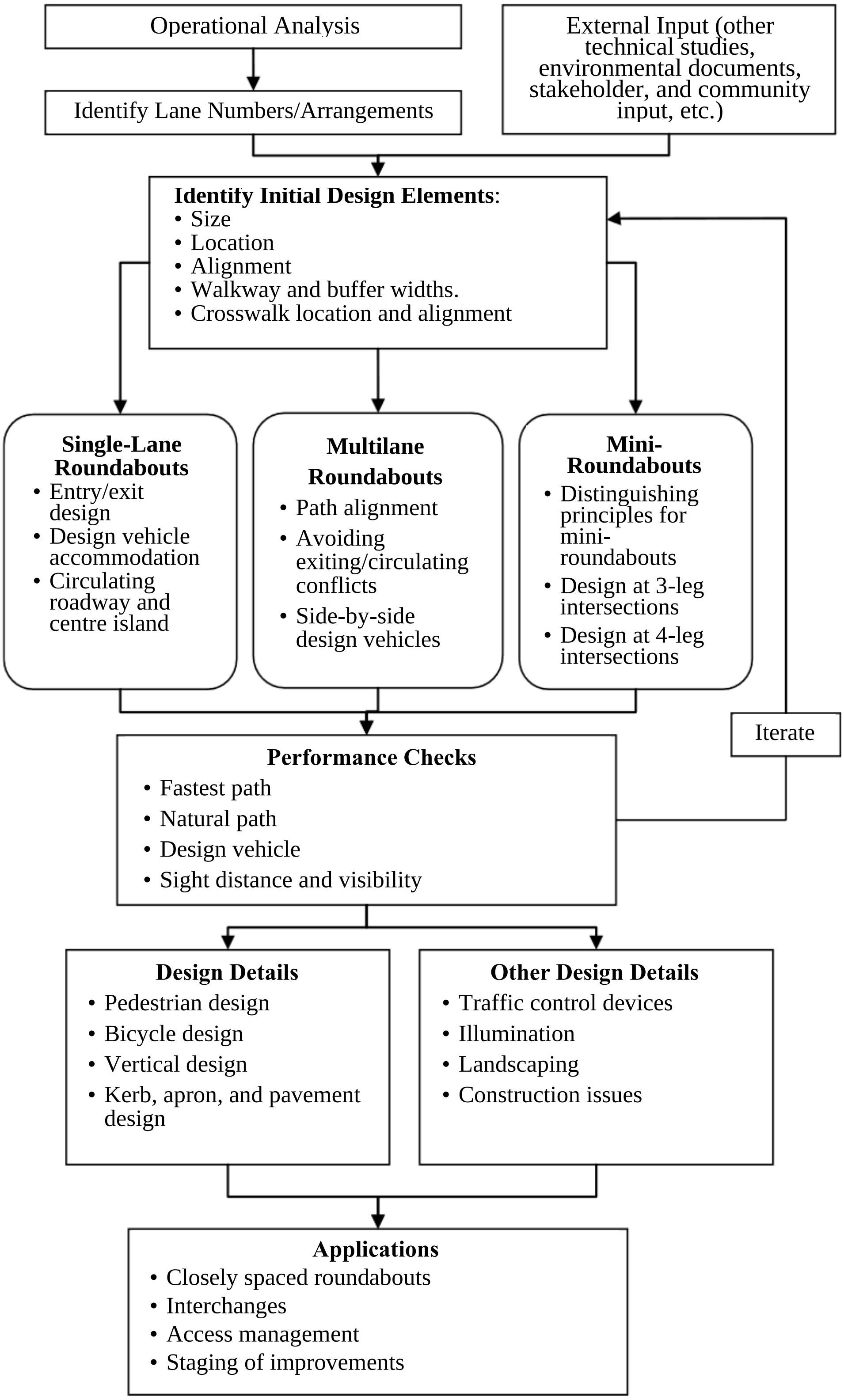


Figure 8.65 Roundabout design process

Because roundabout design is such an iterative process, in which small changes in geometry can result in substantial changes to operational and safety performance, it may be advisable to prepare the initial layout drawings at a sketch level of detail. Although it is easy to get caught into the desire to design each of the individual components of the geometry such that it complies with the specifications provided in this section, it is much more important that the individual components are compatible with each other so that the roundabout will meet its overall performance objectives. Before the details of the geometry are defined, three fundamental elements must be determined in the preliminary design stage:

i. The optimal roundabout size.
ii. The optimal position.
iii. The optimal alignment and arrangement of approach legs.

8.11.6.2 GENERAL DESIGN PRINCIPLES

This section describes the fundamental design principles common among all categories of roundabouts. Guidelines for the design of each geometric element are provided in the following section. Further guidelines specific to double-lane roundabouts, rural roundabouts, and mini-roundabouts are given in subsequent sections. Note that double-lane roundabout design is significantly different from single-lane roundabout design, and many of the techniques used in single-lane roundabout design do not directly transfer to double-lane design.

8.11.6.2.1 DESIGN VEHICLE

One important factor determining a roundabout's layout is the need to accommodate the largest motorized vehicle likely to use the intersection. The turning path requirements of this vehicle, termed hereafter the design vehicle, will dictate many of the roundabout's dimensions. Before beginning the design process, the designer must be conscious of the design vehicle and possess the appropriate vehicle turning templates or a CAD-based vehicle turning path program to determine the vehicle's swept path.

The choice of design vehicle will vary depending upon the approaching roadway types and the surrounding land use characteristics.

In general, larger roundabouts need to be used to accommodate large vehicles while maintaining low speeds for passenger vehicles. However, in some cases, land constraints may limit the ability to accommodate large trailer combinations while achieving adequate deflection for small vehicles. At such times, a truck apron may be used to provide additional traversable area around the central island for large semi-trailers. Truck aprons, though, provide a lower level of operation than standard non-mountable islands and should be used only when there is no other means of providing adequate deflection while accommodating the design vehicle. **Figure 8.66** and **Figure 8.67** demonstrate the use of a CAD-based computer program to determine the vehicle's swept path through the critical turning movements.

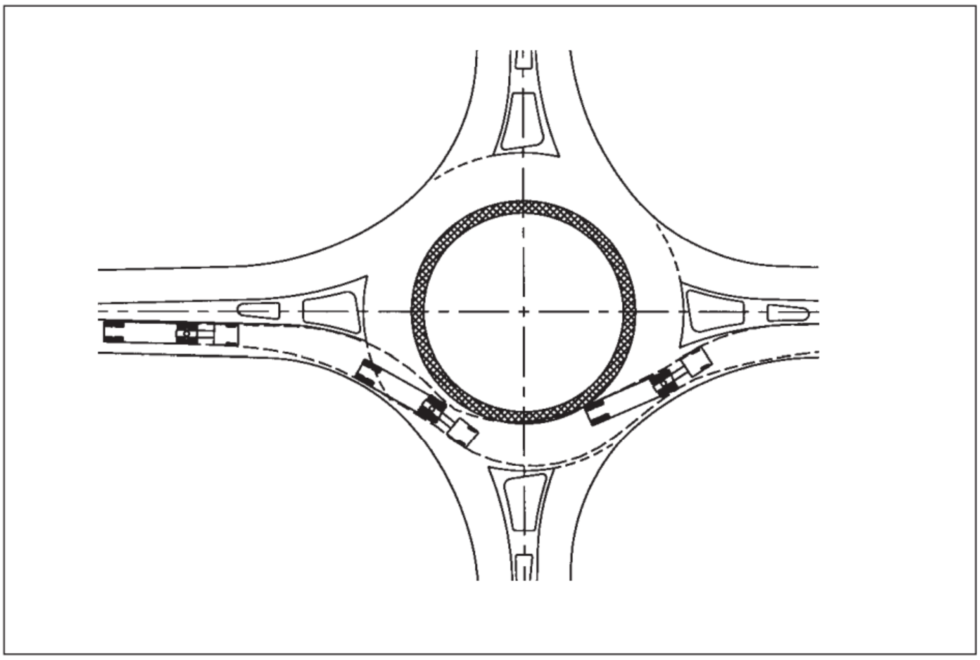


Figure 8.66 Through movement swept path of a trailer

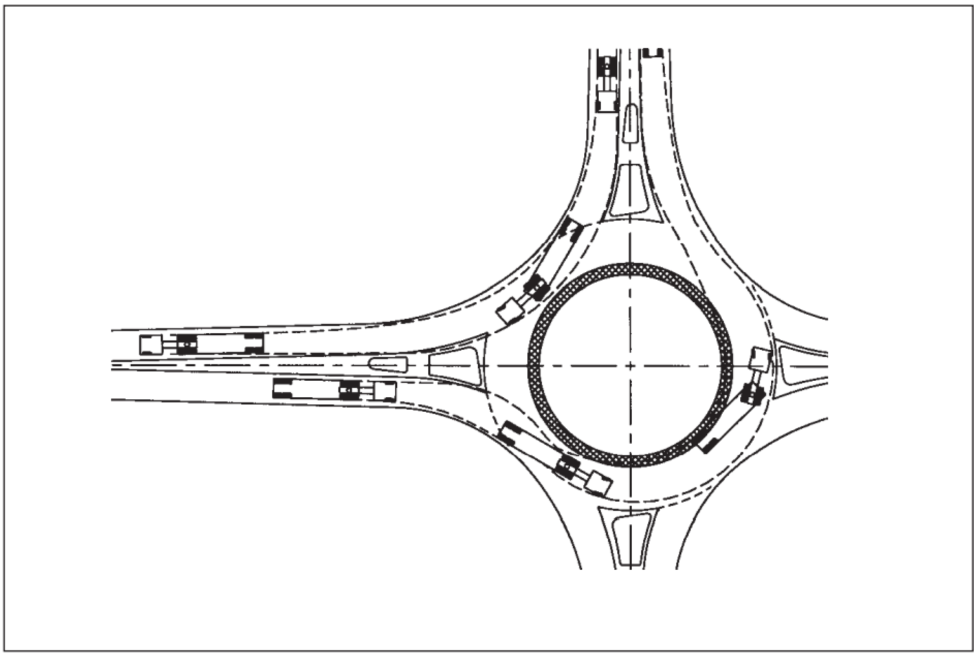


Figure 8.67 Left-turn and right-turn swept paths of a trailer

8.11.6.2.2 NON-MOTORISED DESIGN USERS

Like the motorized design vehicle, the design criteria of nonmotorized potential roundabout users (bicyclists, pedestrians, skaters, wheelchair users, strollers, etc.) should be considered when developing many of the geometric elements of a roundabout design. These users span a wide range of ages and abilities that can have a significant effect on the design of a facility. The basic design dimensions for various design users are given in **Table 8.35**.

Table 8.35 Key dimensions of nonmotorized design users

User	Dimension (m)		Affected Roundabout Features
Bicycles	Length	1.8	Splitter island width at crosswalk
	Minimum operating width	1.5	Bike lane width
	Lateral clearance on each side	0.6	Shared bicycle-pedestrian path width
		1.0 to obstructions	
Pedestrian (walking)	Width	0.5	Width of walkway and crosswalk
Wheelchair	Minimum width	0.75	Width of walkway and crosswalk
	Operating width	0.90	Width of walkway and crosswalk
Person pushing stroller	Length	1.70	Width of walkway and crosswalk
Skaters	Typical operating width	1.8	Width of walkway

8.11.6.2.3 SPEED THROUGH THE ROUNDABOUT

Because it has profound impacts on safety, achieving appropriate vehicular speeds through the roundabout is the most critical design objective. A well-designed roundabout reduces the relative speeds between conflicting traffic streams by requiring vehicles to negotiate the roundabout along a curved path.

A. SPEED PROFILES

Figure 8.68 shows the operating speeds of typical vehicles approaching and negotiating a roundabout. Approach speeds of 40, 55, and 70 km/h about 100 m from the centre of the roundabout are shown. Deceleration begins before this time, with circulating drivers operating at approximately the same speed on the roundabout. The relatively uniform negotiation speed of all drivers on the roundabout means that drivers are able to more easily choose their desired paths in a safe and efficient manner.

B. DESIGN SPEED

International studies have shown that increasing the vehicle path curvature decreases the relative speed between entering and circulating vehicles and thus usually results in decreases in the entering-circulating and exiting-circulating vehicle crash rates. However, at multilane roundabouts, increasing vehicle path curvature creates greater side friction between adjacent traffic streams and can result in more vehicles cutting across lanes and higher potential for sideswipe crashes. Thus, for each roundabout, there exists an optimum design speed to minimize crashes.

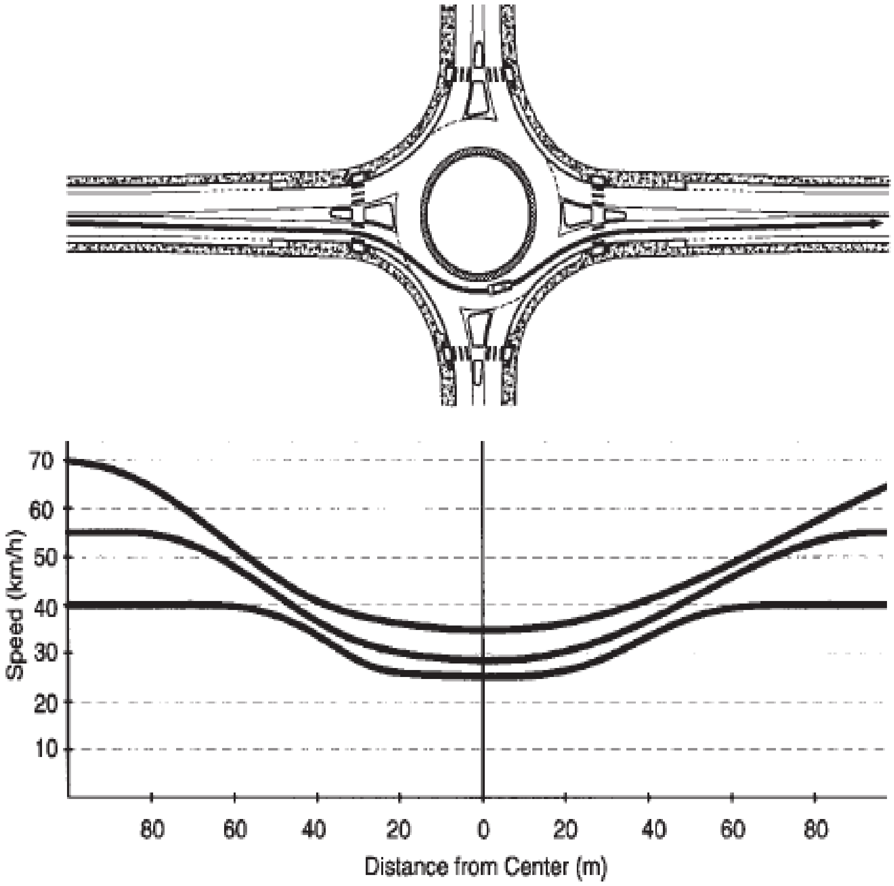


Figure 8.68 Sample Theoretical Speed Profile (Urban Compact Roundabout)

Recommended maximum entry design speeds for roundabouts at various intersection site categories are provided in **Table 8.36**.

Table 8.36 Recommended Maximum Entry Design Speed

Site Category	Recommended Maximum Entry Design Speed (km/h)
Mini-Roundabout	25
Urban Compact	25
Urban Single Lane	35
Urban Double Lane	40
Rural Single Lane	40
Rural Double Lane	50

C. VEHICLE PATHS

To determine the speed of a roundabout, the fastest path allowed by the geometry is drawn. This is the smoothest, flattest path possible for a single vehicle, in the absence of other traffic and ignoring all lane markings, traversing through the entry, around the central island, and out the exit. Usually, the fastest possible path is the through movement, but in some cases, it may be a right turn movement. A vehicle is assumed to be 2m wide and to maintain a minimum clearance of 0.5m from a roadway centreline or concrete kerb and flush with a painted edge line (2). Thus, the centreline of the vehicle path is drawn with the following distances to the particular geometric features:

i. 1.5 m from a concrete kerb,

ii. 1.5 m from a roadway centreline, and

iii. 1.0 m from a painted edge line.

Figure 8.69 and **Figure 8.70** illustrate the construction of the fastest vehicle paths at a single-lane roundabout and at a double-lane roundabout, respectively.

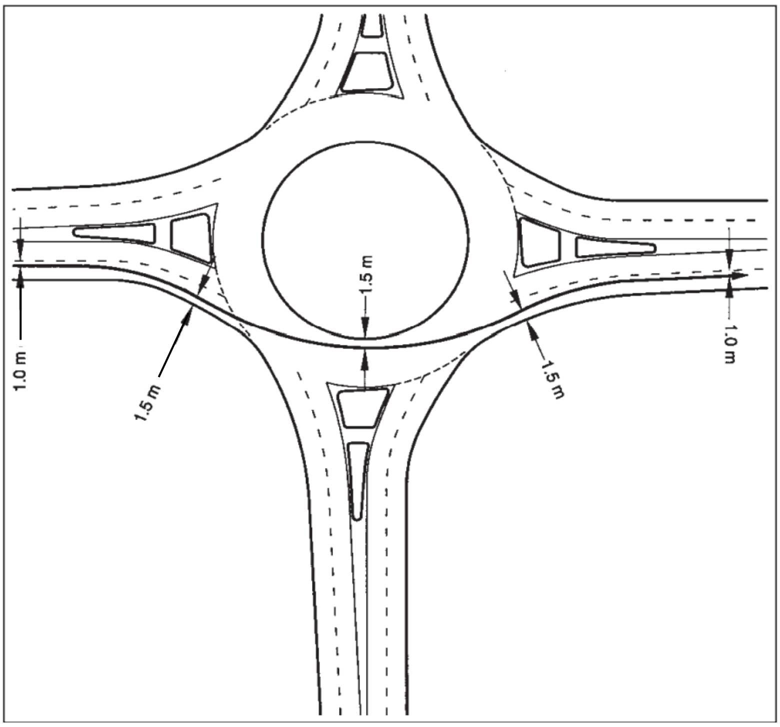


Figure 8.69 Fastest vehicle path through double-lane roundabout

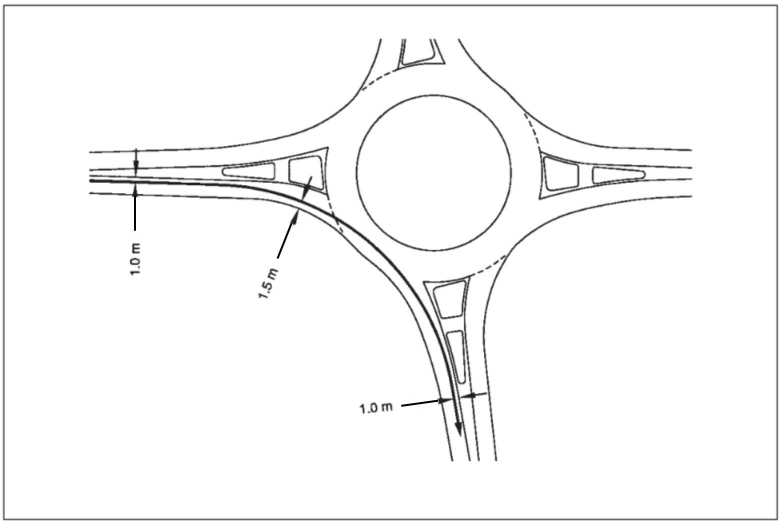


Figure 8.70 Example of critical right-turn movement

As shown in **Figure 8.69** and **Figure 8.70**, the fastest path for the through movement is a series of reverse curves (i.e., a curve to the right, followed by a curve to the left, followed by a curve to the right). When drawing the path, a short length of tangent should be drawn between consecutive curves to account for the time it takes for a driver to turn the steering wheel. It may be initially better to draw the path freehand, rather than using drafting templates or a computer-aided design (CAD) program. The freehand technique may provide a more natural representation of the way a driver negotiates the roundabout, with smooth transitions connecting curves and tangents. Having sketched the fastest path, the designer can then measure the minimum radii using suitable curve templates or by replicating the path in CAD and using it to determine the radii.

The design speed of the roundabout is determined from the smallest radius along the fastest allowable path. The smallest radius usually occurs on the circulatory roadway as the vehicle curves to the left around the central island. However, it is important when designing the roundabout geometry that the radius of the entry path (i.e., as the vehicle curves to the right through entry geometry) not be significantly larger than the circulatory path radius. The fastest path should be drawn for all approaches of the roundabout. Because the construction of the fastest path is a subjective process requiring a certain amount of personal judgment, it may be advisable to obtain a second opinion.

D. SPEED-CURVE RELATIONSHIP

The relationship between travel speed and horizontal curvature is documented in **Section 7.2.3** of this guide. **Equation 7.1** can be used to calculate the design speed for a given travel path radius. Superelevation values are usually assumed to be +0.025 for entry and exit curves and -0.025 for curves around the central island.

E. SPEED CONSISTENCY

In addition to achieving an appropriate design speed for the fastest movements, another important objective is to achieve consistent speeds for all movements. Along with overall reductions in speed, speed consistency can help to minimize the crash rate and severity between conflicting streams of vehicles. It also simplifies the task of merging into the conflicting traffic stream, minimizing critical gaps, thus optimizing entry capacity. This principle has two implications:

i. The relative speeds between consecutive geometric elements should be minimized.

ii. The relative speeds between conflicting traffic streams should be minimized.

As shown in **Figure 8.71,** five critical path radii must be checked for each approach. R_1, the entry path radius, is the minimum radius on the fastest through path prior to the yield line. R_2, the circulating path radius, is the minimum radius on the fastest through path around the central island. R_3, the exit path radius, is the minimum radius on the fastest through path into the exit. R_4, the left-turn path radius, is the minimum radius on the path of the conflicting left-turn movement. R_5, the right-turn path radius, is the minimum radius on the fastest path of a right-turning vehicle. It is important to note that these vehicular path radii are not the same as the kerb radii. First, the basic kerb geometry is laid out, and then the vehicle paths are drawn in accordance with the procedures described in **C** (vehicle paths) above.

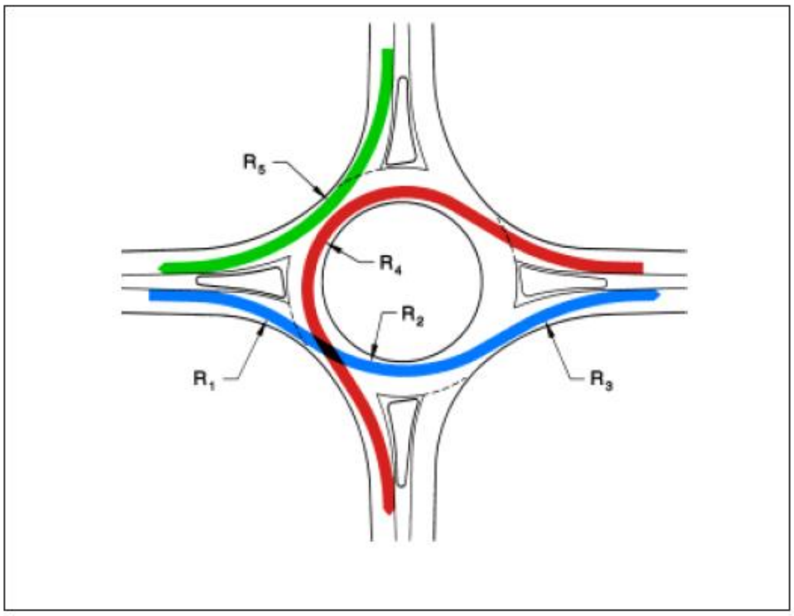


Figure 8.71 Vehicle path radii

On the fastest path, it is desirable for R_1 to be smaller than R_2, which in turn should be smaller than R_3. This ensures that speeds will be reduced to their lowest level at the roundabout entry and will thereby reduce the likelihood of loss-of-control crashes. It also helps to reduce the speed differential between entering and circulating traffic, thereby reducing the entering-circulating vehicle crash rate. However, in some cases it may not be possible to achieve an R_1 value less than R_2 within given right-off-way or topographic constraints. In such cases, it is acceptable for R_1 to be greater than R_2, provided the relative difference in speeds is less than

20 km/h and preferably less than 10 km/h.

At single-lane roundabouts, it is relatively simple to reduce the value of R_1. The kerb radius at the entry can be reduced or the alignment of the approach can be shifted further to the left to achieve a slower entry speed (with the potential for higher exit speeds that may put pedestrians at risk). However, at double-lane roundabouts, it is generally more difficult as overly small entry curves can cause the natural path of adjacent traffic streams to overlap. Path overlap happens when the geometry leads a vehicle in the left approach lane to naturally sweep across the right approach lane just before the approach line to avoid the central island. It may also happen within the circulatory roadway when a vehicle entering from the righthand lane naturally cuts across the left side of the circulatory roadway close to the central island. When path overlap occurs at double-lane roundabouts, it may reduce capacity and increase crash risk. Therefore, care must be taken when designing double-lane roundabouts to achieve ideal values for R_1, R_2, and R_3.

The exit radius, R_3, should not be less than R_1 or R_2 in order to minimize loss-of control crashes. At single-lane roundabouts with pedestrian activity, exit radii may still be small (the same or slightly larger than R_2) in order to minimize exit speeds. However, at double-lane roundabouts, additional care must be taken to minimize the likelihood of exiting path overlap. Exit path overlap can occur at the exit when a vehicle on the left side of the circulatory roadway (next to the central island) exits into the right-hand exit lane. Where no pedestrians are expected, the exit radii should be just large enough to minimize the likelihood of exiting path overlap. Where pedestrians are present, tighter exit curvature may be necessary to ensure sufficiently low speeds at the downstream pedestrian crossing.

The radius of the conflicting left-turn movement, R_4, must be evaluated in order to ensure that the maximum speed differential between entering and circulating traffic is not more than 20km/h. The left-turn movement is the critical traffic stream because it has the lowest circulating speed. Large differentials between entry and circulating speeds may result in an increase in single-vehicle crashes due to loss of control. Generally, R_4 can be determined by adding 1.5m to the central island radius. Based on this assumption, **Table 8.37** show approximate R_4 values and corresponding maximum R_1 values for various inscribed circle diameters.

Table 8.37 Approximated R_4 values and corresponding R_1 values

Inscribed Circle Diameter (m)		Approximate R_4 Value		Maximum R_1 Value	
		Radius (m)	Speed (km/h)	Radius (m)	Speed (km/h)
Single-Lane Roundabout	30	11	21	54	41
	35	13	23	61	43
	40	16	25	69	45
	45	19	26	73	46
Double-Lane Roundabout	45	15	24	65	44
	50	17	25	69	45
	55	20	27	78	47
	60	23	28	83	48
	65	25	29	88	49
	70	28	30	93	50

Finally, the radius of the fastest possible right-turn path, R_5, is evaluated. Like R_1, the right-turn radius should have a design speed at or below the maximum design speed of the roundabout and no more than 20 km/h above the conflicting R_4 design speed.

8.11.6.2.4 ALIGNMENT APPROACHES AND ENTRIES

In general, the roundabout is optimally located when the centrelines of all approach legs pass through the centre of the inscribed circle. This location usually allows the geometry to be adequately designed so that vehicles will maintain slow speeds through both the entries and the exits. The radial alignment also makes the central island more conspicuous to approaching drivers.

If it is not possible to align the legs through the centre point, a slight offset to the left (i.e., the centreline passes to the left of the roundabout's centre point) is acceptable. This alignment will still allow sufficient curvature to be achieved at the entry, which is of supreme importance. In some cases (particularly when the inscribed circle is relatively small), it may be beneficial to introduce a slight offset of the approaches to the left in order to enhance the entry curvature.

However, care must be taken to ensure that such an approach offset does not produce an excessively tangential exit. Especially in urban environments, it is important that the exit geometry produce a sufficiently curved exit path in order to keep vehicle speeds low and reduce the risk for pedestrians. It is almost never acceptable for an approach alignment to be offset to the right of the roundabout's centre point. This alignment brings the approach in at a more tangential angle and reduces the opportunity to provide sufficient entry curvature. Vehicles will be able to enter the roundabout too fast, resulting in more loss-of control crashes and higher crash rates between entering and circulating vehicles. **Figure 8.72** illustrates the preferred radial alignment of entries.

In addition, it is desirable to equally space the angles between entries. This provides optimal separation between successive entries and exits. This results in optimal angles of 90 degrees for four-leg roundabouts, 72 degrees for five-leg roundabouts, and so on.

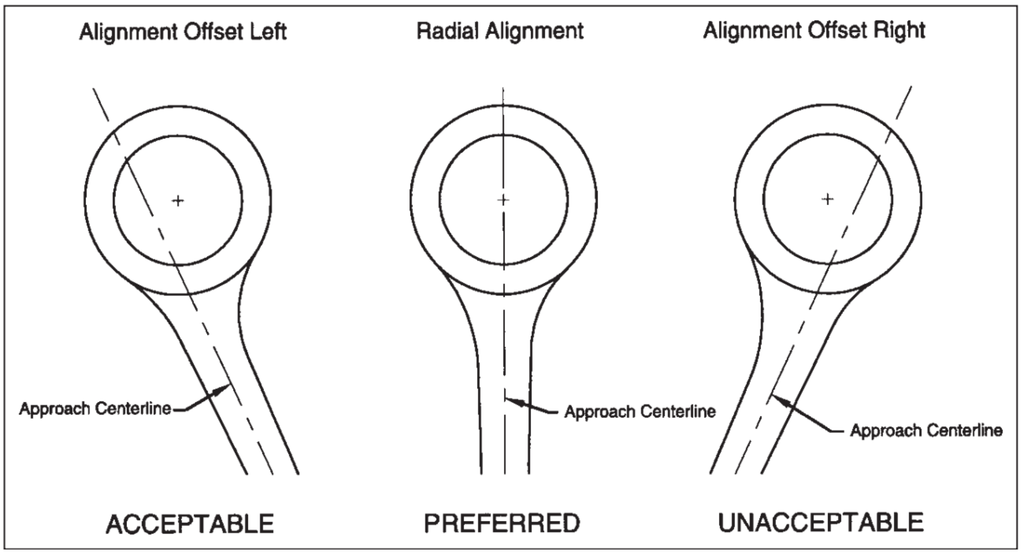


Figure 8.72 Radial alignment of entries

8.11.6.2.5 GEOMETRIC ELEMENTS

This section presents specific parameters and guidelines for the design of each geometric element of a roundabout. The designer must keep in mind, however, that these components are not independent of each other. The interaction between the components of the geometry is far more important than the individual pieces. Care must be taken to ensure that the geometric elements are all compatible with each other so that the overall safety and capacity objectives are met.

A. INSCRIBED CIRCLE DIAMETER

The inscribed circle diameter is the distance across the circle inscribed by the outer kerb (or edge) of the circulatory roadway. As illustrated in **Figure 8.52**, it is the sum of the central island diameter (which includes the apron, if present) and twice the circulatory roadway. The inscribed circle diameter is determined by a number of design objectives. The designer often has to experiment with varying diameters before determining the optimal size at a given location.

At single-lane roundabouts, the size of the inscribed circle is largely dependent upon the turning requirements of the design vehicle. The diameter must be large enough to accommodate the design vehicle while maintaining adequate deflection curvature to ensure safe travel speeds for smaller vehicles. However, the circulatory roadway width, entry and exit widths, entry and exit radii, and entry and exit angles also play a significant role in accommodating the design vehicle and providing deflection. Careful selection of these geometric elements may allow a smaller inscribed circle diameter to be used in constrained locations. In general, the inscribed circle diameter should be a minimum of 30 m to accommodate a T-17 design vehicle. Smaller roundabouts can be used for some local road or collector road intersections, where the design vehicle may be an L-9.

At double-lane roundabouts, accommodating the design vehicle is usually not a constraint. The size of the roundabout is usually determined either by the need to achieve deflection or by the need to fit the entries and exits around the circumference with reasonable entry and exit radii between them. Generally, the inscribed circle diameter of a double-lane roundabout should be a minimum of 45 m.

In general, smaller inscribed diameters are better for overall safety because they help to maintain lower speeds. In high-speed environments, however, the design of the approach geometry is more critical than in low-speed environments. Larger inscribed diameters generally allow for the provision of better approach geometry, which leads to a decrease in vehicle approach speeds. Larger inscribed diameters also reduce the angle formed between entering and circulating vehicle paths, thereby reducing the relative speed between these vehicles and leading to reduced entering-circulating crash rates. Therefore, roundabouts in high-speed environments may require diameters that are somewhat larger than those recommended for low-speed environments. Very large diameters (greater than 60 m), however, should generally not be used because they will have high circulating speeds and more crashes with greater severity. **Table 8.38** provides recommended ranges of inscribed circle diameters for various site locations.

Table 8.38 Recommended inscribed circle diameter ranges.

Site Category	Typical Design Vehicle	Inscribed Circle Diameter Range (m)*
Mini-Roundabout	L-9	13–25
Urban Compact	L-9	25–30
Urban Single Lane	T-17	30–40
Urban Double Lane	T-17	45–55
Rural Single Lane	T-21	35–40
Rural Double Lane	T-21	55–60

* Assumes 90-degree angles between entries and no more than four legs.

B. ENTRY WIDTH

Entry width is the largest determinant of a roundabout's capacity. The capacity of an approach is not dependent merely on the number of entering lanes, but on the total width of the entry. In other words, the entry capacity increases steadily with incremental increases to the entry width. Therefore, the basic sizes of entries and circulatory roadways are generally described in terms of width, not number of lanes. Entries that are of sufficient width to accommodate multiple traffic streams (at least 6.0 m) are striped to designate separate lanes. However, the circulatory roadway is usually not striped, even when more than one lane of traffic is expected to circulate (for more details related to roadway markings, see **Section 12.3**).

As shown in **Figure 8.52**, entry width is measured from the point where the yield line intersects the left edge of the travelled-way to the right edge of the travelled-way, along a line perpendicular to the right kerb line. The width of each entry is dictated by the needs of the entering traffic stream. It is based on design traffic volumes and can be determined in terms of the number of entry lanes. The circulatory roadway must be at least as wide as the widest entry and must maintain a constant width throughout.

To maximize the roundabout's safety, entry widths should be kept to a minimum. The capacity requirements and performance objectives will dictate that each entry be a certain width, with a number of entry lanes. In addition, the turning requirements of the design vehicle may require that the entry be wider still. However, larger entry and circulatory widths increase crash frequency. Therefore, determining the entry width and circulatory roadway width involves a trade-off between capacity and safety. The design should provide the minimum width necessary for capacity and accommodation of the design vehicle in order to maintain the highest level of safety. Typical entry widths for single-lane entrances range from 4.3 to 4.9 m, however, values higher or lower than this range may be required for site-specific design vehicle and speed requirements for critical vehicle paths.

When the capacity requirements can only be met by increasing the entry width, this can be done in two ways:

i. By adding a full lane upstream of the roundabout and maintaining parallel lanes through the entry geometry; or
ii. By widening the approach gradually (flaring) through the entry geometry.

Figure 8.73 and **Figure 8.74** illustrate these two widening options.

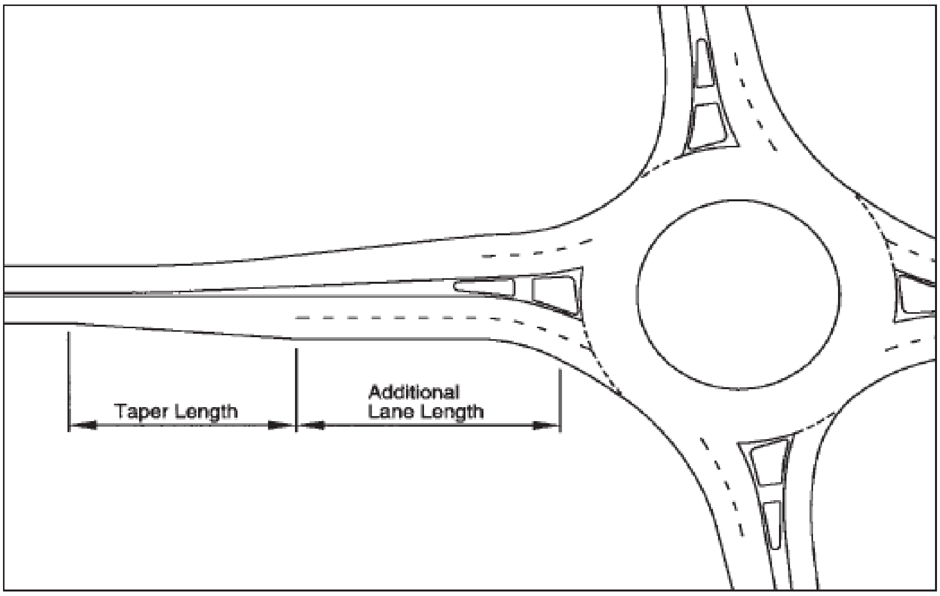


Figure 8.73 Approach widening by adding full lane

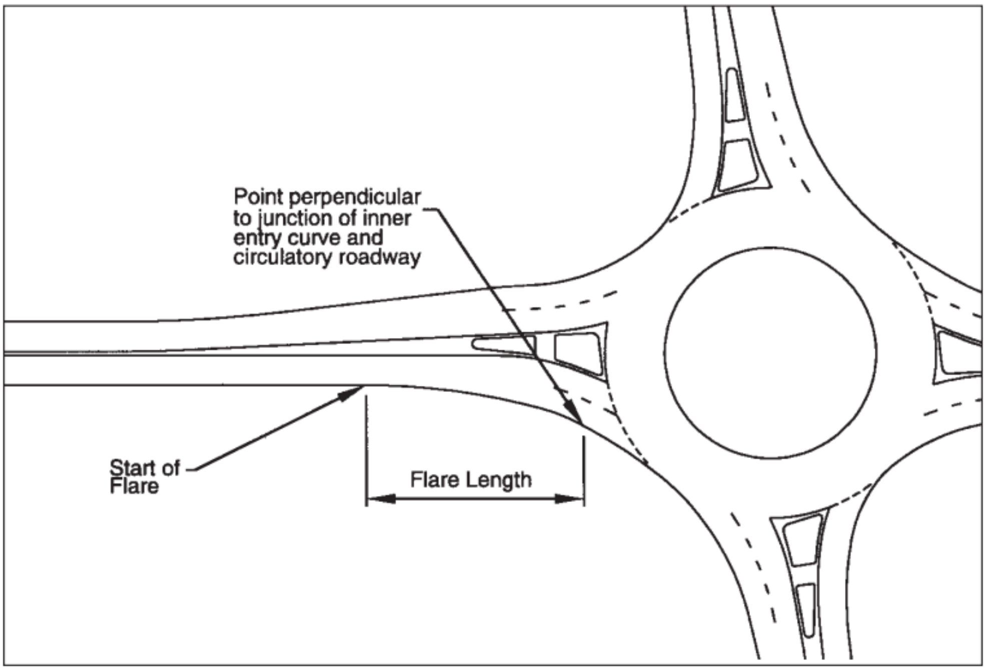


Figure 8.74 Approach widening by entry flaring

Flaring is an effective means of increasing capacity without requiring as much right-of-way as a full lane addition. While increasing the length of flare increases capacity, it does not increase crash frequency. Consequently, the crash frequency for two approaches with the same entry width will be essentially the same, whether they have parallel entry lanes or flared entry designs. Entry widths should therefore be minimized, and flare lengths maximized to achieve the desired capacity with minimal effect on crashes. Generally, flare lengths should be a minimum of 25m in urban areas and 40m in rural areas. However, if right-of-way is constrained, shorter lengths can be used with noticeable effects on capacity.

In some cases, a roundabout designed to accommodate design year traffic volumes, typically projected 20 years from the present, can result in substantially wider entries and circulatory roadway than needed in the earlier years of operation. Because safety will be significantly reduced by the increase in entry width, the designer may wish to consider a two-phase design solution. In this case, the first phase design would provide the entry width requirements for near-term traffic volumes with the ability to easily expand the entries and circulatory roadway to accommodate future traffic volumes. The interim solution should be accomplished by first laying out the ultimate plan, then designing the first phase within the ultimate kerb lines. The interim roundabout is often constructed with the ultimate inscribed circle diameter, but with a larger central island and splitter islands. At the time additional capacity is needed, the splitter and central islands can be reduced in size to provide additional widths at the entries, exits, and circulatory roadway.

C. CIRCULATORY ROADWAY WIDTH

The required width of the circulatory roadway is determined from the width of the entries and the turning requirements of the design vehicle. In general, it should always be at least as wide as the maximum entry width (up to 120 percent of the maximum entry width) and should remain constant throughout the roundabout .

i. Single lane roundabouts

At single-lane roundabouts, the circulatory roadway should just accommodate the design vehicle. Appropriate vehicle-turning templates or a CAD-based computer program should be used to determine the swept path of the design vehicle through each of the turning movements. Usually the left-turn movement is the critical path for determining circulatory roadway width. In accordance with best practice, a minimum clearance of 0.6 m should be provided between the outside edge of the vehicle's tire track and the kerb line. **Table 5.26** provides derived widths required for various radii for each standard design vehicle.

In some cases (particularly where the inscribed diameter is small, or the design vehicle is large) the turning requirements of the design vehicle may dictate that the circulatory roadway be so wide that the amount of deflection necessary to slow passenger vehicles is compromised. In such cases, the circulatory roadway width can be reduced and a truck apron, placed behind a mountable kerb on the central island, can be used to accommodate larger vehicles. However, truck aprons generally provide a lower level of operation than standard non-mountable islands. They are sometimes driven over by four-wheel drive vehicles, may surprise inattentive motorcyclists, and can cause load shifting on trucks. They should, therefore, be used only when there is no other means of providing adequate deflection while accommodating the design vehicle.

ii. Double-lane roundabouts

At double-lane roundabouts, the circulatory roadway width is usually not governed by the design vehicle. The width required for one, two, or three vehicles, depending on the number of lanes at the widest entry, to travel simultaneously through the roundabout should be used to establish the circulatory roadway width. The combination of vehicle types to be accommodated side-by-side is dependent upon the specific traffic conditions at each site. If the entering traffic is predominantly small vehicles and large vehicles, where trailer traffic is infrequent, it may be appropriate to design the width for two small vehicles or a small and a large vehicle side-by-side. If trailer traffic is relatively frequent (greater than 10 percent), it may be necessary to provide sufficient width for the simultaneous passage of a trailer in combination with a small or large vehicle.

Table 8.39 provides minimum recommended circulatory roadway widths for two lane roundabouts where trailer traffic is relatively infrequent.

Table 8.39 Minimum circulatory lane widths for two-lane roundabouts.

Inscribed Circle Diameter (m)	Minimum Circulatory Lane Width* (m)	Central Island Diameter (m)
45	9.8	25.4
50	9.3	31.4
55	9.1	36.8
60	9.1	41.8
65	8.7	47.6
70	8.7	52.6

*Assumes infrequent semi-trailer use (typically less than 5 percent of the total traffic).

D. CENTRAL ISLAND

The central island of a roundabout is the raised, non-traversable area encompassed by the circulatory roadway; this area may also include a traversable apron. The island is typically landscaped for aesthetic reasons and to enhance driver recognition of the roundabout upon approach. Central islands should always be raised, not depressed, as depressed islands are difficult for approaching drivers to recognize.

In general, the central island should be circular in shape. A circular-shaped central island with a constant-radius circulatory roadway helps promote constant speeds around the central island. Oval or irregular shapes, on the other hand, are more difficult to drive and can promote higher speeds on the straight sections and reduced speeds on the arcs of the oval. This speed differential may make it harder for entering vehicles to judge the speed and acceptability of gaps in the circulatory traffic stream. It can also be deceptive to circulating drivers, leading to more loss-of-control crashes. Non-circular central islands have the above disadvantages to a rapidly increasing degree as they get larger because circulating speeds are higher. Oval shapes are generally not such a problem if they are relatively small, and speeds are low. Raindrop-shaped islands may be used in areas where certain movements do not exist, such as interchanges (see Chapter 9), or at locations where certain turning movements cannot be safely accommodated, such as roundabouts with one approach on a relatively steep grade.

As described in **Section 8.11.6.2.3**, the size of the central island plays a key role in determining the amount of deflection imposed on the through vehicle's path. However, its diameter is entirely dependent upon the inscribed circle diameter and the required circulatory roadway width (see Sub-Sections A and C of **Section 8.11.6.2.5**). Therefore, once the inscribed diameter, circulatory roadway width, and initial entry geometry have been established, the fastest vehicle path must be drawn though the layout, as described in **Sub-Section C** of **Section 8.11.6.2.3,** to determine if the central island size is adequate. If the fastest path exceeds the design speed, the central island size may need to be increased, thus increasing the overall inscribed circle diameter. There may be other methods for increasing deflection without increasing the inscribed diameter, such as offsetting the approach alignment to the left, reducing the entry width, or reducing the entry radius. These treatments, however, may preclude the ability to accommodate the design vehicle.

In cases where right-of-way, topography, or other constraints preclude the ability to expand the inscribed circle diameter, a mountable apron may be added to the outer edge of the central island. This provides additional paved area to allow the over-tracking of large semi-trailer vehicles on the central island without compromising the deflection for smaller vehicles. **Plate 8.9** shows a typical central island with a traversable apron.

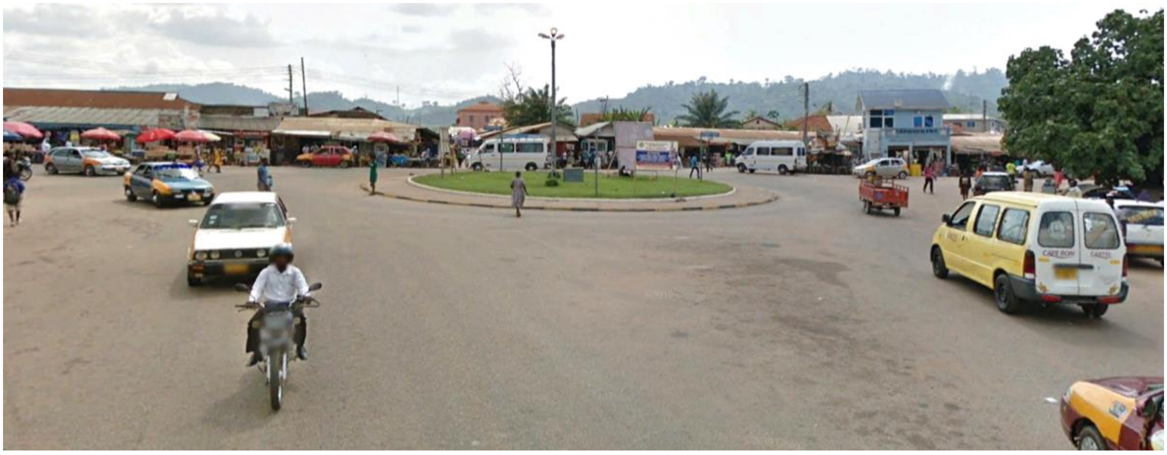


Plate 8.9 Example of central island with a traversable apron

Where aprons are used, they should be designed so that they are traversable by large vehicles but discourage small vehicles from using them. They should generally be 1 to 4 m wide and have a cross slope of 3 to 4 percent away from the central island. To discourage use by small vehicles, the outer edge of the apron should be raised a minimum of 30 mm above the circulatory roadway surface. The apron should be constructed of coloured and/or textured paving materials to differentiate it from the circulatory roadway. Care must be taken to ensure that delivery trucks will not experience load shifting as their rear trailer wheels track across the apron.

In general, roundabouts in rural environments typically need larger central islands than urban roundabouts in order to enhance their visibility and to enable the design of better approach geometry.

E. ENTRY CURVES

As shown in **Figure 8.51**, the entry curves are the set of one or more curves along the right kerb (or edge of pavement) of the entry roadway leading into the circulatory roadway. It should not be confused with the entry path curve, defined by the radius of the fastest vehicular travel path through the entry geometry (R_1 on **Figure 8.71**).

The entry radius is an important factor in determining the operation of a roundabout as it has significant impacts on both capacity and safety. The entry radius, in conjunction with the entry width, the circulatory roadway width, and the central island geometry, controls the amount of deflection imposed on a vehicle's entry path. Larger entry radii produce faster entry speeds and generally result in higher crash rates between entering and circulating vehicles. In contrast, the operational performance of roundabouts benefits from larger entry radii. Research has found that the capacity of an entry increases as its entry radius is increased (up to 20 m), beyond which entry radius has little effect on capacity.

The entry curve is designed curvilinearly tangential to the outside edge of the circulatory roadway. Likewise, the projection of the inside (left) edge of the entry roadway should be curvilinearly tangential to the central island. **Figure 8.75** shows a typical roundabout entrance geometry.

The primary objective in selecting a radius for the entry curve is to achieve the speed objectives, as described in **Section 8.11.6.2.3**. The entry radius should first produce an appropriate design speed on the fastest vehicular path. Second, it should desirably result in an entry path radius (R_1) equal to or less than the circulating path radius (R_2).

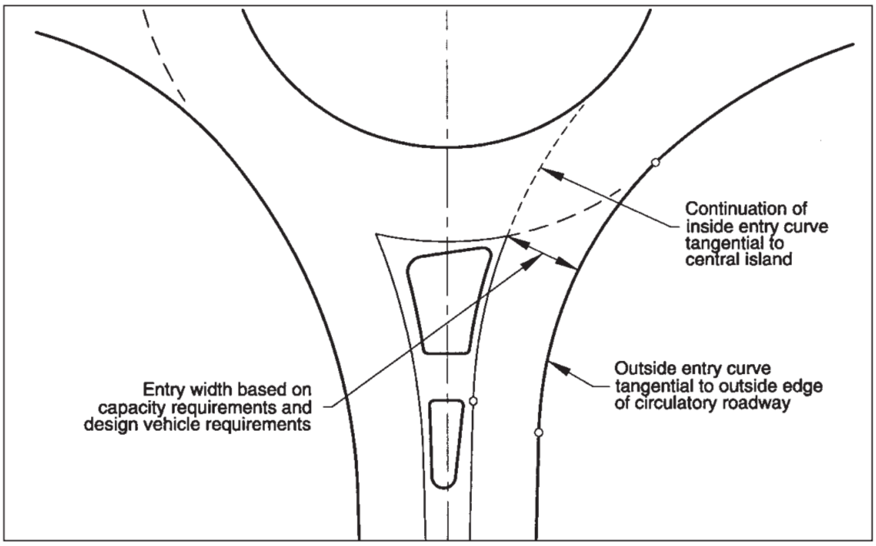


Figure 8.75 Single-lane roundabout entry design

i. Entry curves at single-lane roundabouts

For single-lane roundabouts, it is relatively simple to achieve the entry speed objectives. With a single traffic stream entering and circulating, there is no conflict between traffic in adjacent lanes. Thus, the entry radius can be reduced or increased as necessary to produce the desired entry path radius. Provided sufficient clearance is given for the design vehicle, approaching vehicles will adjust their path accordingly and negotiate through the entry geometry into the circulatory roadway.

Entry radii at urban single-lane roundabouts typically range from 10 to 30 m. Larger radii may be used, but it is important that the radii not be so large as to result in excessive entry speeds. At local street roundabouts, entry radii may be below 10 m if the design vehicle is small.

At rural and suburban locations, consideration should be given to the speed differential between the approaches and entries. If the difference is greater than 20 km/h, it is desirable to introduce approach curves or some other speed reduction measures to reduce the speed of approaching traffic prior to the entry curvature. Further details on rural roundabout design are provided in **Section 8.11.6.2.7**.

ii. Entry curves at double-lane roundabouts

At double-lane roundabouts, the design of the entry curvature is more complicated. Overly small entry radii can result in conflicts between adjacent traffic streams. This conflict usually results in poor lane utilization of one or more lanes and significantly reduces the capacity of the approach. It can also degrade the safety performance as sideswipe crashes may increase. Techniques and guidelines for avoiding conflicts between adjacent entry lanes at double-lane roundabouts are provided in **Section 8.11.6.2.6**.

F. EXIT CURVES

Exit curves usually have larger radii than entry curves to minimize the likelihood of congestion at the exits. This, however, is balanced by the need to maintain low speeds at the pedestrian crossing on exit. The exit curve should produce an exit path radius (R_3 in **Figure 8.71**) no smaller than the circulating path radius (R_2). If the exit path radius is smaller than the circulating path radius, vehicles will be traveling too fast to negotiate the exit geometry and may crash into the splitter island or into oncoming traffic in the adjacent approach lane. Likewise, the exit path radius should not be significantly greater than the circulating path radius to ensure low speeds at the downstream pedestrian crossing.

The exit curve is designed to be curvilinearly tangential to the outside edge of the circulatory roadway. Likewise, the projection of the inside (left) edge of the exit roadway should be curvilinearly tangential to the central island. **Figure 8.76** shows a typical exit layout for a single-lane roundabout.

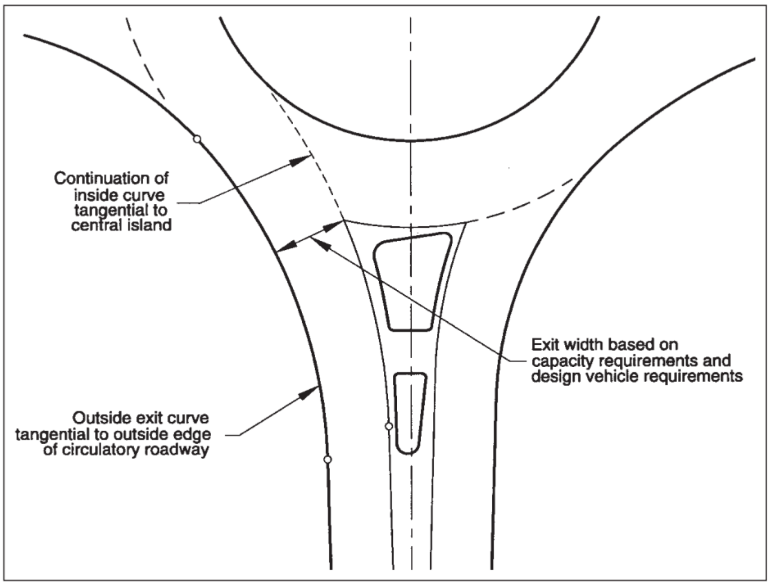


Figure 8.76 Single lane roundabout exit design

i. Exit curves at single-lane roundabouts

At single-lane roundabouts in urban environments, exits should be designed to enforce a curved exit path with a design speed below 40km/h in order to maximize safety for pedestrians crossing the exiting traffic stream. Generally, exit radii should not be less than 15m. However, at locations with pedestrian activity and no large semi-trailer traffic, exit radii may be as low as 10 to 12m. This produces a very slow design speed to maximize safety and comfort for pedestrians. Such low exit radii should only be used in conjunction with similar or smaller entry radii on urban compact roundabouts with inscribed circle diameters below 35m.

In rural locations where there are few pedestrians, exit curvature may be designed with large radii, allowing vehicles to exit quickly and accelerate back to traveling speed. This, however, should not result in a straight path tangential to the central island because many locations that are rural today become urban in the future. Therefore, it is recommended that pedestrian activity be considered at all exits except where separate pedestrian facilities (paths, etc.) or other restrictions eliminate the likelihood of pedestrian activity in the foreseeable future.

ii. Exit curves at double-lane roundabouts

As with the entries, the design of the exit curvature at double-lane roundabouts is more complicated than at single-lane roundabouts. Techniques and guidelines for avoiding conflicts between adjacent exit lanes at double-lane roundabouts are provided in **Section 8.11.6.2.6**.

G. PEDESTRIAN CROSSING LOCATION AND TREATMENTS

Pedestrian crossing locations at roundabouts are a balance among pedestrian convenience, pedestrian safety, and roundabout operations:

i. Pedestrian convenience:

Pedestrians want crossing locations as close to the intersection as possible to minimize out-of-direction travel. The further the crossing is from the roundabout, the more likely that pedestrians will choose a shorter route that may put them in greater danger.

ii. Pedestrian safety:

Both crossing location and crossing distance are important. Crossing distance should be minimized to reduce exposure of pedestrian-vehicle conflicts. Pedestrian safety may also be compromised at a yield-line crosswalk because driver attention is directed to the left to look for gaps in the circulating traffic stream. Crosswalks should be located to take advantage of the splitter island; crosswalks located too far from the yield line require longer splitter islands. Crossings should also be located at distances away from the yield line measured in increments of approximate vehicle length to reduce the chance that vehicles will be queued across the crosswalk.

iii. Roundabout operations:

Roundabout operations (primarily vehicular) can also be affected by crosswalk locations, particularly on the exit. A queuing analysis at the exit crosswalk may determine that a crosswalk location of more than one vehicle length away may be required to reduce to an acceptable level the risk of queuing into the circulatory roadway. Pedestrians may be able to distinguish exiting vehicles from circulating vehicles (both visually and audibly) at crosswalk locations further away from the roundabout, although this has not been confirmed by research.

With these issues in mind, pedestrian crossings should be designed as follows:

i. The pedestrian refuge should be a minimum width of 1.8 m to adequately provide shelter for persons pushing a stroller or walking a bicycle (see **Section 8.11.6.2.2**).

ii. At single-lane roundabouts, the pedestrian crossing should be located one vehicle-length (7.5 m) away from the yield line. At double-lane roundabouts, the pedestrian crossing should be located one, two, or three car lengths (approximately 7.5 m, 15 m, or 22.5 m) away from the yield line.

iii. The pedestrian refuge should be designed at road level, rather than elevated to the height of the splitter island. This eliminates the need for ramps within the refuge area, which can be cumbersome for wheelchairs.

iv. Ramps should be provided on each end of the crosswalk to connect the crosswalk to other crosswalks around the roundabout and to the walkway network.

v. It is recommended that a detectable warning surface be applied to the surface of the refuge within the splitter island as shown in **Figure 8.77**. Where used, a detectable warning surface shall meet the following requirements.

- The detectable warning surface shall consist of raised truncated domes with a nominal diameter of 23mm, a nominal height of 5mm , and a nominal centre-to-centre spacing of 60mm.
- The detectable warning surface shall contrast visually with adjoining surfaces, either light-on-dark or dark-on-light. The material used to provide contrast shall be an integral part of the walking surface.
- The detectable warning surface shall begin at the kerb line and extend into the pedestrian refuge area a distance of 600 mm. This creates a minimum 600mm clear space between detectable warning surfaces for a minimum splitter island width of 1.8 m at the pedestrian crossing.

In urban areas, speed tables (flat-top road humps) could be considered for wheelchair users, provided that good geometric design has reduced absolute vehicle speeds to less than 20km/h near the crossing. Pedestrian crossings across speed tables must have detectable warning material as described above to clearly delineate the edge of the road. Speed tables should generally be used only on roads with approach speeds of 50km/h or less, as the introduction of a raised speed table in higher speed environments may increase the likelihood of single-vehicle crashes and is not consistent with the speed consistency philosophy presented in this document.

H. SPLITTER ISLAND

Splitter islands (also called separator islands or median islands) should be provided on all roundabouts, except those with very small diameters at which the splitter island would obstruct the visibility of the central island. Their purpose is to provide shelter for pedestrians (including wheelchairs, bicycles, and baby strollers), assist in controlling speeds, guide traffic into the roundabout, physically separate entering and exiting traffic streams, and deter wrong-way movements. Additionally, splitter islands can be used as a place for mounting signs. The splitter island envelope is formed by the entry and exit curves on a leg, as shown previously in **Figure 8.75** and **Figure 8.76**. The total length of the island should generally be at least 15m to provide sufficient protection for pedestrians and to alert approaching drivers to the roundabout geometry. Additionally, the splitter island should extend beyond the end of the exit curve to prevent exiting traffic from accidentally crossing into the path of approaching traffic. **Figure 8.77** shows the minimum dimensions for a splitter island at a single lane roundabout, including the location of the pedestrian crossing as discussed in **Sub-Section G** of **Section 8.11.6.2.5**.

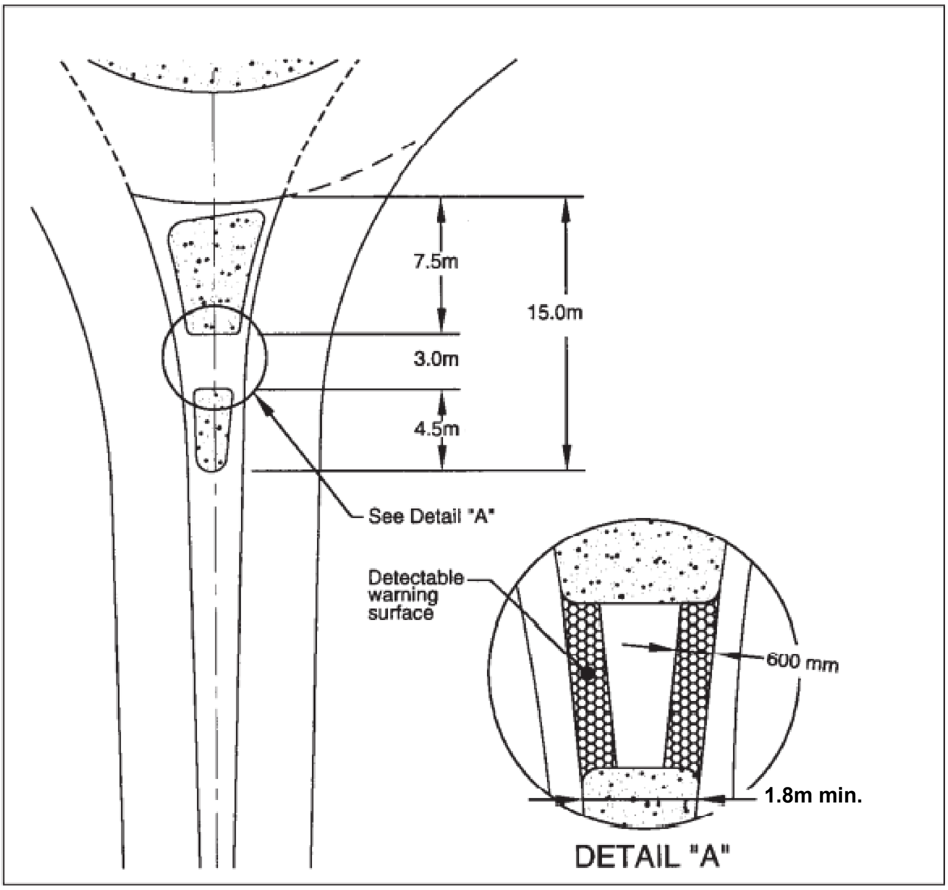


Figure 8.77 Minimum splitter island dimensions.

While **Figure 8.77** provides minimum dimensions for splitter islands, there are benefits to providing larger islands. Increasing the splitter island width results in greater separation between the entering and exiting traffic streams of the same leg and increases the time for approaching drivers to distinguish between exiting and circulating vehicles. In this way, larger splitter islands can help reduce confusion for entering motorists. However, increasing the width of the splitter islands generally requires increasing the inscribed circle diameter. Thus, these safety benefits may be offset by higher construction cost and greater land impacts.

Guidelines for island design as discussed in **Section 8.9.2** should be followed for the splitter island. This includes using larger nose radii at approach corners to maximize island visibility and offsetting kerb lines at the approach ends to create a funnelling effect. The funnelling treatment also aids in reducing speeds as vehicles approach the roundabout. **Figure 8.78** shows minimum splitter island nose radii and offset dimensions from the entry and exit travelled ways.

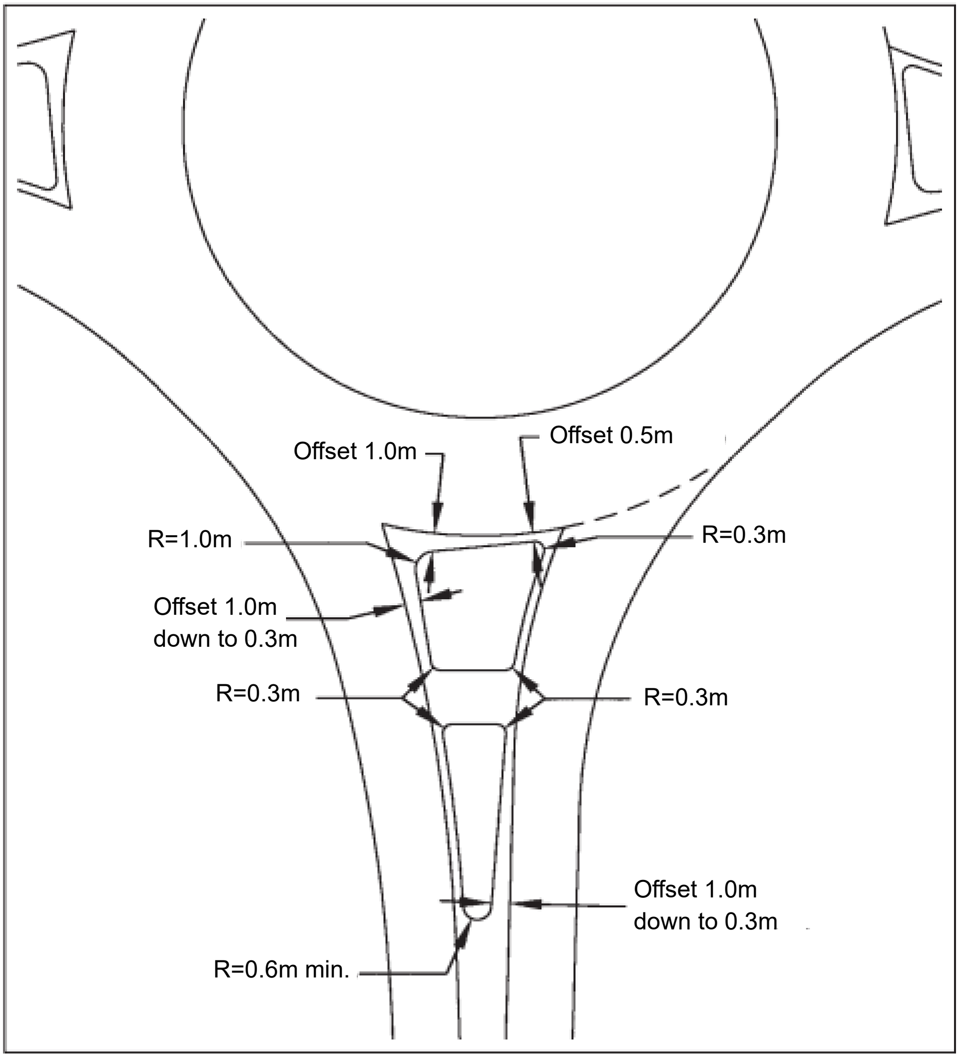


Figure 8.78 Minimum splitter island nose radii and offsets

I. STOPPING SIGHT DISTANCE

Stopping sight distance is the distance along a roadway required for a driver to perceive and react to an object in the roadway and to brake to a complete stop before reaching that object. Stopping sight distance should be provided at every point within a roundabout and on each entering and exiting approach. Refer to **Section 7.2.6.1**. At roundabouts, three critical types of locations should be checked at a minimum:

i. Approach sight distance (Figure 8.79);
ii. Sight distance on circulatory roadway (**Figure 8.80**); and
iii. Sight distance to crosswalk on exit (**Figure 8.81**).

Forward sight distance at entry can also be checked; however, this will typically be satisfied by providing adequate stopping sight distance on the circulatory roadway itself.

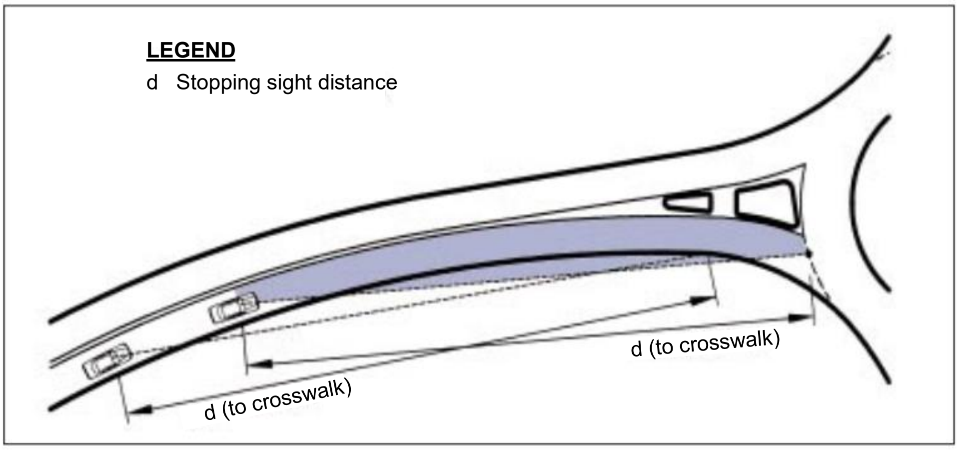


Figure 8.79 Approach sight distance

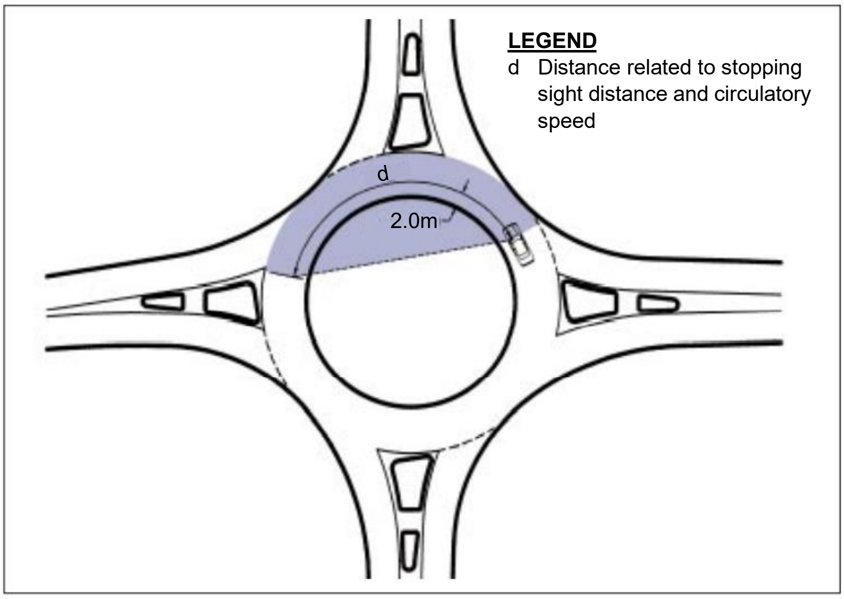


Figure 8.80 Sight distance on circulatory roadway

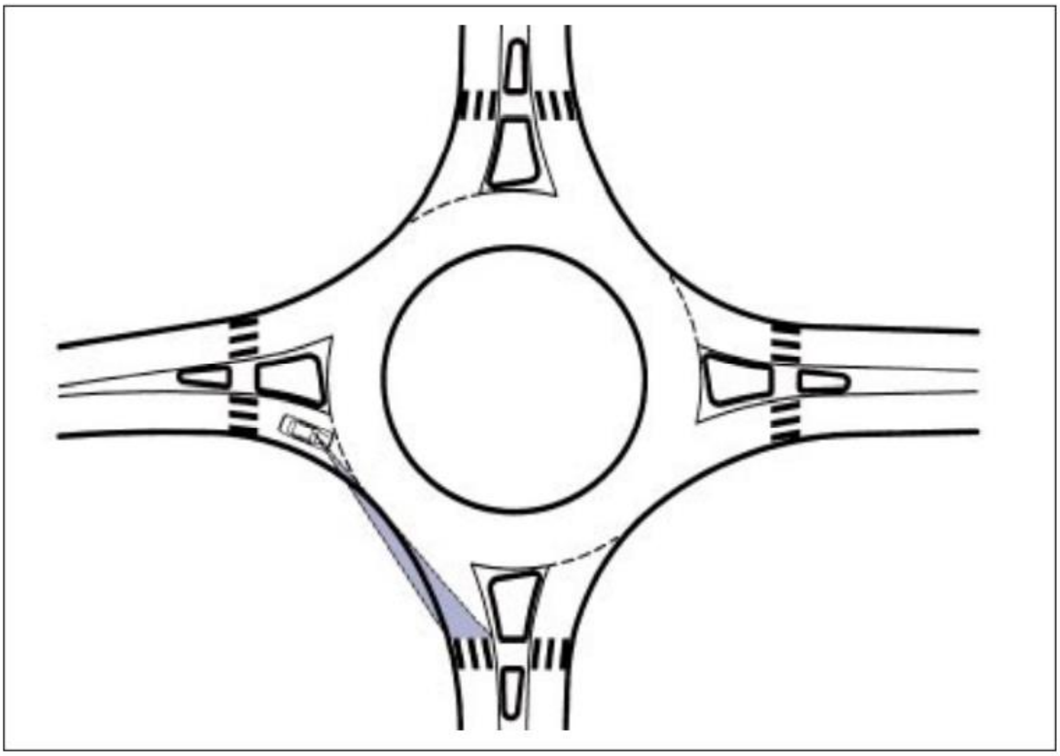


Figure 8.81 Sight distance to crosswalk on exit

J. INTERSECTION SIGHT DISTANCE

As discussed in **Section 7.2.6.4**, intersection sight distance is the distance required for a driver without the right of way to perceive and react to the presence of conflicting vehicles. Intersection sight distance is achieved through the establishment of adequate sight lines that allow a driver to see and safely react to potentially conflicting vehicles. At roundabouts, the only locations requiring evaluation of intersection sight distance are the entries.

Figure 8.82 presents a diagram showing the method for determining intersection sight distance. As can be seen in the figure, the sight distance "triangle" has two conflicting approaches that must be checked independently. The following two subsections discuss the calculation of the length of each of the approaching sight limits.

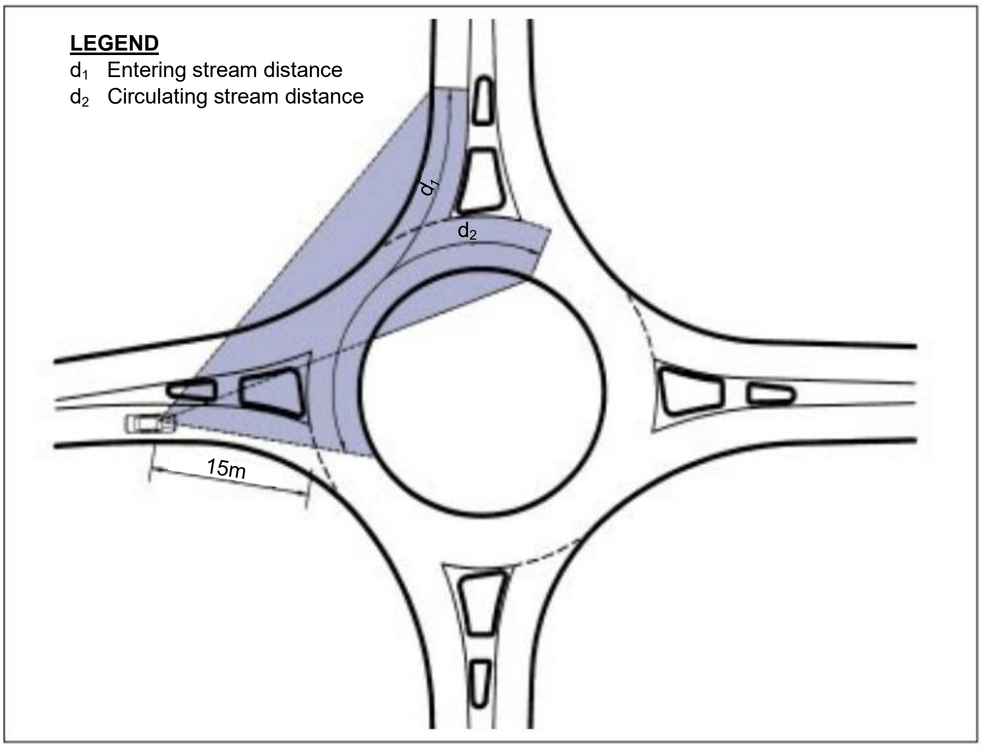


Figure 8.82 Intersection sight distance

i. Length of approach leg of sight triangle

The length of the approach leg of the sight triangle should be limited to 15m. Research has shown that excessive intersection sight distance results in a higher frequency of crashes. This value is intended to require vehicles to slow down prior to entering the roundabout, which allows them to focus on the pedestrian crossing prior to entry. If the approach leg of the sight triangle is greater than 15m, it may be advisable to add landscaping to restrict sight distance to the minimum requirements.

ii. Length of conflicting leg of sight triangle

A vehicle approaching an entry to a roundabout faces conflicting vehicles within the circulatory roadway. The length of the conflicting leg is calculated using **Equation 8.12:**

$$b = 0.278(V_{major})(t_c) \tag{8.12}$$

Where,

b	=	length of conflicting leg of sight triangle, m
V_{major}	=	design speed of conflicting movement, km/h
t_c	=	critical gap for entering the major road, s, equal to 6.5s

Two conflicting traffic streams should be checked at each entry:

i. Entering stream, comprised of vehicles from the immediate upstream entry. The speed for this movement can be approximated by taking the average of the entry path speed (path with radius R_1 from Figure 8.71) and the circulating path speed (path with radius R_2 from Figure 8.71).

ii. Circulating stream, comprised of vehicles that entered the roundabout prior to the immediate upstream entry. This speed can be approximated by taking the speed of left turning vehicles (path with radius R4 from **Figure 8.71**).

The critical gap for entering the major road is based on the amount of time required for a vehicle to turn right while requiring the conflicting stream vehicle not to slow down below 70 percent of initial speed. This is based on research on critical gaps at stop-controlled intersections, adjusted for yield-controlled conditions (Ref). The critical gap value of 6.5s given in **Equation 8.12** is based on the critical gap required for small vehicles, which are assumed to be the most critical design vehicle for intersection sight distance. This assumption holds true for large vehicle and trailer speeds that are at least 10km/h and 15 to 20km/h slower than small vehicles, respectively. Computed length of conflicting leg of intersection sight triangle is shown in **Table 8.40**.

Table 8.40 Computed length of conflicting leg of intersection sight triangle.

Conflicting Approach Speed (km/h)	Computed Distance (m)
20	36.1
25	45.2
30	54.2
35	63.2
40	72.3

In general, it is recommended to provide not more than the minimum required intersection sight distance on each approach. Excessive intersection sight distance can lead to higher vehicle speeds that reduce the safety of the intersection for all road users (vehicles, bicycles, pedestrians). Landscaping can be effective in restricting sight distance to the minimum requirements.

Note that the stopping sight distance on the circulatory roadway (**Figure 8.80**) and the intersection sight distance to the circulating stream (**Figure 8.82**) imply restrictions on the height of the central island, including landscaping and other objects, within these zones. In the remaining central area of the central island, higher landscaping may serve to break the forward vista for through vehicles, thereby contributing to speed reduction. However, should errant vehicles encroach on the central island, **Figure 8.104** shows the recommended maximum grades on the central island to minimize the probability of the vehicles rolling over, causing serious injury.

K. VERTICAL CONSIDERATIONS

Elements of vertical alignment design for roundabouts include profiles, superelevation, approach grades, and drainage.

i. Profiles

The vertical design of a roundabout begins with the development of approach roadway and central island profiles. The development of each profile is an iterative process that involves tying the elevations of the approach roadway profiles into a smooth profile around the central island.

Generally, each approach profile should be designed to the point where the approach baseline intersects with the central island. A profile for the central island is then developed which passes through these four points (in the case of a fourlegged roundabout). The approach roadway profiles are then readjusted as necessary to meet the central island profile. The shape of the central island profile is generally in the form of a sine curve. Examples of how the profile is developed can be found in **Figure 8.83**, **Figure 8.84** and **Figure 8.85**, which consist of a sample plan, profiles on each approach, and a profile along the central island, respectively. Note that the four points where the approach roadway baseline intersects the central island baseline are identified on the central island profile.

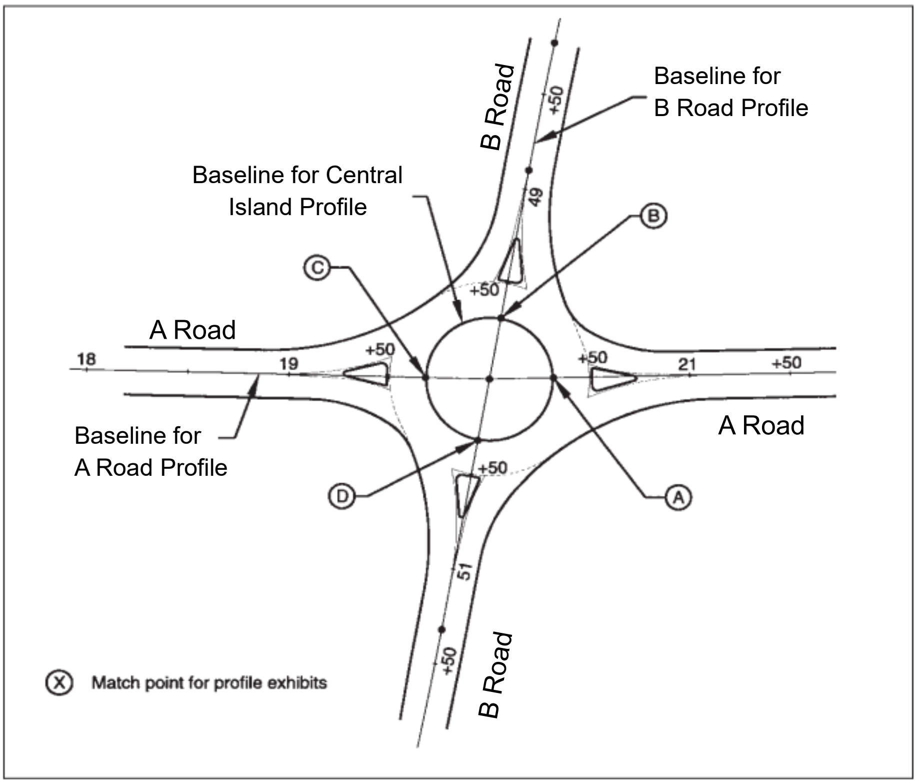


Figure 8.83 Sample plan view

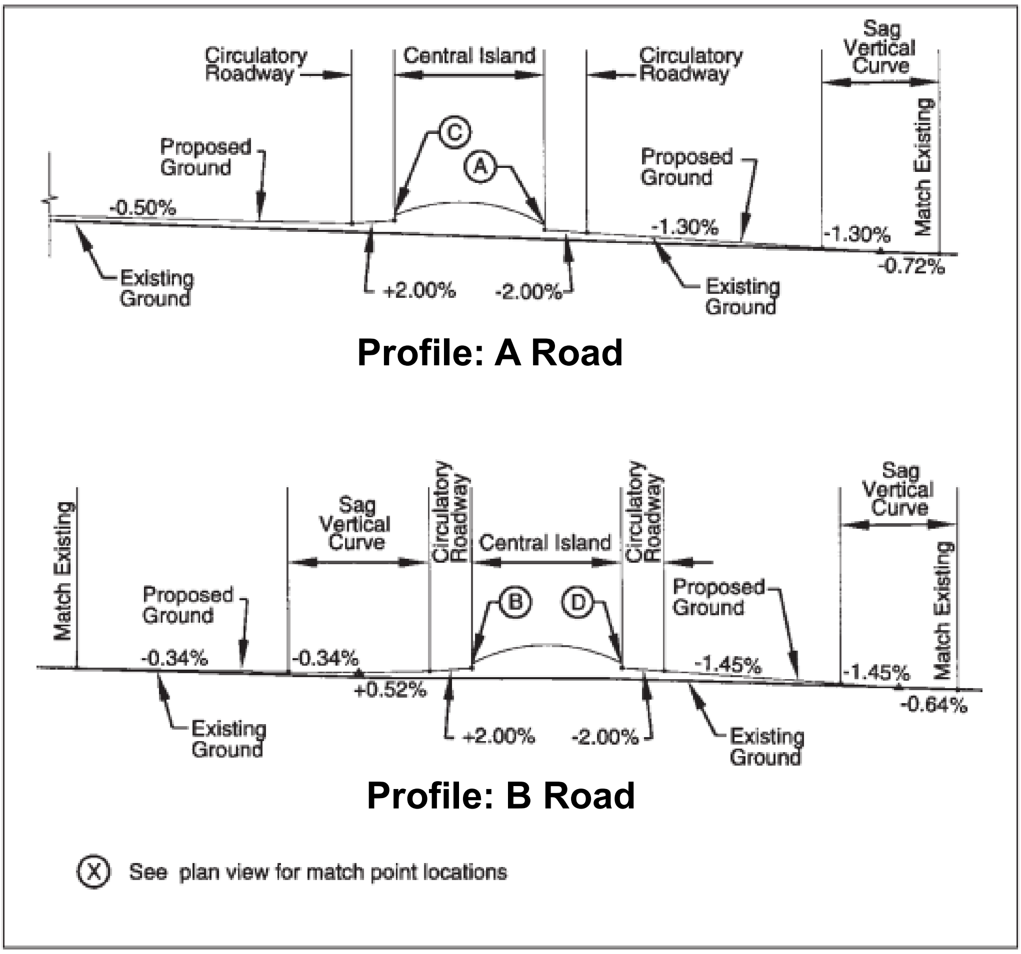


Figure 8.84 Sample approach profile

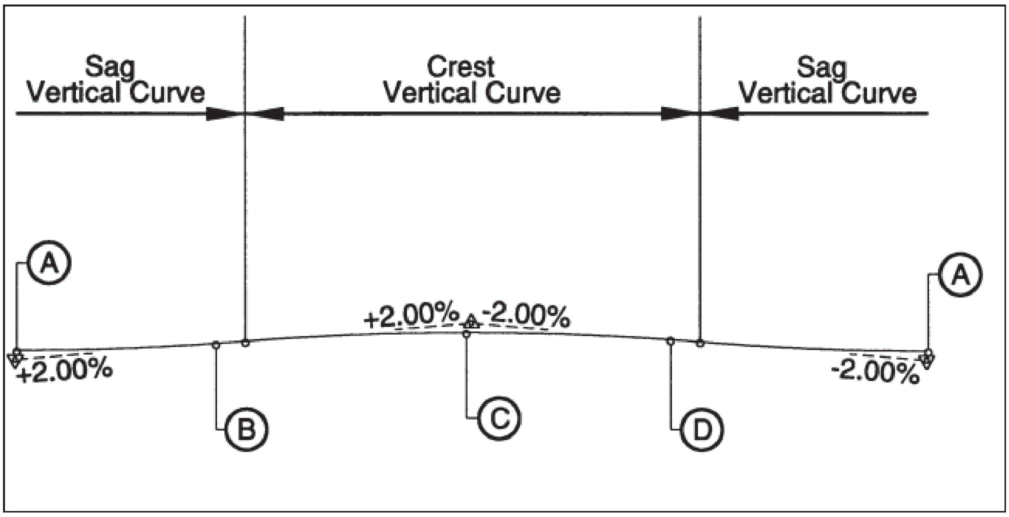


Figure 8.85 Sample central island profile

ii. Superelevation

As a general practice, a cross slope of 2.5 percent away from the central island should be used for the circulatory roadway. This technique of sloping outward is recommended for four main reasons:

a. It promotes safety by raising the elevation of the central island and improving its visibility.
b. It promotes lower circulating speeds.
c. It minimizes breaks in the cross slopes of the entrance and exit lanes.
d. It helps drain surface water to the outside of the roundabout.

The outward cross slope design means vehicles making through and left-turn movements must negotiate the roundabout at negative superelevation. Excessive negative superelevation can result in an increase in single-vehicle crashes and loss-of-load/load shift incidents for trucks, particularly if speeds are high. However, in the intersection environment, drivers will generally expect to travel at slower speeds and will accept the higher side force caused by reasonable adverse superelevation.

Figure 8.86 provides a typical section across the circulatory roadway of a roundabout without a truck apron. **Figure 8.87** provides a typical section for a roundabout with a truck apron. Where truck aprons are used, the slope of the apron should be 3 to 4 percent; greater slopes may increase the likelihood of loss-of-load incidents. Refer to Table 6.2 for values of cross slope based on carriageway surface type.

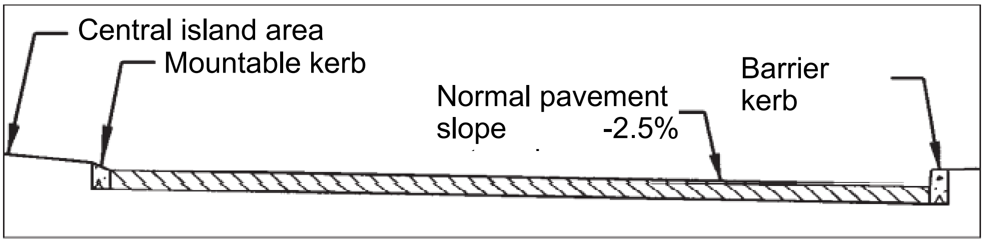


Figure 8.86 Typical circulatory roadway section

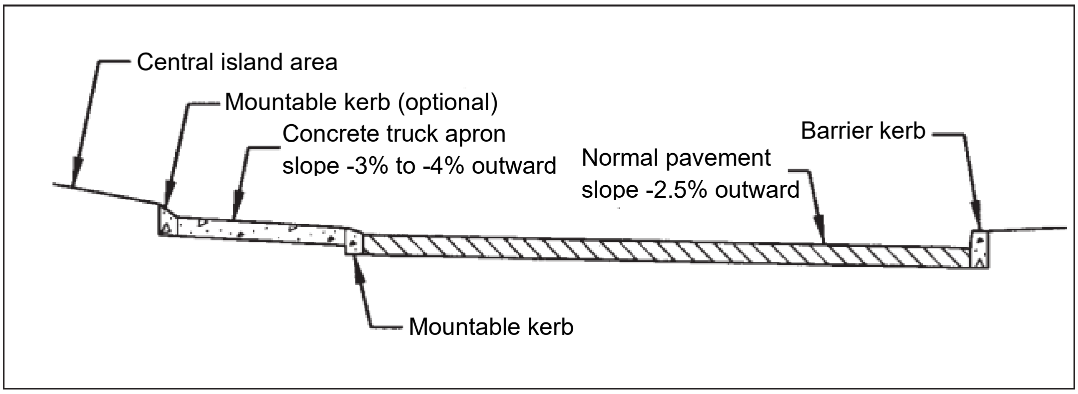


Figure 8.87 Typical section with a truck apron

iii. Locating roundabouts on grades

It is generally not desirable to locate roundabouts in locations where grades through the intersection are greater than 4 percent. The installation of roundabouts on roadways with grades lower than 3 percent is generally not problematic. At locations where a constant grade must be maintained through the intersection, the circulatory roadway may be constructed on a constant-slope plane. This means, for instance, that the cross slope may vary from +3 percent on the high side of the roundabout (sloped toward the central island) to -3 percent on the low side (sloped outward). Note that central island cross slopes will pass through level at a minimum of two locations for roundabouts constructed on a constant grade.

Care must be taken when designing roundabouts on steep grades. On approach roadways with grades steeper than -4 percent, it is more difficult for entering drivers to slow or stop on the approach. At roundabouts on crest vertical curves with steep approaches, a driver's sight lines will be compromised, and the roundabout may violate driver expectancy. However, under the same conditions, other types of at-grade intersections often will not provide better solutions. Therefore, the roundabout should not necessarily be eliminated from consideration at such a location. Rather, the intersection should be relocated or the vertical profile modified, if possible.

iv. Drainage

With the circulatory roadway sloping away from the central island, inlets will generally be placed on the outer kerbline of the roundabout. However, inlets may be required along the central island for a roundabout designed on a constant grade through an intersection. As with any intersection, care should be taken to ensure that low points and inlets are not placed in crosswalks. If the central island is large enough, the designer may consider placing inlets in the central island.

L. BICYCLE PROVISIONS

With regard to bicycle treatments, the designer should strive to provide bicyclists the choice of proceeding through the roundabout as either a vehicle or a pedestrian. In general, bicyclists are better served by treating them as vehicles. However, the best design provides both options to allow cyclists of varying degrees of skill to choose their more comfortable method of navigating the roundabout.

To accommodate bicyclists traveling as vehicles, bike lanes should be terminated in advance of the roundabout to encourage cyclists to mix with vehicle traffic. Under this treatment, it is recommended that bike lanes end 30 m upstream of the yield line to allow for merging with vehicles. This method is most successful at smaller roundabouts with speeds below 30 km/h, where bicycle speeds can more closely match vehicle speeds.

To accommodate bicyclists who prefer not to use the circulatory roadway, a widened walkway or a shared bicycle/pedestrian lane may be provided physically separated from the circulatory roadway (not as a bike lane within the circulatory roadway). Ramps or other suitable connections can then be provided between this walkway or lane and the bike lanes, shoulders, or road surface on the approaching and departing roadways. The designer should exercise care in locating and designing the bicycle ramps so that they are not misconstrued by pedestrians as an unmarked pedestrian crossing. Nor should the exits from the roadway onto a shared path allow cyclists to enter the shared path at excessive speeds. **Figure 8.88** illustrates a possible design of this treatment.

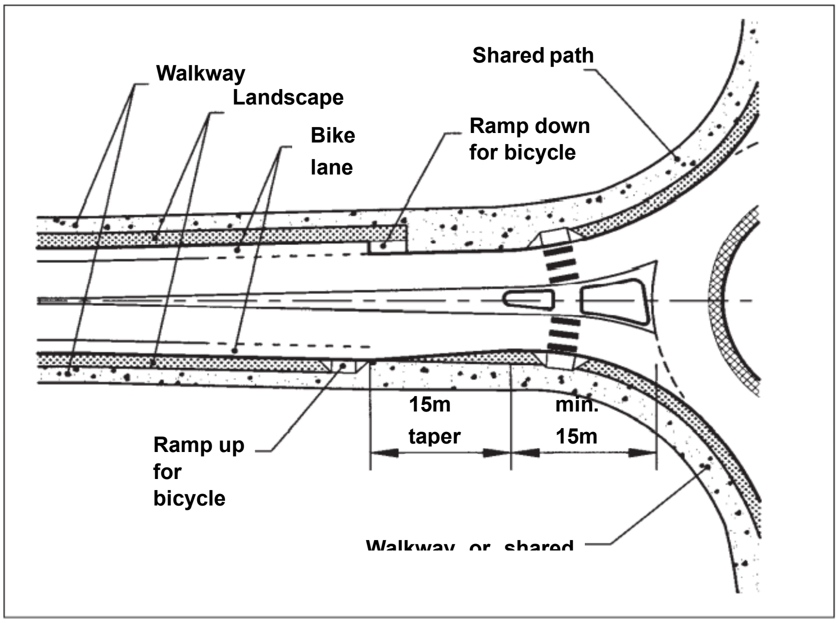


Figure 8.88 Possible provisions for bicycles.

M. WALKWAY TREATMENTS

Where possible, walkways should be set back from the edge of the circulatory roadway in order to discourage pedestrians from crossing to the central island, particularly when an apron is present or a monument on the central island. Equally important, the design should help pedestrians with visual impairments to recognize that they should not attempt to cross roads from corner to corner but at designated crossing points. To achieve these goals, the walkway should be designed so that pedestrians will be able to clearly find the intended path to the crosswalks. A recommended set back distance of 1.5m (minimum 0.6 m) should be used, and the area between the walkway and kerb can be planted with low shrubs or grass. **Figure 8.89** shows this technique.

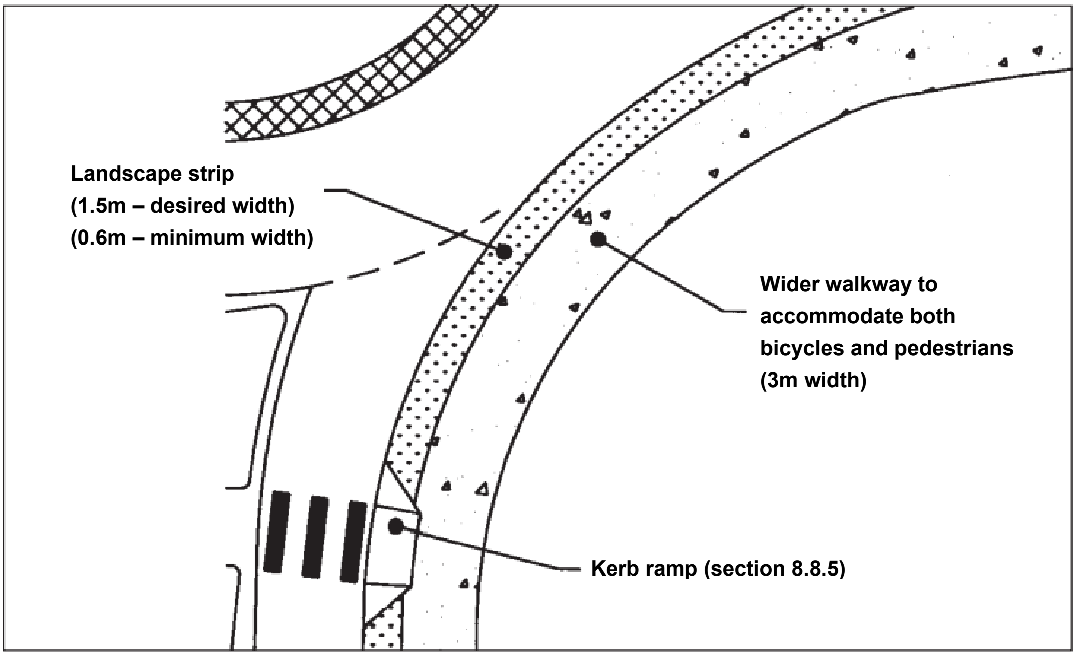


Figure 8.89 Walkway treatments

N. PARKING CONSIDERATIONS AND BUS STOP LOCATIONS

Parking or stopping in the circulatory roadway is not conducive to proper roundabout operations and should be prohibited. Parking on entries and exits should also be set back as far as possible so as not to hinder roundabout operations or to impair the visibility of pedestrians. Parking should end at least 15m from the crosswalk of an intersection. Kerb extensions or "bulb-outs" can be used to clearly mark the limit of permitted parking and reduce the width of the entries and exits.

For safety and operational reasons, bus stops should be located as far away from entries and exits as possible, and never in the circulatory roadway.

i. **Near-side stops:** If a bus stop is to be provided on the near side of a roundabout, it should be located far enough away from the splitter island so that a vehicle overtaking a stationary bus is in no danger of being forced into the splitter island, especially if the bus starts to pull away from the stop. If an approach has only one lane and capacity is not an issue on that entry, the bus stop could be located at the pedestrian crossing in the lane of traffic. This is not recommended for entries with more than one lane, because vehicles in the lane next to the bus may not see pedestrians.

ii. **Far-side stops:** Bus stops on the far side of a roundabout should be constructed with pull-outs to minimize queuing into the roundabout. These stops should be located beyond the pedestrian crossing to improve visibility of pedestrians to other exiting vehicles.

O. RIGHT-TURN BYPASS LANES

In general, right-turn bypass lanes (or right-turn slip lanes) should be avoided, especially in urban areas with bicycle and pedestrian activity. The entries and exits of bypass lanes can increase conflicts with bicyclists. The generally higher speeds of bypass lanes and the lower expectation of drivers to stop increases the risk of collisions with pedestrians. However, in locations with minimal pedestrian and bicycle activity, right-turn bypass lanes can be used to improve capacity where there is heavy right turning traffic.

The provision of a right-turn bypass lane allows right-turning traffic to bypass the roundabout, providing additional capacity for the through and left-turn movements at the approach. They are most beneficial when the demand of an approach exceeds its capacity and a significant proportion of the traffic is turning right. However, it is important to consider the reversal of traffic patterns during the opposite peak time period. In some cases, the use of a right-turn bypass lane can avoid the need to build an additional entry lane and thus a larger roundabout. To determine if a right-turn bypass lane should be used, capacity and delay calculations should be performed. Right-turn bypass lanes can also be used in locations where the geometry for right turns is too tight to allow trucks to turn within the roundabout. **Plate 8.10** shows an example of right turn bypass lane.

Plate 8.10 Example of right-turn bypass lane

There are two design options for right-turn bypass lanes.

i. The first option, shown in Figure 8.90, is to carry the bypass lane parallel to the adjacent exit roadway, and then merge it into the main exit lane. Under this option, the bypass lane should be carried alongside the main roadway for a sufficient distance to allow vehicles in the bypass lane and vehicles exiting the roundabout to accelerate to comparable speeds. The bypass lane is then merged at a taper rate according to **Section 8.8** for the appropriate design speed.

ii. The second design option for a right-turn bypass lane, shown in **Figure 8.91,** is to provide a yield-controlled entrance onto the adjacent exit roadway.

The first option provides better operational performance than the second does. However, the second option generally requires less construction and right-of-way than the first.

The option of providing yield control on a bypass lane is generally better for both bicyclists and pedestrians and is recommended as the preferred option in urban areas where pedestrians and bicyclists are prevalent. Acceleration lanes can be problematic for bicyclists because they end up being to the left of accelerating motor vehicles. In addition, yield control at the end of a bypass lane tends to slow motorists down, whereas an acceleration lane at the end of a bypass lane tends to promote higher speeds.

The radius of the right-turn bypass lane should not be significantly larger than the radius of the fastest entry path provided at the roundabout. This will ensure vehicle speeds on the bypass lane are similar to speeds through the roundabout, resulting in safe merging of the two roadways. Providing a small radius also provides greater safety for pedestrians who must cross the right-turn slip lane.

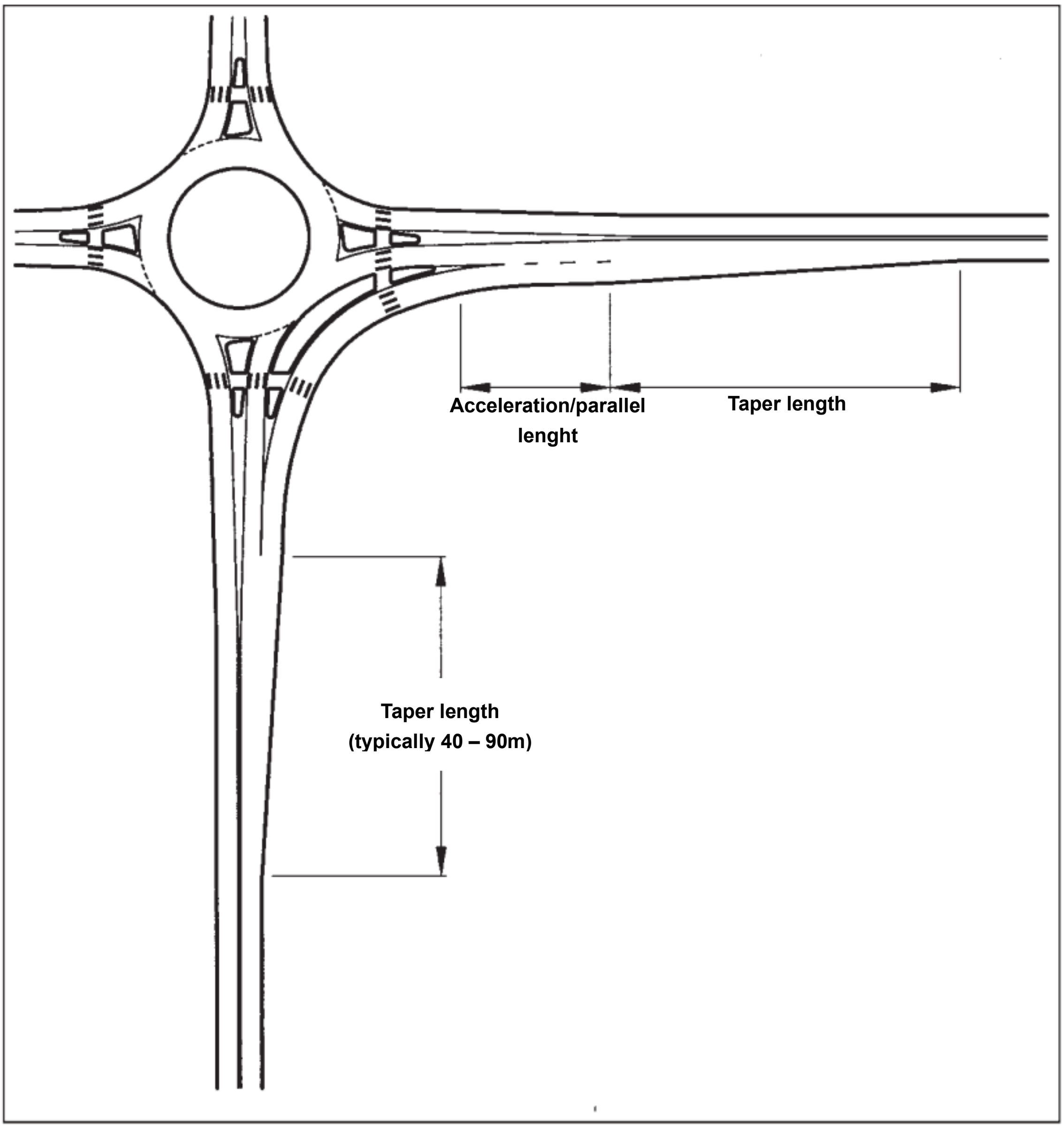


Figure 8.90 Configuration of right-turn bypass lane with acceleration lane

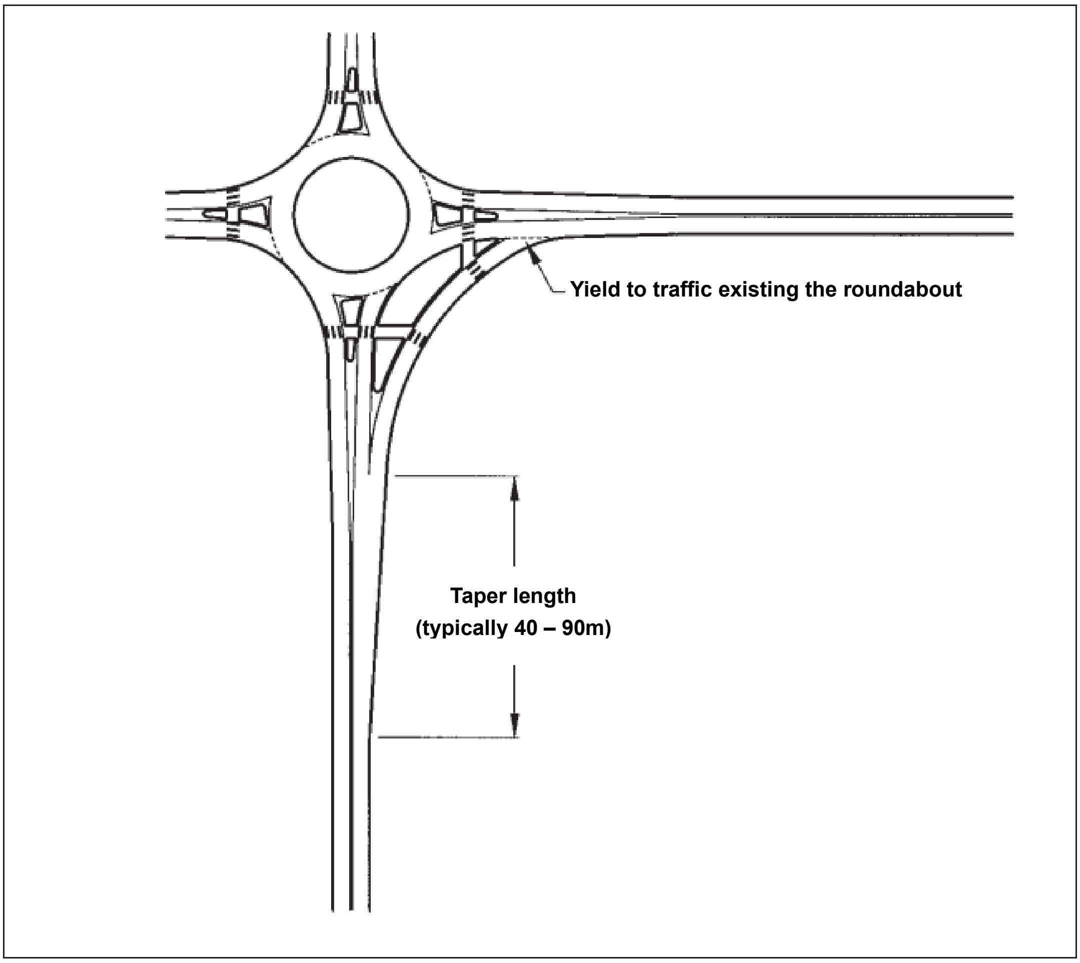


Figure 8.91 Configuration of right-turn bypass with yield at exit leg

8.11.6.2.6 DOUBLE LANE ROUNDABOUTS

While the fundamental principles described above apply to double-lane roundabouts as well as single-lane roundabouts, designing the geometry of double-lane roundabouts is more complicated. Because multiple traffic streams may enter, circulate through, and exit the roundabout side-by-side, consideration must be given to how these adjacent traffic streams interact with each other. Vehicles in adjacent entry lanes must be able to negotiate the roundabout geometry without competing for the same space. Otherwise, operational and/or safety deficiencies can occur.

A. THE NATURAL VEHICLE PATH

As discussed in **Section 8.11.6.2.3**, the fastest path through the roundabout is drawn to ensure the geometry imposes sufficient curvature to achieve a safe design speed. This path is drawn assuming the roundabout is vacant of all other traffic and the vehicle cuts across adjacent travel lanes, ignoring all lane markings. In addition to evaluating the fastest path at double-lane roundabouts, the designer must also evaluate the natural vehicle paths. This is the path an approaching vehicle will naturally take, assuming there is traffic in all approach lanes, through the roundabout geometry.

As two traffic streams approach the roundabout in adjacent lanes, they will be forced to stay in their lanes up to the yield line. At the yield point, vehicles will continue along their natural trajectory into the circulatory roadway, then curve around the central island, and curve again into the opposite exit roadway. The speed and orientation of the vehicle at the yield line determines its natural path. If the natural path of one lane interferes or overlaps with the natural path of the adjacent lane, the roundabout will not operate as safely or efficiently as possible.

The key principle in drawing the natural path is to remember that drivers cannot change the direction of their vehicle instantaneously. Neither can they change their speed instantaneously. This means that the natural path does not have sudden changes in curvature; it has transitions between tangents and curves and between consecutive reversing curves. Secondly, it means that consecutive curves should be of similar radius. If a second curve has a significantly smaller radius than the first curve, the driver will be traveling too fast to negotiate the turn and may lose control of the vehicle. If the radius of one curve is drawn significantly smaller than the radius of the previous curve, the path should be adjusted.

To identify the natural path of a given design, it may be advisable to sketch the natural paths over the geometric layout, rather than use a computer drafting program or manual drafting equipment. In sketching the path, the designer will naturally draw transitions between consecutive curves and tangents, similar to the way a driver would negotiate a vehicle. Freehand sketching also enables the designer to feel how changes in one curve affect the radius and orientation of the next curve. In general, the sketch technique allows the designer to quickly obtain a smooth, natural path through the geometry that may be more difficult to obtain using a computer. **Figure 8.92** illustrates a sketched natural path of a vehicle through a typical double lane roundabout.

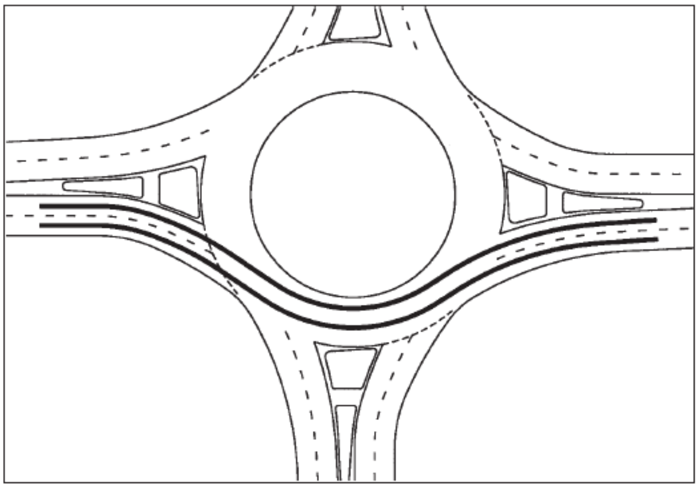


Figure 8.92 Sketched natural paths through a double-lane roundabout

B. VEHICLE PATH OVERLAP

Vehicle path overlap occurs when the natural path through the roundabout of one traffic stream overlaps the path of another. This can happen to varying degrees. It can reduce capacity, as vehicles will avoid using one or more of the entry lanes. It can also create safety problems, as the potential for sideswipe and single-vehicle crashes is increased. The most common type of path overlap is where vehicles in the left lane on entry are cut off by vehicles in the right lane, as shown in **Figure 8.93**.

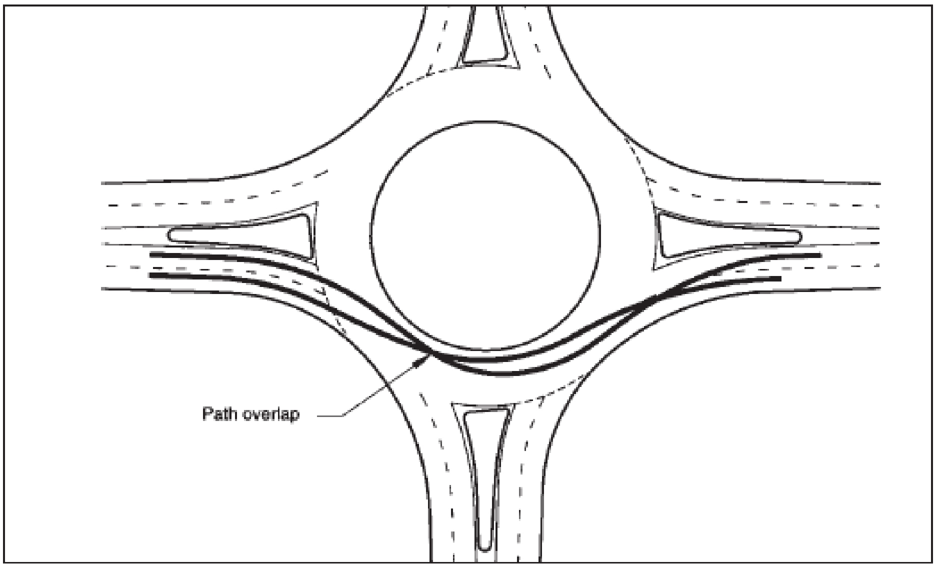


Figure 8.93 Path overlap at a double-lane roundabout

C. DESIGN METHOD TO AVOID PATH OVERLAP

Achieving a reasonably low design speed at a double-lane roundabout while avoiding vehicle path overlap can be difficult because of conflicting interaction between the various geometric parameters. Providing small entry radii can produce low entry speeds, but often leads to path overlap on the entry, as vehicles will cut across lanes to avoid running into the central island. Likewise, providing small exit radii can aid in keeping circulating speeds low, but may result in path overlap at the exits.

i. Entry curves

At double-lane entries, the designer needs to balance the need to control entry speed with the need to minimize path overlap. This can be done in a variety of ways that will vary significantly depending on site-specific conditions, and it is thus inappropriate to specify a single method for designing double-lane roundabouts. Regardless of the specific design method employed, the designer should maintain the overall design principles of speed control and speed consistency presented in **Section 8.11.6.2.2**.

One method to avoid path overlap on entry is to start with an inner entry curve that is curvilinearly tangential to the central island and then draw parallel alignments to determine the position of the outside edge of each entry lane. These curves can range from 30 to 60 m in urban environments and 40 to 80 m in rural environments. These curves should extend approximately 30 m to provide clear indication of the curvature to the driver. The designer should check the critical vehicle paths to ensure that speeds are sufficiently low and consistent between vehicle streams. The designer should also ensure that the portion of the splitter island in front of the crosswalk meets the recommendations for minimum size in **Section 8.9.2**. **Figure 8.94** demonstrates this method of design.

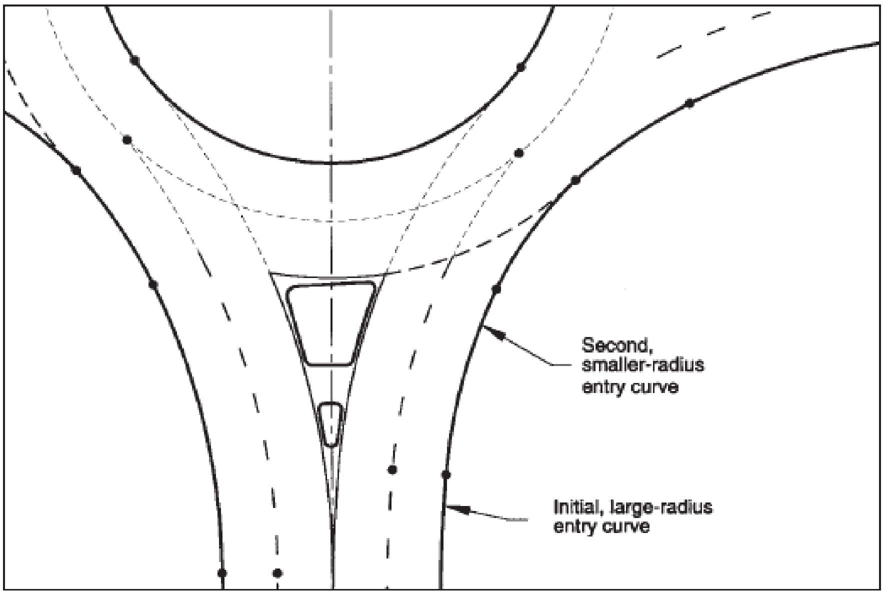


Figure 8.94 One method of entry design to avoid path overlap at double-lane roundabouts

Another method to reduce entry speeds and avoid path overlap is to use a small radius (generally 15 to 30 m) curve, approximately 10 to 15 m upstream of the yield line. A second, larger-radius curve (or even a tangent) is then fitted between the first curve and the edge of the circulatory roadway. In this way, vehicles will still be slowed by the small-radius approach curve, and they will be directed along a path that is tangential to the central island at the time they reach the yield line. **Figure 8.95** demonstrates this alternate method of design.

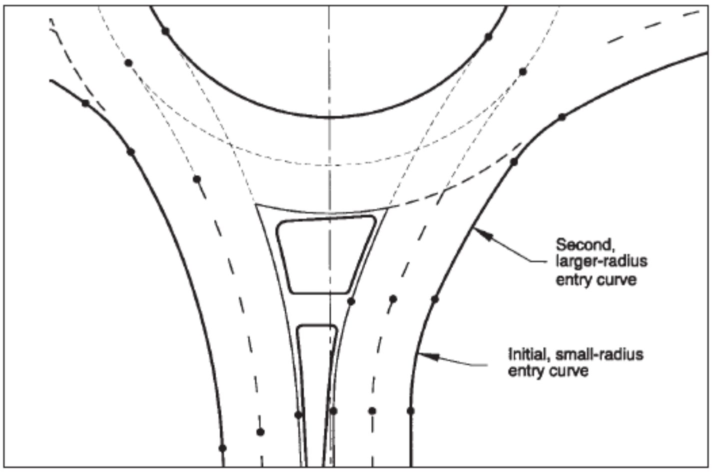


Figure 8.95 Alternate method of entry design to avoid path overlap at double-lane roundabouts

As in the case of single-lane roundabouts, it is a primary objective to ensure that the entry path radius along the fastest path is not substantially larger than the circulating path radius. Referring to **Figure 8.71,** it is desirable for R_1 to be less than or approximately equal to R_2. At double-lane roundabouts, however, R_1 should not be excessively small. If R_1 is too small, vehicle path overlap may result, reducing the operational efficiency and increasing potential for crashes. Values for R_1 in the range of 40 to 70 m are generally preferable. This results in a design speed of 35 to 45 km/h.

The entry path radius, R_1, is controlled by the offset between the right kerb line on the entry roadway and the kerb line of the central island (on the driver's left). If the initial layout produces an entry path radius above the preferred design speed, one way to reduce it is to gradually shift the approach to the left to increase the offset; however, this may increase adjacent exit speeds. Another method to reduce the entry path radius is to move the initial, small-radius entry curve closer to the circulatory roadway. This will decrease the length of the second, larger-radius curve and increase the deflection for entering traffic. However, care must be taken to ensure this adjustment does not produce overlapping natural paths.

ii. Exit curves

To avoid path overlap on the exit, it is important that the exit radius at a double-lane roundabout is not too small. At single-lane roundabouts, it is acceptable to use a minimal exit radius in order to control exit speeds and maximize pedestrian safety. However, the same is not necessarily true at double-lane roundabouts. If the exit radius is too small, traffic on the inside of the circulatory roadway will tend to exit into the outside exit lane on a more comfortable turning radius.

At double-lane roundabouts in urban environments, the principle for maximizing pedestrian safety is to reduce vehicle speeds prior to the yield and maintain similar (or slightly lower) speeds within the circulatory roadway. At the exit points, traffic will still be traveling slowly, as there is insufficient distance to accelerate significantly. If the entry and circulating path radii (R_1 and R_2, as shown on **Figure 8.71** are each 50 m, exit speeds will generally be below 40 km/h regardless of the exit radius.

To achieve exit speeds slower than 40 km/h, as is often desirable in environments with significant pedestrian activity, it may be necessary to tighten the exit radius. This may improve safety for pedestrians at the possible expense of increased vehicle-vehicle collisions.

8.11.6.2.7 RURAL ROUNDABOUTS

Roundabouts located on rural roads often have special design considerations because approach speeds are higher than urban or local roads and drivers generally do not expect to encounter speed interruptions. The primary safety concern in rural locations is to make drivers aware of the roundabout with ample distance to comfortably decelerate to the appropriate speed. This section provides design guidelines for providing additional speed-reduction measures on rural roundabout approaches.

A. VISIBILITY

Perhaps the most important element affecting safety at rural intersections is the visibility of the intersection itself. Roundabouts are no different from stop-controlled or signalized intersections in this respect except for the presence of kerbing along roadways that are typically not kerbed. Therefore, although the number and severity of multiple-vehicle collisions at roundabouts may decrease (as discussed previously), the number of single-vehicle crashes may increase. This potential can be minimized with attention to proper visibility of the roundabout and its approaches. Where possible, the geometric alignment of approach roadways should be constructed to maximize the visibility of the central island and the general shape of the roundabout. Where adequate visibility cannot be provided solely through geometric alignment, additional treatments (signing, pavement markings, advanced warning beacons, etc.) should be considered (refer to **Chapter 12**). Note that many of these treatments are similar to those that would be applied to rural stop-controlled or signalized intersections.

B. KERBING

On an open rural highway, changes in the roadway's cross-section can be an effective means to help approaching drivers recognize the need to reduce their speed. Rural highways typically have no outside kerbs with wide paved or gravel shoulders. Narrow shoulder widths and kerbs on the outside edges of pavement, on the other hand, generally give drivers a sense they are entering a more urbanized setting, causing them to naturally slow down. Thus, consideration should be given to reducing shoulder widths and introducing kerbs when installing a roundabout on an open rural highway.

Kerbs help to improve delineation and to prevent "corner cutting," which helps to ensure low speeds. In this way, kerbs help to confine vehicles to the intended design path. The designer should carefully consider all likely design vehicles, including farm equipment, when setting kerb locations. Limited research has been performed to date regarding the length of kerbing required in advance of a rural roundabout. In general, it may be desirable to extend the kerbing from the approach for at least the length of the required deceleration distance to the roundabout.

C. SPLITTER ISLANDS

Another effective cross-section treatment to reduce approach speeds is to use longer splitter islands on the approaches. Splitter islands should generally be extended upstream of the yield bar to the point at which entering drivers are expected to begin decelerating comfortably. A minimum length of 60 m is recommended. **Figure 8.96** provides a diagram of such a splitter island design. The length of the splitter island may differ depending upon the approach speed. The recommendations for required braking distance with an alert driver should be applied to determine the ideal splitter island length for rural roundabout approaches.

A further speed-reduction technique is the use of landscaping on the extended splitter island and roadside to create a "tunnel" effect. If such a technique is used, the stopping and intersection sight distance requirements (refer to **Section 8.11.6.2.5**) will dictate the maximum extent of such landscaping.

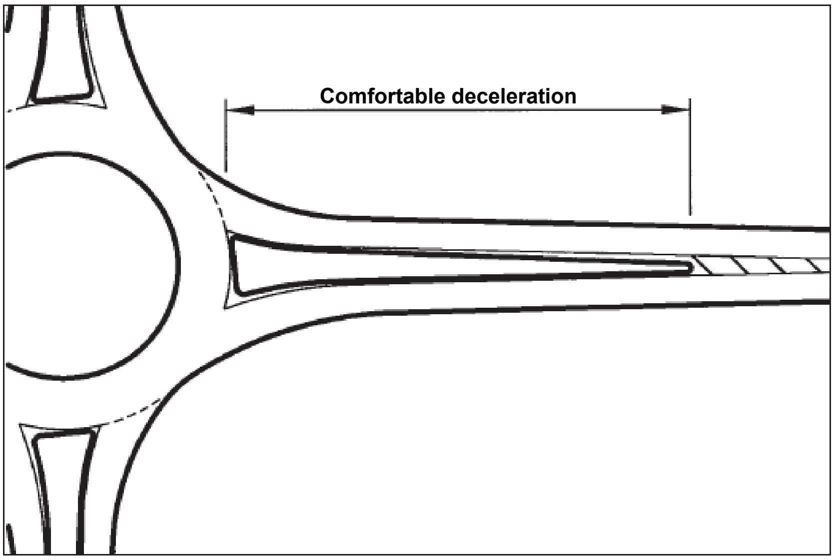


Figure 8.96 Extended splitter island treatment

D. APPROACH CURVES

Roundabouts on high-speed roads (speeds of 80 km/h or higher), despite extra signing efforts, may not be expected by approaching drivers, resulting in erratic behaviour and an increase in single-vehicle crashes. Good design encourages drivers to slow down before reaching the roundabout, and this can be most effectively achieved through a combination of geometric design and other design treatments. Where approach speeds are high, speed consistency on the approach needs to be addressed to avoid forcing all of the reduction in speed to be completed through the curvature at the roundabout.

The radius of an approach curve (and subsequent vehicular speeds) has a direct impact on the frequency of crashes at a roundabout (ref). On the other hand, decreasing the radius of an approach curve may increase the single-vehicle crash rate on the curve, particularly when the required side-friction for the vehicle to maintain its path is too high. This may encourage drivers to cut across lanes and increase sideswipe crash rates on the approach curve.

One method to achieve speed reduction that reduces crashes at the roundabout while minimizing single-vehicle crashes is the use of successive curves on approaches (ref). It is recommended that approach speeds immediately prior to the entry curves of the roundabout be limited to 60 km/h to minimize high-speed rear-end and entering-circulating vehicle crashes.

Figure 8.97 shows a typical rural roundabout design with a succession of three curves prior to the yield line. As shown in the figure, these approach curves should be successively smaller radii in order to minimize the reduction in design speed between successive curves. Studies have shown that shifting the approaching roadway laterally by 7m usually enables adequate curvature to be obtained while keeping the curve lengths to a minimum. If the lateral shift is too small, drivers are more likely to cut into the adjacent lane (ref).

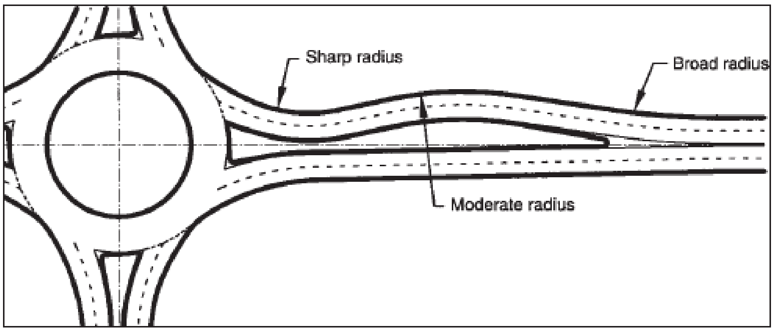


Figure 8.97 Use of successive curves on high speed approaches

Equations 8.13 and **8.14** can be used to estimate the operating speed of two-lane rural roads as a function of degree of curvature. **Equation 8.15** can be used similarly for four-lane rural roads.

Two-lane rural roads:

$$V_{85} = 103.66 - 1.95D, D \geq 3° \quad (8.13)$$

$$V_{85} = 97.9, D < 3° \quad (8.14)$$

where:

V_{85} = 85th-percentile speed, km/h; and

D = degree of curvature, degrees = 1746.38 / R

R = radius of curve, m

Four-lane rural roads:

$$V_{85} = 103.66 - 1.95D \quad (8.15)$$

where:

V_{85} = 85th-percentile speed, km/h; and

D = degree of curvature, degrees = 1746.38 / R

R = radius of curve, m

8.11.6.2.8 MINI-ROUNDABOUTS

A mini-roundabout is an intersection design alternative that can be used in place of stop control or signalization at physically constrained intersections to help improve safety problems and excessive delays at minor approaches. Mini-roundabouts are not traffic calming devices but rather are a form of roundabout intersection. **Figure 8.98** presents an example of a mini-roundabout.

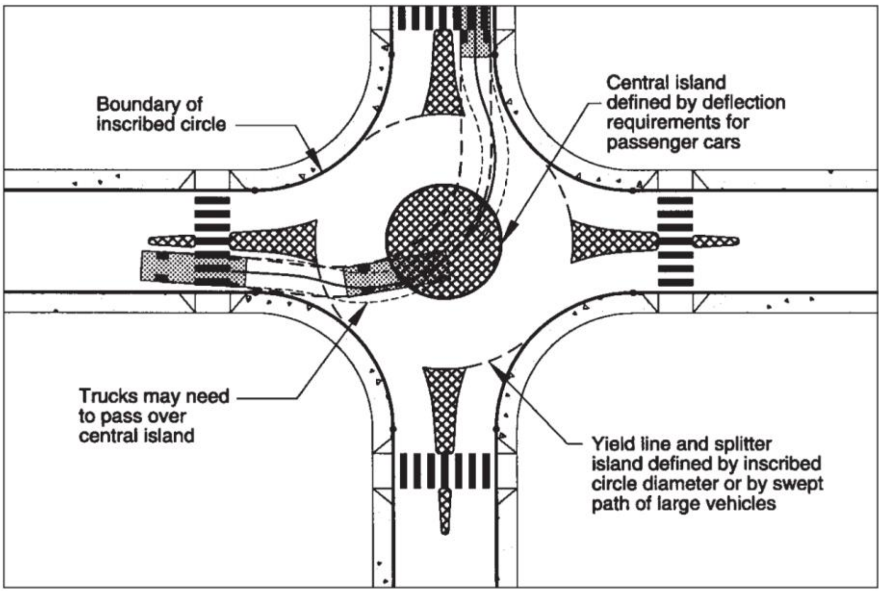


Figure 8.98 Example of a mini-roundabout

Mini-roundabouts should only be considered in areas where all approaching roadways have an 85th-percentile speed of less than 50 km/h. In addition, mini-roundabouts are not recommended in locations in which high U-turn traffic is expected, such as at the ends of road segments with access restrictions. Mini roundabouts are not well suited for high volumes of trucks, as trucks will occupy most of the intersection when turning.

The design of the central island of a mini-roundabout is defined primarily by the requirement to achieve speed reduction for small vehicles. As discussed previously in **Section 8.11.6.2**, speed reduction for entering vehicles and speed consistency with circulating vehicles are important. Therefore, the location and size of the central island are dictated by the inside of the swept paths of small vehicles that is needed to achieve a maximum recommended entry speed of 25 km/h. The central island of a mini-roundabout is typically a minimum of 4m in diameter and is fully mountable by large vehicles and trailers. Composed of asphalt, concrete, or other paving material, the central island should be domed at a height of 25 to 30 mm per 1 m diameter, with a maximum height of 125mm. Although fully mountable and relatively small, it is essential that the central island be clear and conspicuous.

The outer swept path of small vehicles and large vehicles is typically used to define the location of the yield line and boundary of each splitter island with the circulatory roadway. Given the small size of a mini-roundabout, the outer swept path of large vehicles may not be coincident with the inscribed circle of the roundabout, which is defined by the outer kerbs. Therefore, the splitter islands and yield line may extend into the inscribed circle for some approach geometries. On the other hand, for very small mini-roundabouts, such as the one shown in **Figure 8.98**, all turning trucks/trailers will pass directly over the central island while not encroaching on the circulating roadway to the left which may have opposing traffic. In these cases, the yield line and splitter island should be set coincident with the inscribed circle.

8.11.7 CLOSELY SPACED ROUNDABOUTS

It is sometimes desirable to consider the operation of two or more roundabouts in close proximity to each other. In these cases, the expected queue length at each roundabout becomes important. **Plate 8.11A** presents an example of closely spaced T-intersections. The engineer should compute the 95th-percentile queues for each approach to check that sufficient queuing space is provided for vehicles between the roundabouts. If there is insufficient space, then drivers will occasionally queue into the upstream roundabout and may cause it to lock.

Closely spaced roundabouts may improve safety by calming the traffic on the major road. Drivers may be reluctant to accelerate to the expected speed on the arterial if they are also required to slow again for the next close roundabout. This may benefit nearby residents. Roundabouts may also provide benefit for other closely spaced intersections. Short delay and queuing for vehicles at roundabouts allow for tighter spacing of intersections without providing a significant operational detriment to the other intersection, provided that adequate capacity is available at both intersections.

Plate 8.11B illustrates two closely spaced roundabouts at an interchange ramp and nearby frontage road. The two roundabouts work together as a system to effectively serve the traffic demands. Due care must be given to a system of roundabouts with this complexity to ensure that the design objectives are met, that each approach leg has sufficient capacity, and that the lane numbers and arrangements work together to allow a driver to intuitively navigate the intersection without lane changes or weaving.

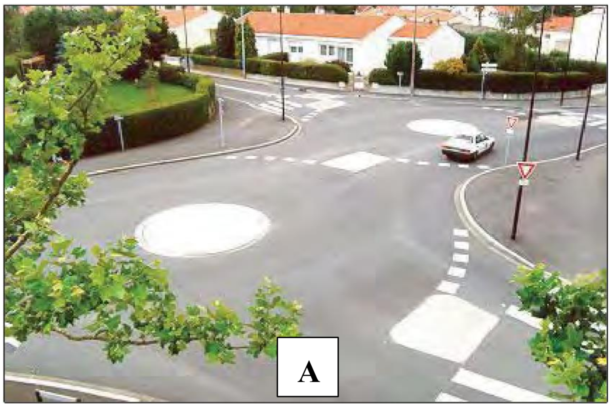


Plate 8.11 Examples of Closely Spaced Roundabouts

8.11.8 ACCESS MANAGEMENT

Access points near an intersection or along an arterial create additional conflicts within the roadway system that affect operations and safety. Managing access points can improve the overall effectiveness of the system by streamlining the roadway operations and reducing the number of conflicts. Roundabouts can provide a useful tool within an access management program to provide U-turn opportunities at the intersections, thereby allowing for a reduction of full access points along the roadway segment. However, within the vicinity of an individual roundabout intersection, property access must also be carefully evaluated.

Access management at roundabouts follows many of the principles used for access management at conventional intersections. For public and private access points near a roundabout, two scenarios commonly occur:

i. Access into the roundabout itself or
ii. Access near the roundabout.

8.11.8.1 ACCESS INTO THE ROUNDABOUT

It is preferable to avoid locating accesses where they must take direct access to a roundabout. Accesses introduce conflicts into the circulatory roadway, including acceleration and deceleration. Traditional access designs do not discourage wrong way movements as a splitter island does.

Nonetheless, site constraints sometimes make it necessary to consider providing direct access into a roundabout. **Plate 8.12** shows example where one or two residential houses have been provided direct access into a roundabout. These accesses have been designed with traditional concrete access aprons to provide a clear visual and tactile indication that these are private accesses not to be confused with public roadways.

For a driveway to be located where it takes direct access to the circulatory roadway of a roundabout, it should satisfy the following criteria:

i. No alternative access point is reasonable.
ii. Traffic volumes are sufficiently low to make the likelihood of errant vehicle behaviour minimal. Accesses carrying the trip generation associated with a very small number of single-family houses are typically acceptable; accesses with higher traffic volumes should be designed as a regular approach with a splitter island. In addition, if a high proportion of unfamiliar drivers are expected at the access, the engineer should consider providing more positive guidance.
iii. The access design should enable vehicles to exit facing forward with a hammerhead design or other area on-site where vehicles can turn around. Driveways that only allow backing manoeuvres into the roundabout should be discouraged in all but very low-volume environments.
iv. The access design should enable proper intersection sight distance from the access location and adequate stopping sight distance for vehicles approaching the access traveling along the primary roadway.

Plate 8.12 Example of residential access into circulatory roadway

8.11.8.2 ACCESS NEAR THE ROUNDABOUT

Public and private access points near a roundabout often have restricted operations due to the channelization of the roundabout. Access between the crosswalk and entrance line complicate the pedestrian ramp treatments and introduce conflicts in an area critical to operations of the roundabout. **Plate 8.13** shows examples of access challenges of this type. Accesses blocked by the splitter island will be restricted to right-in/right-out operation and are best avoided altogether unless the impact is expected to be minimal and/or no reasonable alternatives are available.

The ability to provide an access point that allows all ingress and egress movements (hereafter referred to as full access) is governed by a number of factors:

i. The capacity of the minor movements at the access point. A standard unsignalized intersection capacity analysis should be performed to assess the operational effectiveness of an access point with full access. Unlike the platooned flow typically downstream of a signalized intersection, traffic passing in front of an access point downstream of a roundabout will be more randomly distributed. As a result, an access point downstream of a roundabout may have less capacity and higher delay than one downstream of a traffic signal. Queuing from nearby intersections (the roundabout or others nearby) should be checked to see if the operation of the access point will be affected.

ii. The need to provide left-turn storage on the major street to serve the access point. For all but low-volume accesses it is often desirable to provide separate left-turn storage for access points downstream of a roundabout to minimize the likelihood that a left-turning vehicle will block the major street traffic flow. If quantification is desired, a probability analysis can be used to determine the likelihood of an impeding left-turning vehicle, and a queuing analysis can be used to determine the length of the queue behind the impeding left-turning vehicle. If the number of left-turning vehicles is sufficiently small and/or the distance between the access point and the roundabout is sufficiently large, a left-turn pocket may not be necessary.

iii. The available space between the access point and the roundabout. **Figure 8.99** presents a figure showing typical dimensions associated with a roundabout and left-turn storage for a downstream minor street. As the figure demonstrates, a minimum distance is required to provide adequate roundabout splitter island design and left-turn pocket channelization. In addition, access is restricted along the entire length of the splitter island and left-turn pocket channelization.

iv. Sight distance needs. A driver at the access point should have proper intersection sight distance and should be visible when approaching or departing the roundabout, as applicable.

(a) Access between crosswalk and roundabout (b) Access aligned with crosswalk

Plate 8.13 Example of Access Challenges near Roundabout

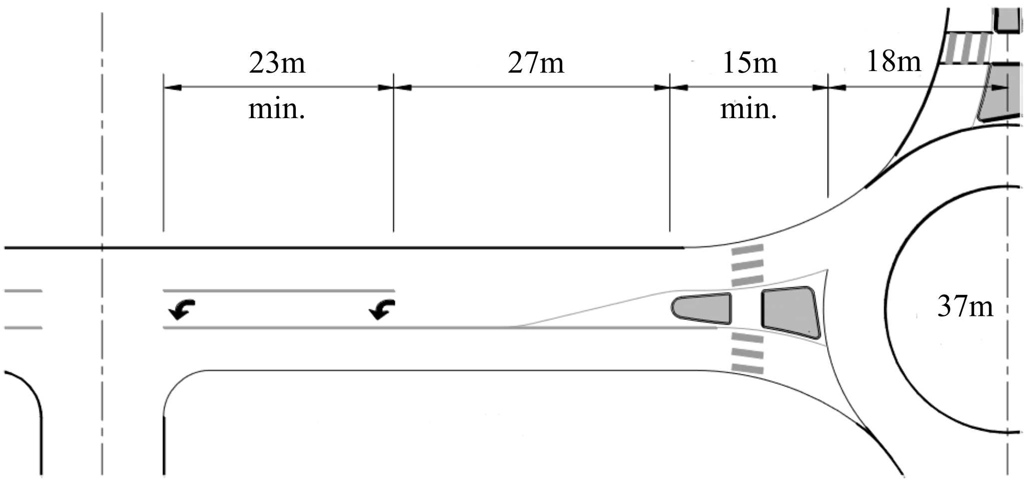


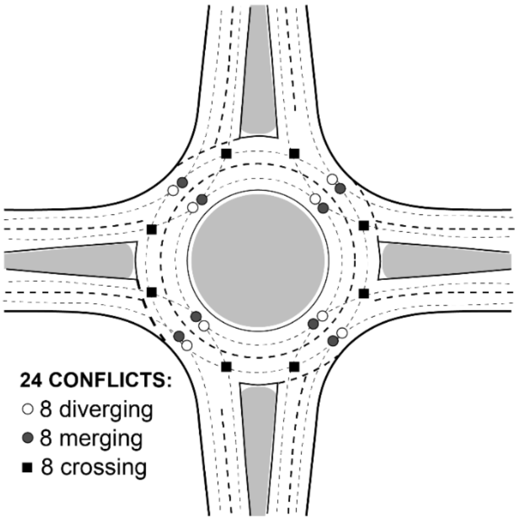
Figure 8.99 Typical dimensions for left-turn access near roundabouts

8.11.9 TURBO ROUNDABOUTS

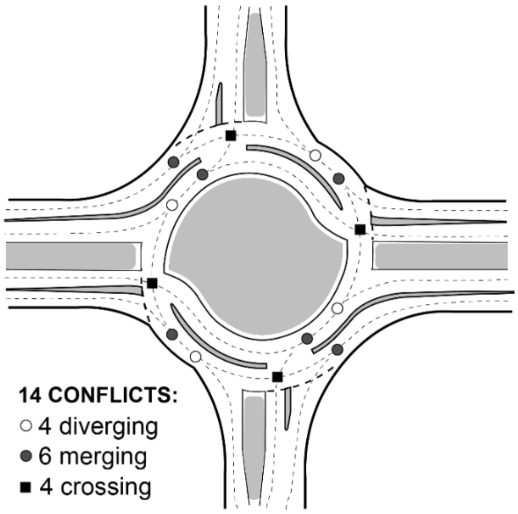
Turbo roundabouts are multilane roundabouts where vehicles are required to enter in specific lanes
depending on which exit they wish to take. Raised line markings can be used to further discourage lane changing and to encourage lower speeds (**Plate 8.14**).

Turbo roundabouts were developed to improve the capacity of two-lane roundabouts. The conventional two-lane roundabouts were found to not be adequate when there were equal flows, i.e. a dominant flow in one direction, and drivers were undertaking weaving manoeuvres within the circulating lanes to change lanes, which was impacting on the capacity of the roundabout. The effect of converting existing roundabouts into turbo roundabouts was a radical improvement in driver lane discipline and subsequent reductions in crashes.

Case studies indicate that turbo roundabouts result in reduced speeds, up to a 70% reduction in crashes, a reduction in the number of conflicts (see **Figure 8.100**) due to elimination of weaving traffic within the roundabout. In addition, the geometric characteristics of the turbo roundabout result in operational outcomes that should help address lane selection, lane changing, and entering and exiting behaviors that can lead to the lower severity, multiple-vehicle crashes in 2 x 2 multilane roundabouts. In general, capacity of a turbo roundabout will be similar to a conventional multilane roundabout (FHWA 2019), but an increase of up to 25–35% may occur depending on the balance of traffic volumes on the approaches (Engelsman & Uken 2007).



A. Conflict point frequency for modern multilane roundabout.



B. Conflict point frequency for turbo roundabout.

Figure 8.100 Conflict point frequency for modern multilane and turbo roundabouts

Plate 8.14 Example of a turbo roundabout

8.11.9.1 APPLICABILITY AND IMPLEMENTATION ISSUES

i. Generally applied at high-capacity and high-speed intersections on high-order arterial roads, with traffic volumes of up to 35,000 vehicles per day.

ii. Should not be used on high-cyclist-volume roads. If applied on these roads, cyclist lanes should be considered.

iii. Requires clear line/lane marking for circulating vehicles.

iv. Consideration of the likely maximum design vehicle is needed as longer vehicles may have difficulty navigating around the roundabout.

v. Possible drainage issues at separator islands.

vi. Additional signage on approaches, painted destination pavement markings and lane markings with directional arrows may be required for navigation, appropriate lane selection and to minimise the risk of late lane changes.

8.11.9.2 CHARACTERISTICS OF A TURBO ROUNDABOUT

Features (illustrated in **Plate 8.15**) that characterize turbo roundabouts include the following:

i. A second circulatory lane is inserted opposite of at least one entry lane.

ii. Traffic approaching the roundabout on at least one leg must yield to traffic in two, and no more than two, circulatory lanes in the roundabout.

iii. Smooth flow is encouraged by a spiral alignment.

iv. Lane dividers discourage lane changing within the roundabout. Drivers, therefore, select the proper lane prior to entering the roundabout. Internationally, options for lane separation have included raised, mountable lane dividers; flush lane dividers; or solid pavement markings.

v. Each segment of the roundabout includes one circulatory lane from which drivers can choose whether to exit or continue around the roundabout.

vi. At least two exit legs are two-lane.

vii. The diameter of the roundabout is kept small to encourage lower speeds through the roundabout.

viii. Approach legs and entry are typically at right angles to the roundabout.

ix. Roundabout directional arrow signs direct drivers and increase conspicuity of the central island.

x. Mountable aprons offer sufficient manoeuvring space for longer vehicles.

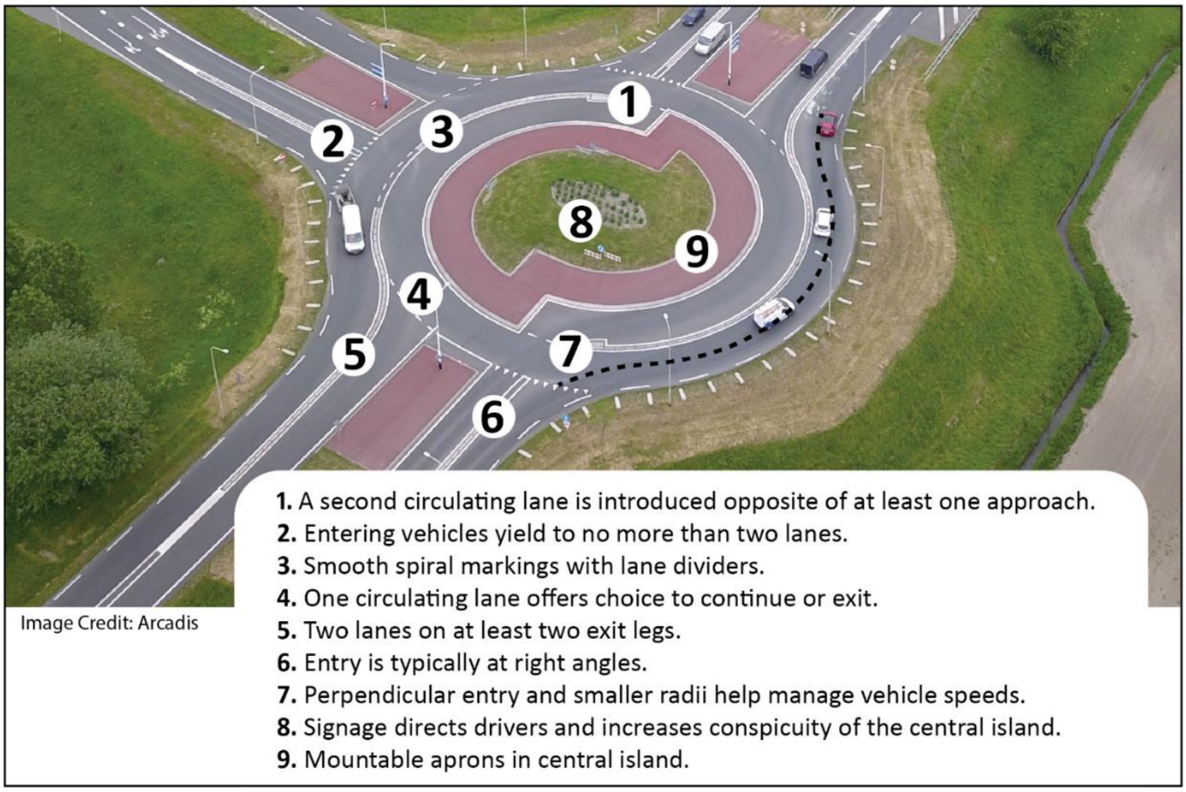


Plate 8.15 Turbo roundabout features. Image based on Fortuijn, 2009

There are different types of turbo roundabouts, including the basic, egg, knee, spiral, and rotor turbo roundabouts. These options differ with respect to central island design, number of circulating lanes, and number of approach lanes, as described below:

A. **Basic** – inside lane added on major approaches, two lanes on each approach (see **Figure 8.101A**).

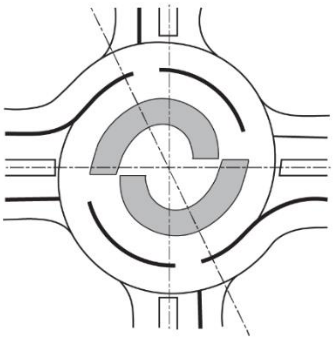
B. **Egg** – similar to a basic turbo roundabout, but with only one approach lane on minor approaches (see **Figure 8.101B**).

C. **Knee** – the inside lane is only added on one approach, two lanes on each approach (see **Figure 8.101C**).

D. **Spiral** – three circulatory lanes, inside lane only added on two approaches, two approaches with three lanes and two approaches with two lanes (see **Figure 8.101D**).

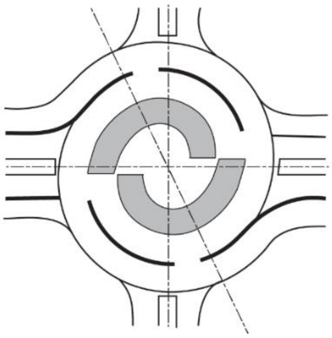
E. **Rotor** – three circulatory lanes, inside lane added on each approach, three lanes on each approach (see **Figure 8.101E**).

The variations in turbo roundabout designs differ in terms of total capacity available, so the type selected may be dictated by intersection demand. The capacity values provided in **Figure 8.101** represent capacity in the Netherlands and are not necessarily reflective of expected capacity values elsewhere.



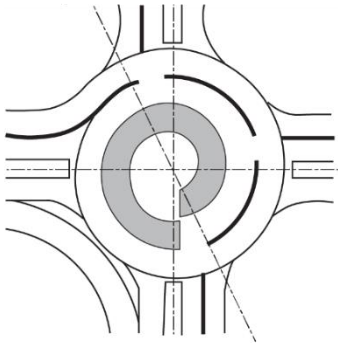
Capacity: 3500pcu/h

A. BASIC



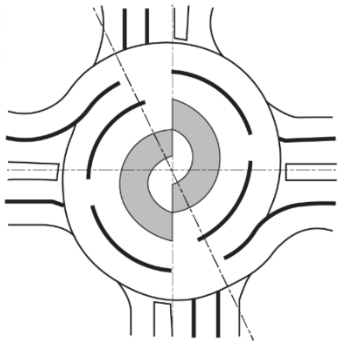
Capacity: 2800pcu/h

B. EGG



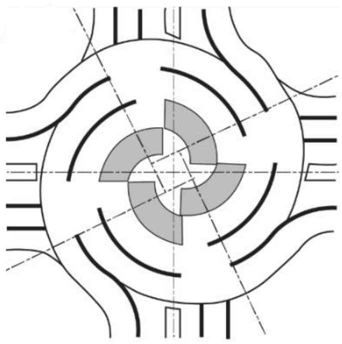
Capacity: 3500pcu/h

C. KNEE



Capacity: 4000pcu/h

D. SPIRAL



Capacity: 4500pcu/h

E. ROTOR

Figure 8.101 Basic types of turbo roundabouts

8.11.9.3 USER CONSIDERATIONS

It is important to consider how various user groups are accommodated at turbo roundabouts given the intersection type's key features. Five primary user groups – motorists, pedestrians, bicyclists, motorcyclists, and freight/large vehicles – are discussed in this section.

A. Motorists

Turbo roundabouts rely on more direct entry geometry and enhanced delineation of lanes that can make it easier for motorists to successfully navigate them. Signage and supplemental pavement markings are provided in advance on approaches so drivers are given enough time to select their desired lane. When locating signs, designers should consider decision and stopping sight distance as well as potential queue lengths to provide drivers with adequate advance notice. At the entrance to the roundabout, drivers are required to identify acceptable gaps in no more than two conflicting lanes. A roundabout directional arrow sign placed directly in the drivers' field of view directs drivers to enter the circulatory roadway in the appropriate direction.

Internationally, these signs are also recommended to increase the conspicuity of the central island and communicate to drivers the need to slow and turn into the roundabout. Landscaping can also be used to increase central island conspicuity. Finally, the spiral geometry and enhanced delineation reinforce the appropriate manoeuvres from each lane inside the roundabout.

One notable difference between turbo roundabouts and other modern roundabouts is the ability to complete U-turns. Modern roundabouts allow vehicles from all approaches to complete U-turn manoeuvres. The lane arrangement of a turbo roundabout prohibits vehicles that enter on some approaches from completing U-turns. The approaches and lanes from which vehicles can and cannot perform U-turns vary based on the type of turbo roundabout. For instance, vehicles entering from the inside lane of the left and right approaches (the major road approaches) in **Figure 8.100B** (a "Basic" turbo roundabout) can complete a U-turn; while vehicles approaching from the top and bottom approaches (the minor road approaches) cannot. As a result, it is important for analysts to consider the frequency of U-turn manoeuvres at an intersection when evaluating turbo roundabouts as a potential alternative.

B. Pedestrians

The navigation through a turbo roundabout by a pedestrian does not differ from single-lane and multilane roundabouts. As a result, designers can follow the following guidance for pedestrian facilities at roundabouts:

i. Keep walkways along the perimeter of the roundabout, separated from the edge of the circulatory roadway with a landscaped strip or buffer.

ii. Where crosswalks are provided, locate them for pedestrian convenience and safety, where drivers can be expected to yield the right-of-way, and where the crossing will be less likely to be blocked by queued vehicles.

iii. Provide a splitter island sufficiently wide to accommodate a crossing that is accessible to pedestrians with disabilities as well as wide enough for comfortable queueing.

C. Bicyclists

The decision of whether to provide separated bicycle facilities at turbo roundabouts depends on context, considering factors such as bicycle volume, the presence of existing bicycle facilities, motor vehicle volume, complexity of the roundabout, adjacent infrastructure and land use, and right-of-way availability. Bicycle features at turbo roundabouts are not expected to differ from traditional roundabouts, and features designers can consider to better accommodate bicyclists include:

i. Keeping radii small to reduce vehicle speeds, which can make bicyclists more comfortable if they ride in the roundabout.
ii. Terminating bicycle lanes before the edge of the circulatory roadway and crosswalks with enough length remaining for bicyclists to merge into traffic.
iii. Introducing bicycle lanes on exit legs downstream of crosswalks.
iv. If bicyclists are required to utilize the sidewalk, designing sidewalks to meet shared use path width requirements.
v. If the intent is for bicyclists to cross at-grade on approaches, whether on a designated crossing or on a pedestrian crosswalk, a pavement-level cut-through of the splitter island can be provided. The cut-through can be designed to include a chicane to encourage a two-stage crossing for bicyclists and provide more time for approaching drivers to identify crossing bicyclists. This is a commonly used treatment in the Netherlands.

D. Motorcyclists

While fatal crashes at roundabouts are much less likely than at traditional three- and four-leg intersections, motorcyclists are overrepresented in those fatal crashes. Roadway features that can have a significant impact on motorcycle safety performance at roundabouts include the presence and location of raised lane dividers and kerbing, surface friction, pavement markings, drainage, sight distance (especially rider conspicuity), radii, the roadside environment, and surface conditions. Specific concerns for motorcyclists in turbo roundabouts are the truck apron and lane divider options that are raised.

Sloped kerbing with minimal vertical reveal can provide a more forgiving environment to motorcycles compared to vertical or rolled curbing. Designers can also provide supplemental signage alerting motorcyclists to these elements of turbo roundabouts. Potential alternatives to the raised lane dividers include striping and colorized and/or textured pavement,

E. Freight/Large Vehicles

The design of some turbo roundabout features is influenced by the physical dimensions and turning characteristics of the larger vehicles that will use the intersection. The lane widths of turbo roundabouts are determined with consideration of the design vehicle, typically the largest vehicle anticipated to regularly navigate the intersection.

Starting the lane divider of a turbo roundabout as near as possible to the vehicle entry point is necessary to prevent vehicles circulating in the outside lane from changing to the inside lane at these locations. However, large vehicles entering the inside lane from an approach need a wider opening to account for their larger swept paths. Where a raised lane divider option is used, a traversable, demarcating feature can be provided at the origin of the raised divider to ease the entrance of larger vehicles.

A central truck apron is provided in turbo roundabouts to help accommodate larger vehicles that need to navigate the intersection. Aprons can also be provided on the perimeter of the roundabout to provide additional turning space for large vehicles.

8.11.9.4 LOCATION CONSIDERATIONS

Modern roundabouts can be among the safest feasible intersection alternatives in a wide variety of settings and contexts – low-speed urban, high-speed rural, at isolated intersections, as corridor treatments, and even at interchange ramp terminal intersections. Relevant site characteristics that can influence whether a roundabout is a feasible alternative include right-of-way limitations, intersection skew, adjacent traffic generators or sites that require pre-emption, and downstream bottlenecks.

Turbo roundabouts may be considered at any intersection where a roundabout is a potential alternative, particularly where traffic demand indicates the need for a multilane roundabout. Their design provides similar capacity to multilane roundabouts while reducing conflict points, discouraging lane changes, and maintaining the speed reduction characteristics of single-lane roundabouts.

8.11.9.5 OPERATIONAL ANALYSIS

For a turbo roundabout to be successful, it is important to verify the design can accommodate the projected traffic volumes at the intersection. At modern multilane roundabouts, the capacity of one entry lane ranges from 300 to 1,100 passenger cars per hour (pc/h), depending on conflicting flow in the circulatory roadway, implying a total approach capacity ranging from approximately 600 to 2,200 pc/h for a two-lane approach. As with modern roundabouts, turbo roundabout capacity is measured at the approach level. Operational performance models for turbo roundabouts have not yet been developed for, or adapted to, the context of Ghanaian driving population. International research suggests basic turbo roundabouts have similar capacities as multilane roundabouts with two entry and two circulating lanes. One such study from the Netherlands estimated a capacity for a basic turbo roundabout design of approximately 3,500 pc/h for all entries combined, assuming conflicting traffic volumes between 1,900 and 2,100 pc/h.

Gap-acceptance models that consider critical headway, critical follow-up time, and conflicting traffic appear adequate for estimating turbo roundabout capacity. Research in Poland found the Highway Capacity Manual (HCM) capacity models for roundabouts produced capacity estimates for Polish turbo roundabouts that were comparable to estimates from Polish-specific turbo roundabout capacity models. The roundabout capacity models of the HCM are likely to represent reasonable capacity estimates for turbo roundabout approaches with up to two lanes. As with single and multilane roundabouts, analysts would apply the HCM models to each lane of each approach, given the specific characteristics of the lane and approach (e.g., number of entry lanes, number of conflicting lanes, conflicting flow).

8.11.9.6 DESIGN CONSIDERATIONS

The geometric design of a turbo roundabout is driven by the desired capacity and the desired characteristics of a design vehicle's horizontal swept path. The projected demand and cross sections on the approach roadways inform the number of lanes/lane arrangement decisions, which dictate the type of turbo roundabout to be built. Once the type is selected, a horizontal swept path analysis of the design vehicle informs lane width decisions along with other lane width-related considerations (e.g., right-of-way, performance for all vehicle types and users). The turbo roundabout type and lane widths are combined to construct the turbo block, which guides the geometric design of the circulatory roadway.

8.11.9.6.1 HORIZONTAL DESIGN

A. Turbo Block

The spiral alignment of a turbo roundabout is generated from the "turbo block," a series of circular arcs with centres located at various points along a reference line known as a "translation axis." The turbo block consists of arcs that represent the inner and outer edges of each lane.

The inner radius of the turbo block, which represents the radius of the central island, is selected based on the anticipated size of the turbo roundabout. The shift along the translation axis from the centre is the width of the lane represented by the arc. The turbo block and angle of the translation axis differs for each turbo roundabout type. **Figure 8.102** is a sample turbo block for a basic turbo roundabout with the major roadway oriented horizontally.

The turbo block is defined by the characteristics shown in **Figure 8.102**. First is the centre point (CG), which is the intersection of the approach centrelines. Second is the orientation of the translation axis, which is defined in relation to the major road approaches. Assuming the major road is oriented with the x-axis in **Figure 8.102**, the right side of the translation axis is rotated 57.5 degrees around the centre below the x-axis for a four-leg intersection, and the left side of the translation axis is rotated 65 degrees around the centre below the x-axis for a three-leg intersection. The angle of rotation for the translation axis can be tweaked to provide smooth, spiraled vehicle paths for all vehicle movements. Third are the radii of the circles (TR1, TR2, TR3, and TR4). TR1 defines the radius of the inside edge of the inside roadway. TR2 defines the outside edge of the inside roadway; with the difference between TR2 and TR1 equal to the width of the inside travel lane plus additional width for the edge lines delineating the raised lane divider. TR3 defines the inside edge of the outside roadway. The difference between TR2 and TR3 is the width of the lane divider. TR4 defines the outside edge of the outside roadway.

The fourth key set of dimensions defining the turbo block is the distances between the centre points of the arcs. The circles corresponding to the four radii are split along the translation axis, and the resulting arcs are slid along the translation axis in opposing directions by half the distance defined as the shift. The shift is the distance between the centres of the arcs. The shift can differ for the TR1 centres and the TR2/3/4 centres if the inside roadway width is different than the outside roadway width. The shift for the TR1 centers ($\Delta\upsilon$ in **Figure 8.102**) is equal to the difference between the inside edge of the inside roadway and the inside edge of the outside roadway (also the difference between the values used for TR3 and TR1). The shift for the TR1 centres is achieved by sliding the two arcs defined by TR1 in opposing directions away from

CG, each by Δυ/2. In international practice, Δυ/2 ranges from between 2.5 and 3.0 m (for total shifts ranging between 5.0 and 6.0 m), as shown in **Figure 8.102**. The shift for the TR2/3/4 centres (Δu in **Figure 8.102**) is the distance between the outside edge of the inside roadway and the outside edge of the outside roadway (also the difference between the values used for TR4 and TR2).

The shift for the TR2/3/4 centres is achieved by sliding the arcs defined by TR2/3/4 in opposing directions away from CG by Δu/2, as shown in figure 10. This value (Δu/2) typically ranges from between 2.3 and 2.6 m (for a total shift of 4.5 to 5.0 m). If the inside and outside roadways are the same width, the shift value for all radii are the same (Δυ = Δu).

Internationally, the radii (TR1, TR2, TR3, and TR4) for basic turbo roundabouts have ranges as follows:

• 10 to 20 m for TR1.

• 15 to 25 m for TR2.

• 16 to 25 m for TR3.

• 21 to 30 m for TR4.

With the offset arcs making up the turbo roundabout, the nominal diameter of the turbo roundabout is twice the value TR4 plus the width of the TR2/3/4 shift, Δu. Assuming a shift of 4.5m, the inscribed circle for basic turbo roundabouts ranges from 47 to 66 m.

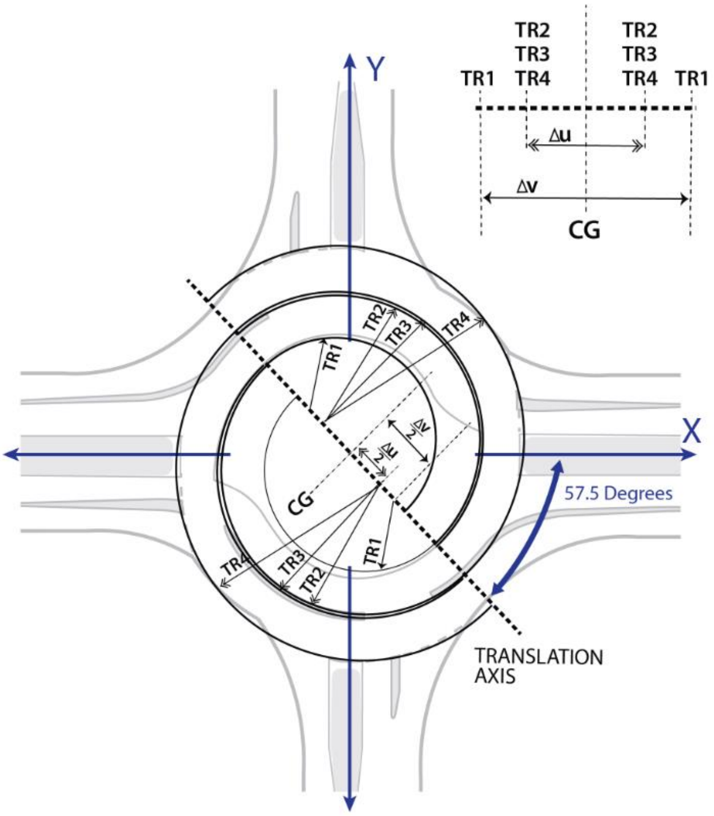


Figure 8.102 Sample turbo block

B. Lane and Roadway Width

Determining the width of each lane of a turbo roundabout is informed by a horizontal swept path analysis of the design vehicle. The inside lane is often wider than the outside lane to compensate for the design vehicle manoeuvring a smaller radius. Internationally, inside lane width ranges from between 4.3 and 4.9 m, while outside lane width ranges from between 4.0 and 4.4 m. The inner roadway width, defined as the distance from the central island to the lane divider (TR2 minus TR1), including the inside and outside edge line pavement markings ranges from between 16 and 18 feet. The outer roadway width, defined as the distance from the

lane divider to the outer edge of the roundabout (TR4 minus TR3), again including the inside

and outside edge line pavement markings ranges from between 4.6 and 5.0 m.

C. Central Island

The central island is defined by the innermost radius of the turbo block (TR1) and consists of a traversable portion (mountable apron) and a non-traversable portion. The non-traversable portion is typically used for signage, specifically a roundabout directional arrow sign. There are cutouts in the central island to introduce the inside lane of the turbo roundabout on the applicable approaches. There are two developed methods for design of these cutouts and beginning the inner lane.

i. A curved entry, shown in **Plate 8.16**, provides a smooth path for approaching vehicles, but may result in a greater chance of circulating vehicles entering the inside lane.

ii. A flat entry, shown in **Plate 8.17**, helps to discourage this movement from circulating vehicles.

Designers should check that objects on the central island do not restrict sight distance along the circulatory roadway.

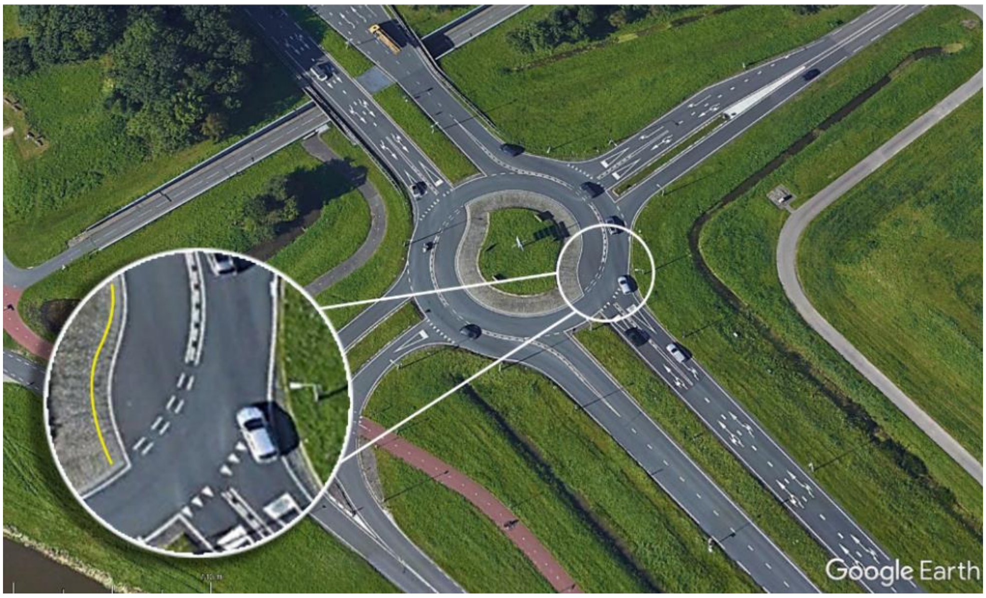


Plate 8.16 Original design used in the Netherlands for introducing the inner lane

Plate 8.17 Revised design used in the Netherlands for introducing the inner lane

D. Lane Divider

One important feature of the turbo roundabout is a lane divider between each circulating lane. In the Netherlands, this lane divider is raised but mountable, designed with little vertical profile and a rather flat slope to provide forgiveness for errant vehicles (as shown in **Plate 8.18**). Often, the raised lane divider is introduced with a traversable, demarcating feature to allow tracking by large vehicles (see **Plate 8.19**). Some countries (including Poland, Germany, and Canada) have implemented turbo roundabouts without raised lane dividers, in part due to possible challenges these dividers present to motorcyclists. Alternatives to the raised lane divider include striping and colorized or textured pavement, as shown in **Plate 8.20** from a turbo roundabout in Canada. While these options do not provide a physical barrier to lane changing, they still communicate this message to the driver both visually, and in the case of textured pavement, through audible and tactile mediums. Other alternatives to consider include:

i. Milled rumble strips or rumble stripes, which provide more intense feedback to drivers than textured pavement.

ii. A double solid white lane, which is described as a standard approach when crossing the lane lines are prohibited.

iii. Raised pavement markers which can provide visual and tactile feedback.

Plate 8.18 Raised lane divider in a turbo roundabout in the Netherlands.

Plate 8.19 Example introduction of the raised lane divider.

Plate 8.20 Lane divider for turbo roundabout at Victoria International Airport.

E. Approach Geometry

Turbo roundabouts are constructed with radial approaches, which have the benefit of reducing changes to the alignment along the approach roadway and maintaining exit curvature that encourages drivers to maintain slower speeds through the exit of the roundabout. Additionally, turbo roundabouts are built with little or no flare or deflection and smaller entry radii. The angle between entering traffic and circulating traffic is therefore larger (closer to a perpendicular entry) for a turbo roundabout than for other modern multilane roundabouts. These approach features differ from modern multilane roundabouts, which typically include flare to gain some capacity increase and deflection to align entering vehicles "to the right of" the central island in the desired direction of travel. The entry geometry of a turbo roundabout generally does not channelize drivers into the circulatory roadway to the right of the central island and the splitter islands generally do not have enough curvature to block a direct path of approaching vehicles to the central island. This approach geometry is based on the premise that it will be clearer to drivers that they are approaching an intersection that should be negotiated at lower speeds. Potential disadvantages include drivers errantly hitting the central island, making wrong-way left turn manoeuvres to enter the roundabout, and making wrong-way exit manoeuvres into entrance approach lanes. International literature emphasizes the importance of a roundabout directional arrow sign, placed in the central island in the line of sight of approaching drivers, that directs drivers to turn right and increases the conspicuity of the central island. It also emphasizes the need for a forgiving design of the central island and sign in the case that either is struck.

Internationally, turbo roundabout entry radii range from 12 to 15 m. For comparison, modern multilane roundabouts are designed with entry radii exceeding 20m, and even single-lane roundabouts have entry radii ranging from 15 to 30 m.

8.11.9.7 SIGHT DISTANCE AND VISIBILITY

Adequate stopping and decision sight distance should be provided for all users of the turbo roundabout. Stopping and intersection sight distance should be provided at all approaches.

8.11.9.8 SIGNAGE AND PAVEMENT MARKINGS

There are a few differences in the traffic control devices within the circulatory roadway of turbo roundabouts compared to modern multilane roundabouts. For modern multilane roundabouts, lanes are separated using either a single dashed or solid white line. As discussed earlier, these are replaced with lane dividers in turbo roundabouts. Potential advantages of the lane divider compared to single dashed or solid white lines include less ambiguous and more intuitive messaging to drivers on lane selection, lane keeping, and the appropriate manoeuvres from each lane. Given the operational characteristic of prohibited lane changes within the circulatory roadway of a turbo roundabout, signage and pavement markings on the approaches, especially for lane selection, are critical for motorists to identify and select their desired lane before entering the roundabout. Signage can also direct pedestrians and bicyclists to designated facilities, drivers to their desired lanes, and communicate the presence of raised curbing, such as a raised lane divider (if one is used). Pavement markings shall be used to delineate the edges of the approach and circulatory lanes. Additionally, supplemental delineation can be achieved using reflectors or Light Emitting Diodes (LEDs) to illuminate the edges of the apron and lane dividers. Finally, given the important role signage and pavement markings play for all users of turbo roundabouts, it is important that all traffic control devices are compliant with the MRH specifications.

8.11.9.9 PEDESTRIAN DESIGN TREATMENTS

Pedestrian accommodations for turbo roundabouts do not differ from modern roundabouts. Crossings should be kept at the perimeter of the intersection, with crosswalks and splitter islands on the approaches to provide two stage crossings. All walkways, crosswalks, and kerb ramps should be accessible to and usable by pedestrians with disabilities. The crosswalk should be placed far enough (minimum of 6 m, or one vehicle-length) from the circulatory roadway so a motorist can exit the roundabout and then stop before reaching any potential pedestrians in the crosswalk.

8.11.9.10 BICYCLE DESIGN TREATMENTS

Bicycle guidance for turbo roundabouts is the same as for modern roundabouts. A bicyclist can either mix with motor vehicle traffic or, when available, utilize separated facilities. The decision as to which treatment is adopted is based on context, weighing factors such as bicyclist volume, motor vehicle volume, complexity of the roundabout, adjacent infrastructure and land use, and available right-of-way. In the Netherlands, separate bicycle paths outside of the roundabout are recommended where possible, including for turbo roundabouts. Dutch guidance recommends adding kerb cuts with chicanes in splitter islands for bicycle crossings (**Plate 8.21**). The kerb cuts encourage bicyclists to use the crossing, while the chicane encourages the crossing to be taken in two stages.

Plate 8.21 Example of a chicane in a splitter island at a turbo roundabout in the Netherlands

8.11.9.11 VERTICAL DESIGN

Vertical alignment considerations are the same as other modern roundabouts. The geometry should not restrict sight distance throughout the intersection area, including decision sight distance on the approaches when selecting lanes, stopping sight distance on the approach and on the circulatory roadway, and intersection sight distance at the entrances to the circulatory roadway.

8.11.9.12 LIGHTING

The use of proper lighting is encouraged to improve the visibility of the middle island and raised lane divider. Lighting should also be provided to give adequate visibility for pedestrian and bicycle facilities, especially crossings, though it is important that designers are careful to avoid creating negative contrast lighting and shadowing.

8.11.9.13 OTHER DESIGN CONSIDERATIONS

Other design considerations, such as bypass lanes, access management, at-grade rail crossings, evacuation routes, and bus stops, should be addressed the same as they are for modern roundabouts.

8.11.10 LANDSCAPING

Landscaping is one of the distinguishing features that give roundabouts an aesthetic advantage over traditional intersections. Landscaping in the central island, splitter islands (where appropriate), and along the approaches can benefit both public safety and community enhancement. In addition to landscaping, some local assemblies use the central island of a roundabout as an opportunity to display local art or other gateway features. To determine the type and quantity of landscaping or other material to incorporate into a roundabout design, maintenance, sight distance, and the available planting zones should all be considered. The primary objectives and considerations of incorporating landscaping or art into a roundabout design are to:

i. Make the central island more conspicuous, thus improving safety;
ii. Improve the aesthetics of the area while complementing surrounding streetscapes as much as possible;
iii. Make decisions regarding placement of fixed objects (e.g., trees, poles, walls, guide rail, statues, or large rocks) that are sensitive to the speed environment in which the roundabout is located;
iv. Avoid obscuring the form of the roundabout or the signing to the driver;
v. Maintain adequate sight distances;
vi. Clearly indicate to drivers that they cannot pass straight through the intersection;
vii. Discourage pedestrian traffic through the central island; and
viii. Help pedestrians who are visually impaired locate walkways and crosswalks.

Landscaping should be limited to the non-traversable portion of the central island and splitter islands and not hinder stopping sight distance around the circulatory roadway. If sprinklers are used to maintain landscaping, designers should consider the impacts of irrigation runoff onto the circular roadway, as unexpected wet pavement can introduce another potential risk to users of the intersection.

8.11.10.1 PRINCIPLES

Landscaping should be designed to ensure that vehicles can observe the signing and shape of the roundabout as they approach and have adequate visibility for making decisions within the roundabout. The sight distance requirements at the roundabout dictate the size and types of landscaping materials appropriate for the various areas within and adjacent to the roundabout. Landscaping within the critical visibility areas must be limited to a height of 0.6 m to ensure adequate sight distance. The appropriate planting zones within a roundabout and the types of landscaping for each zone are described below.

The overall speed environment of the roadway is another important consideration when selecting plant material and other landscape features. Within lower-speed urban environments [typically 55 km/h or less], there is generally more flexibility than in higher-speed suburban and rural environ-
ments [typically 65 km/h or greater] where drivers are traveling at greater speeds upstream of the roundabout. Therefore, the types and location of landscape features are dependent on operating environment and the potential risk.

Figure 8.103 illustrates the typical landscaping zones within a roundabout.

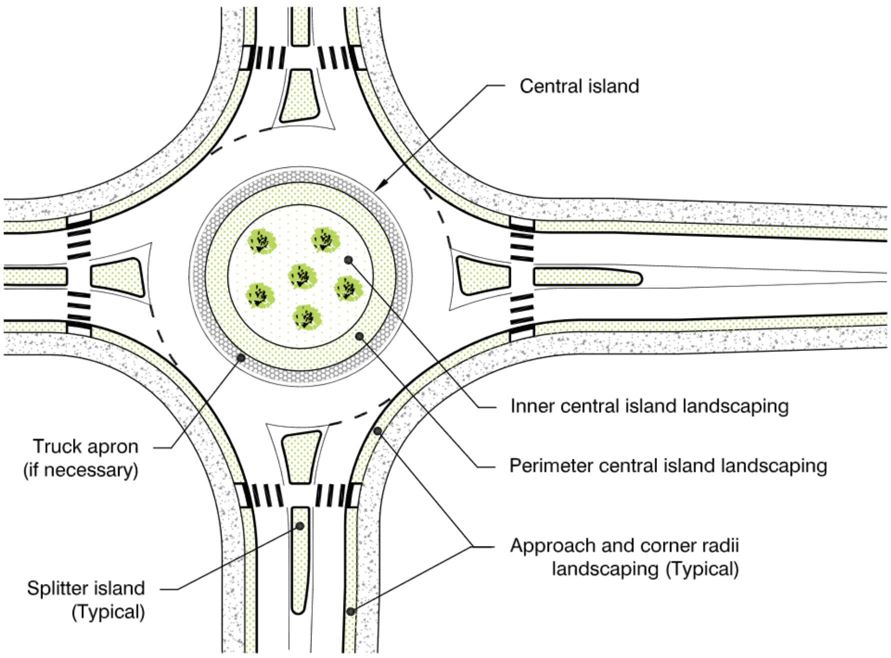


Figure 8.103 Summary of Roundabout Landscaping Zones

8.11.10.2 CENTRAL ISLAND LANDSCAPING

The landscaping of the central island can enhance the safety of the intersection by making the intersection a focal point, by promoting lower speeds, and by breaking the headlight glare of oncoming vehicles. Landscaping elements should be selected so that sight distance is maintained where required. Conversely, the landscaping should also be strategically located to limit the amount of excess sight distance to help encourage slow speeds. This typically results in different types of landscaping being considered for the inner and outer portion of the central island, as described below. Landscaping plans must give consideration of future maintenance requirements to ensure adequate sight distance for the life of the project. It is desirable to create

a domed or mounded central island to increase the visibility of the intersection on the approach. A minimum elevation of
1.0 m and a maximum elevation of 1.8 m is recommended for the domed area on the central island. In addition, the slope of the central island should not exceed a horizontal-to-vertical ratio of 6:1 in order to enable errant vehicles to recover.
The size of the roundabout can influence the type and location of landscaping. Large and small diameter roundabouts have unique landscaping trade-offs that should be considered, as seen in **Table 8.41**.

Table 8.41 Landscaping Considerations as a Function of Diameter

Large Diameter	Small Diameter
• There is more surface area for landscaping features. • A greater focal point for visibility is available as drivers approach the intersection. • There is greater opportunity to create a gateway feature for community enhancement. • A greater amount of landscaping is required, which requires initial installation cost and ongoing maintenance. • Local Assembly often cannot provide ongoing maintenance of roundabouts; therefore, an agreement with a local civic group and/or garden club may be necessary. • If limited maintenance is desired, hardscape features may be installed. • Central island landscaping features (trees, gateway features, hardscape) can create a potential fixed-object conflict, particularly for the high-speed approaching vehicles.	• There is less surface area for landscaping features. • The limited surface area would likely require a lower initial installation cost and less ongoing maintenance. • Central island landscaping is likely not feasible, and the focus should be on the perimeter of the roundabout. • Perimeter landscaping does not typically provide the same visibility benefits to drivers approaching the roundabout. • A small central island provides less opportunity for gateway features in the centre of the roundabout. • Less concern for fixed-object conflicts exists when trees and gateway features are not placed within the central island.

Care is needed when considering landscaping that introduces fixed objects within the central island, particularly in environments with higher approach speeds. While it is important to provide features that increase the visibility of the roundabout to approaching drivers, fixed objects such as trees, poles, walls, guide rail, statues, and large rocks can introduce potential safety concerns for errant vehicles. In most cases, fixed objects should be minimized, particularly in the perimeter area of the central island. If used, fixed objects should preferably be placed in a location where the geometry of the roundabout deflects approaching vehicles away from the object.

In some cases, trees, shrubs, statues, and other larger items can be placed on the inner central island to help obscure the line of sight straight through the roundabout to provide drivers an indication that they cannot pass straight through the intersection. In addition, landscaping in this planting zone can make the roundabout more visible at night with the vehicle headlights illuminating the central island.

The perimeter portion of the central island can be landscaped with low-level shrubs, grass, or groundcover to ensure that stopping sight distance requirements are maintained for vehicles within the circulatory roadway and at the entrance line of the roundabout. The planting zone width around the perimeter of the central island will vary depending on the size of the roundabout and the required sight triangles. **Figure 8.104** illustrates the two potential landscaping zones and possible landscape features within the central island. **Plate 8.22** shows an example of proper landscaping within the central island.

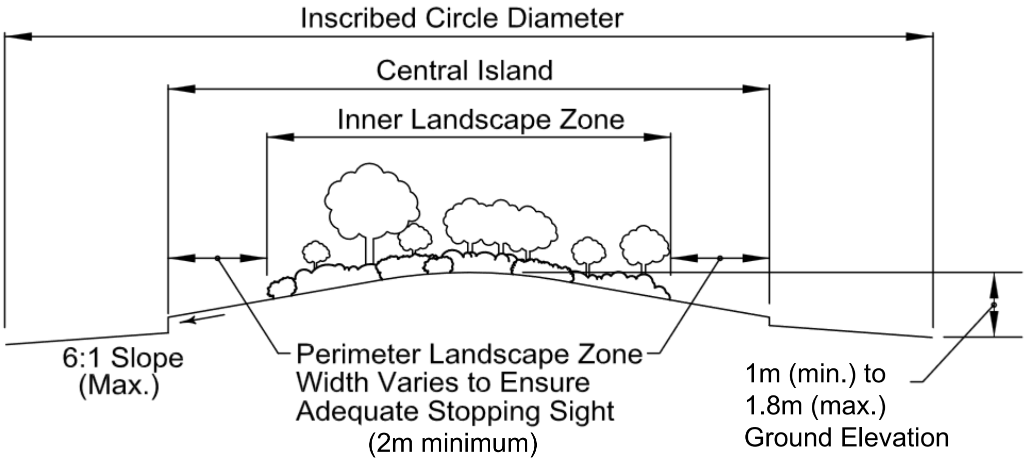


Figure 8.104 Central Island Landscaping Profile

Plate 8.22 Example of Central Island Landscaping

Landscaping within the central island should discourage pedestrian traffic to and through the central island. Street furniture that may attract pedestrian traffic to the central island, such as benches or monuments with small text, should be avoided.

Communities commonly desire to place public art or other large aesthetic objects within the central island, including statues, fountains, monuments, and other gateway features for community enhancement. This type of landscaping is acceptable provided that the objects are located outside the sight triangles and minimize the likelihood of a fixed-object conflict for errant vehicles. In addition, the central island features should not impact the vehicles circulating the roundabout. For example, fountains in windy areas can generate water spray that impacts drivers' visibility through the intersection.

In some areas, a roundabout design can help define a community, township, or region by displaying a piece of art that represents local heritage. **Plate 8.23** illustrates examples of central island art.

Plate 8.23 Example of Central Island Art

8.11.10.3 SPLITTER ISLAND AND APPROACH LANDSCAPING

When designing landscaping for the splitter islands and along the outside edges of the approach, care should be taken with the landscaping to avoid obstructing sight distance since the splitter islands are usually located within the critical sight triangles. **Plate 8.24** gives an example where the vegetation in the splitter island is beginning to encroach on driver sight lines. In addition, landscaping should not obscure the form of the roundabout or signing to an approaching vehicle. Therefore, the size of the splitter islands and location of the roundabout are determining factors in assessing whether to provide landscaping within the splitter islands.

Plate 8.24 Example of Splitter Island Landscaping Encroaching on Sight Lines

Landscaping on the approaches to the roundabout can enhance safety by making the intersection a focal point and by reducing the perception of a high-speed through-traffic movement. Plant material in the splitter islands (where appropriate) and on the right and left side of the approaches can help to create a funnelling effect and induce drivers to slow down when approaching the roundabout. Landscaping between the sidewalk and the circulatory roadway will help to channelize pedestrians to the crosswalk areas and discourage pedestrian crossing to the central island. Because a portion of the splitter island and the area between the walkway and the circulatory roadway are typically situated within the critical sight triangles, the landscaping in these areas may be constructed with low-growth plants or grass.

Grass or low shrubs are also desirable due to their ability to blend well with nearby streetscapes and the fact that they require only limited maintenance. Splitter islands should generally not contain trees, planter boxes, or light poles. Hardscape treatments like a simple patterned concrete or paver surface may be used on splitter islands in lieu of landscaping.

8.12 REFERENCES

1. Ghana Highway Authority Road Design Guide, 1991.
2. Japanese Road Structure Ordinance, April 2021.
3. Explanation and Operation of the Road Structure Ordinance.
4. Geometric Design Manual of Uganda, (2005).
5. A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.

6. Asian Highway Design Standard for Road Safety, 2017.
7. Austroads. 2015. Guide to Road Design Part 1, AGRD01-15, Austroads, Sydney, Australia. Austroads. 2019.
8. Austroads 2009a, Guide to road design part 4: intersections and crossings: general, AGRD04-09, Austroads, Sydney, NSW.
9. South African Geometric Design Guidelines (2003).
10. Geometric Design Manual, Federal Democratic Republic of Ethiopia, Ethiopian Roads Authority, 2013.
11. Rodegerdts, L. A., J. Bansen, C Tiesler, J. Knudsen, E. Myers, M. Johnson, M. Moule, B. Persaud, C. Lyon, S. Hallmark, H. Isebrands, R. B. Crown, B. Guichet, and A. O'Brien. National Cooperative Highway Research Program Report 672: Roundabouts: An Informational Guide, Second Edition. NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2010.
12. Federal Highway Administration. (2019). Advancing Turbo Roundabouts in the United States: Synthesis Report. Federal Highway Administration, Report No. FHWA-SA-19 027, Washington, D.C.
13. Engelsman, J. C., & Uken, M. (2007). Turbo roundabouts as an alternative to two lane roundabouts. SATC 2007.
14. Fortuijn, L. and Harte, V. (1997). Multilane Roundabouts: An Exploration of New Forms (in Dutch). Verkeerskundige werkdagen, CROW, Ede, Netherlands.
15. Džambas, T., S. Ahac, and V. Dragčević. (2017). Geometric Design of Turbo Roundabouts. Tehnički vjesnik, 24(1), 309-318.
16. Rodegerdts, L., Bansen, J., Tiesler, C., Knudsen, J., Myers, E., Johnson, M., … O'Brien, A.. (2010). Roundabouts: An Informational Guide. Transportation Research Board, NCHRP 672, National Research Council, Washington, DC.
17. Overkamp, D. P., & van der Wijk, W. (2009). Roundabouts-Application and design, A practical manual, Royal Haskoning DHV. Ministry of Transport. Public Works and Water management, Partners for Roads.
18. Macioszek, E. (2016). The application of HCM 2010 in the determination of capacity of traffic lanes at turbo roundabout entries. Transport Problems, 11.
19. Highway Capacity Manual (HCM), 2016.

CHAPTER 9

GRADE SEPARATED INTERSECTIONS

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CHAPTER 9 GRADE SEPARATED INTERSECTIONS

9.1 INTRODUCTION

The ability to accommodate high volumes of traffic safely and efficiently through intersections depends largely on the arrangements provided for handling intersecting traffic. The greatest efficiency, safety, and capacity are attained when the intersecting travelled ways are grade separated.

An interchange is a system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways or highways on different levels.

The selection of the appropriate type of grade separation and interchange, along with its design is influenced by many factors, such as road classification, character and composition of traffic, design speed, and degree of access control. In addition to these controls, signing needs, economics, terrain, and right-of-way are of great importance in designing facilities with adequate capacity to accommodate traffic demands. Essential interchange elements include the main line, cross roads, median, ramps, and auxiliary lanes.

To reduce conflicts between vehicles, pedestrians, or bicycles within interchanges, it is preferable to separate their movements. When separation of pedestrians and bicycle movements from vehicle traffic is not practical, each interchange site should be studied, and alternate designs considered to determine the most appropriate arrangement of structures and ramps to accommodate bicycle and pedestrian traffic through the interchange area.

Interchanges vary from single ramps connecting local roads to complex and comprehensive layouts involving two or more roadways. The basic interchange configurations are shown in **Figure 9.1**. Any one configuration can vary extensively in shape and scope, and there are numerous combinations of interchange types that are difficult to designate by separate names. An important element of interchange design is the assembly of one or more of the basic types of ramps, which are discussed in **Section 9.4.4**. The layout for any specific ramp and type of traffic movement will reflect surrounding topography and culture, cost, and degree of flexibility in desired traffic operation. The practical aspects of topography, culture, and cost may be determining factors in the configuration and nature of ramps, but the desired traffic operation should predominate in design.

Figure 9.1A and **Figure 9.1B** illustrate typical three-leg interchanges. **Figure 9.1A** is a trumpet interchange, named for the trumpet or jug-handle ramp configuration. **Figure 9.1B** is a three-level, directional, three leg interchange. With ramps in one quadrant, the interchange in **Figure 9.1C** is not suitable for expressway/motorway systems but becomes very practical for an interchange between a major road and an access road. This design is appropriate for access roads because design speeds are usually lower, large trucks are prohibited, and turning movements are light. A typical diamond interchange is illustrated in **Figure 9.1D**. Diamond interchanges have numerous other configurations incorporating frontage roads and continuous collector or distributor roads. **Figure 9.1E** is a Single-Point Diamond Interchange (SPDI).

The SPDI is a form of a diamond interchange with a single signalized intersection through which all left turns utilizing the interchange must travel. All right turns into and out of ramp approaches are generally free flow. **Figure 9.1F** presents a partial cloverleaf that contains two cloverleaf loops and four diagonal ramps. Varying configurations favour heavier traffic movements. A full cloverleaf, as shown in **Figure 9.1G**, gives each interchanging movement an independent ramp; however, it generates weaving manoeuvres that occur either in the area adjacent to the through lanes or on collector-distributor roads. **Figure 9.1H** illustrates a fully directional interchange.

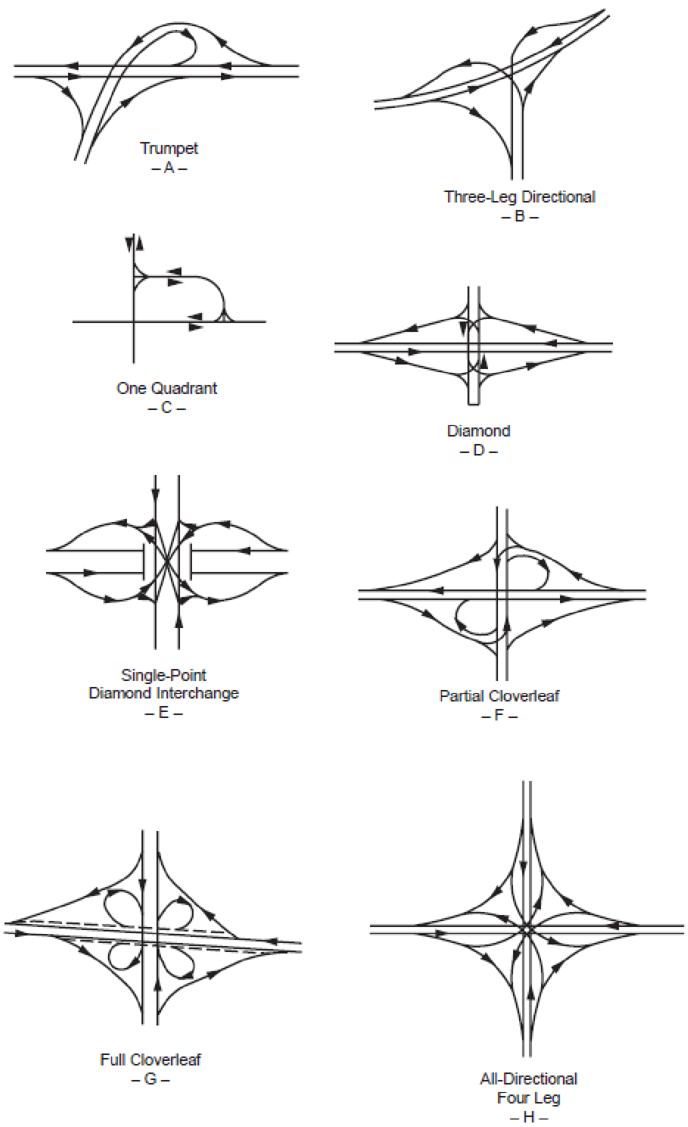


Figure 9.1 Interchange Configuration

9.2 WARRANTS FOR INTERCHANGES AND GRADE SEPARATIONS

An interchange can be a useful and an adaptable solution to improve many intersection conditions either by reducing existing traffic bottlenecks or by reducing crash frequency. However, the high cost of constructing an interchange limits its use to those cases where the additional expenditure can be justified. An enumeration of the specific conditions or warrants justifying an interchange at a given intersection is difficult and, in some instances, cannot be conclusively stated. Because of the wide variety of site conditions, traffic volumes, roadway types, and interchange layouts, the warrants that justify an interchange may differ at each location. The following six conditions, or warrants, should be considered when determining if an interchange is justified at a particular site:

i. **Design designation**

The determination to develop a roadway with full access control between selected terminals becomes the warrant for providing roadway grade separations or interchanges for all intersecting roads crossing the roadway.

ii. **Reduction of bottlenecks or spot congestion**

Insufficient capacity at the intersection of heavily traveled routes results in intolerable congestion on one or all approaches. Inability to provide essential capacity with an at-grade facility provides a warrant for an interchange where development and available right-of-way permit.

iii. **Reduction of crash frequency and severity**

Some at-grade intersections have a disproportionate frequency of serious crashes. If inexpensive methods of reducing crashes are likely to be ineffective or impractical, a roadway grade separation or interchange may be warranted.

iv. **Site topography**

At some sites, grade-separation designs are the only type of intersection that can be constructed economically. The topography at the site may be such that, to satisfy appropriate design criteria, any other type of intersection is physically impossible to develop or is equal to or greater than the cost of a grade-separated design.

v. **Road-user benefits**

The road-user costs from delays at congested at-grade intersections are large. Road-user costs, such as fuel and oil usage, wear on tires, repairs, delay to motorists, and crashes that result from speed changes, stops, and waiting, are well in excess of those for intersections permitting uninterrupted or continuous operation.

vi. **Traffic volume warrant**

A traffic volume warrant for interchange treatment may be the most tangible of any interchange warrant. Although a specific volume of traffic at an intersection cannot be completely rationalized as the warrant for an interchange, it is an important guide, particularly when combined with the traffic distribution pattern and the effect of traffic behaviour. However, volumes in excess of the capacity of an at grade intersection would certainly be a warrant. Interchanges are desirable at cross roads with heavy traffic volumes because the elimination of conflicts due to high crossing volume greatly improves the movement of traffic.

9.3 GRADE SEPARATION STRUCTURES

Grade separations are facilities provided for the purpose of ensuring smooth and uninterrupted traffic flow by eliminating or reducing the impact of intersecting traffic.

In planning, comprehensive examination is conducted on the road traffic volume and traffic capacity, road specifications, road functions, terrain, roadside conditions, land use situation and environmental conditions.

In situations where roads with four (4) or more lanes intersect, except where there are unavoidable reasons, grade separation should be adopted. The detailed standards are as follows.

i. Grade separation should be adopted in all cases of intersection between Design Class A Roads.
ii. Grade separation should be adopted in all cases of intersection between Design Class A Roads and other Design Classes.
iii. Grade separation should be adopted in cases of intersection between Design Class B Roads.
iv. Grade separation should be adopted in cases of intersection between Design Class B Roads and other Design Classes.
v. Grade separation should be adopted in cases of intersection between roads that have 4 lanes or more.

However, even if these standards are met except for i and ii, at-grade intersections are permitted in cases where:

i. the intersecting traffic volume is low.
ii. there are no issues with traffic safety.
iii. the interval with preceding and succeeding intersections is long.
iv. there are terrain issues or other reasons.

9.3.1 TYPES OF SEPARATION STRUCTURES

Grade-separation structures are identified by three general types: deck type, through, and partial through. The deck type is most common for grade separations. However, the through and partial through types are appropriate for railroad structures. In special cases where the spans are long and the difference in elevation between the roadways is to be severely limited, truss bridges may be used. Through girder bridges, in comparison to through deck-type bridges, will decrease vertical restrictions.

In the case where the upper roadway extends from hilltop to hilltop and vertical clearance is not a concern, deck-type structures, such as trusses, arches, girders, etc., may be appropriate. A through plate girder bridge is often used for railroad separations when the railroad overpasses the roadway or road.

The through plate girder and through truss bridges produce a greater sense of visual restriction than deck type structures; therefore, lateral offset from the edge of lane should be as great as practical.

In any single separation structure, care should be exercised in maintaining a constant clear roadway width and a uniform protective railing or parapet. The type of structure best suited to grade separations is one that gives drivers little sense of restriction. Where drivers take little notice of a structure over which they are crossing, their behavior is the same or nearly the same as at other points on the roadway, and sudden, erratic changes in speed and direction are unlikely. On the other hand, it is virtually impossible for drivers not to notice a structure overpassing the roadway being used. For this reason, every effort should be made to design a structure that fits the environment in a pleasing and functional manner without drawing excessive or distracting attention.

Collaboration between the bridge and roadway engineers throughout the various stages of planning and design can provide excellent results in this regard. Overpass structures should have liberal lateral offset on the roadways at each level. All piers and abutment walls should be suitably offset from the travelled way. The finished underpass roadway median and off-shoulder slopes should be rounded, and there should be a transition to backslopes to redirect errant vehicles away from protected or unprotected structural elements.

For the underpass roadway, the most desirable structure from the standpoint of vehicular operation is one that will span the entire roadway cross section and provide a lateral offset of structural supports from the edge of roadway that is consistent with good roadside design. The lateral offset between the edge of roadway and the structural supports should be as wide and flat as practical to provide usable recovery space for errant vehicles and to prevent distraction in the motorist’s peripheral field of vision.

In the case of depressed roadways, lateral offset may be reduced. On divided roadway, centre supports should be used only where the median is wide enough to provide sufficient lateral offset or narrow enough to need protective barriers. The usual lateral offset of an underpass at piers or abutments may allow sufficient room to construct additional lanes under the structure in the future, but at a sacrifice of recovery space. In anticipation of future widening, the piers or abutment design should provide footings with sufficient cover after widening. The bridge engineer should be advised when future widening is contemplated.

A greater sense of openness results with end spans than with full-depth abutments. Perched stub or semi-stub abutments can also provide appropriate visual clearance. In urban areas, although not all crossroad are important enough to warrant interchange ramps with the main line, a sufficient number of crossroads should be separated in grade to preserve the continuity of traffic flow on the local road system. As a matter of economics, however, it is seldom practical to continue all crossroad across the main line. Most roads that cross the major roadway, whether or not they connect with it, experience a rapid increase in traffic after construction of the major roadway as a result of intensified land development and local road closures within the main-line corridor.

Terminated and through roads may be intercepted by one-way frontage roads on each side of the main facility. Access between the main roadway and frontage roads can be provided by slip ramps at prescribed intervals to serve traffic demands. On elevated facilities with viaduct construction, crossroad are relatively undisturbed; however, on all other types of roadways, considerable savings can be achieved by terminating some of the less important crossroad.

Special consideration is needed relative to the spacing and treatment of crossroad on these roadways. Arterials and other major crossroad should continue across the main line without interruption or deviation. Grade separations should be of sufficient number and capacity to accommodate not only the normal cross traffic but also the traffic diverted from the other roads terminated by the main facility and the traffic generated by access connections to and from the main line.

Thus, determination of the number and location of crossroad to be separated in grade needs a thorough analysis of traffic on the road system, in addition to that on the main line and its interchanges. Insofar as expressway/motorway operation is concerned, there is no minimum spacing or limit to the number of grade separated crossroad. The number and their location along one corridor are governed by the local road system, existing or planned. **Plate 9.1** shows a typical grade separation structure with open-end span.

Plate 9.1 Typical Grade Separation with Open-End Span (Tomei Expressway, Japan)

9.3.2 OVERPASS VERSUS UNDERPASS ROADWAYS

9.3.2.1 GENERAL DESIGN CONSIDERATIONS

A detailed study should be made at each proposed roadway grade separation to determine whether the major roadway should be carried over or under the crossroad. Often this decision is based on features such as topography or road classification. It may be appropriate to make several nearly complete preliminary layout plans before an appropriate decision can be reached. General guidelines for over-versus-under preference follow, but such guidelines should be used in combination with detailed studies of the grade separation as a whole.

In principle, a design that best fits the existing topography is also the most pleasing and economical to construct and maintain, and this factor becomes the first consideration in design. Where topography does not govern, as is common in the case of flat topography, it may be appropriate to study secondary factors, and the following general guidelines should be examined:

i. It is appropriate to consider alternatives in the interchange area as a whole to decide whether the major road should go over or under the crossroad.
ii. In addition to the intersecting roadways, consider how the ramps and slopes within the whole of the interchange area fit the existing topography.
iii. An undercrossing roadway has a general advantage in that an approaching interchange may be easily seen by drivers. As a driver approaches, the structure appears ahead, making the presence of the upper-level crossroad obvious, and providing advance warning of the likely presence of interchange ramps.

iv. Through traffic is given aesthetic preference by a layout in which the more important road is the overpass. A wide overlook can be provided from the structure and its approaches, giving drivers a minimum feeling of restriction.

v. Where turning traffic is significant, the ramp profiles are best fitted when the major road is at the lower level. The ramp grades then assist turning vehicles to decelerate as they leave the major roadways and to accelerate as they approach it, rather than the reverse. In addition, for diamond interchanges, the ramp terminal is visible to drivers as they exit the major roadways.

vi. In rolling topography or in rugged terrain, major-road overcrossings may be attainable only by a forced alignment and rolling grade line. Where there otherwise is no pronounced advantage to the selection of either an underpass or an overpass, the design that provides the better sight distance on the major road (desirably passing distance if the road is two-lane) should be preferred.

vii. Available sight distance on the major and minor roadways through a grade separation should be at least as long as that needed for stopping sight distance and preferably longer. Where exits are involved, decision sight distance is preferred, although not always practical.

viii. An overpass offers the best possibility for stage construction, both in the roadway and structure, with minimum impairment of the original investment. The initial development of only part of the ultimate width is a complete structure and roadway in itself. By lateral extension of both or construction of a separate structure and roadway for a divided roadway, the ultimate development is reached without loss of the initial facility.

ix. Troublesome drainage challenges may be reduced by carrying the major roadway over the crossroad without altering the crossroad grade. In some cases, drainage concerns alone may be sufficient reason for choosing to carry the major roadway over rather than under the crossroad.

x. Where topography control is secondary, the cost of bridges and approaches may determine whether the major roadway underpasses or overpasses the minor facility. A cost analysis that takes into account the bridge type, span length, roadway cross-section, angle of skew, soil conditions, and cost of approaches will determine which of the two intersecting roadways should be placed on structure.

xi. An underpass may be more advantageous where the major road can be built close to the existing ground, with continuous gradient and with no pronounced grade changes. Where the widths of the roads differ greatly, the quantity of earthwork makes this arrangement more economical. Because the minor road usually is built to less generous design criteria than the major road, grades on it may be steeper and sight distances shorter, with resultant economy in grading volume and pavement area on the shorter length of road to be rebuilt above the general level of the adjoining land.

xii. If a grade-separated intersection is adopted, constraints are imposed on roadside traffic to the section between intersections. In urban areas where there is major impact on traffic capacity at intersections with heavy traffic volume, all the intersections should be studied for possible grade separation.

xiii. Frequently, the choice of an underpass at a particular location is determined not by conditions at that location, but by the design of the roadway as a whole. Grade separations near urban areas constructed as parts of a depressed expressway/motorway, or as one raised above the general level of adjoining roads, are good examples of cases where decisions regarding individual grade separations are subordinated to the general development.

xiv. Where a new roadway crosses an existing route carrying a large volume of traffic, an overcrossing by the new roadway causes less disturbance to the existing route and a detour is usually not needed.

xv. The overcrossing structure has no limitation as to vertical clearance, which can be a significant advantage in the case of oversized loads requiring special permits on a major roadway or route.

xvi. Desirably, the roadway carrying the highest traffic volume should have the fewest number of bridges for better rideability and fewer conflicts when repair and reconstruction are needed.

xvii. In some instances, it may be appropriate to have the higher volume facility depressed and crossing under the lower volume facility to reduce noise impact.

xviii. In some instances, the lower volume facility should be carried over if there is a pronounced economic advantage.

xix. Bicyclists and pedestrians benefit most from a flat profile, which is a factor that should be considered, particularly in urban area conditions.

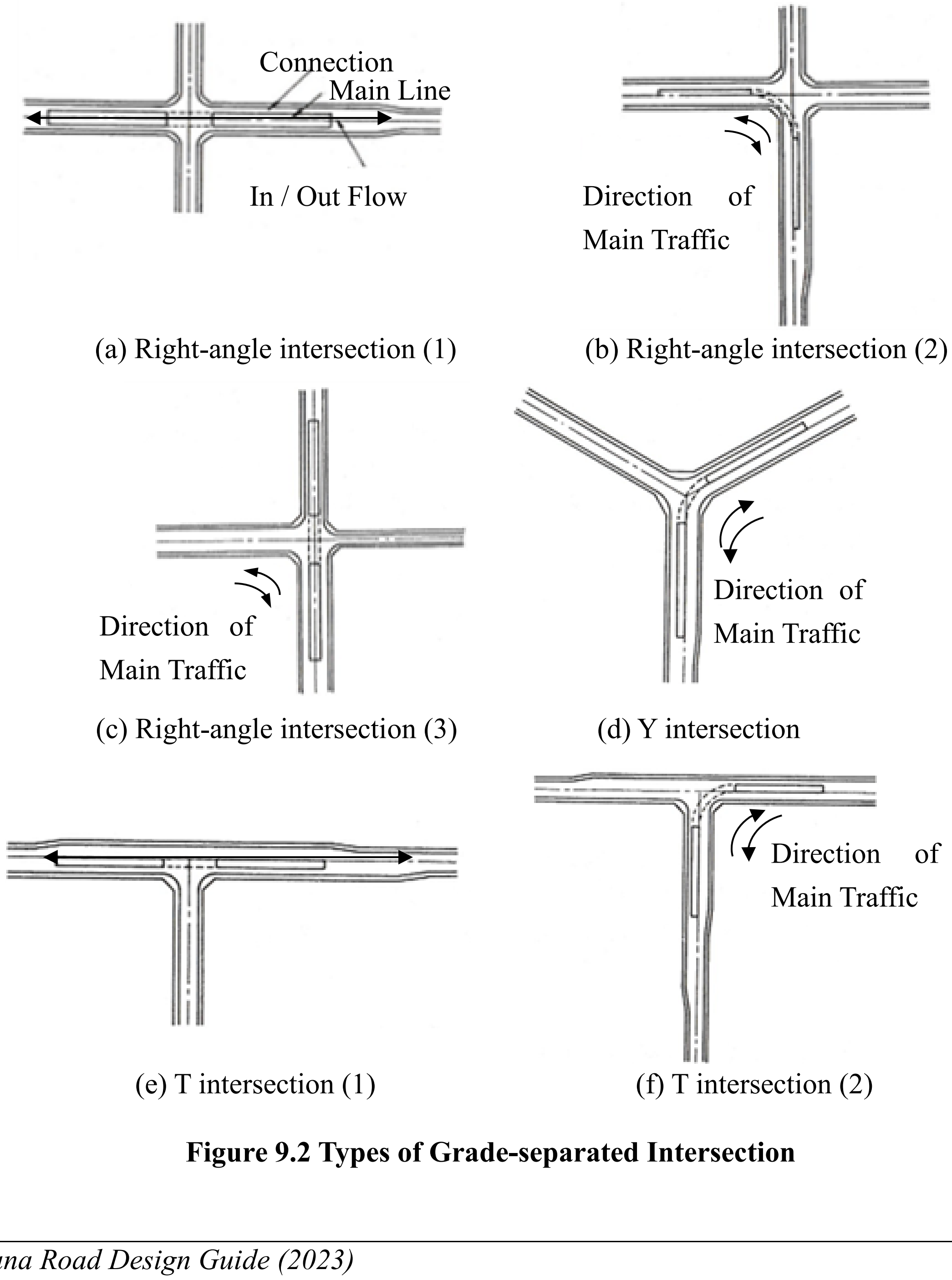
9.3.2.2 STRUCTURE WIDTHS

Roadways with wide shoulders, wide gutters, and flat slopes have the fewest severe crashes. Poles, walkways, bridge columns, bridge railing, and parapets located close to the travelled way are potential obstructions and cause drivers to shy away from them. For this reason, the clear width on bridges should preferably be as wide as the approach roadway to give drivers a sense of openness and continuity.

On long bridges, particularly on long-span structures where cost per square meter is greater than the cost on short-span structures, widths that are less than ideal may be acceptable; however, economy alone should not be the governing factor in determining structure widths. Traffic characteristics, potential crash frequency and severity, emergency contingencies, maintenance needs, and benefit/cost ratios should be fully considered before the desirable structure width is reduced.

9.3.3 PLANNING OF OVERPASS/UNDERPASS INTERSECTION

Grade-separated intersections are mainly provided in urban areas that have communities along the roads. The traffic flow that needs to be elevated is the direction with the heaviest traffic flow (main flow direction). A comprehensive study should be conducted on terrain, land use conditions, situation regarding other facilities, construction cost and other constraints. **Figure 9.2** shows the types of grade sepearated intersection.



(a) Right-angle intersection (1)
(b) Right-angle intersection (2)
(c) Right-angle intersection (3)
(d) Y intersection
(e) T intersection (1)
(f) T intersection (2)

Figure 9.2 Types of Grade-separated Intersection

Also, when planning an overpass/underpass intersection, in addition to the overall design conditions of the intersecting roads, consideration must be given to safety and smooth traffic flow on the ramp and connecting road.

9.3.3.1 LATERAL OFFSET AT UNDERPASSES

For a two-lane roadway or an undivided multilane roadway, the cross-section width at underpasses will vary, depending on the design criteria appropriate for the particular functional classification and traffic volume. The minimum lateral offset from the edge of the travelled way to the face of the protective barrier should be the normal shoulder width.

On divided roadways, the offset on the left side of each roadway is usually governed by the median width. A minimum median width of 3.0m may be used on a four-lane roadway to provide 1.2m shoulders and a rigid median barrier. For a roadway with six or more lanes, the minimum median width should be 6.6m to provide 3.0m shoulders and a rigid median barrier. **Figure 9.3A** shows the minimum lateral offset to a continuous median barrier, either concrete or metal, for the basic roadway section and for an underpass where there is no centre support. The same offset dimensions are applicable for a continuous wall on the left. Where a concrete median barrier is used, its base should be aligned with respect to the travelled way, as shown in **Figure 9.3A**.

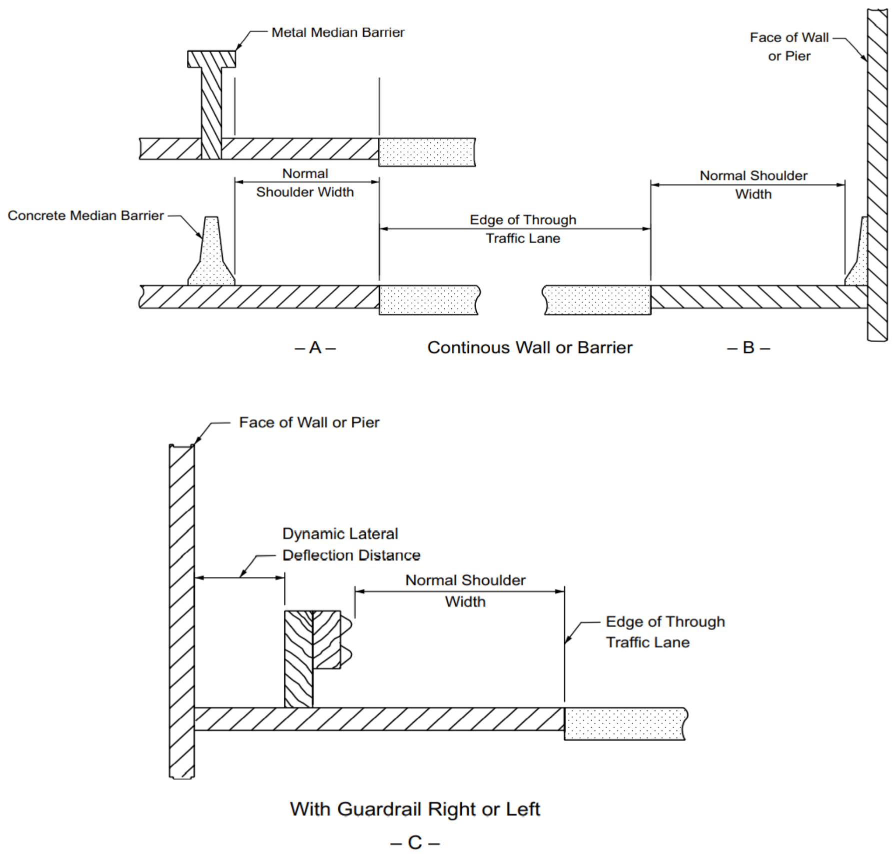


Figure 9.3 Lateral Offset for Major Roadway Underpasses

Figure 9.3B shows the minimum lateral offset on the right side of the roadway as applicable to a continuous wall section. A concrete barrier is constructed integrally with the wall. For this situation, the lateral offset on the right should be measured to the base of the barrier. For designs with a continuous concrete barrier on the right, usually a section similar to a median barrier, **Figure 9.3B** is applicable. The same type of barrier may be used as an introduced feature where conditions lead to structure design with full depth abutments. The shoulder on high-speed roadways should flush with the traveled way. Continuous kerbs on high-speed roadways should be limited to special situations, such as drainage systems on the outside of shoulders. Such kerbs should be carried through the underpass. Where walkways are provided, the full shoulder section should be maintained through the underpass and the span increased by the width of the walk. Where a kerb is needed along solid abutments or walls, a concrete barrier may be used.

Where conditions preclude the clear roadside design concept, all abutments, piers, and columns should be shielded with suitable protective devices unless they are so situated that they cannot be hit by out-of-control vehicles. Protective devices are usually not needed along continuously walled sections.

Guardrail installed along the face of an exposed pier or abutment should have an offset appropriate to the dynamic lateral deflection of the particular rail type. The rail cannot cushion and deflect an errant vehicle unless there is sufficient lateral space clear of the bridge support. **Figure 9.3C** shows the limits of the dynamic lateral deflection distance between the face of bridge support and the back of the rail system.

Guardrail attached flush with the exposed faces of piers, abutments, and bridge railings should be stiffened preceding the obstruction to avoid snagging an errant vehicle. This may be accomplished by one or more of the following techniques:

i. reducing the post spacing;
ii. increasing the post embedment;
iii. increasing the rail section modulus; or
iv. transitioning to a different, stiffer barrier (i.e., metal to concrete).

The rail should be fastened securely enough to develop its full strength longitudinally. For further details, see the MRH Standard Details, Road Signs and Markings for Urban and Trunk Roads, 1991.

Where the horizontal lateral offset through an underpass is reduced for structural design or cost reasons, the change in lateral width should be accomplished through gradual adjustments in the cross section of the approach roadway rather than abruptly at the structure. Such transitions in width should have a gradual rate of 50:1 or more (longitudinal: lateral).

9.3.3.2 LATERAL OFFSET ON OVERPASSES

On overpass structures, it is desirable to carry the full width of the approach roadway across all structures. For facilities other than sxpressways/motorways, exception may be made on major structures with a high unit cost. The selection of cross-section dimensions that are different from those on the approach roadway should be subject to individual economic studies.

In the case of a kerbed roadway, the minimum structure width should match the kerbed approach roadway. When the full approach roadway width is continued across the structure, the parapet rail, both left and right, should align with the guardrail on the approach roadway.

At some interchanges, additional width for speed-change lanes or weaving sections is needed across overpass structures. Where the auxiliary lane is a continuation of a ramp, the lateral offset to the bridge rail should be at least equal to the width of shoulder on the approach ramp. Where the auxiliary lane is a weaving lane connecting entrance and exit ramps or is a parallel-type speed-change lane across the entire structure, the offset to the parapet should be of uniform width and be at least equal to the shoulder width on the ramp.

9.3.3.3 MEDIANS

On a divided roadway with a wide median or one being developed in stages, the overpass will likely be built as two parallel structures. The approach width of each roadway should be carried across each individual structure. If separate parallel structures are used, the width of opening between structures is unimportant.

Where the approach is a multilane, undivided roadway or one with a flush median less than 1.2m wide, a raised median is considered unnecessary on short bridges of about 30m in length but is desirable on bridges of 120m or more in length. On bridges between 30m and 120m in length, local conditions such as traffic volume, speed, sight distance, need for luminaire supports, future improvement, approach cross section, number of lanes, and whether the roadway is to be divided determine whether or not medians are warranted.

Where there are medians of narrow or moderate width on approaches to long single structures, the structure should be wide enough to accommodate the same type of median barrier as is used in the median of the approach roadway.

9.3.4 ENTRY AND EXIT OF OVERPASS/UNDERPASS INTERSECTION

Entry and exit sections of a grade separated intersection is shown in **Figure 9.4**. In these sections, in order to maintain safe and smooth traffic flow, it is necessary to pay utmost attention to diverging/merging section to connecting roads.

The basic approach to runoff grade separation entry and exit is shown in **Figure 9.4**. A desirable length (L) should be provided for the parallel section between the connecting roads and the main line. L= 20m when the main line design speed is 60km/h[a] to ensure the safety of diverging/merging and smooth traffic flow. The runoff factor for widening should be determined according to **Section 7.2.9**.

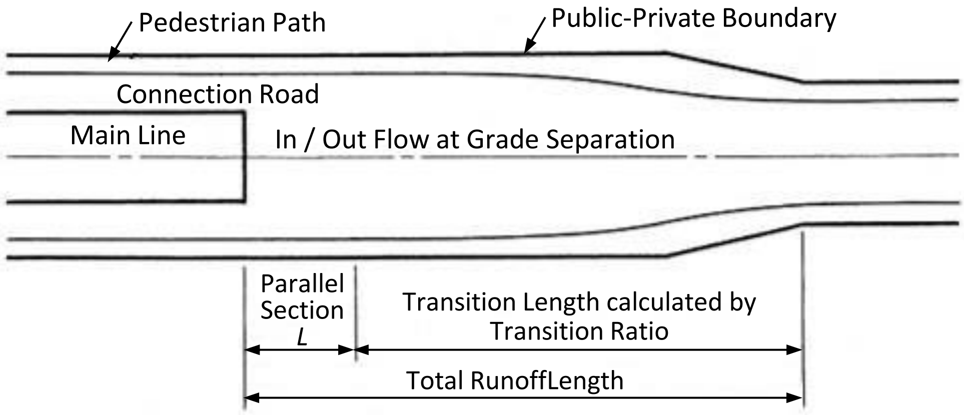


Figure 9.4 Runoff of Grade Separation Inflow and Outflow Sections

9.3.5 GRADE SEPARATIONS WITHOUT RAMPS

There are many situations where grade separations are constructed without the provision of ramps. For example, some major arterials intersecting the existing roadway need to be kept open for access but carry only low traffic volumes. Lacking a suitable relocation plan for the crossroad, a roadway grade separation without ramps may be provided. All drivers desiring to turn to or from that road are required to use other existing routes and enter or leave the roadway at other locations. In some instances, these vehicles may have to travel a considerable extra distance, particularly in rural areas.

In other situations, despite sufficient traffic demand, ramps may be omitted

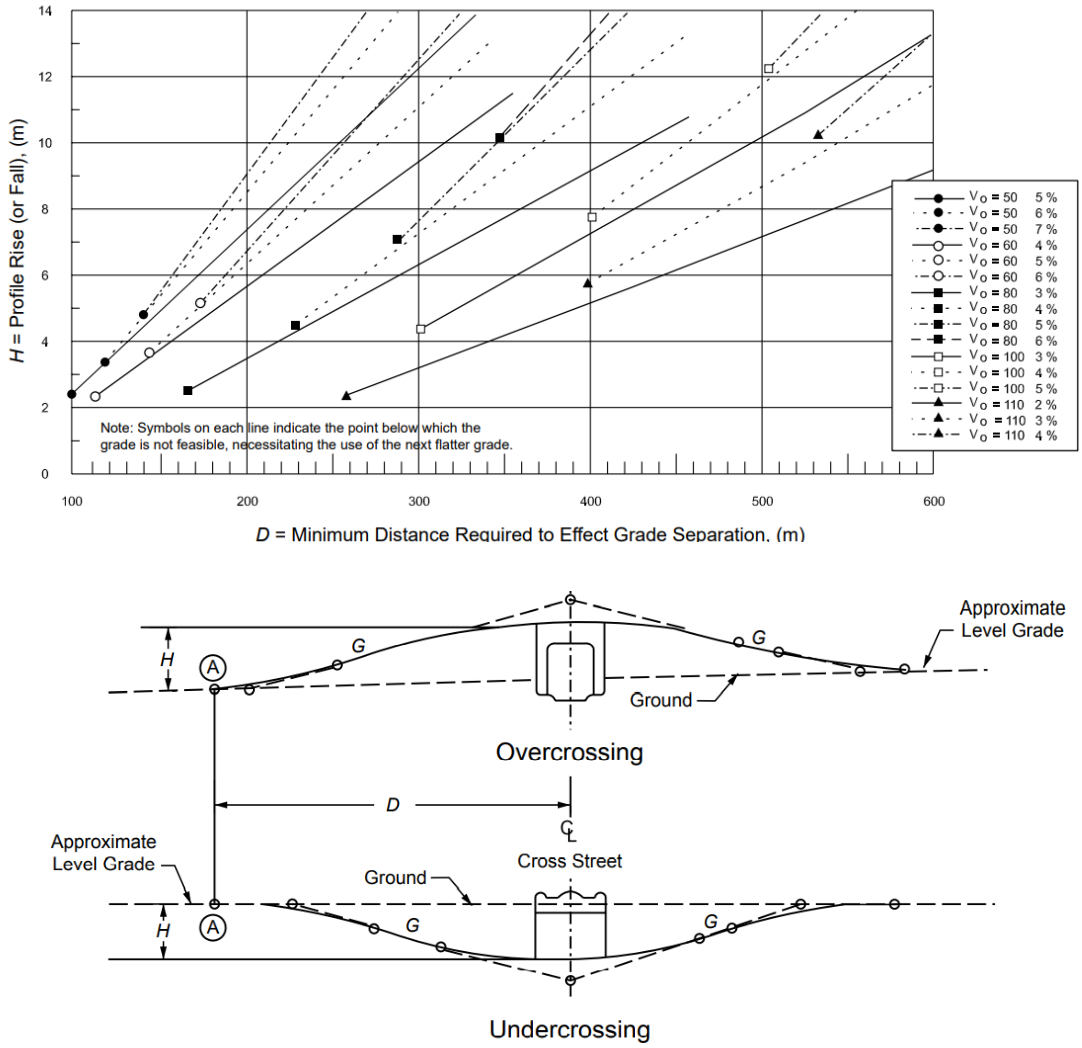
i. to avoid having interchanges so close to each other that signing and operation would be difficult,
ii. to eliminate interference with large roadway traffic volumes, and
iii. to increase mobility and reduce crashes by concentrating turning traffic where it is practical to provide adequate ramp systems.

On the other hand, undue concentration of turning movements at one location should be avoided where it would be better to provide several interchanges.

In rugged topography, the site conditions at an intersection may be more favorable for provision of a grade separation than an at-grade intersection. If ramp connections are difficult or costly, it may be practical to omit them and accommodate turning movements at other intersecting roads.

9.3.6 LONGITUDINAL DISTANCE TO ATTAIN GRADE SEPARATION

The longitudinal distance needed for adequate design of a grade separation depends on the design speed, the roadway gradient, and the amount of rise or fall needed to achieve the separation. **Figure 9.5** shows the horizontal distances needed in flat terrain. It may be used as a guide for preliminary design to determine quickly whether or not a grade separation is practical for given conditions, what gradients may be involved, and what profile adjustments, if any, may be needed on the crossroad. These data also may serve as a general guide in other than flat terrain, and adjustments can be made in the length of the terminal vertical curves. The chart is useful where the profile is rolled to overpass some crossroads and to underpass others, and it is useful for design of an occasional grade separation on a facility located at ground level, such as a major road or at-grade mororway/expressway.



NB: Minimum Vertical Clearance should be checked under the Outside Edge of the Overcrossing Structure

Figure 9.5 Flat Terrain, Distance Needed to Achieve Grade Separation

The distance needed to achieve a grade separation can be determined from **Figure 9.5** for gradients ranging from 2 to 7 percent and for design speeds (V_o) ranging from 50 to 110 km/h. Design speeds (V_o) of 80 to 110 km/h are applicable to expressways/motorways in urban areas, and 60 km/h (50 km/h in special cases) is used on major arterials. The curves are derived with the same approach gradient on each side of the structure. However, values of D from **Figure 9.5** also are applicable to combinations of unequal gradients. Distance D is equal to the length of the initial vertical curve, plus one-half the central vertical curve, plus the length of tangent between the curves. Lengths of vertical curves, both sag and crest, are minimums based on the minimum stopping sight distance. Longer curves are desirable. Length D applies equally to an overpass or an underpass, despite the fact that the central crest vertical curve may be longer than the central sag vertical curve for comparable values of H and G.

Certain characteristics and relations in **Figure 9.5** are worthy of note:

i. For the usual profile rise (or fall) needed for a grade separation H of 7.5m or less, gradients greater than 3 percent for a design speed of 110 km/h, 4 percent for 100 km/h, 5 percent 80 km/h, and 6 percent for 60 km/h cannot be used. For values of H less than 7.5m, flatter gradients than those just cited should generally be used. The lower terminal of the gradient lines on the chart, marked by a small circle, indicates the point where the tangent between curves is zero and below which a design for the given grade is not feasible (i.e., a profile condition where the minimum central and end curves for the gradient would overlap).
ii. For given H and design speed, distance D is shortened a negligible amount by increasing the gradient above 4 percent for a design speed of 80 km/h and above 5 percent for 60 and 50km/h. Distance D varies to a greater extent, for given H and G, with changes in design speed.

A 7.0 – 8.0 m difference in elevation is usually needed at a grade separation of two roadways for essential vertical clearance and structural thickness. The same dimension generally applies to a roadway undercrossing a railroad, but about 8.4m is needed for a roadway overcrossing a main-line railroad. In level terrain, these vertical dimensions correspond to H, the rise or fall needed to achieve a grade separation. In practice, however, H may vary over a wide range because of topography. Where a relatively short distance is available for a grade separation, it may be appropriate to reduce H to keep D within the distance available. This reduction is accomplished by raising or lowering the intersecting road or railroad.

Figure 9.6 shows the required distance between a grade separation and an at-grade intersection.

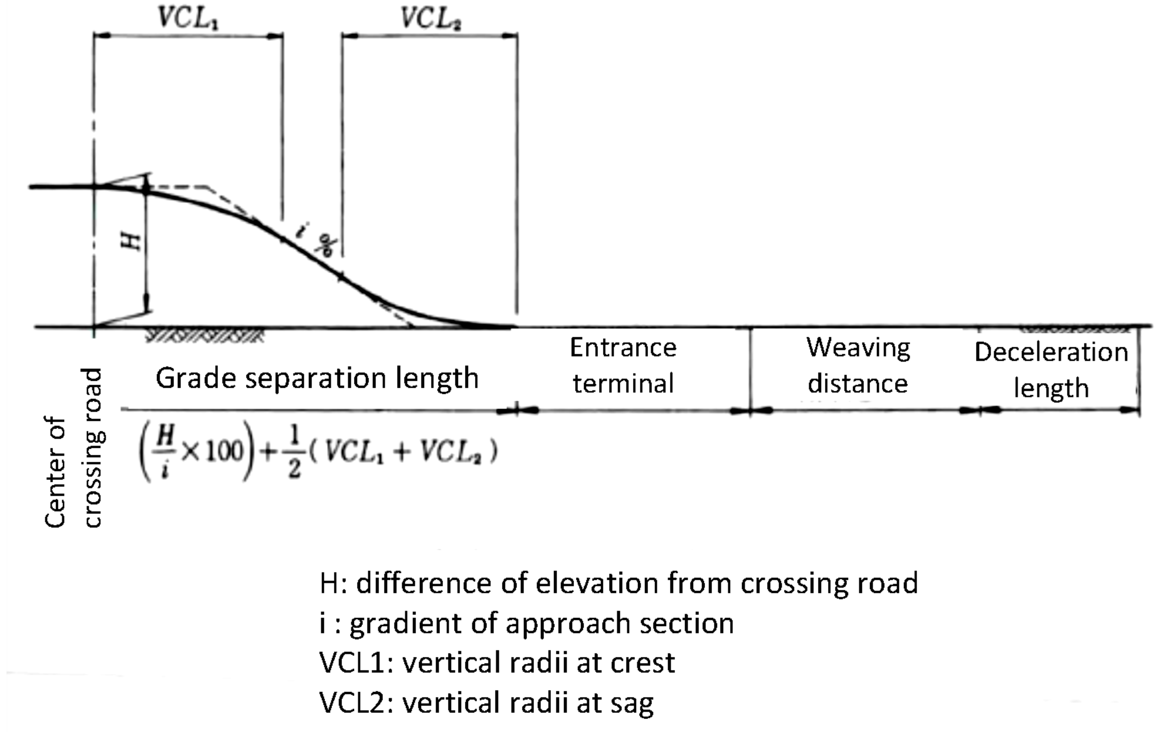


Figure 9.6 Required distance between at-grade and grade separated intersections

9.4 INTERCHANGES

9.4.1 GENERAL CONSIDERATIONS

There are several basic interchange configurations to accommodate turning movements at a grade separation. The type of configuration used at a particular site is determined by the number of intersection legs, expected volumes of through and turning movements, type of large vehicle and trailer traffic, topography, culture, design controls, and proper signing. The designer's initiative also plays an important role.

While interchanges are custom designed to fit specific site conditions, it is desirable that the overall pattern of exits along the expressway/motorway have some degree of uniformity. Furthermore, from the standpoint of driver expectancy, it is desirable that all interchanges have one point of exit located in advance of the crossroad wherever feasible.

Signing and operations are major considerations in the design of the interchanges. The signing of each design should be tested to determine if it can provide for effective flow of traffic. The need to simplify interchange design from the standpoint of signing and driver understanding cannot be overstated. To prevent wrong-way movements, all expressway/motorway interchanges with non-access-controlled roadways should provide ramps to serve all basic directions. Drivers expect expressway/motorway -to- expressway/motorway interchanges to provide all directional movements. As a special case treatment, an expressway/motorway -to-expressway/motorway movement may be omitted if the turning traffic is minor and can be accommodated by other nearby expressway/motorway facilities.

The accommodation of pedestrians and bicyclists also should be considered in the selection of an interchange configuration. For convenience, examples of interchange configurations are illustrated in the following discussion in general terms for three- and four-leg intersections and for special designs involving two or more structures. The general interchange configurations are shown either schematically or as examples of existing facilities.

9.4.1.1 THREE-LEG DESIGNS

An interchange with three intersecting legs consists of one or more roadway grade separations and one way roadways for all traffic movements. When two of the three intersection legs form a through road and the angle of intersection is not acute, the term “T-interchange” applies. When all three intersection legs have a through character or the intersection angle with the third intersection leg is small, the interchange may be considered a Y-configuration. A clear distinction between the T- and Y-configurations is not important. Regardless of the intersection angle and through-road character, any basic interchange pattern may apply for a wide variety of conditions. Three-leg interchanges should only be considered when future expansion to the unused quadrant is either impossible or highly unlikely. This is partly due to the fact that three-leg interchanges are very difficult to expand or modify in the future.

Figure 9.7 illustrates patterns of three-leg interchanges with one grade separation. **Figure 9.7A** and **Figure 9.7B** show the widely used trumpet pattern. Through-traffic movements, from points a to c, are on direct alignment. A criterion for selection of either design is the relative volumes of the left-turning movements, the more direct alignment favoring the heavier volume and the loop favouring the lesser volume. Skewed crossings are more desirable than right-angle crossings because the skewed crossing has a somewhat shorter travel distance and flatter turning radius for the heavier left-turning volume, and there is less angle of turn for both left turns. In **Figure 9.7A**, the curvature of the loop b-a begins before the structure, warning the driver to anticipate a major break in curvature. The transition spirals provide for a smooth speed change and steering manoeuvre both into the loop and onto the high-speed facility. The oblong shape of the loop allows the curvature of the high-volume left turn, c to b, to be flattened, allowing higher operating speeds to be attained. The exit to the loop ramp of **Figure 9.7B** is placed well in advance of the structure to provide sufficient deceleration length in the approach to the break in curvature. Curves with spiral transitions are effective in developing the desired shape of ramps. The curvature of the left turn, b-a, is initiated in advance of the structure for driver anticipation.

The other type of three-leg single-structure interchange shown in **Figure 9.7C** is less common, with loops for both left-turning movements. The interchange in **Figure 9.7C** has an excellent field of usage as the initial stage of an ultimate cloverleaf. A collector-distributor road is provided to eliminate weaving on the main road. In the second stage, the roadway forming the fourth leg opposite the stem of the "T" is developed, and the remaining ramps are added. With respect to traffic, this type of interchange is inferior to those in **Figure 9.7A** and **Figure 9.7B** because both left-turn movements use loops and weave across each other. Furthermore, the small-radius loop ramps are not considered an appropriate method of terminating a roadway. Although the pattern is appropriate for interchanges where the left-turning volumes are not great, the configurations in **Figure 9.7A** and **Figure 9.7B** are preferable if they are equally adaptable to the site conditions. For comparable conditions, construction costs for **Figure 9.7A** and **Figure 9.7B** should be about the same.

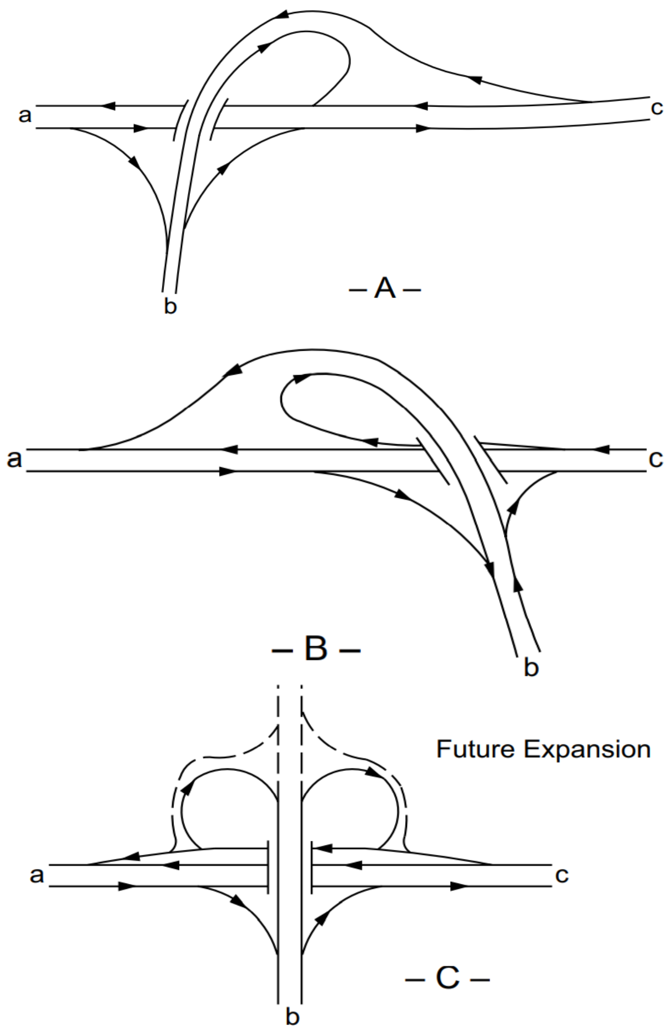


Figure 9.7 Three-Leg Interchanges with Single Structures

Figure 9.8 illustrates high-type T- and Y-interchanges that provide for all of the movements without loops, each with more than one structure or with one three-level structure. These configurations are more costly than single-structure configurations and are justified only where all movements are large.

In **Figure 9.8A**, all movements are directional, three structures are needed, and weaving is avoided. This plan is suitable for the intersection of a through freeway with the terminal of another major freeway. Some or all of the interchanging movements will need at least two-lane roadways. All entrances and exits are designed as branch connections or major forks, as discussed in **Section 9.7.2**. The alignment of this interchange may be adjusted to reduce the right-of-way needs, forming an interchange with only one three-level structure, as illustrated in **Figure 9.8B**.

Operationally, the configuration in **Figure 9.8A** might be superior to the configuration in **Figure 9.8B** because of the inherent sharp curvature on movement c-b in **Figure 9.8B.** While complete cost comparison involves a special analysis, there usually is little difference in cost. In some cases, the more complex three-level structure has been found to be less costly.

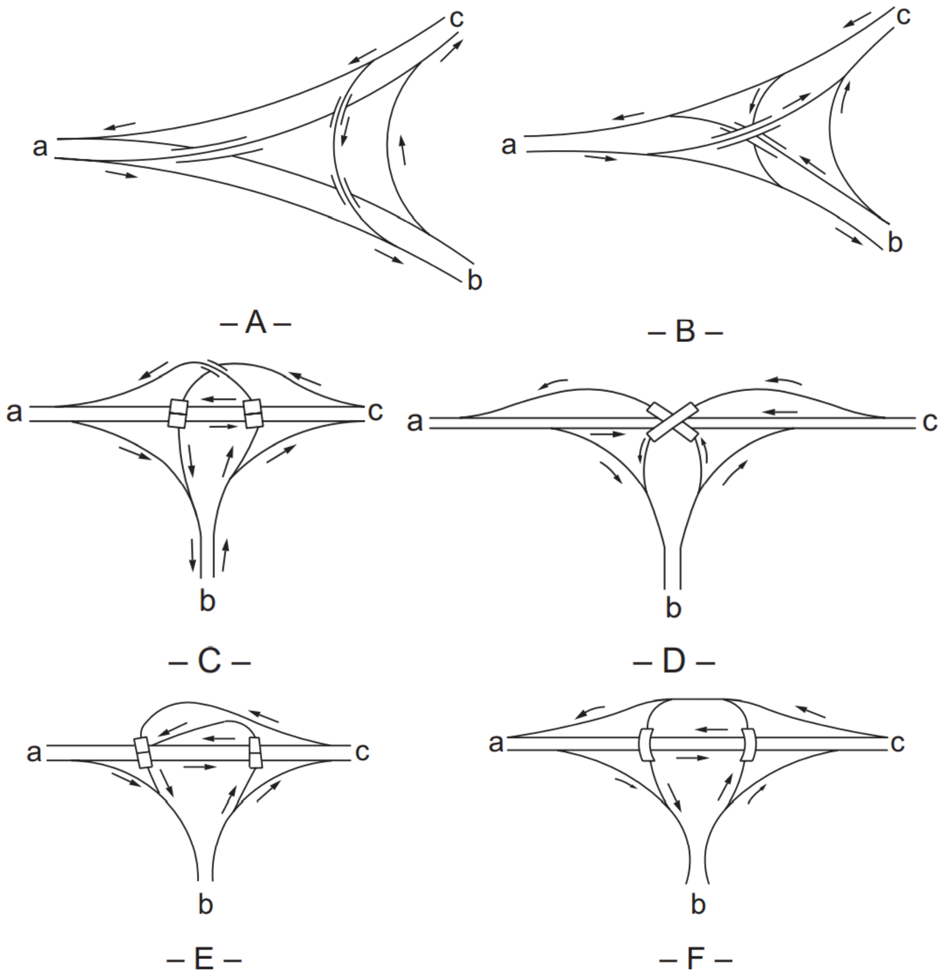


Figure 9.8 Three-Leg Interchanges with Multiple Structures

Figure 9.8C illustrates a three-leg interchange with a double jug-handle pattern. This pattern applies where it is appropriate to carry one of the freeways through the interchange with minimal deviation in alignment but where the intersecting radius is also considerably important. Interchanging traffic enters and exits the freeway on the right, and ramps are usually only single-lane roadways. This pattern involves the use of three structures, at least two of which span double roadways. As shown in **Figure 9.8D**, the basic pattern can be arranged so that the two left-turn ramps and the through roadway meet at a common point where a three-level structure replaces the three structures shown.

Figure 9.8E is another variation of the configuration in **Figure 9.8C** and **Figure 9.8D**. Separate roadways are provided for each left-turning movement with two two-level structures separating the ramps from the through movements. The grade separation structures should be spaced sufficiently far apart to permit the placement of the separate ramp, b-a, between them, thus avoiding the third structure of **Figure 9.8C**.

This design may be altered, as shown in **Figure 9.8F**. This arrangement provides smoother alignment on the ramps, but successful operation depends on provision of a weaving section that is suitably long for these two movements.

Figure 9.9 shows a trumpet interchange at the junction of an expressway/motorway and a major local road in a rural area. A unique feature of this configuration is that the local road overpasses one roadway of the expressway/motorway and underpasses the other because of the steep slope on the terrain. This pattern also explains the relatively sharp radius on the loop. The design favors the heavier traffic movement that is provided by the semidirect connection, and the loop handles the lighter volume.

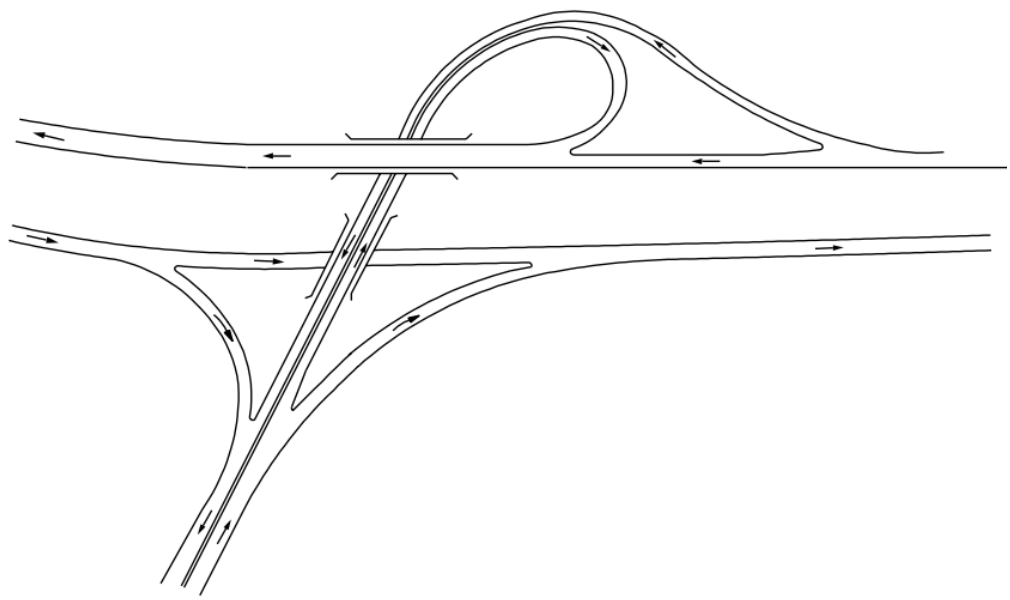


Figure 9.9 Three-Leg Interchange (T-Type or Trumpet)

Figure 9.10 shows an interchange between two expressway/motorway in a rural area. The directional design with large radii permits high-speed operation for all movements. The separation distance between major forks and the ramp terminals that follow should be sufficient to provide for smooth traffic operations. There are five separate structures in this configuration.

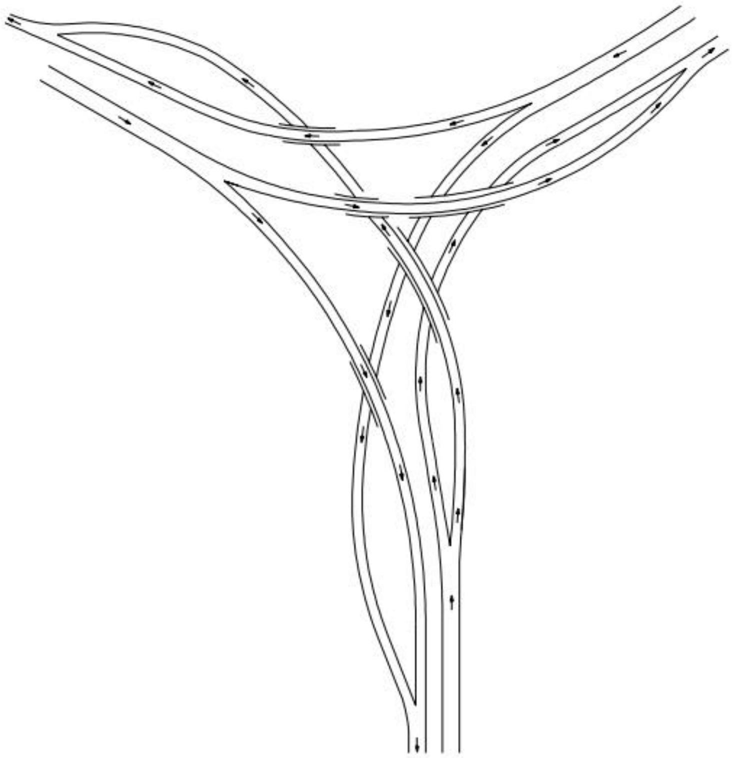


Figure 9.10 Three-Leg Interchange Directional Design

Plate 9.2 illustrates a trumpet-type interchange.

Plate 9.2 Trumpet Interchange (Mallam)

9.4.1.2 FOUR-LEG DESIGNS

Interchanges with four intersection legs may be grouped under six general configurations:

i. ramps in one quadrant,
ii. diamond interchanges,
iii. roundabout interchanges,
iv. single-point diamond interchanges (SPDIs),
v. diverging diamond interchanges,
vi. full or partial cloverleafs (including ramps in two or three quadrants), and
vii. directional interchanges.

9.4.1.2.1 RAMPS IN ONE QUADRANT

Interchanges with ramps in only one quadrant have application for an intersection of roadways with low traffic volumes. Where a grade separation is provided at an intersection because of topography, even though volumes do not justify the structure, a single two-way ramp of near-minimum design usually will suffice for all turning traffic. The ramp terminals may be simple T intersections. Appropriate locations for this type of interchange are very limited. A typical location would be at the intersection of an access to a national park and a two-lane rual roadway where turning movements are light, there is minimal large vehicle and trailer traffic, and the terrain and preservation of natural environment typically take precedence over providing additional ramps.

At some interchanges it may be appropriate to limit ramp development to one quadrant because of topography, culture, or other controls, even though the traffic volumes justify more extensive turning facilities. With ramps in only one quadrant, a high degree of channelization at the ramp terminals, at the median, and at the left-turn lanes on the through facilities is normally needed to control turning movements properly.

In some instances, a one-quadrant interchange may be constructed as the first step in a stage construction program. In this case, the initial ramps should be designed as a part of the ultimate development.

Figure 9.11A illustrates a one-quadrant interchange at the intersection of a rural minor arterial and an access road located in a rural mountainous area. The elongated shape of the ramp was determined largely by topography. Traffic entering both through roadways is under stop-sign control. Although traffic volumes are low, the turning traffic consists of a substantial proportion of the total volume.

Figure 9.11B is a one-quadrant interchange designed to function as an early phase of stage construction. On future construction, it is readily adaptable to become a part of a full or partial cloverleaf interchange without major renovation. The channelization, although elaborate, is conducive to reducing intersection conflicts and crashes and also to providing attractive landscaping.

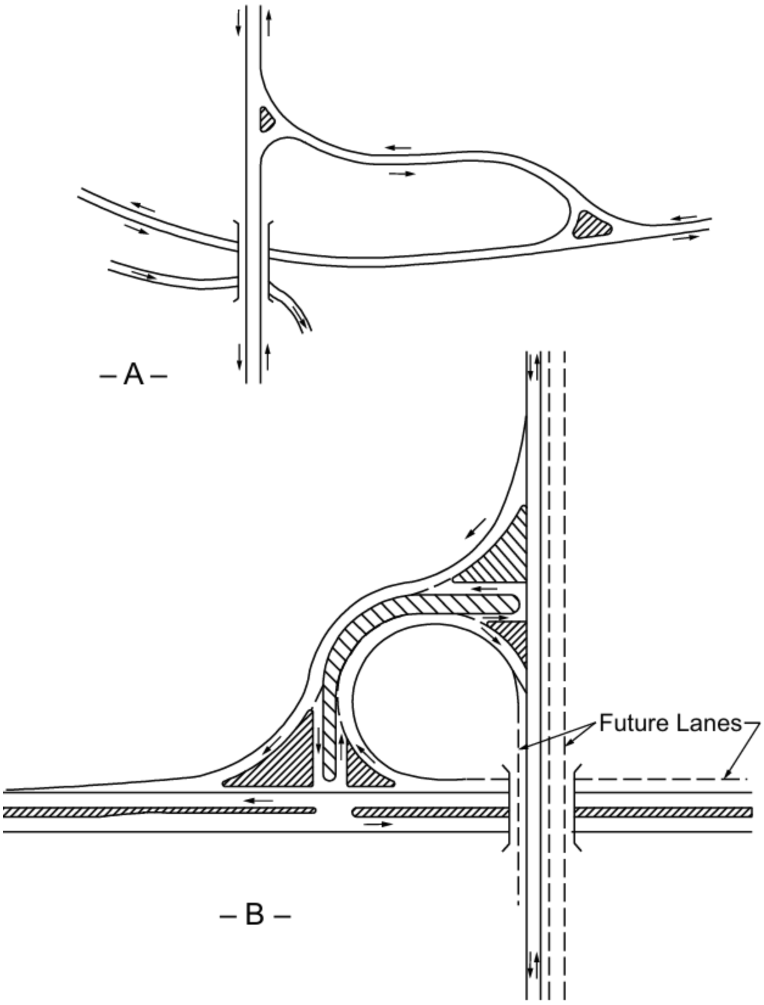


Figure 9.11 Four-Leg Interchanges, Ramps in One Quadrant

9.4.1.2.2 DIAMOND INTERCHANGES

The simplest and perhaps most common interchange configuration is the diamond. A full diamond interchange is formed when a one-way diagonal ramp is provided in each quadrant. The ramps are aligned with free-flow terminals on the major roadway, and the left turns at grade are confined to the crossroad. The diamond interchange has several advantages over a comparable partial cloverleaf:

viii. all traffic can enter and leave the major road at relatively high speeds,

ix. left-turning manoeuvres entail little extra travel, and

x. a relatively narrow band of right-of-way is needed, sometimes no more than that needed for the roadway alone.

Plate 9.3 shows a typical diamond interchange with some surrounding development.

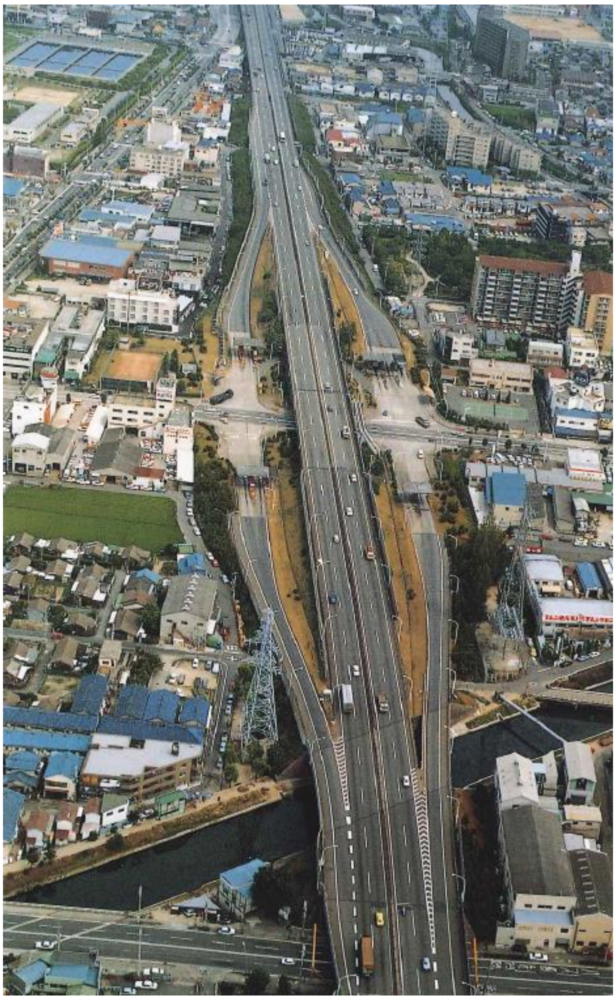


Plate 9.3 Typical Four -Leg Diamond Interchange (Amagasaki, Japan)

Diamond interchanges have application in both rural and urban areas. They are particularly adaptable to major-minor crossings where left turns at grade on the minor road are fitting and can be handled with minimal interference to traffic approaching the intersection from either direction. However, because these intersections have four legs, two of which are one-way, they present a challenge in traffic control to prevent wrong-way entry from the crossroad. For this reason, a median should be provided on the crossroad to facilitate proper channelization. While this median can be a painted median, a depressed or raised median with a sloping kerb is preferred. In most cases, additional signing to help prevent improper use of the ramps should be incorporated in the interchange design. Wrong-way entry concerns are further discussed in "Wrong-Way Entry" of **Section 9.4.3.13**.

Diamond interchanges usually need signalization where the crossroad carries moderate-to-large traffic volumes. The capacity of the ramps and that of the crossroad may be determined by the signal-controlled ramp terminals. In such a case, roadway widening may be needed on the ramps or on the crossroad through the interchange area, or both. While a single-lane ramp may adequately serve traffic from the expressway/motorway, it may have to either be widened to two or three lanes or be channelized for storage at the crossroad, or both, in order to provide the capacity needed for the at-grade condition. This design would prevent stored vehicles from extending too far along the ramps or onto the expressway/motorway. Left-turning movements in the most common diamond interchange configurations, as shown in **Plate 9.3**, usually need multiphase control.

Figure 9.12 through **Figure 9.14** illustrate a variety of diamond interchange configurations. These interchanges may be designed with or without frontage roads. Designs with frontage roads are common in built-up areas, often as part of a series of such interchanges along a expressway/motorway. Ramps should connect to the frontage road at a minimum distance of 100 m from the crossroad. Greater distances are desirable to provide adequate weaving length, space for vehicle storage, and turn lanes at the crossroad. **Figure 9.12C** is a spread diamond rural interchange with the potential for conversion to a cloverleaf.

In a diamond interchange, the greatest impediment to smooth operations is left-turning traffic at the crossroad terminal. Arrangements that may be suitable to reducing traffic conflicts are shown in **Figure 9.13** and **Figure 9.14**. By using a split diamond (i.e., each pair of ramps connected to a separate crossroad about a block apart), as shown in **Figure 9.13A**, conflicts are minimized by handling the same traffic at four rather than two crossroad intersections, reducing the left-turn movements at each intersection from two to one. A disadvantage with this arrangement is that traffic leaving the expressway/motorway cannot return to the freeway at the same interchange. Frontage roads (shown as dashed lines) are optional.

Figure 9.13B shows a split diamond in conjunction with a pair of one-way crossroad and one-way frontage roads. Simplicity of layout and operation of both the crossroad and the at-grade terminals result. Traffic leaving the expressway/motorway is afforded easy access to return to the freeway and continue the journey in the same direction.

Figure 9.13C shows a diamond interchange with frontage roads and separate turnaround provisions. These are highly desirable if the crossroad has heavy traffic volumes and there is considerable demand for the U-turning movement. The turnaround roadways are adjacent to the cross street with additional width provided beneath the structure or, if the cross street overpasses the expressway/motorway, on top of the structure. As an alternative, separate structures may be provided for the U-turn movements.

Figure 9.14 shows diamond interchanges with more than one structure. The layout in **Figure 9.14A** and the "criss-cross" arrangement in **Figure 9.14B** are sometimes dictated by topographic conditions or right-of-way restrictions. The operational performance of the interchanges in **Figure 9.14A** and **Figure 9.14B** are the same as those shown in **Figure 9.13A**.

The layout of **Figure 9.14B** also may be used to eliminate weaving between two closely spaced interchanges.

These layouts may be further modified by the use of one-way operation on the cross streets. The deficiency of both layouts in **Figure 9.14A** and **Figure 9.14B** is that traffic that has left the roadway cannot return directly to it and continue in the same direction. The spacing of the crossroads is determined primarily by grade constraints and acceleration and deceleration lengths.

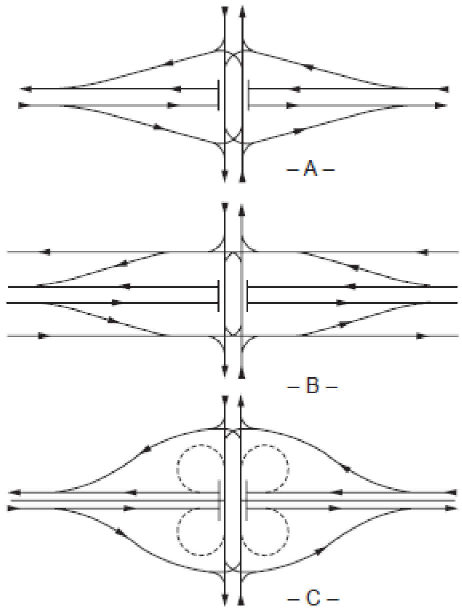


Figure 9.12 Diamond Interchanges, Conventional Arrangements

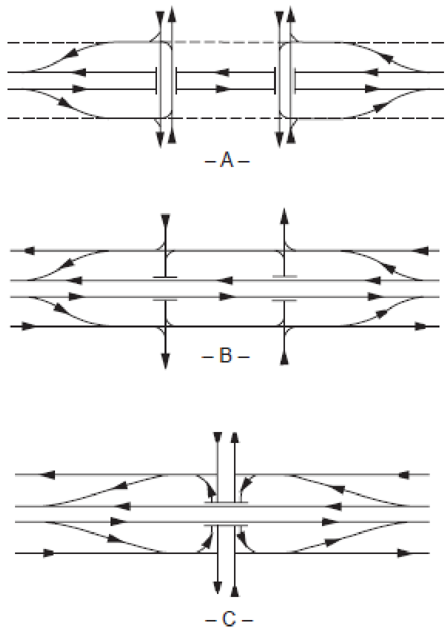


Figure 9.13 Diamond Interchange Arrangements to Reduce Traffic Conflicts

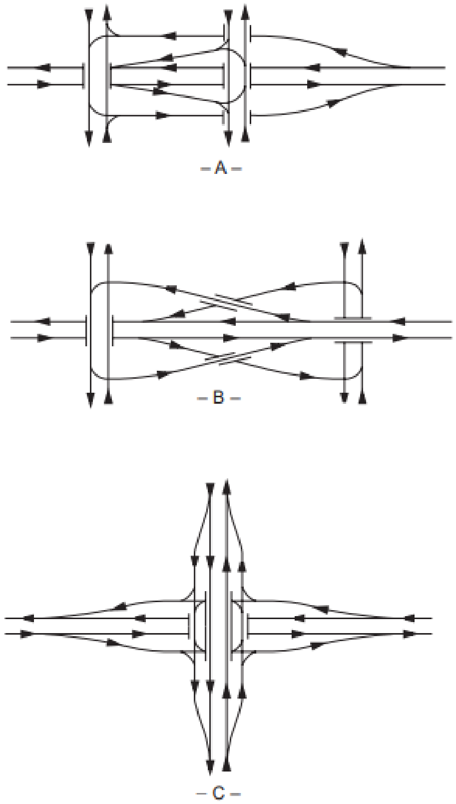


Figure 9.14 Diamond Interchanges with Additional Structures

The double or three-level diamond in **Figure 9.14C**, which has a third-level structure and four pairs of ramps, provides uninterrupted flow of through traffic on both of the intersecting roadways. Only the left turning movements cross at grade. This arrangement is applicable where the crossroad carries large traffic volumes and topography is favorable. The right-of-way needed is much less than that for other layouts having comparable capacity. Although large through and turning volumes can be handled, it is disadvantageous for intersections of two expressway/motorway in that some of the turning movements must either stop or slow down substantially. Signals are used in high-volume situations, and their efficiency is dependent on the relative balance in left-turn volumes. They are normally synchronized to provide continuous movement through a series of left turns once the area is entered.

Plate 9.4 presents an example of a diamond interchange configuration that is somewhat different from the conventional application—a three-level diamond interchange. In urban areas, where a crossing street carries a high volume of traffic, the three-level diamond interchange may be appropriate.

Plate 9.4 Three-Level Diamond Interchange (Ako Adjei)

It may be beneficial to consider the use of "X" pattern ramps at diamond interchanges in urban areas. With this ramp pattern, the entrance occurs prior to the intersection while the exit occurs after the crossroad. This configuration, as shown in **Figure 9.15**, can improve traffic flow characteristics for the through roadways around diamond interchanges. However, driver expectancy should be considered.

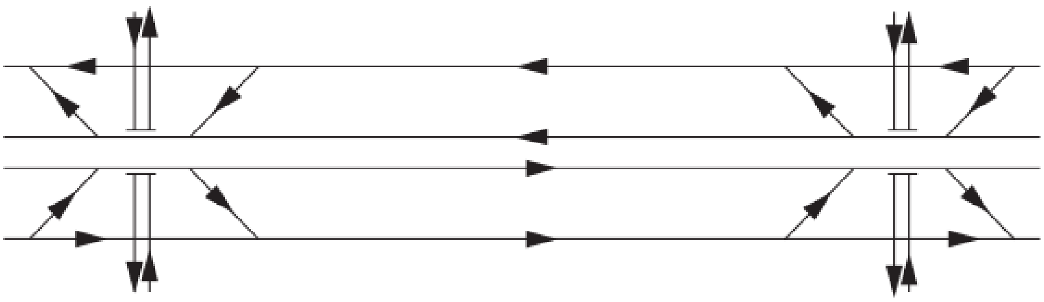


Figure 9.15 X-Pattern Ramp Arrangement

9.4.1.2.3 ROUNDABOUT INTERCHANGES

Plate 9.5 shows a diamond interchange with roundabouts at each crossroad ramp terminal. All through and turning movements on the crossroad and ramps are provided by using single-lane or multilane roundabouts. The design provides a narrower bridge (no storage turn lanes) and the elimination of signal control at the interchange. Consideration need to be given to the crossroad traffic volumes and expressway/motorway ramp volumes, when analysing the roundabout operations. Bicycle and pedestrian accommodation should be addressed taking into account the crossing locations and single-lane or multilane roundabouts. As with any form of interchange, grades should be taken into consideration. **Section 8.11** provides additional guidance on roundabouts.

Plate 9.5 Diamond Interchange with Roundabouts at the Crossroad Ramp Terminals

9.4.1.2.4 SINGLE-POINT DIAMOND INTERCHANGES

The single-point diamond interchanges (SPDIs) also known as the single-point urban interchanges (SPUIs) are typically characterized by narrow right-of-way, high construction costs, and greater capacity than conventional tight diamond interchanges. These interchanges can be constructed either with or without frontage roads. They are primarily suited for urban areas where right-of-way is restricted but may also be applicable to rural settings where it is undesirable to utilize adjacent right-of-way due to environmental, geographical, or other constraints.

SPDIs offer several advantages. These include construction in a relatively narrow right-of-way, resulting in potentially significant cost reductions. The primary operational advantage of this interchange configuration is that vehicles making opposing left turns pass to the left of each other rather than to the right, so their paths do not intersect. In addition, the right-turn movements from the exit ramps are typically free flow or yield control and only the left turns pass through the signalized intersection. As a result, a major source of traffic conflict is eliminated, increasing overall intersection efficiency, and reducing the traffic signal phasing needed from four-phase to three-phase operation. Since the SPDI has only one intersection, as opposed to two for a diamond interchange, the operation of the single traffic signal on the crossroad may result in reduced delay through the intersection area when compared to a diamond interchange.

The turning angle and curve radii for left-turn movements through the intersection are significantly flatter than at conventional intersections and, therefore, the left turns move at higher speeds. The left turn angle is typically 45 to 60 degrees with a minimum radius of 45m to 60m. The above-mentioned operations may result in a higher capacity than a conventional tight diamond interchange.

The primary disadvantage of SPDIs is high construction costs associated with bridges. Overpass SPDIs need long bridges to span the large intersection below. A two-span structure is not a design option because a centre column would conflict with traffic movements. Single-span overpass bridges are typically 65m in length, while three-span bridges often exceed 120m.

As shown in **Figure 9.16,** the SPDI underpass tends to be wide and often is “butterfly” in shape, resulting in high costs. Rectangular SPDI structures, while resulting in unused deck area, may provide additional area for maintenance of traffic and simplified construction. Where right-of-way is constrained, SPDIs typically utilize extensive retaining walls, further adding to the cost. However, the higher construction cost of SPDIs is often offset by the reduced right-of-way cost. **Figure 9.17** shows an underpass SPDI in restricted right-of-way.

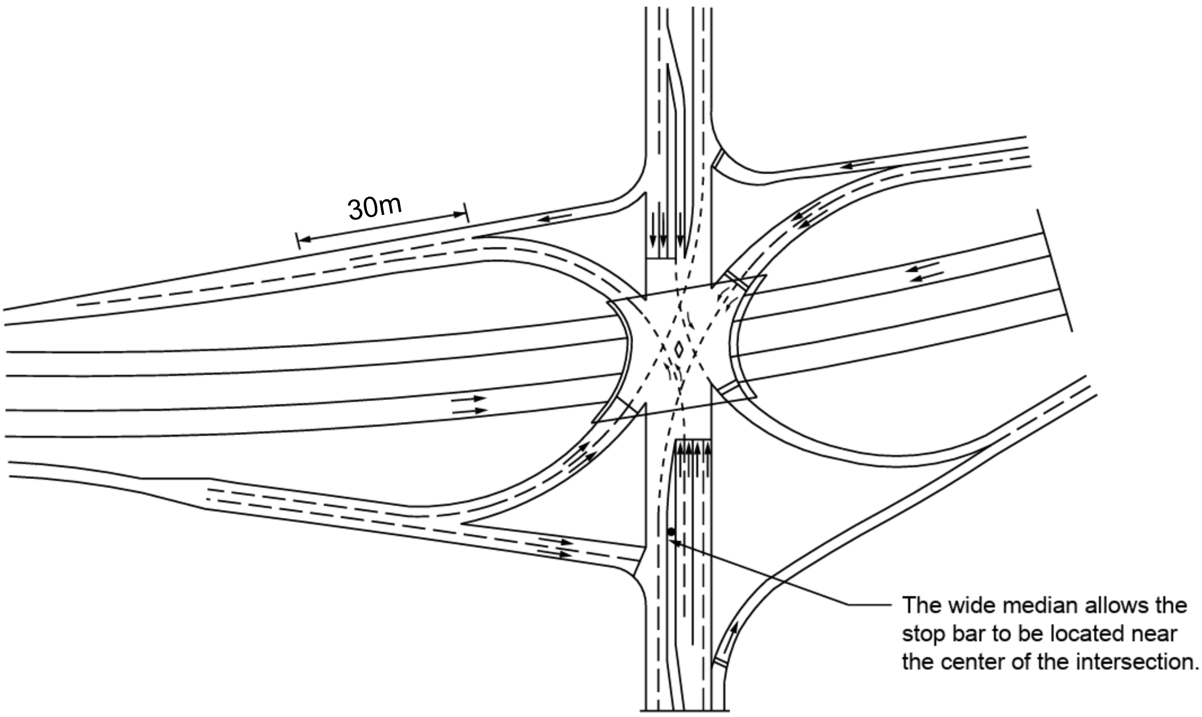


Figure 9.16 Underpass Single-Point Diamond Interchange

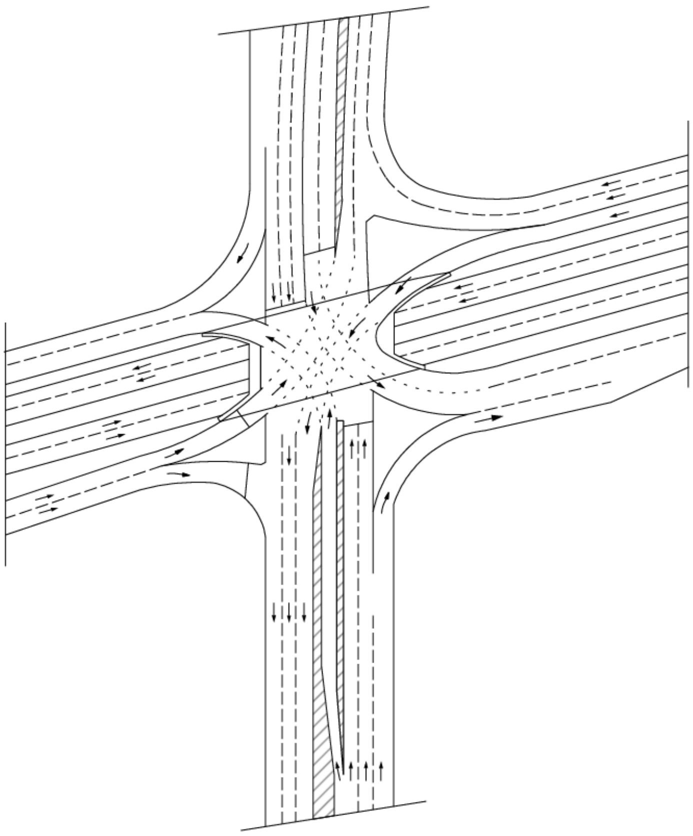


Figure 9.17 Typical SPDI Underpass Configuration in Restricted Right-of-Way

A second potential disadvantage of SPDIs is the length and geometry of the path for left-turning vehicles through the intersection. Like most typical intersections, left-turning vehicles pass to the left of opposing left-turning vehicles. However, due to the size and distance between opposing approaches, the path of left turning vehicles does not resemble a quarter of a circle found at typical intersections, but rather resembles a quarter of an ellipse. To provide positive guidance for this non-traditional path, various features have been developed. At a minimum, 0.6m dashed lane lines should be painted through the intersection.

A skew angle between the two roadway alignments has an adverse effect on SPDIs because it increases clearance distances and adversely affects sight distance. Severe skew in alignments may also increase the length of the bridge and widen the distance between the stop bars on the local roadss. Extreme care should be exercised in planning SPDIs when the skew angle approaches 30 degrees. It is important to provide visibility between exit ramp traffic and crossroad traffic approaching from the left. For left-turn movements from the main line's ramp to the crossroad, provide a clear cornering sight line with no obstructions from bridge abutments, pilasters, signal/light poles, signing, or landscaping.

Several basic design considerations can optimize the geometrics and operation of an SPDI:

i. It is desirable that the left-turn curve be a single radius. This will, however, typically result in additional right-of-way, a larger bridge structure, or both. Where it is not practical to provide a single radius and curves are compounded from a larger to a smaller radius, the second curve should be at least half the radius of the first.

ii. Provision of stopping sight distance on the left-turn movements equal to or exceeding the design speed for the curve radius involved.

iii. Provision of additional median width on the crossroad. The stop bar location on the crossroad is dependent on the wheel tracks from the opposing ramp left-turn movement (refer to **Figure 9.16**). By widening the median, the stop bar on the crossroad can be moved forward, thus reducing the size of the intersection and the distance each vehicle travels through the intersection. The results include greater available green time and less potential driver confusion due to an expansive intersection area.

iv. Provision of a minimum clear distance of 3.0m between opposing left turns within the intersection.

An SPDI with frontage roads, as illustrated in **Figure 9.18**, introduces additional considerations into the design. Frontage roads should be one-way in the direction of the ramp traffic. A slip ramp from the main line to the frontage road provides access to and from the intersection. This ramp should connect to the frontage road at least 200m, and preferably greater than 300m, from the crossroad. The traffic signal needs a fourth phase to provide through movements on the frontage roads. A free-flow U-turn movement may be desirable to expedite movements from one direction on the frontage road to the other. The combination of SPDIs and frontage roads may result in additional signal phases, increased intersection size, increased vehicle clearance times, and an impact on access control measures.

Because of the size, shape, and operational characteristics of SPDIs, pedestrian and bicycle movement through the intersection should be given careful consideration. Pedestrian crossing of the local road at ramp terminals typically adds a signal phase and uses considerable green time, resulting in reduced operational efficiency. Therefore, the overall design should include provision for pedestrian crossings at adjacent intersections instead of at the ramp terminal intersection. Pedestrian movements parallel to the local road are more readily handled. If, however, crosswalks are provided at ramps, they should be perpendicular to the ramp direction of travel and near to the local road. Perpendicular crosswalks minimize the length of the crossing and therefore minimize conflicting movements. Crosswalks located near the local road meet driver expectation and allow good sight distance to the pedestrian crossing. Crossing distances should be as short as practical. Design consideration for bicyclists should include provision of a direct route through the intersection and the development of right-turn channelization at SPDIs.

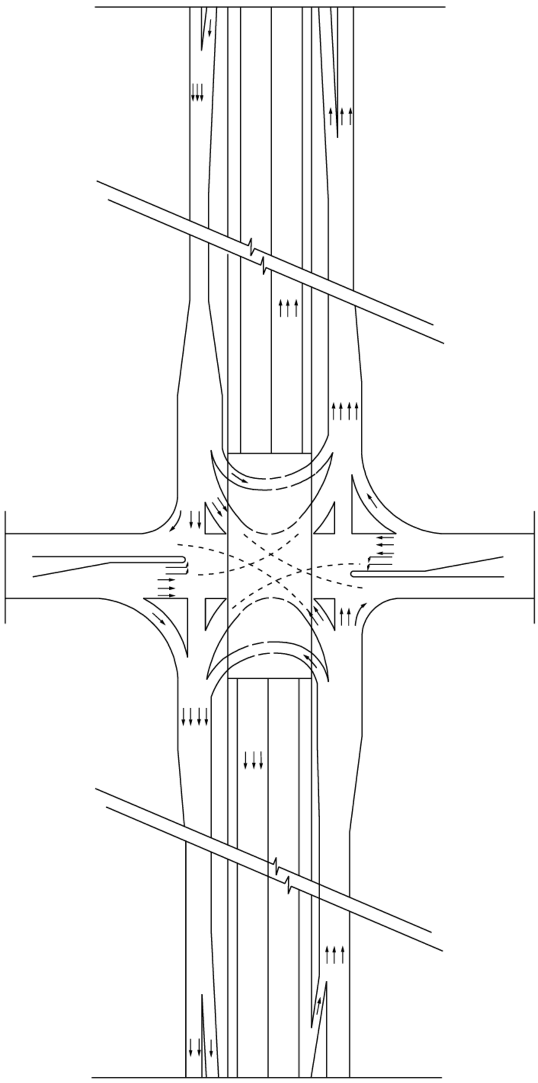


Figure 9.18 Overpass Layout for an SPDI with a Frontage Road and a Separate U-Turn Movement

Right-turn lanes at SPDIs are typically separated from the left-turn lanes, often by a considerable distance. The exit ramp right turn can be a free or controlled movement. The design of free right turns should include an additional lane on the crossroad beginning at the free right-turn lane for at least 60 m before being merged. Free-flow right turns from the exit ramp to an arterial crossroad are not desirable when the nearest intersection on the crossroad is within 150m, because there may be inadequate weaving distance between the exit ramp and the adjacent intersection. Heavy pedestrian traffic also can diminish the desirability of free right-turn lanes by adding a potential conflict with non-controlled vehicular traffic. Where the right-turn movement is controlled by a stop sign or traffic signal, adequate right-turn storage on the exit ramp should be provided to prevent blockage of vehicles turning left or traveling straight. Free-flow right turns on entrance ramps pose little operational concern, assuming adequate merge length is provided on the entrance ramp. As shown at the upper left portion of **Figure 9.16** the right-turn lane should extend at least 30m beyond the convergence point before beginning the merge. **Figure 9.19** is a single point urban interchange at the Tema roundabout.

Figure 9.19 Artistic impression of a SPDI (Tema roundabout)

9.4.1.2.5 DIVERGING DIAMOND INTERCHANGES

The Diverging Diamond Interchanges (DDI) uses directional crossover intersections to shift traffic on the cross street to the left-hand side between the ramp terminals within the interchange. Crossing the through movements to the opposite side replaces left-turn conflicts with same-direction merge/diverge movements and eliminates the need for exclusive left-turn signal phases to and from the ramp terminals. All connections from the ramps to and from the cross street are joined outside of the cross-over intersections, and these connections can be controlled by two-phase signals, have stop or yield control, or be free flowing.

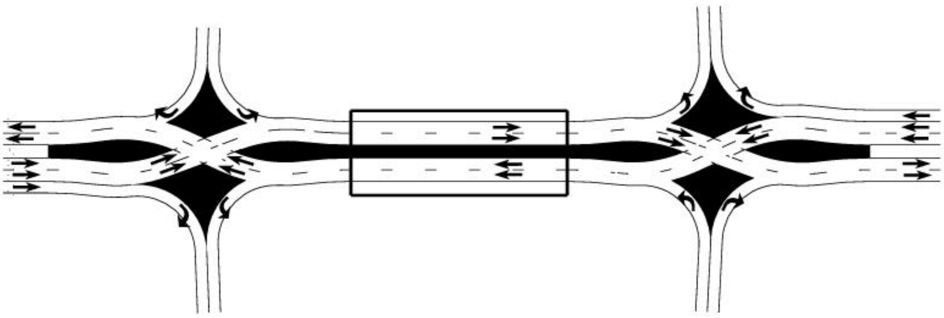
The DDI offers several advantages in comparison to a conventional diamond interchange. By allowing the ramp-terminal intersections to operate with simple, two-phase signal operations, the design provides flexibility to accommodate varying traffic patterns. The DDI design has significantly fewer vehicle-to-vehicle, vehicle-to-pedestrian, and vehicle-to-bike conflict points compared to a conventional diamond interchange. Left-turn volume capacity at a DDI is generally higher, and fewer and shorter signal phases are needed to accommodate both motorized and nonmotorized movements. Overall operations of a DDI may be greater compared to a conventional signalized diamond interchange due to shorter cycle lengths, reduced time lost per cycle phase, reduced stops and delay, and shorter queue lengths. The DDI also reduces the number and severity of conflict points for both motorized and nonmotorized users. The crossing distances for pedestrians are comparatively shorter, and usually involve traffic approaching from only one direction at a time. The cross-sectional characteristics of a DDI provide multiple options for facilitating convenient pedestrian and bicycle movements, and the geometry of the crossover intersections have an added benefit of reducing motorized vehicle speeds through the interchange, resulting in a traffic calming effect which may reduce crashes.

At an existing conventional diamond interchange where additional capacity is needed, it may be advantageous to convert the interchange into a DDI. Retrofitting to a DDI may be less costly than options involving widening the crossroad near the interchange (including widening the bridge) and adding additional lanes to the ramps. For new interchanges, the operational efficiency of a DDI may allow for a smaller structural footprint since fewer lanes are generally needed to accommodate the traffic demands. In some contexts, the DDI may allow for reduced right-of-way needs and construction costs compared to other interchange forms.

A DDI may be designed with the crossroad as either an underpass or overpass (see **Figure 9.20**), depending on site conditions. In some conditions it may be advantageous to use multiple structures at the grade separation, especially where the skew angle between facilities is significant. The spacing between ramp intersections is also a key consideration as this will impact signal design and operations on the crossroad corridor. Crossroads that are heavily skewed to the main facility typically need greater intersection spacing. On the other hand, very tight spacing between ramp intersections may constrain the design of the crossovers and limit queue storage and signal timing options.

– A – Underpass DDI
Source: Oregon DOT

– B – Overpass DDI
Source: Tennessee DOT



– C – DDI Diagram

Figure 9.20 Underpass and Overpass Diverging Diamond Interchanges

The proximity to a DDI of adjacent signalized intersections along the crossroad may impact the performance of the DDI at a given location. If an adjacent signal is too close and the queue storage length is inadequate, the traffic spillback may inhibit the movement of traffic along the crossroad, and potentially block traffic from the exit ramps. Modifications to adjacent signalized intersections along the crossroad may be necessary to maintain the overall signal progression along the corridor and reduce potential effects of queue spillback. Although this consideration is not unique to the DDI, the potential operational benefits of the DDI ramp intersections may be overshadowed by poor operational performance of nearby signals on the crossroad. Another operational consideration is that the DDI form does not accommodate typical “up and over” exit to entrance movements for oversized vehicles or authorized vehicles during maintenance or emergency situations.

Several key design elements of a DDI are interrelated, and the overall design should collectively consider the combinations of the related dimensions for application to specific sites. The appropriate choices for design elements such as design speed, reverse curve radii, lane widths, median widths, and other features will vary from one application to another.

Since the crossover area of a DDI tends to operate best at lower speeds, design speeds for crossover alignments should be in the range of 30 to 60 km/h, resulting in crossover radii in the range of 30 to 150 m depending upon chosen cross slope (which is typically in the range of plus or minus 2 percent). Along higher speed crossroads, it is appropriate to lower speeds in advance of the DDI crossover area with advance warning signs and geometric features. The reverse curves at the crossovers should have an appropriate combination of radius and length, as geometry that is too abrupt can make it difficult, especially for large vehicles, to maintain a natural driving path in their own lane. Providing a tangent alignment between the crossover intersections assists drivers in maintaining the desired vehicle tracking and the curve-tangent-curve sequence promotes driving at the desired target speed. Using an alignment that provides approximately 15 to 30 m of tangent between sets of reversing curves through the crossover is recommended to provide positive guidance through the crossover intersections. The considerations for designing curvature radii at the exit and entrance ramp movements are similar to other interchange forms and include the turning path of the design vehicle, sight distance needs, pedestrian and bicycle crossing conditions, and intersection traffic control type.

In addition to selecting appropriate combinations of crossover radii for the reversing curves and the tangent length between them, the crossover angle is a design element that needs consideration of the trade-offs involved. The crossing angle is the acute angle between lanes of opposing traffic within the crossover based on the tangent sections or lines perpendicular to the radii at points of reverse curvature. The greater the crossover angle, the more the crossover will appear like a “normal” intersection of two different cross routes and decrease the likelihood of a driver making a wrong-way movement. However, greater crossing angles generally result in larger footprints and may be constrained in a DDI retrofit of an existing interchange. Also, larger crossing angles in combination with sharp reverse curves can increase the potential for overturning of vehicles with high centres of gravity and excessive driver discomfort through the crossovers. The recommended approach is to attain the largest crossing angle possible that is in balance with the other geometric parameters and site constraints. The crossover angle of a DDI is generally between 30 to 50 degrees. Crossover angles less than 30 degrees may increase the potential for wrong-way movements. Additional features, such as supplemental signs and pavement markings, should be used at a DDI to minimize the likelihood of a wrong-way movement.

Appropriate lane widths along the crossroad of a DDI typically range from 3.6 to 4.6 m depending on site location and consideration for design vehicles traveling side by side through the crossover area. Tapering to provide wider lane width typically occurs prior to and after the crossover curves. The additional lane width is typically not continued between the two crossovers. Shoulders may or may not be present along the crossroad leading to a DDI. As with other interchange forms, designs should reduce the potential for wrong-way manoeuvres. For most interchange configurations the outside shoulder is typically used for emergency response. Due to the crossover design, the inside, as opposed to the outside shoulder should be wide enough for emergency vehicle accommodation. Marked bicycle lanes may be provided along the crossroad to the right side of traffic through a DDI. Bicycle movements through a DDI are similar to motor vehicle traffic in that they perform the same crossover movements as other vehicles. If bicycles are legally permitted to use the limited access facility, the turn movements to enter or exit the limited access facility may be served by the interchange ramps.

In some situations, it is advantageous to add auxiliary lanes in advance of the crossover to reduce lane changing between ramps. Lanes added or dropped in the interchange area may take various forms. These can be lanes dedicated for left or right turns, shared through/left lanes, or exclusive through lanes. When lanes are added in advance of crossovers, together with overhead signs, it allows drivers to select appropriate lanes ahead of time, reducing lane changes and confusion through the middle of the interchange. When lanes are dropped within the interchange or beyond the outbound crossover, it should be done at a place that drivers can easily recognize; such as left turns onto the entrance ramp or a lane reduction beyond the crossover area.

Alternative methods for developing the directional alignments on the crossroad through a DDI exist. The considerations for determining which method is most suitable for a specific site include: the desire to minimize the cross-section under or over a bridge (common in retrofit situations); minimizing the distance between crossovers or matching existing ramp spacing on the crossroad (new and retrofit situations); minimizing the amount of reverse curvature and/or right-of-way at crossovers; and constructability issues. Methods that are typically used to develop the crossover include: symmetrical alignments using reversed curves, often used in retrofit situations; offset alignment, where one direction is held basically as-is and the other is deflected with appropriate combinations of curves; and shifted alignments, where both directional alignments move sideways – often to avoid a specific impact or facilitate staged construction.

Other design elements that should be taken into consideration at DDIs include sight distance for both the crossover intersections and ramp terminals, signalization of certain ramp movements, and signing and pavement markings. Sight distance at DDIs is important for both vehicles manoeuvring through the crossovers or turning left and right from the ramp terminals onto the cross street, especially when the turning ramp terminal traffic is under yield control. Typical DDI design includes a concrete median between the signalized crossovers that can be used for pedestrians, while raised islands are typically used in the ramp terminal areas. Sight distance for right turning vehicles from the ramp terminal should be reviewed so that oncoming cross over traffic and pedestrians in the median are adequately seen. Visual obstructions created by bridges, signal and illumination poles, signing, landscaping, and other potential objects should be considered when determining available cornering sight distance.

The DDI form offers excellent opportunities to integrate multimodal facilities into an interchange. It is possible to integrate pedestrian facilities along the outside of the crossroad through lanes and in the median between the signalized ramp terminals. Designing a DDI with a centre walkway minimizes the overall number of conflict points, including accelerating conflicts, while providing full access of pedestrians to also cross the arterial street. With a centre walkway, vehicular left turns to the entrance ramps can be made freely without conflict with pedestrian crossings. Lines of sight to and from the pedestrian crossings may also be improved by using a centre walkway. Pedestrian facilities on the outside of the crossroad are also possible but require pedestrians to cross both left and right turns from the ramp terminals. Regardless of the crossing strategy used, the channelization at a DDI for right- and left-turns to and from the crossroad presents an opportunity to utilize pedestrian-focused design choices through the use of appropriate curve radii and refuge areas for multi-stage pedestrian crossings.

A disadvantage of the DDI design is the inability to route oversized trucks or bus rapid transit from the exit ramp directly through the intersection and onto the entrance ramp. Consideration of oversize loads and bus transit stops is key when evaluating the DDI as an optional interchange form.

9.4.1.2.6 CLOVERLEAFS

Cloverleafs are four-leg interchanges that employ loop ramps to accommodate left-turning movements. Interchanges with loops in all four quadrants are referred to as “full cloverleafs” and all others are referred to as “partial cloverleafs.” A full cloverleaf may not be warranted at major-minor crossings where, with the provision of only two loops, freedom of movement for traffic on the major road can be maintained by confining the direct at-grade left turns to the minor road. The principal disadvantages of the cloverleaf are the additional travel distance for left-turning traffic, the weaving manoeuvre generated, the very short weaving length typically available, and the relatively large right-of-way areas needed. When collector distributor roads are not used, further disadvantages include weaving on the main line, the double exit on the main line, and difficulties in placing signing for the second exit. Because cloverleafs are considerably more expansive than diamond interchanges, they are less common in urban areas and are better adapted to suburban or rural areas where space is available.

The advantages of increased speed should be weighed against the disadvantages of increased travel time, distance, and right-of-way. It should also be noted that large vehicles and trailers may not be able to operate as efficiently on smaller radii curves. Considering all factors, experience shows that the practical size of loops resolves into approximate radii of 30 to 50 m for minor movements on roadways with design speeds of 80 km/h or less and 50 to 75 m for more important movements on roadways with higher design speeds. A continuous additional lane is needed for deceleration, acceleration, and weaving between the on- and off-loop ramps. Additional structure width or length is usually needed for this lane.

The cloverleaf involves weaving manoeuvres as discussed in “Weaving Sections” of **Section 9.4.3.10**. The presence of weaving manoeuvres is not objectionable when the left-turning movements are relatively light, but when the sum of traffic on two adjoining loops approaches about 1,000 vph, interference mounts rapidly, which results in a reduction in speed of through traffic. When the weaving volume in a particular weaving section exceeds 1,000 vph, the quality of service on the main facility deteriorates rapidly, thus generating a need to transfer the weaving section from the through lanes to a collector-distributor road. A loop rarely operates with more than a single line of vehicles, regardless of the roadway width, and thus has a design capacity limit of 800 to 1,200 vph, the higher figure being applicable only where there are no large vehicles and trailers, and where the design speed for the ramp is 50 km/h or higher. Loop ramp capacity is, therefore, a major control in cloverleaf designs.

Loops may be made to operate with two lanes abreast, but only by careful attention to design of the terminals and design for weaving, which would need widening by at least two additional lanes through the separation structure. To accomplish this type of design, the terminals should be separated by such great distances and the loop radii should be made so large that cloverleafs with two-lane loops generally are not economical from the standpoint of right-of-way, construction, cost, and amount of out-of-direction travel. Loops that operate with two lanes of traffic, therefore, are considered exceptional cases.

Where no direct left turns are permitted on either the main facility or the crossroad, but all turning movements are to be accommodated, a four-quadrant cloverleaf interchange is the minimum interchange configuration that will suffice. When a full cloverleaf interchange is used in conjunction with a expressway/motorway and the sum of the traffic on two adjoining cloverleaf loops approaches about 1,000 vph, collector-distributor roads should be considered. Collector-distributor roads are generally not cost-effective where the ramp volumes are low and are not expected to increase significantly. The use of acceleration or deceleration lanes with cloverleaf interchanges is one possible alternative to collector-distributor roads.

Plate 9.6 shows an existing partial cloverleaf interchange at Apenkwa. **Plate 9.7** and **Plate 9.8** shows a full cloverleaf interchange between two major arterials.

Plate 9.6 Partial Cloverleaf Interchange (Apenkwa)

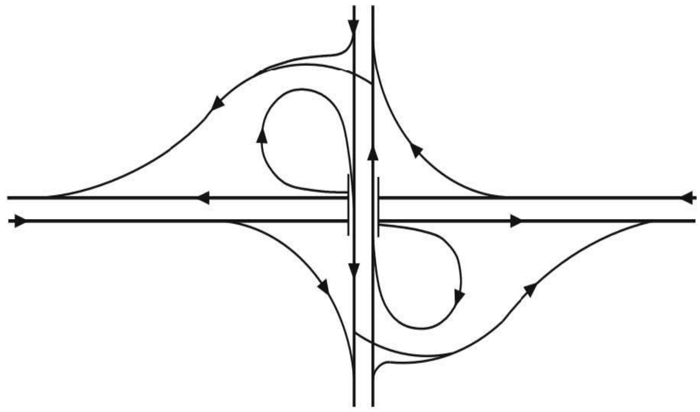
Plate 9.7 Full Cloverleaf (Sofo-line)

Plate 9.8 Full Cloverleaf (Tetteh Quarshie)

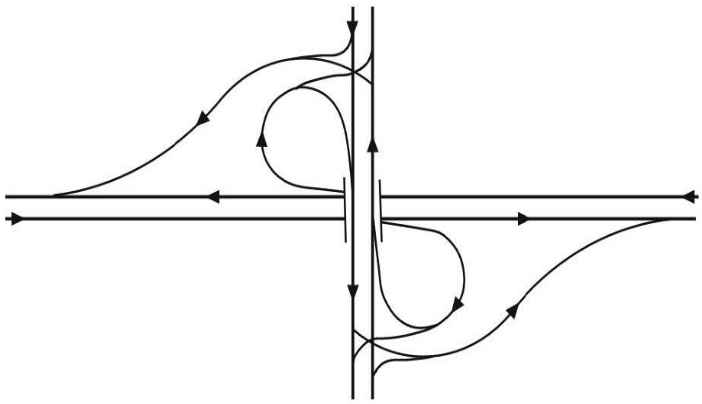
9.4.1.2.7 PARTIAL CLOVERLEAF RAMP ARRANGEMENTS

Par-Clo interchanges derive their name as a contraction of PARtial CLOverleaf, mainly because of their appearance, but also because they were frequently a first stage development of a

Cloverleaf Interchange. These interchanges are preferred where there are difficulties to obtain land in some quadrants of interchanges i.e., Diamonds cannot be used. Three configurations of Par-Clo Interchange are possible: the Par-Clo A (**Figure 9.21**), the Par-Clo B (**Figure 9.22**) and the Par-Clo AB (**Figure 9.23**). The letters have the significance of the loops being in advance (A) of or beyond (B) the structure. The Par-Clo AB configuration has the loop in advance of the structure for the one direction of travel and beyond the structure for the other. In all cases, the loops are on opposite sides of the main road.

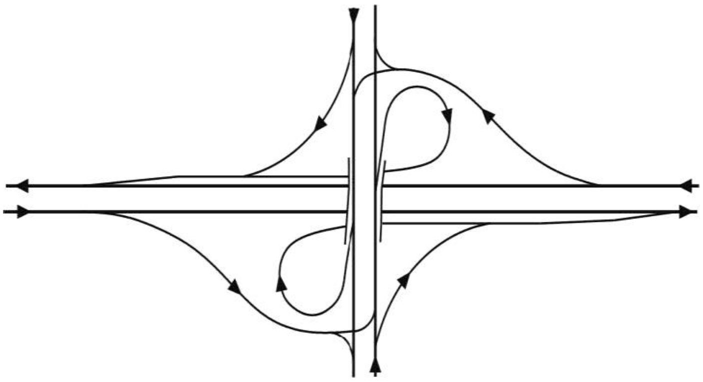


A4 (the two (2) exits from the major road should be on a C-D road)

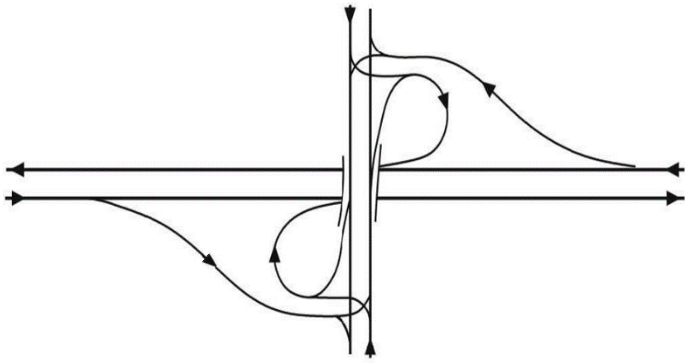


A2

Figure 9.21 Par-Clo A Interchange



B4



B2

Figure 9.22 Par-Clo B interchanges

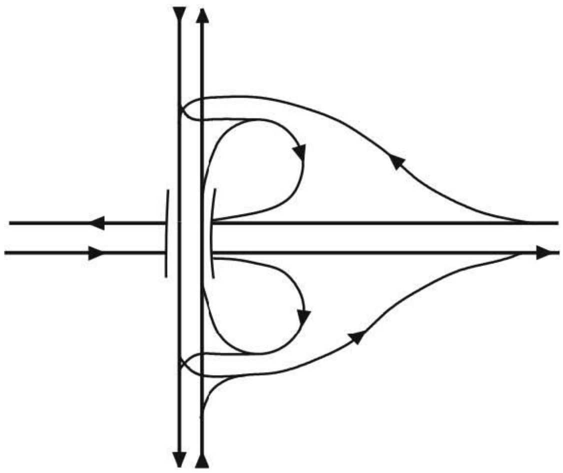


Figure 9.23 Par-Clo AB interchange

Both the Par-Clo A and the Par-Clo B have alternative configurations: the A2 and A4 and the B2 and B4.

Internationally, the Par-Clo B4 is generally regarded as being the preferred option for an interchange between an expressway/motorway and a heavily trafficked arterial. In the first instance, the loops serve vehicles entering the expressway/motorway whereas, in the case of the Par-Clo A, the high-speed vehicles exiting the expressway/motorway are confronted by the loop. This tends to surprise many drivers and loops carrying exiting traffic have higher accident rates than the alternative layout. Secondly, the left turn from the crossing road is remote from the intersections on the crossing road and the only conflict is between right-turning vehicles exiting from the expressway/motorway and through traffic on the crossing road. This makes two-phase signal control possible.

The Par-Clo AB is particularly useful in the situation where there are property or environmental restrictions in two adjacent quadrants on the same side of the crossing road.

In the design of partial cloverleafs, the site conditions may offer a choice of quadrants to use. However, at a particular interchange site, topography and culture may be the factors that determine the quadrants in which the ramps and loops can be developed. There is considerable operational advantage in certain arrangements of ramps. These are discussed and summarized in the following analysis. Ramps should be arranged so that the entrance and exit turns create the least impediment to the traffic flow on the major roadway. The following guidelines should be considered in the arrangement of the ramps at partial cloverleafs:

i. The ramp arrangement should enable major turning movements to be made by right-turn exits and entrances.

ii. Where through-traffic volume on the major road is decidedly greater than that on the intersecting minor road, preference should be for an arrangement that places the right turns (either exit or entrance) on the major road, even though this results in a direct left turn off the crossroad.

These controls do not always lead to the most direct turning movements. Instead, drivers frequently may need to first turn away from or drive beyond the road that is their intended destination. Such arrangements cannot be avoided if the through-traffic movements, for which the separation is provided, are to be facilitated to the extent practical.

Figure 9.24 illustrates the manner in which the turning movements are made for various two- and three quadrant cloverleaf arrangements. When ramps in two quadrants are adjacent and on the same side of the minor road, as shown in **Figure 9.24A** and **Figure 9.24B**, or diagonally opposite each other, as shown in **Figure 9.24E** and **Figure 9.24F**, all turning movements to and from the major road are accomplished by right turns. Any decision between the arrangement in **Figure 9.24A** and its alternate arrangement (ramps in the other two quadrants) will depend on the predominant turning movements or the availability of right of-way, or both. When the ramps in two quadrants are adjacent but on the same side of the major road (**Figure 9.24B** and **Figure 9.24D**), four direct left turns fall on the major road. This arrangement and its alternate are the least desirable of the six possible arrangements, and their use should be avoided.

The arrangement with ramps in diagonally opposite quadrants is advantageous because the turning movements in both directions in the quadrants that contain the ramps are made by desirable right-turn exits and entrances. At interchanges where turning movements in one quadrant predominate, the best two quadrant arrangement has ramps in that quadrant and in the quadrant diagonally opposite. Where turning movements in two adjacent quadrants are of nearly the same importance, arrangements shown in **Figure 9.24A**, **Figure 9.24E**, and **Figure 9.24F** are applicable in that all turns to and from the major road are on the right. However, the arrangement in **Figure 9.24E** is preferable because the ramps are on the near side of the structure as drivers approach on the major road. With this plan, it may be practical to provide for high-speed turns from the major road, and drivers desiring to turn are not confused by ramps that may be hidden by the structure, as shown in **Figure 9.24F**.

There are four possible arrangements for ramps in three quadrants, including the arrangement in **Figure 9.24G** and the alternate arrangements in which each of the other three quadrants has no ramps. In an arrangement with ramps in three quadrants, six of the eight turning movements can be made by right-turn exits and entrances, and the other two are made by right turns on the major road and corresponding left turns on the minor road. The determination of which quadrant is to be without ramps is usually dependent on the availability of right-of-way and the predominant turning movements to be handled.

In some cases, it is desirable to provide diagonal ramps in all four quadrants, but with loops in one, two, or three of the quadrants. **Figure 9.24H** shows a design with loops in diagonally opposite quadrants. This design has the advantage of providing all right exits. Storage of vehicles waiting to make the left turn at the at-grade intersections occurs on the ramp and not on either of the through roads. In addition, there is no weaving on the major road.

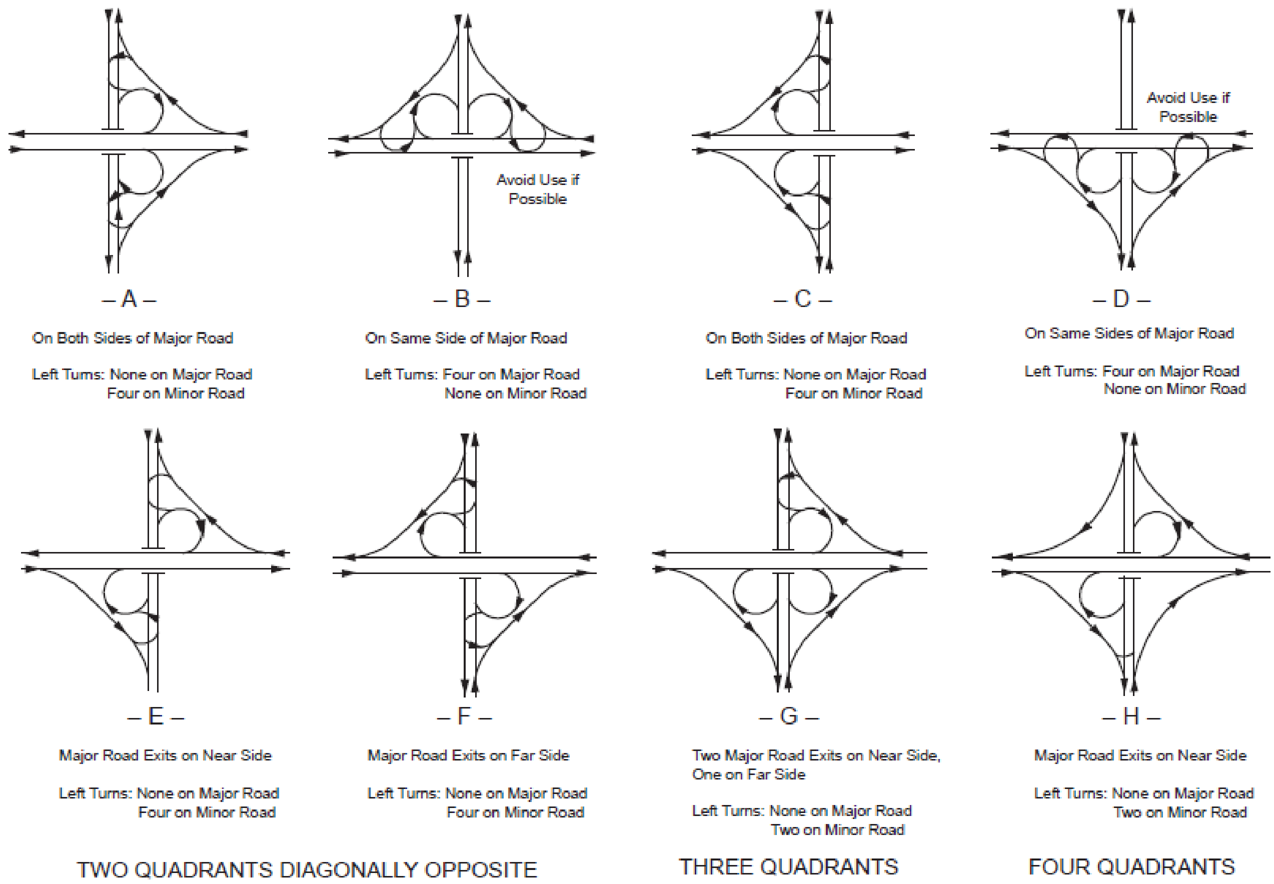


Figure 9.24 Schematic of Partial Cloverleaf Ramp Arrangements, Exit and Entrance Turns

9.4.1.2.8 DIRECTIONAL INTERCHANGES

Direct or semidirect connections are used for important turning movements to reduce travel distance, increase speed and capacity, eliminate weaving, and avoid the need for out-of-direction travel in driving on a loop. Higher levels of service can be realized on direct connections and, in some instances, on semidirect ramps because of relatively high speeds and the likelihood of better terminal design. Often a direct connection is designed with two lanes. In such cases, the ramp capacity may approach the capacity of an equivalent number of lanes on the through road. In rural areas, there rarely is a volume justification for provision of direct connections in more than one or two quadrants. The remaining left-turning movements usually are handled satisfactorily by loops or at-grade intersections. At least two structures are needed for such an interchange.

There are many possible arrangements with direct and semidirect connections, but only the more basic arrangements are discussed herein.

A direct connection is defined as a ramp that does not deviate greatly from the intended direction of travel. Interchanges that use direct connections for the major left-turn movements are directional interchanges. Direct connections for one or all left-turn movements would qualify an interchange to be also considered directional even if the minor left-turn movements are accommodated on loops. Direct connections are generally designed with higher design speeds than semidirect connections.

A semidirect connection is defined as a ramp where the driver exits to the right first, heading away from the intended direction of travel, gradually reversing, and then passing around other interchange ramps before entering the other road. Semidirect connections for one or all left-turn movements also qualify an interchange as directional even if the minor left-turn movements are accommodated on loops.

Semidirect or direct connections for one or more left-turning movements are often appropriate at major interchanges in urban areas. In fact, interchanges involving two express/motorway almost always need directional layouts. In such cases, turning movements in one or two quadrants often are comparable in volume to through movements. In comparison to loops, direct or semidirect connections have shorter travel distance, higher speeds of operation, a higher level of service, and they often avoid the need for weaving.

There are many configurations for directional interchanges that use various combinations of direct and semidirect connections, and loop ramps. Any one of them may be appropriate for a certain set of conditions, but only a limited number of patterns are generally used. The most common configurations fill the least space, have the fewest or least complex structures, minimize internal weaving, and fit the common terrain and traffic conditions. Basic patterns of selected directional interchanges are illustrated in **Figure 9.25** through **Figure 9.27**, with distinctions made as to configurations with and without weaving.

A. WITH LOOPS AND WEAVING

Common arrangements where turning movements in one quadrant predominate are shown in **Figure 9.25A** and **Figure 9.25B**. The predominant turning movement bypasses the central portion of the interchange via semidirect or direct ramps. The minor turning movements pass through weaving sections between loops on each roadway. In both figures, direct and semidirect connections are used without affecting the alignment of the intersecting roadways. Both arrangements involve three structures, and the area occupied is about the same as or somewhat greater than a full cloverleaf.

The efficiency and capacity of all the layouts shown in **Figure 9.25** may be improved by eliminating weaving on the main roadways through the use of a collector–distributor road, as shown dashed in **Figure 9.25**.

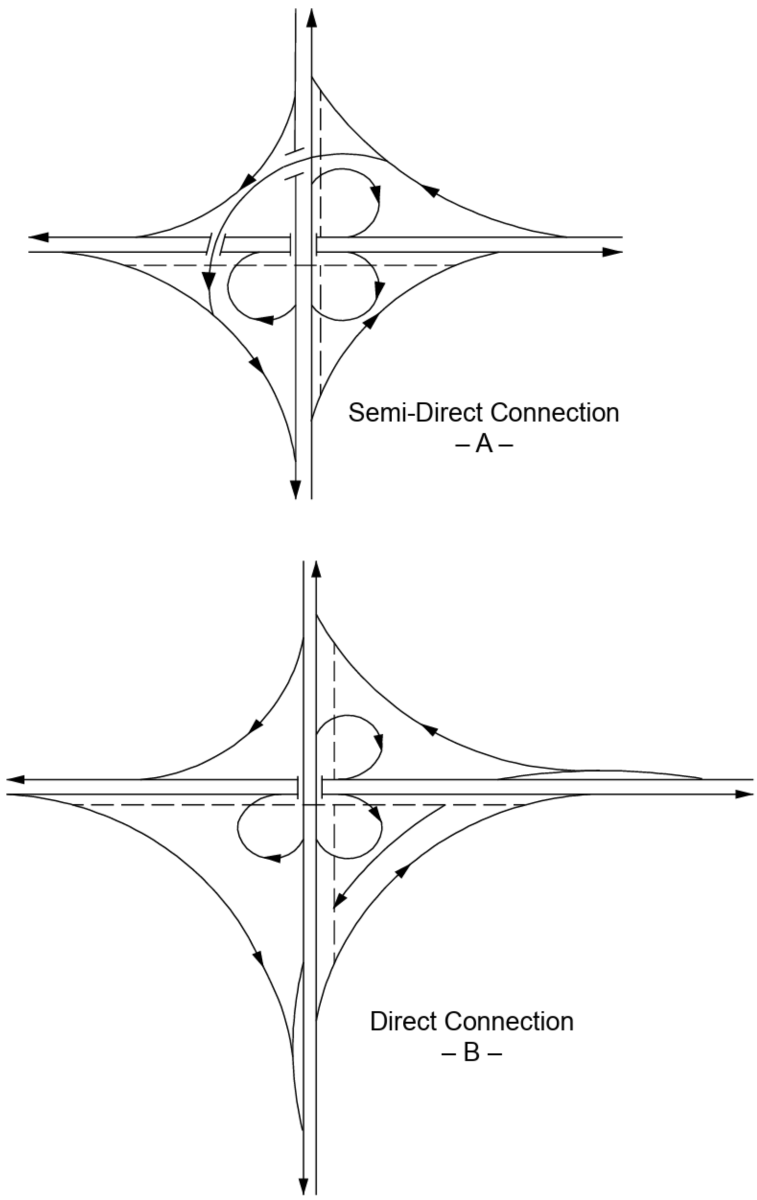


Figure 9.25 Directional Interchanges with Weaving Areas

B. WITH LOOPS AND NO WEAVING

Directional interchanges that do not involve weaving but include loops are shown in . The through lanes do not need to be spread apart for any of these configurations; however, four or more structures are needed. Single exits on the right side along with right-hand entrances enhance the operational characteristics of these designs.

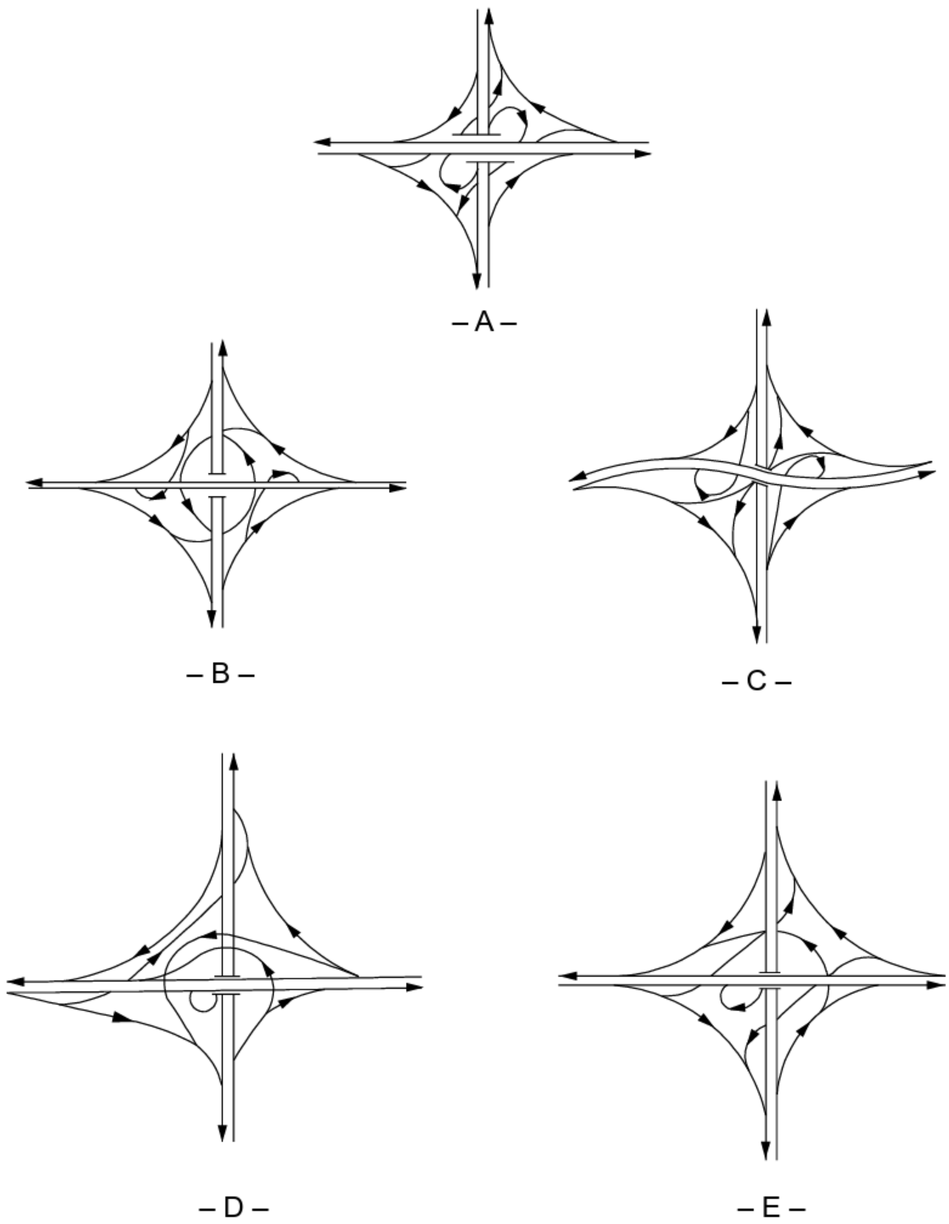


Figure 9.26 Directional Interchanges with no Weaving

C. FULLY DIRECTIONAL

Fully directional interchanges are generally preferred where two high-volume expressway/motorway intersect. Since traffic movements between the two express/motoway are free-flow with this interchange configuration, there are no at-grade intersections, only direct or semidirect ramp connections from one express/motoway to the other. Fully directional interchanges are costly to construct due to the increased number and length of ramps and the increased number of bridge crossings, but they offer high-capacity movements for both through and turning traffic with comparatively little additional area needed for construction.

The configuration and design of each interchange is uniquely based on the traffic volumes and patterns, environmental considerations, costs, etc. As a result, detailed and time-consuming studies are usually needed for each interchange and should include a study of all likely alternatives. **Figure 9.27A** through **Figure 9.27C** show diagrammatic layouts of a fully directional interchanges.

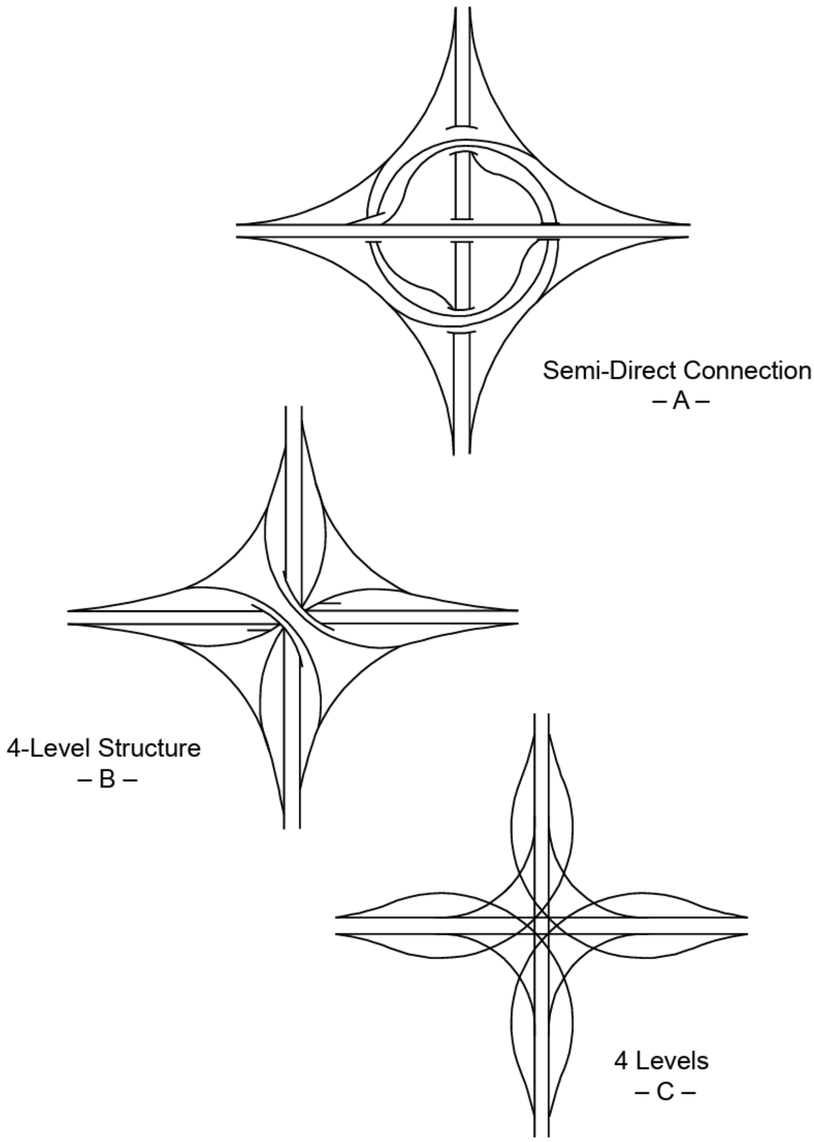


Figure 9.27 Directional Interchanges with Multilevel Structures

Weaving, left-side exits, and left-side entrances are undesirable within directional interchanges; however, there may be instances where they cannot be reasonably avoided because of site restrictions or other considerations.

With heavy left-turn movements, the terminals should be designed as major forks and branch connections, as covered in **Section 9.4.5.9**. The most widely used directional interchange configuration is the four-level layout shown in **Figure 9.27B**.

A variation of this configuration is the four-level interchange with two exits from both major roadways, as shown in **Figure 9.27C**. **Figure 9.28** shows a diagram of an existing interchange between two high-volume expressway/motorway in a suburban area. Other examples of directional interchanges are shown in **Plate 9.9, Plate 9.10** and **Plate 9.11**.

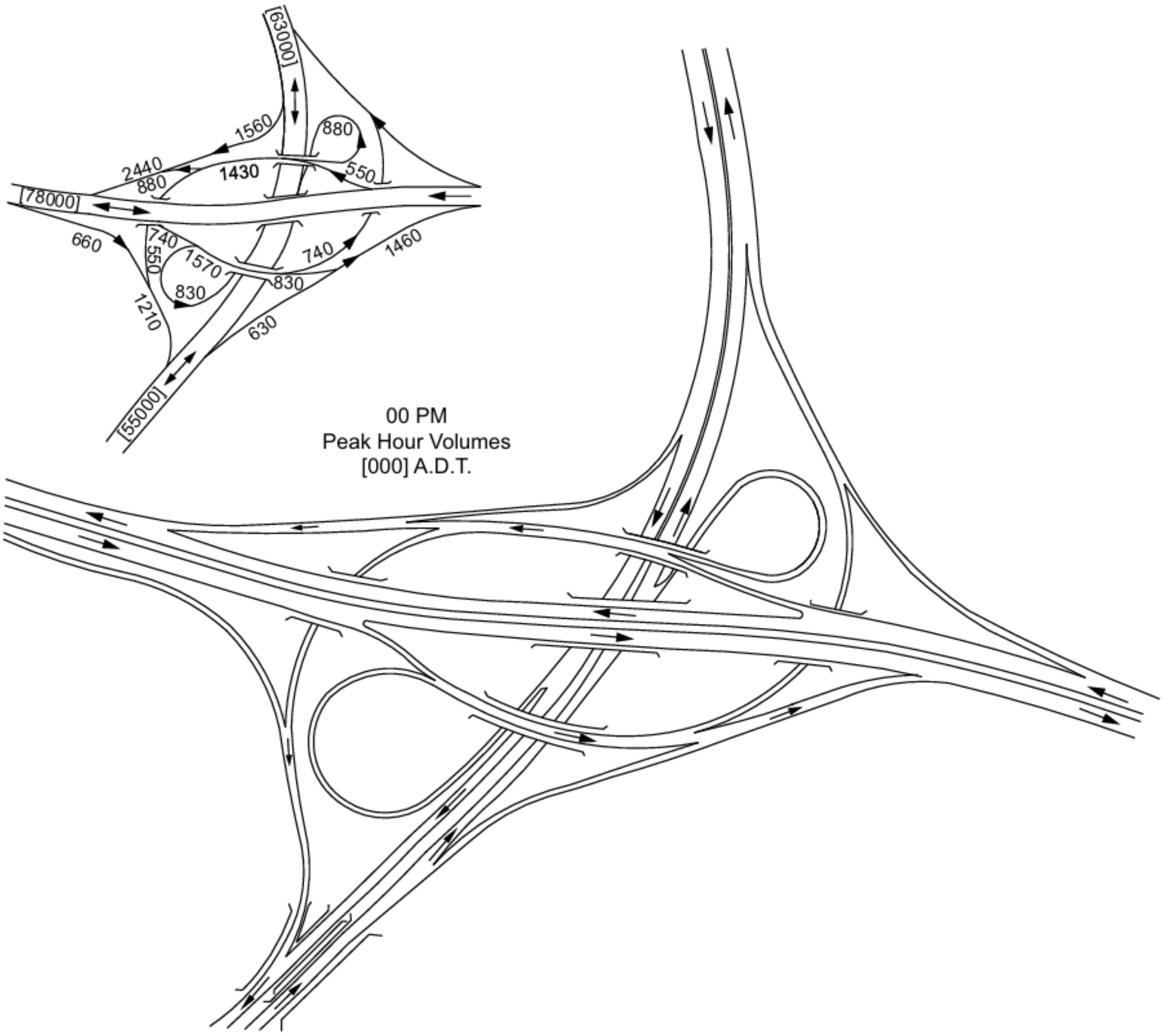


Figure 9.28 Directional Interchange, Two Semidirect Connections

Plate 9.9 Four-Level Directional Interchange (Source: Georgia DOT)

Plate 9.10 Four-Level Directional Interchange (Pokuase)

Plate 9.11 Directional Interchange with Semidirect Connection and Loops (Source: Maryland SHA)

9.4.2 DETERMINATION OF INTERCHANGE CONFIGURATION

The appropriate form of interchange is generally that which maintains the operational capacity under the predicted demand conditions while retaining the principles of consistency and route continuity. From an operational perspective, the form of interchange adopted at a particular site will depend on the:

i. functional classification of the roads and the importance of the intersection in the road network
ii. volume and characteristics of traffic to be accommodated
iii. need for ramp metering either now or in the future
iv. desired level of service generally or for a particular movement.

Interchange configurations are covered in two categories:

i. **Service interchanges** – a service (access) interchange is an interchange between a major and a minor road. A minor road typically refers to a road, arterial or sub-arterial that contains at-grade intersections. Major road/minor road interchanges consist of a major road carrying high traffic volumes crossing a minor road carrying low to moderate traffic volumes. At access and service interchanges, vehicles entering from the crossing road may be doing so from a stopped condition, so that it is necessary to provide acceleration lanes to ensure that they enter the expressway/motorway at or near expressway/motorway speeds.

Access interchanges normally provide for all turning movements. If, for any reason, it is deemed necessary to eliminate some of the turning movements, the return movement, for any movement that is provided, should also be provided. Movements excluded from a particular interchange should, desirably, be provided at the next interchange upstream or downstream as, without this provision, the community served loses amenity.

There are only two basic interchange types that are appropriate to access and service interchanges. These are:

- the Diamond and
- the Par-Clo interchanges (a partial cloverleaf interchange) interchanges.

Each has a variety of possible configurations. Trumpet interchanges used to be considered suitable in cases where access was to be provided to one side only, for example to a bypass of a town or village. In practice, however, once a bypass has been built it does not take long before development starts taking place on the other side of the bypass. The three-legged interchange then has to be converted into a four-legged interchange. Conversion to a Par-Clo can be achieved at relatively low cost. Other than in the case of the Par-Clo AB, one of the major movements is forced onto a loop ramp. The resulting configuration is thus not appropriate to the circumstances. In practice, the interchange should be planned as a Diamond in the first instance, even though the crossing road, at the time of construction, stops immediately beyond the interchange.

ii. **System interchanges** – a system interchange is an interchange between two major roads. A major road typically refers to an expressway/motorway, major arterial or a major road that does not contain at-grade intersections. At interchanges of major roads, high traffic volumes usually exist on both roadways. System interchanges aim to provide free flow for both major movements and for the interconnecting ramps.

Directional interchanges provide high-speed connections to left and to right provided that the ramp exits and entrances are on the left of the through lanes. Where turning volumes are low or space is limited, provision of loops for right turning traffic can be considered. Directional interchanges that include one or more loops are referred to as being partially-directional. If all right turns are required to take place on loops, the cloverleaf configuration emerges.

The primary difference between system and service interchanges is that the ramps on system interchanges have free-flowing terminals at both ends, whereas the intersecting road ramp terminals on a service interchange are typically in the form of at-grade intersections.

In rural areas, interchange configurations are selected primarily on the basis of service demand.

When the intersecting roadways are expressway/motorways, directional interchanges may be needed for high turning volumes.

A combination of directional, semi-directional, and loop ramps may be appropriate where turning volumes are high for some movements and low for others. When loop ramps are used in combination with direct and semidirect ramp designs, it is desirable that the loops be arranged in such a way that weaving sections are avoided. A cloverleaf interchange is the minimum design that can be used at the intersection of two fully controlled access facilities or where left turns at grade are prohibited.

A cloverleaf interchange is adaptable in a rural environment where right-of-way is not prohibitive, and weaving is minimal. When designing a cloverleaf interchange, careful attention should be given to the potential improvement in operational quality that would be realized if the design included collector–distributor roads on the major roadway.

A simple diamond interchange is the most common interchange configuration for the intersection of a major roadway with a minor facility. The capacity of a diamond interchange is limited by the capacity of the at-grade terminals of the ramps at the crossroad. High through and turning volumes could preclude the use of a simple diamond unless signalization is used. Partial cloverleaf designs with loops in opposite quadrants eliminate the weaving associated with the full cloverleaf designs. They may also provide superior capacity to other interchange configurations.

Partial cloverleaf designs are appropriate where rights-of-way are not available (or are prohibitively expensive) in one or more quadrants or some of the movements are disproportionate to the others. This is especially true for heavy left-turn volumes where loop ramps may be utilized to accommodate the left-turn movements.

Generally, interchanges in rural areas are widely spaced and can be designed on an individual basis without any appreciable effect from other interchanges within the system. However, the final configuration of an interchange may be determined by the need for route continuity, uniformity of exit patterns, single exits in advance of the separation structure, elimination of weaving on the main facility, signing potential, and availability of right-of-way. Sight distance on the highways through a grade separation should be at least as long as that needed for stopping and preferably longer. Where exits are involved, decision sight distance is preferred, although not always practical.

Selecting an appropriate interchange configuration in an urban environment involves considerable analysis of prevailing conditions so that the most practical interchange configuration alternatives can be developed. At a new location, it is desirable that the interchange be planned into the location study so that the final alignment is compatible, both horizontally and vertically, with the interchange site. Generally, in urban areas, interchanges are so closely spaced that each interchange may be influenced directly by the preceding or following interchange to the extent that additional traffic lanes may be needed to satisfy capacity, weaving, and lane balance.

On a continuous urban route, all the interchanges should be integrated into a system design rather than considered on an individual basis. Line sketches for the entire urban corridor can be prepared, and several alternate interchange combinations developed for analysis and comparisons.

During the analysis procedure, a thorough study of the crossroad should be made to determine its potential for handling the heavier volume of traffic that an interchange would discharge. The ability of the crossroad to receive traffic from and discharge traffic to the main roadway has considerable bearing on the interchange geometry. For example, loop ramps may be needed to eliminate heavy left turns on a conventional diamond interchange.

In the process of developing preliminary line-sketch studies, systems interchanges may be inserted at freeway-to-freeway crossings and varying combinations of service interchanges developed for lesser crossroads. Generally, cloverleaf interchanges with or without collector–distributor roads are not practical for urban construction because of the excessive right-of-way needs.

Once several alternatives have been prepared for the system design， they can be compared on the following principles:

i. capacity
ii. route continuity
iii. uniformity of exit patterns
iv. single exits in advance of the separation structure
v. with or without weaving
vi. potential for signing
vii. cost
viii. availability of right-of-way
ix. potential for stage construction
x. compatibility with the environment.

Table 9.1 summarizes the relative capacity, right-of-way, and cost characteristics of the diamond, SPUIs, partial cloverleafs, full cloverleafs, trumpets, and directional interchanges.

Table 9.1 Summary of Interchange Characteristics

Interchange Type	Right-of-Way Required	Capacity	Cost	Notes
Diamond	Low	Low	Low	Simplest interchange
SPUI	Low	Moderate	Low-Moderate	Designed for urban use, problems accommodating pedestrians
Partial Cloverleaf	Moderate	Moderate	Moderate	Loops should be arranged to serve largest left turning movements
Full Cloverleaf	High	Moderate	High	Weaving areas are safety and capacity concerns
Trumpet	Moderate-High	Moderate	Moderate-High	Should be used when 3 legs are present
Directional	Very High	High	Very High	Preferred interchange for expressway/motorway to expressway/motorway connections

The most desirable alternatives can be retained for plan development. In the case of an isolated interchange well removed from the influence of other interchanges， the criteria set forth for rural interchange determination apply. The recommended preliminary interchange type based on the interchange location, type of intersecting facility, and total interchange traffic are shown in **Table 9.2**.

Table 9.2 Preliminary Interchange Selection Table

<table>
<tr><th>Interchange Location</th><th colspan="2">Design Class of Intersecting Facility</th><th>Total Interchanging Traffic (VPD)</th><th>Recommended Interchange Type (Preliminary)</th></tr>
<tr><td rowspan="8">Rural</td><td rowspan="3">System Interchange</td><td rowspan="3">A</td><td>Light < 15000 AADT</td><td>Cloverleaf</td></tr>
<tr><td>Moderate 15000 to 25000 AADT</td><td>Cloverleaf with C-D roads, semi-directional</td></tr>
<tr><td>Heavy > 25000 AADT</td><td>Semi-directional, full directional</td></tr>
<tr><td rowspan="5">Service Interchange</td><td rowspan="3">B, C</td><td>Light < 15000 AADT</td><td>Diamond</td></tr>
<tr><td>Moderate 15000 to 25000 AADT</td><td>Partial cloverleaf, cloverleaf, trumpet</td></tr>
<tr><td>Heavy > 25000 AADT</td><td>Cloverleaf with C-D roads, semi-directional</td></tr>
<tr><td rowspan="2">D, E</td><td>Light < 10000 AADT</td><td>Diamond</td></tr>
<tr><td>Moderate 10000 to 20000 AADT</td><td>Trumpet to cloverleaf</td></tr>
<tr><td rowspan="8">Urban</td><td rowspan="2">System Interchange</td><td rowspan="2">A</td><td>Moderate 20000 to 35000 AADT</td><td>Cloverleaf with C-D roads, semi-directional</td></tr>
<tr><td>Heavy > 35000 AADT</td><td>Semi-directional, full directional</td></tr>
<tr><td rowspan="6">Service Interchange</td><td rowspan="3">B, C</td><td>Light < 20000 AADT</td><td>Diamond, split diamond</td></tr>
<tr><td>Moderate 20000 to 35000 AADT</td><td>Urban diamond, partial cloverleaf, full cloverleaf</td></tr>
<tr><td>Heavy > 35000 AADT</td><td>Cloverleaf with C-D roads, semi-directional</td></tr>
<tr><td rowspan="3">D, E</td><td>Light < 15000 AADT</td><td>Diamond, split diamond</td></tr>
<tr><td>Moderate 15000 to 30000 AADT</td><td>Urban diamond, partial cloverleaf</td></tr>
<tr><td>Heavy > 30000 AADT</td><td>Cloverleaf with C-D roads</td></tr>
</table>

9.4.3 DESIGN PRINCIPLES OF INTERCHANGES

Manoeuvres in an interchange area occur at high speeds close to the main line and over relatively short distances. It is therefore important that drivers should experience no difficulty in recognizing their route through the interchange irrespective of whether that route traverses the interchange on the main line or divert from the main line to a destination that may be to the left or the right of the main line. In following their selected route, drivers should be disturbed as little as possible by other traffic. These requirements can be met through the application of the basic principles of interchange design.

The driver has a number of tasks to execute successfully to avoid being a hazard to other traffic. It is necessary to:

i. select a suitable speed and accelerate or decelerate to the selected speed within the available distance.
ii. select the appropriate lane and carry out the necessary weaving manoeuvres to effect lane changes if necessary.
iii. diverge towards an off-ramp or merge from an on-ramp with the through traffic.

To maintain safety in carrying out these tasks, the driver must be able to understand the operation of the interchange and should not be surprised or misled by an unusual design characteristic. Understanding is best promoted by consistency and uniformity in the selection of types and in the design of particular features of the interchange.

i. Interchange exits and entrances should always be located on the right-hand side. The application of left-hand entrances and exits should only be considered under extremely limiting circumstances. Even in the case of a major fork where two expressways/motorways are diverging, the lesser movement should, for preference, be on the right.
ii. Uniformity of signing practice is an important aspect of consistent design and reference should be made to the MRH Standard Details, Road Signs and Markings for Urban and Trunk Roads, 1991.
iii. Ideally, an interchange should have only a single exit for each direction of flow with this being located in advance of the interchange structure. The directing of traffic to alternative destinations on either side of the main line should take place clear of the main line itself.
iv. Single entrances are to be preferred, also in support of operational efficiency of the interchange.
v. From the standpoint of convenience and safety, in particular prevention of wrong-way movements, interchanges should provide ramps to serve all turning movements.
vi. The choice of whether the crossing road should be taken over or under the main line depends on a number of factors, not the least of which is the matter of terrain and construction costs. There are, however, a number of advantages in carrying the crossing road over the main line. These are:
    a. Exit ramps on up-grades assist deceleration and entrance ramps on downgrades assist acceleration and have a beneficial effect on truck noise.
    b. Rising exit ramps are highly visible to drivers who may wish to exit from the main line.
    c. The structure has target value, i.e., it provides advance warning of the possibility of an interchange ahead necessitating a decision from the driver whether to stay on the main line or perhaps to change lanes with a view to the impending departure

from the main line.

d. Dropping the main line into cut reduces noise levels to surrounding communities and also reduces visual intrusion.

e. For the long-distance driver on a rural expressway, a crossing road on a structure may represent an interesting change of view; and,

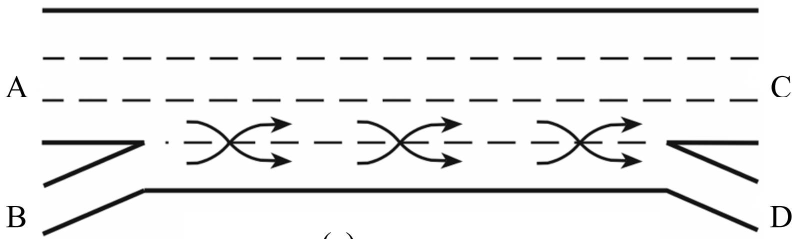
f. The crossing road ramp terminals may include right and left turn lanes, traffic signals and other traffic control devices. Not being obstructed by bridge piers and the like, these would be rendered more visible by taking the crossing road over the main line.

The other design principles, being continuity of basic lanes, lane balance and lane drops are discussed hereafter as matters of detailed design.

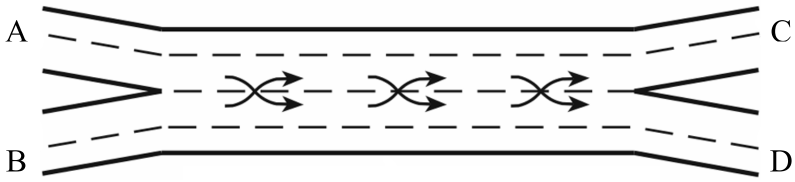
9.4.3.1 WEAVING

The Highway Capacity Manual (2016) defines weaving as the crossing of two or more traffic streams travelling in the same general direction without the aid of traffic control devices but then goes to address the merge-diverge as a separate issue. However, the merge-diverge operation, associated with successive single-lane on-and off-ramps where there is no auxiliary lane, does have two streams that, in fact, are crossing. Reference to weaving should thus include the merge-diverge.

Three types of weaves are illustrated in **Figure 9.29**. Type A weave requires all weaving vehicles to execute one lane change. Type B weaving occurs when one of the weaving streams does not have to change lanes but the other has to undertake at most one lane change. Type C weaving allows one stream to weave without making a lane change, whereas the other stream has to undertake two or more lane changes.

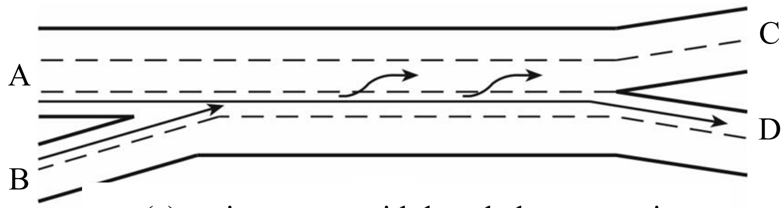


(a) ramp-weave

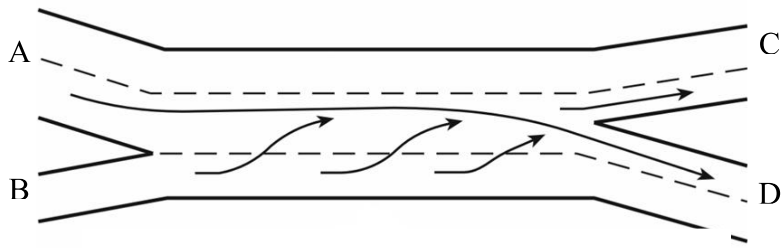


(b) major weave with crown line

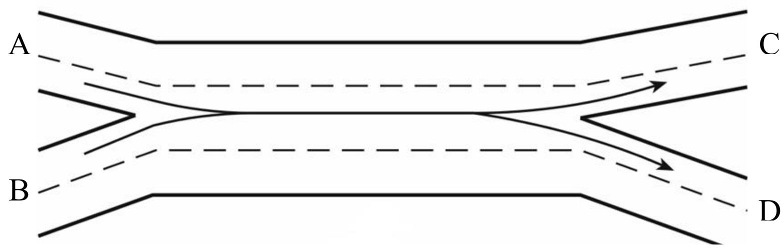
Type A weaves



(a) major weave with lane balance at exit

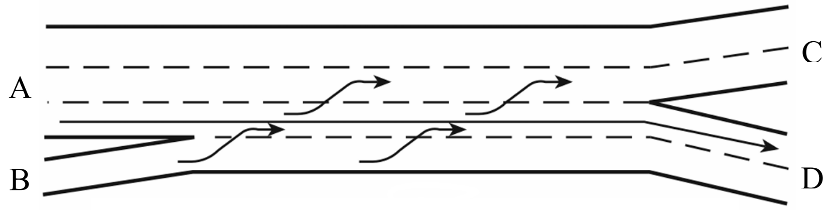


(b) major weave with merging at entrance

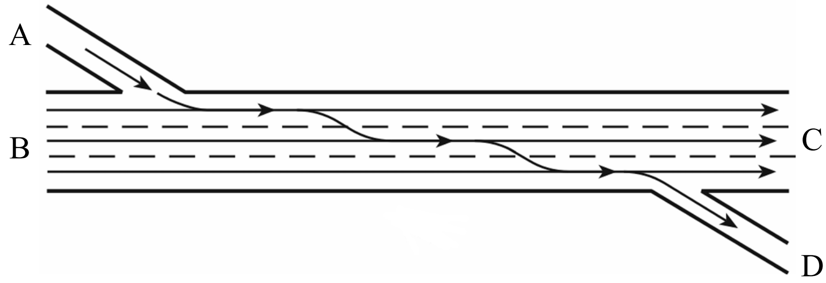


(c) major weave with merging at entrance and lane balance at exit

Type B weaves



(a) major weave without lane balance



(b) two-sided weave

Type C weaves

Figure 9.29 Types of weaves

The Type B weave is, in essence, a Type A weave but with the auxiliary lane extending either up- or downstream of the weaving area and with an additional lane being provided either to the on- or to the off-ramp. It follows that a Type A weaving section can be easily converted into a Type B weave. At any site at which a Type A weave appears, it would thus be prudent to check the operation at the site for both types of weaves.

9.4.3.2 LOCATION AND SPACING OF INTERCHANGES

Selection of interchange location should be examined to satisfy each of the following three basic conditions:

i. Traffic conditions
ii. Land use and other social conditions
iii. Terrain, geology and other natural conditions

The above conditions should be considered while referring to the following interchange positioning elements:

i. Estimation of the traffic that uses the interchange (human traffic such as buses, and physical traffic such as freight)
ii. Examination of the economy of interchange construction
iii. Interval with adjoining interchanges
iv. Selection of road for connecting to the interchange

v. Natural and environmental conditions in roadside areas

Rural interchanges are typically spaced at distances of five (5) to ten (10) kilometres apart or more . For example, 5km in USA, 7km in Germany, 10km in Japan. This distance is measured from centreline to centreline of crossing roads.

The generous spacing applied to rural interchanges would not be able to serve intensively developed urban areas adequately.

As an illustration of context sensitive design, trip lengths are shorter and speeds lower on urban expressways/motorways than on rural expressways/motorways. As drivers are accustomed to taking a variety of alternative actions in rapid succession a spacing of closer than eight (8) kilometres can be considered.

At spacing appropriate to the urban environment, reference to a centreline-to-centreline distance is too coarse to be practical. The point at issue is that weaving takes place between interchanges and the available distance is a function of the layout of successive interchanges. Weaving distance is defined in the Highway Capacity Manual and other sources as the distance between the point at which the separation between the ramp and the adjacent lane is 0.5m to the point at the following off-ramp at which the distance between ramp and lane is 3.65m as illustrated in **Figure 9.30**.

Three criteria for the spacing of interchanges can be considered. In the first instance, the distance required for adequate signage should ideally dictate spacing of successive interchanges. If it is not possible to achieve these distances, consideration can be given to a relaxation based on achieving Level of Service (LOS) D conditions on the expressway/motorway. The third criterion is that of turbulence, which is applied to the merge-diverge situation.

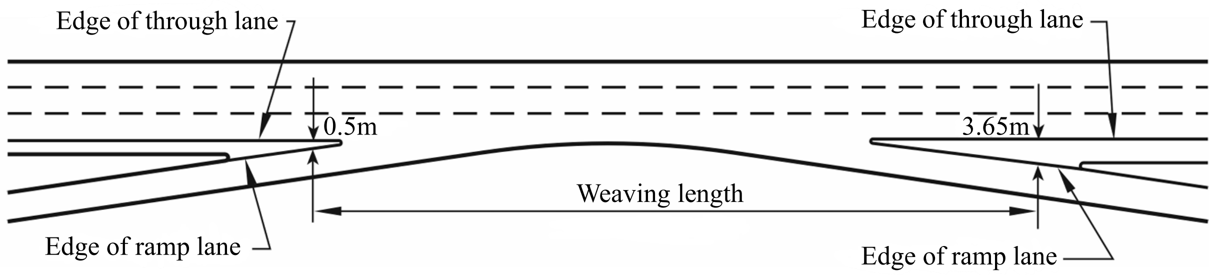


Figure 9.30 Weaving distance

9.4.3.3 UNIFORMITY OF INTERCHANGE PATTERNS

When a series of interchanges are being designed, attention should be given to the group of interchanges as a whole, as well as to each individual interchange. Interchange uniformity and route continuity are interrelated concepts, and both can be obtained under ideal conditions.

Considering the need for high capacity, appropriate level of service, and low crash frequencies in conjunction with expressway/motorway operations, it is desirable to provide uniformity in exit and entrance patterns. Because interchanges are closely spaced in urban areas, shorter distances are available in which to inform drivers of the course to follow when exiting an expressway/motorway. An inconsistent arrangement of exits between successive interchanges causes driver confusion, resulting in drivers slowing down on high-speed lanes and making unexpected manoeuvres. Examples of inconsistent exit arrangements are illustrated in **Figure 9.31A,** and include inconsistency of exit ramp locations with respect to the structure (near and far side of structure) and exit ramps on the left side of the travelled way. The difficulty of left-entrance merging with high-speed through traffic and the lane changing to reach left-exit ramps make these layouts undesirable. Except in highly special cases, all entrance and exit ramps should be on the right. To the extent practical, all interchanges along an expressway/motorway should be reasonably uniform in geometric layout and general appearance, as shown in **Figure 9.31B**.

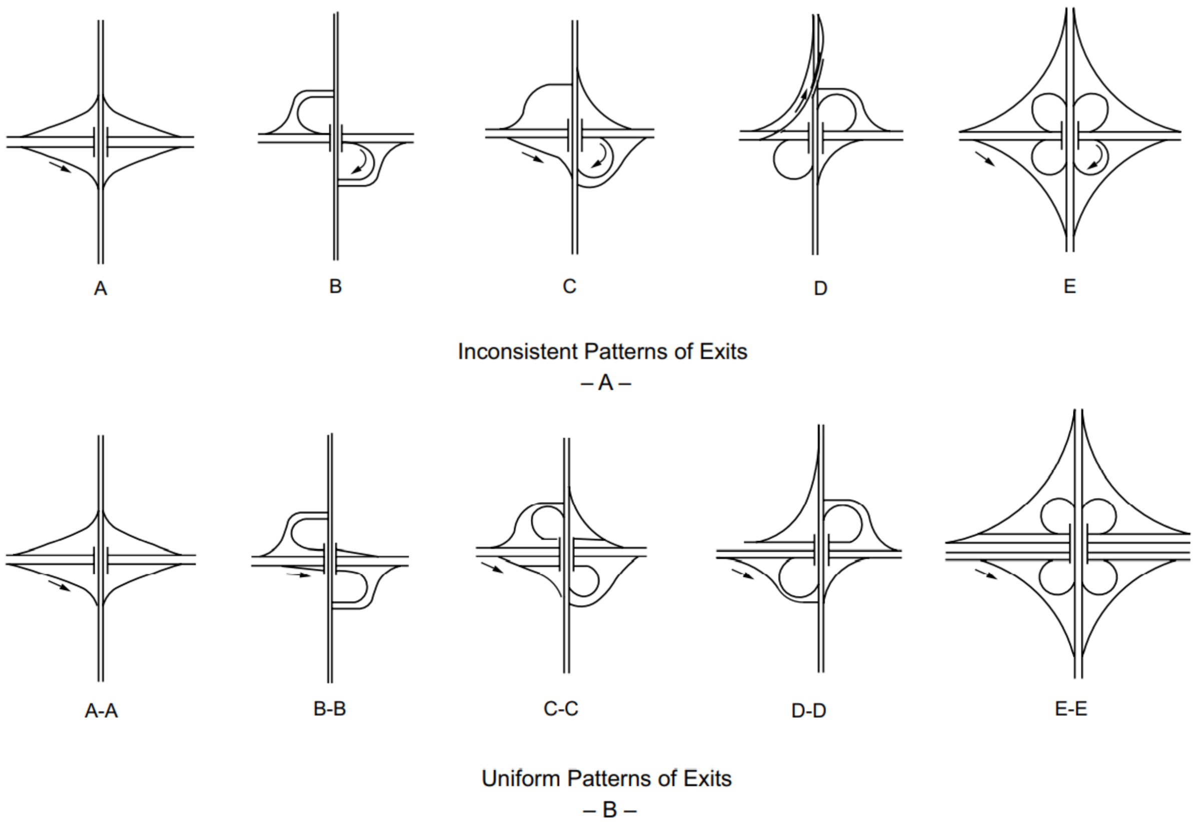


Figure 9.31 Arrangement of Exits between Successive Interchanges

9.4.3.4 ROUTE CONTINUITY

Route continuity refers to the provision of a directional path along and throughout the length of a designated route. The designation pertains to a route number or a name of a major highway.

Route continuity is an extension of the principle of operational uniformity coupled with the application of proper lane balance and the principle of maintaining a basic number of lanes.

The principle of route continuity simplifies the driving task in that it reduces lane changes, simplifies signing, delineates the through route, and reduces the driver's search for directional signing.

Desirably, the through driver, especially one unfamiliar with the route, should be provided a continuous through route on which changing lanes is not needed to continue on the through route.

In the process of maintaining route continuity, particularly through cities and bypasses, interchange configurations need not always favour the heavy movement but rather the through route. In this situation, heavy movements can be designed on flat curves with reasonably direct connections and auxiliary lanes, which are operationally equivalent to through movements. In conditions where the exiting manoeuvre may have higher volume than the through movement that is being provided route continuity, the design alignments of the main line and exit should be such that from a driver's perspective it is apparent which roadway is the main line and which roadway is the exit.

Figure 9.32 illustrates the principle of route continuity as applied to a hypothetical route, N15, as it intersects other major high-volume routes (service interchanges not shown). In **Figure 9.32A** route continuity is maintained on the designated route by keeping it on the left of all other entering or exiting routes. In **Figure 9.32B**, route continuity is disrupted by other routes exiting or entering on the left, except for the northbound direction of the last interchange.

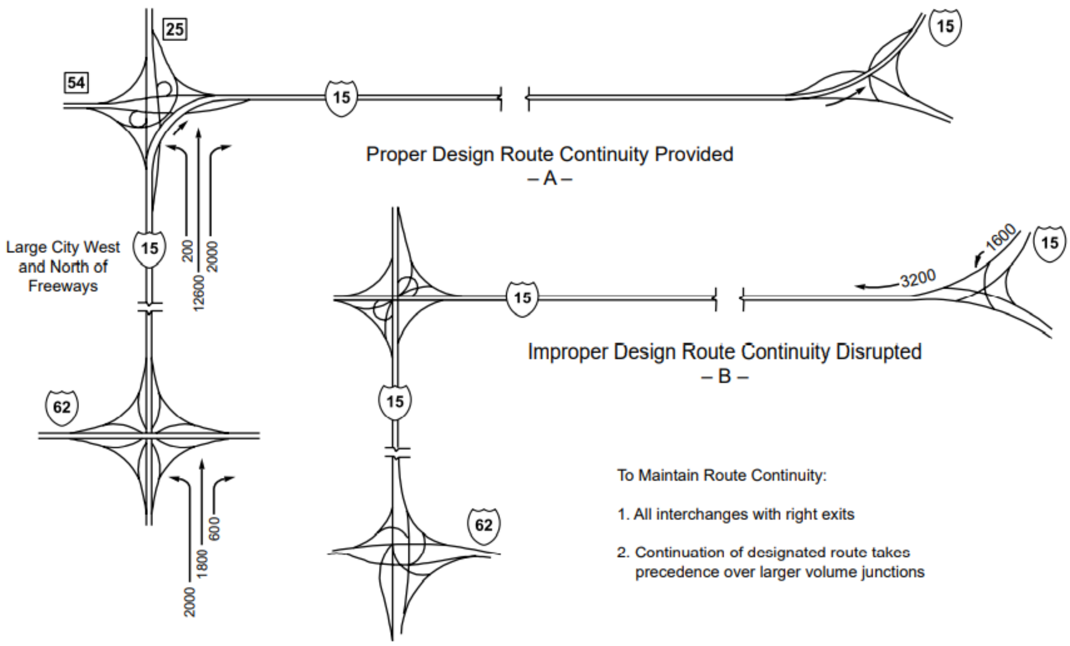


Figure 9.32 Interchange Forms to Maintain Route Continuity

9.4.3.5 OVERLAPPING ROUTES

In some situations, two or more routes share a single roadway within a corridor. In rural areas, overlapping routes are generally addressed by providing adequate signing and maintaining route continuity. In urban areas, the complexity of addressing overlapping routes increases with the probability of weaving and the need for additional capacity and lane balance.

In urban areas, it is preferable not to have overlapping routes, especially for only short distances. When routes overlap, signing is more complicated, and the decision process for the driver is more demanding.

The provision for route continuity through overlapping sections is essential. However, in some instances, this provision poses a challenge in determining which route should take precedence, and this challenge is especially acute when both routes have the same classification. Through a process of subclassification, a priority may be established for one of the overlapping roadways. All other factors being equal, priority should be assigned to the route that handles the highest volume of through traffic.

Once priority for one of the overlapping roadways has been established, basic lanes, lane balance, and other principles of interchange design can be applied to the design of the overlapping section. The lower classified facility should enter and exit on the right, thus conforming to the concept of route continuity.

On overlapping roadways, weaving is usually involved. However, on longer overlaps, the presence of weaving is minimized. Where the overlap is short, such as between successive interchanges, attention should be given to the design of weaving sections and lane.

In a situation where a major arterial would be overlapped by a lesser roadway, the minor facility may be designed as a collector–distributor road with transfer roads connecting the two facilities, as shown in **Figure 9.33**. This design removes weaving from the major roadway and transfers it to the minor facility. (See **Section 9.4.3.11**)

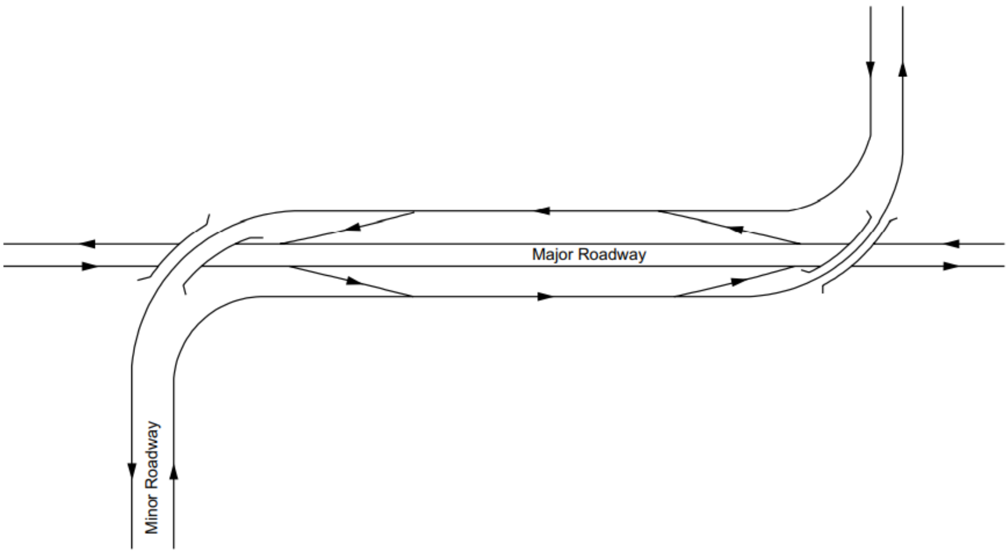


Figure 9.33 Collector–Distributor Road on Major–Minor Roadway Overlap

9.4.3.6 BASIC NUMBER OF LANES

Designation of the basic number of lanes is fundamental to establishing the number and arrangement of lanes on an expressway/motorway. Consistency should be maintained in the number of lanes provided along any route of arterial character. Thus, the basic number of lanes is defined as a minimum number of lanes designated and maintained over a significant length of a route, irrespective of changes in traffic volume and lane-balance needs. Stating it another way, the basic number of lanes is a constant number of lanes assigned to a route, exclusive of auxiliary lanes.

As illustrated in **Figure 9.34**, the basic number of lanes on expressways/motorways is maintained over significant lengths of the routes, as A to B or C to D. The number of lanes is predicated on the general volume level of traffic over a substantial length of the facility. The volume considered here is the DHV (normally, representative of the morning or evening weekday peak).

Localized variations are ignored, so short sections of roadway that carry lower volumes would theoretically have reserve capacity, and short sections of roadway carrying somewhat higher volumes would be augmented by the addition of auxiliary lanes within these sections.

An increase in the basic number of lanes is needed where traffic volume builds up sufficiently over a substantial length of the facility to justify an additional lane.

The basic number of lanes may be decreased where traffic volumes are significantly reduced for a substantial length of highway. Lane reductions are discussed in **Section 9.4.3.9**.

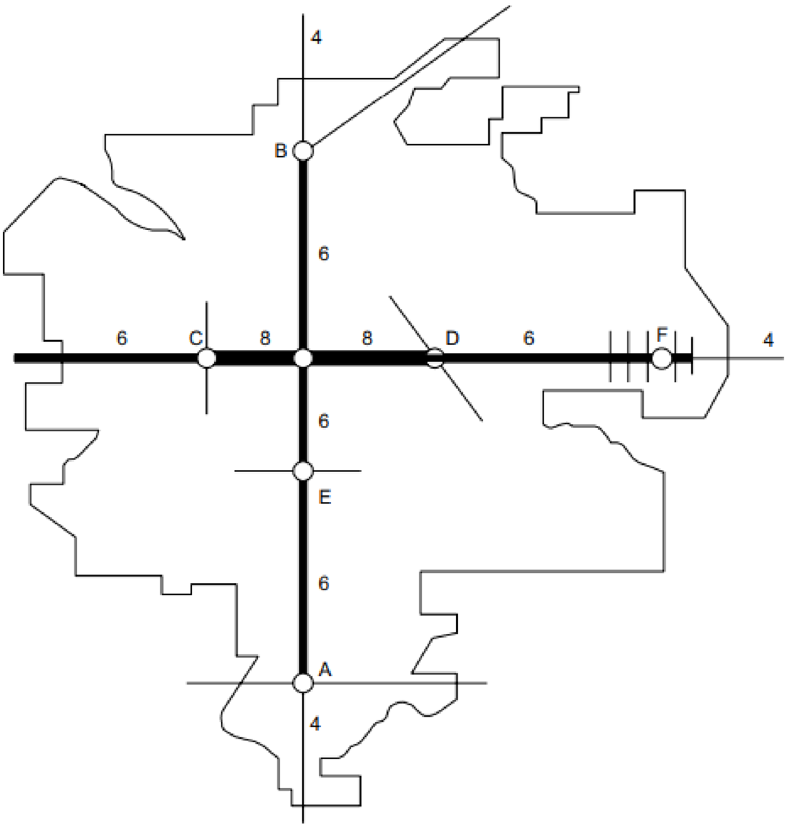


Figure 9.34 Schematic of Basic Number of Lanes

9.4.3.7 COORDINATION OF LANE BALANCE AND BASIC NUMBER OF LANES

To realize efficient traffic operation through and beyond an interchange, there should be balance in the number of traffic lanes on the expressway/motorway and ramps. Design traffic volumes and a capacity analysis determine the basic number of lanes to be used on the roadway and the minimum number of lanes on the ramps. The basic number of lanes should be established for a substantial length of expressway/motorway and should not be changed through pairs of interchanges, simply because there are substantial volumes of traffic entering and leaving the mainline. In other words, there should be continuity in the basic number of lanes. As described later in this section, variations in traffic demand should be accommodated by auxiliary lanes where needed.

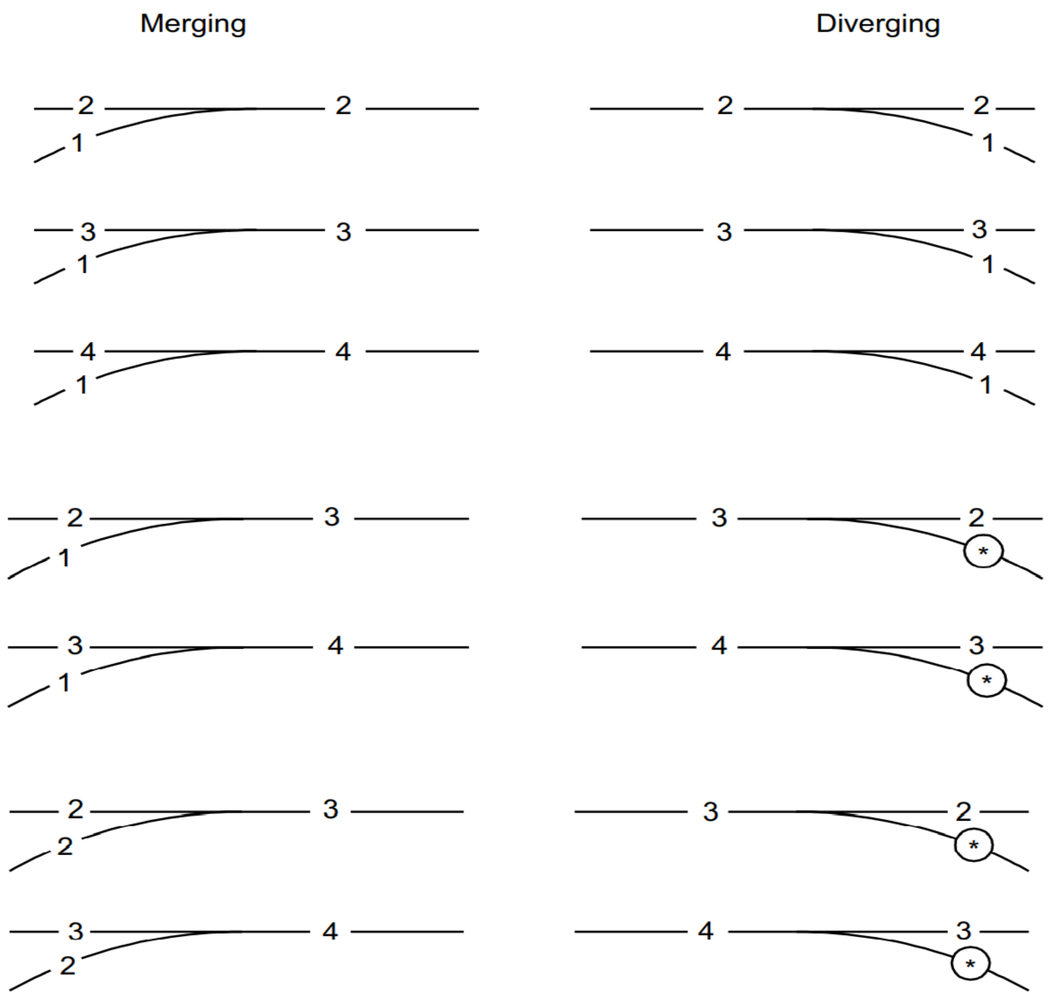
After the basic number of lanes is determined for each roadway, the balance in the number of lanes should be confirmed on the basis of the following principles:

i. At entrances, the number of lanes beyond the merging of two traffic streams should not be less than the sum of all traffic lanes on the merging roadways minus one, but may be equal to the sum of all traffic lanes on the merging roadways (refer to **Figure 9.35**).

iii. At exits, the number of approach lanes on the roadway should be equal to the number of lanes on the roadway beyond the exit, plus the number of lanes on the exit, minus one. Exceptions to this principle occur at cloverleaf loop-ramp exits that follow a loop-ramp entrance and at exits between closely spaced interchanges (closely spaced interchanges are those where the distance between the end of the taper of the entrance terminal and the beginning of the taper of the exit terminal is less than 450m, and a continuous auxiliary lane between the terminals is being used). In these cases, the auxiliary lane may be dropped in a single-lane exit such that the number of lanes on the approach roadway is equal to the number of through lanes beyond the exit plus the lane on the exit.

iv. The travelled way of the roadway should be reduced by not more than one traffic lane at a time.

Typical examples of lane balance are shown in **Figure 9.35.**



(*) One lane under special conditions of Principle (ii) in **Section 9.4.3.7**.

Figure 9.35 Typical Examples of Lane Balance

The principles of lane balance can seem to conflict with the concept of continuity in the basic number of lanes, as illustrated in **Figure 9.36**. The figure shows three different arrangements where a four-lane expressway/motorway in one direction of travel has a two-lane exit followed by a two-lane entrance. In **Figure 9.36A**, lane ba1ance is maintained, but there is no compliance with the basic number of 1anes. This pattern may cause confusion and erratic operations for through traffic on the mainline. Even though traffic volumes are reduced through the interchange, there is no assurance that traffic demand wi11 not increase under certain circumstances. Unduly large concentrations of through traffic may be caused by special events, by closures, or by reduction in capacity of other parallel faci1ities that results from crashes or maintenance operations. Under such circumstances, bottlenecks may occur where any lanes have been dropped on a mainline between interchanges.

The arrangement shown in **Figure 9.36B** provides continuity in the basic number of lanes but does not conform with the principles of lane balance. With this arrangement, the large exiting or entering traffic volume that needs two lanes would have difficulty in either diverging from or merging with the main line flow.

Figure 9.36C illustrates an arrangement in which the concepts of lane balance and basic number of lanes are brought into harmony by building on the basic number of lanes (i.e., by adding auxiliary lanes or removing them from the basic width of the travelled way). Auxiliary lanes may be added to satisfy capacity and weaving needs between interchanges, to accommodate traffic pattern variations at interchanges, and for simplification of operations (such as reducing lane changing). The principles of lane balance should be applied in the use of auxiliary lanes. In this manner, the appropriate balance between traffic load and capacity is provided, and lane balance and operational flexibility are realized.

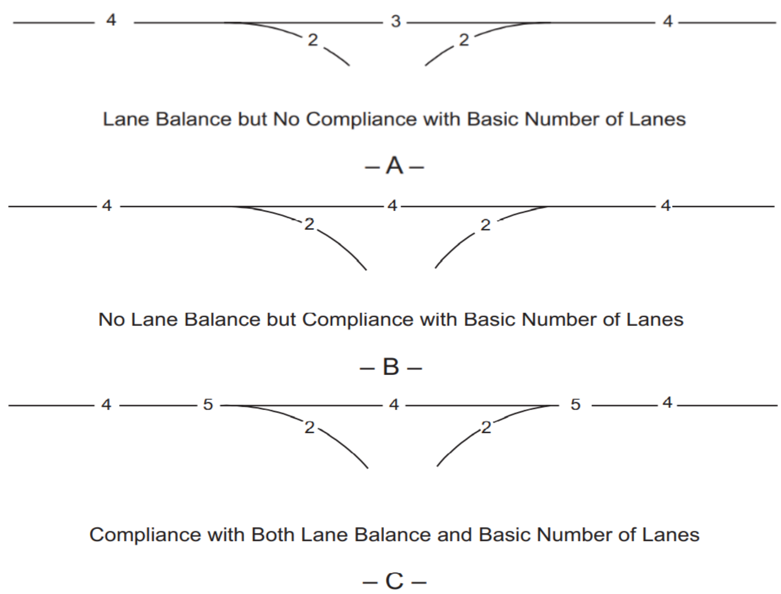


Figure 9.36 Coordination of Lane Balance and Basic Number of Lanes

9.4.3.8 AUXILIARY LANES

An auxiliary lane is defined as the portion of the roadway adjoining the through lanes for speed change turning, storage for turning, weaving, truck c1imbing, and other purposes that supplement through-traffic movement. The width of an auxiliary lane should be equal to the through lanes. An auxiliary lane may be provided to comply with the concept of lane balance, to comply with capacity needs or to accommodate speed changes weaving and manoeuvring of entering and leaving traffic. Where auxiliary lanes are provided along expressway/motorway main lanes, the adjacent shoulder should desirably be 2.5 to 3.0 m in width, with a minimum 1.75 m wide shoulder considered.

Operational efficiency may be improved by using a continuous auxiliary lane between the entrance and exit terminals where:

i. interchanges are closely spaced,
ii. the distance between the end of the taper on the entrance terminal and the beginning of the taper on the exit terminal is short and/or
iii. local frontage roads do not exist.

An auxiliary lane may be introduced as a single exc1usive lane or in conjunction with a two-lane entrance. The termination of the auxiliary lane may be accomplished by several methods:

i. The auxiliary lane may be dropped in a two-lane exit, as illustrated in **Figure 9.37A**. This treatment complies with the principles of 1ane balance.
ii. The auxiliary lane in a single lane exit may be dropped as illustrated in **Figure 9.37B**. This treatment is in accordance with the exceptions listed under Principle (ii) of lane balance as presented earlier in **Section 9.4.3.7**.
iii. Another method is to carry the full-width auxiliary lane to the physical nose before it is tapered into the through roadway. This design provides a recovery lane for drivers who inadvertently remain in the discontinued lane (see **Figure 9.37C**).

When these methods of terminating the auxiliary lane (**Figure 9.37B** and **Figure 9.37C**) are used, the exit gore should be visible throughout the length of the auxiliary lane.

If local experience with single-exit design indicates a history of turbulence in the traffic flow caused by vehic1es attempting to recover and proceed on the through lanes, the recovery area should be modified to provide a lane reduction at least 450m downstream to allow sufficient space for signing and markings associated with the reduction (refer to **Figure 9.37D**). Within large interchanges, this distance should be increased to 450m. Desirably, the lane reduction should not occur for 750m to minimize turbulence to the true lanes. When an auxiliary lane is carried through one or more interchanges, it may be eliminated by lane reduction beyond the influence of the last interchange, beginning approximately 750 m downstream of the last acceleration lane (see **Figure 9.37D**).

Where interchanges are widely spaced, it may not be practical or necessary to extend the auxiliary lane from one interchange to the next. In such cases the auxiliary lane originating at a two-lane entrance should be carried along the expressway/motorway for an effective distance beyond the merging point as shown in **Figure 9.38A1** and **Figure 9.38A2**. An auxiliary lane introduced for a two-lane exit should be carried along the expressway/motorway for an effective distance in advance of the exit and then extended onto the ramp, as shown in **Figure 9.38B1** and **Figure 9.38B2**. **Figure 9.38A1** and **Figure 9.38B1** show parallel designs, whereas **Figure 9.38A2** and **Figure 9.38B2** show tapered designs.

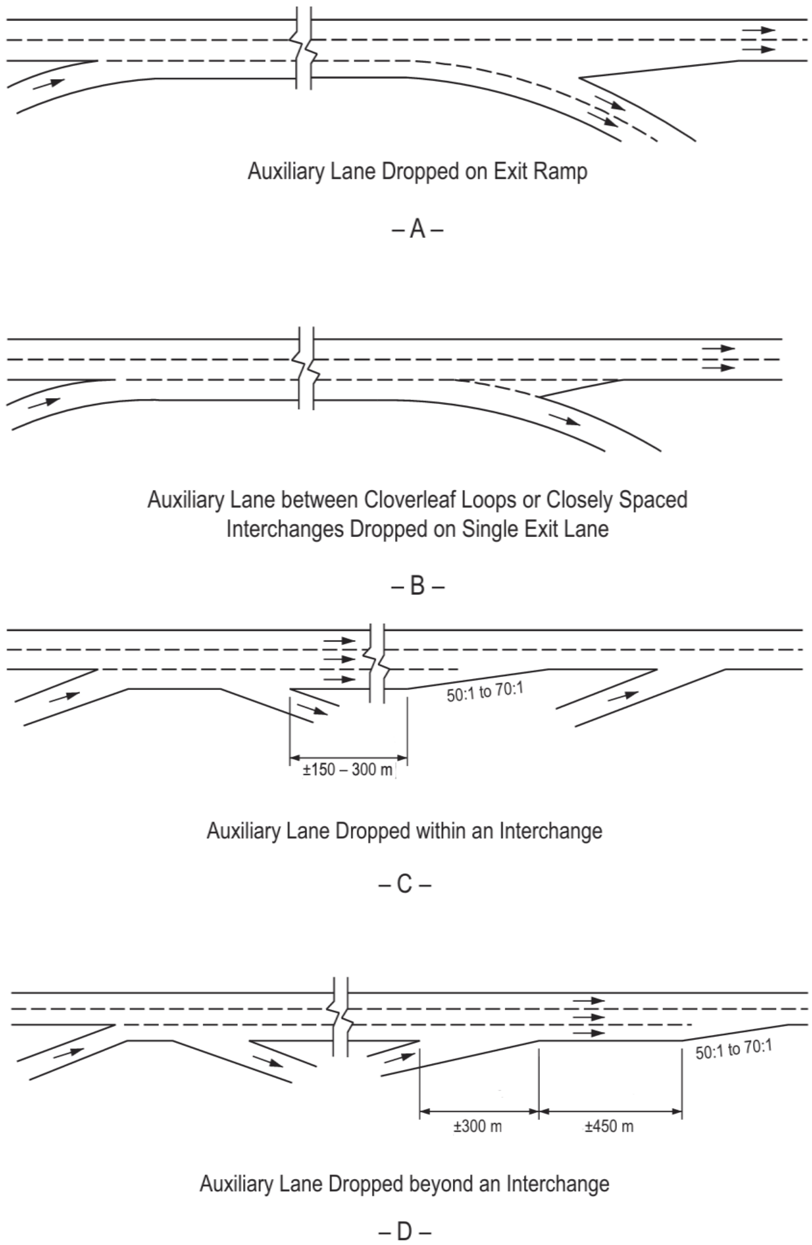


Figure 9.37 Alternative Methods of Dropping Auxiliary Lanes

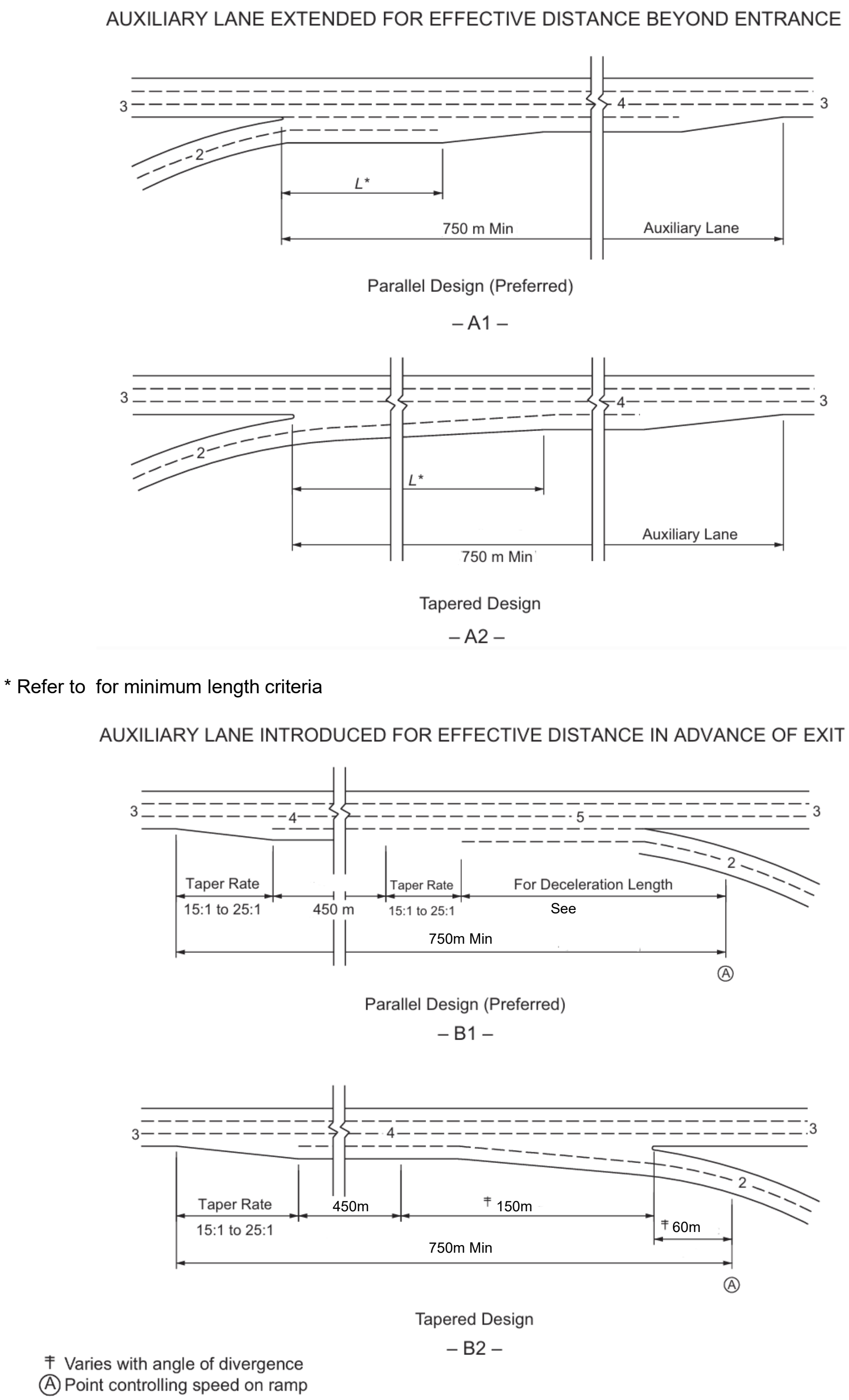


Figure 9.38 Coordination of Lane Balance and Basic Number of Lanes through Application of Auxiliary Lanes

Generally, parallel designs are preferred. The tapered design for a two-lane entrance ramp as shown in **Figure 9.38A2**, creates an undesirable inside merge. Auxiliary lanes should not be shorter than those shown in **Section 9.4.5.8** for single-lane ramps (see with adjustments for grades as suggested in **Table 9.22** and **Table 9.24**). It is not precisely known what the effective length of the introduced auxiliary lane should be under these circumstances. Experience indicates that minimum distances of about 750m produce the desired operational effect and enable achieving the full capacity of two-lane entrances and exits.

For those instances where an auxiliary lane extends for a long distance from an entrance at one interchange to exit at the next interchange, unfamiliar motorists may perceive the auxiliary lane as an additional through lane. For these situations an auxiliary lane may be terminated as discussed in **Section 9.4.3.9** or by providing a two-lane exit.

Auxiliary lanes are used to balance the traffic load and maintain a more uniform level of service on the roadway. They facilitate the positioning of drivers at exits and the merging of drivers at entrances. Thus, the concept is very similar in intent to signing and route continuity. Careful consideration should be given to the design treatment of an auxiliary lane because it may have the potential for trapping a driver at its termination point or the point where it is continued onto a ramp or turning roadway.

Figure 9.39 illustrates the application of an auxiliary lane that is terminated through a multilane exit terminal. The outside basic lane automatically becomes an interior lane with the addition of the auxiliary lane. From this interior lane a driver may exit right or proceed straight ahead. The example complies with the principles of lane balance and basic number of lanes. The design emphasizes the through route and allows drivers to make their decision to travel through or turn right well in advance of the exit point, or fairly close to it as result of the additional manoeuvre area.

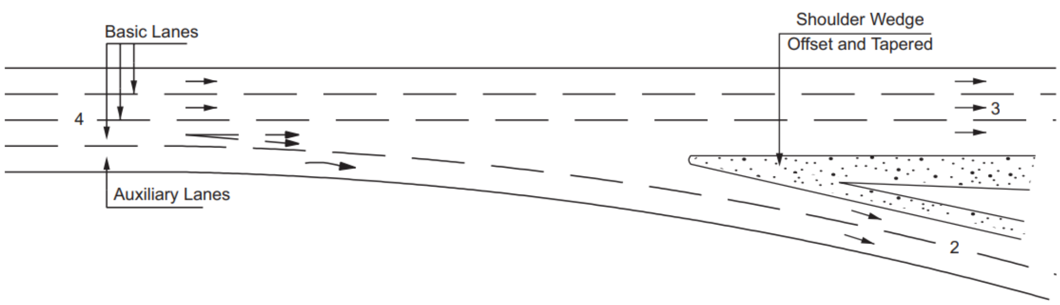


Figure 9.39 Auxiliary Lane Dropped at Two-Lane Exit

9.4.3.9 LANE REDUCTIONS

As discussed earlier, the basic number of lanes should be maintained over a significant length of expressway/motorway. Lane reductions should not be made between and within interchanges simply to accommodate variations in traffic volumes. Instead, auxiliary lanes are added or eliminated from the basic number of lanes as needed.

A reduction in the basic number of lanes may be made beyond a principa1 interchange involving a major fork or at a point downstream from an interchange with another expressway/motorway. This reduction may be made if the exit volume is large enough to change the basic number of 1anes beyond this point on the expressway/motorway route as a whole. Another case where the basic number of lanes may be reduced is where a series of exits such as in outlying areas of the city, causes enough decrease in the traffic load on the expressway/motorway to justify a lower basic number of 1anes. Dropping a basic lane or an auxiliary lane may be accomplished at a two-lane exit ramp or between interchanges.

If a basic lane or an auxiliary lane is to be dropped between interchanges, it should be accomplished at a distance of 600 to 900 m from the preceding interchange to allow for adequate lane reduction signing and marking.

The reduction should not be made so far downstream that motorists become accustomed to a number of lanes and are surprised by the reduction violating their expectation (See). Desirably, the lane reduction transition should be located on tangent horizontal alignment and on the approach side of any crest vertical curve. A sag vertica1 curve is also a good location for a lane reduction because it provides good visibi1ity. Preferably, the lane reduction should be made on the right side following an exit ramp because less traffic is likely in that lane. A right-side lane reduction has advantages in that speeds are generally lower and the merging manoeuvre from the right is more fami1iar to most motorists because it is simi1ar to a merge at an entrance ramp. Left-side Lane reductions may not function as well because of generally higher speeds and the less fami1iar left-side merge.

The end of the lane reduction should be tapered into the highway in a manner similar to that at a ramp entrance. Preferably, the rate of taper should be longer than that for a ramp. The minimum taper rate should be 50:1 and the desirable taper rate is 70:1.

If there is a basic lane or an auxiliary lane dropped within an interchange, it should be made in conjunction with a two-lane exit as shown in **Figure 9.37A**, or in a single-lane exit with an adequate recovery lane, as discussed in **Section 9.4.3.8**.

9.4.3.10 WEAVING SECTION

Weaving sections are highway segments where the pattern of traffic that enters and leaves at contiguous points of access produces vehicle paths that cross each other. Weaving sections may occur within an interchange, between entrance ramps in one interchange and exit ramps in a downstream interchange, or on segments of over1apping roadways.

Because considerable turbulence occurs throughout weaving sections, interchange designs that eliminate weaving entirely or at least remove it from the main facility are desirable. Weaving sections may be eliminated on the main facility by the selection of interchange forms that do not have weaving or by the incorporation of collector-distributor roads. Interchanges that provide all exit movements before any entrance movements will avoid weaving.

Although interchanges without weaving operate better than those with weaving, interchanges with weaving areas generally cost less. Designs that avoid weaving movements may need a greater number of structures or larger and more complex structures, with some direct connections. Joint evaluation of the total interchange cost and the specific volumes to be handled wil1 help reach a sound decision between design alternatives. The partial cloverleaf design with loops in opposite quadrants eliminates the weaving sections, does not involve direct connections or additional structures and has been found to operate superiorly to all other interchanges with a single separation structure.

Where cloverleaf interchanges are used, consideration should be given to the inclusion of collector-distributor roads on the main facility, or possibly both facilities where warranted.

The capacity of weaving sections may be seriously restricted unless the weaving section has adequate length, adequate width, and lane balance. Refer to **Section 8.5.8.1** for procedure in determining weaving lengths.

9.4.3.11 COLLECTOR-DISTRIBUTOR ROADS

A full cloverleaf interchange in an urban or suburban area is a typical example of a single interchange that should be analysed for the need for collector-distributor roads within the interchange. Collector-distributor roads may be one or two lanes in width, depending on capacity needs. Lane balance should be maintained at entrances and exits to and from the main line, but strict adherence is not mandatory on the collector-distributor road proper because weaving is handled at reduced speed. The design speed usually ranges from 60 to 80 km/h but should not be less than 20 km/h below the design speed of the main roadway. Traffic conflicts are likely if collector-distributor roads are not properly signed, especially those servicing more than one interchange.

Outer separations between the main line and the collector-distributor roads should be as wide as practical; however, minimum widths are tolerable. The minimum width should allow for shoulder widths equal to that on the main line and for a suitable barrier to prevent indiscriminate crossovers.

The advantages of using collector-distributor roads within an interchange are that weaving is transferred from the main roadway, single entrances and exits are developed, all main-line exits occur in advance of the structure, and a uniform pattern of exits can be maintained.

9.4.3.12 TWO-EXIT VERSUS SINGLE-EXIT INTERCHANGE DESIGN

In general, interchanges that are designed with single exits are superior to those with two exits especially if one of the exits is a loop ramp or if the second exit is a loop ramp preceded by an entrance loop ramp. Whether used in conjunction with a full cloverleaf or with a partial cloverleaf interchange, the single exit design may improve operational efficiency of the entire facility.

The purposes for developing single exits, where applicable, are to:

i. remove weaving from the main facility and transfer it to a slower speed facility,
ii. provide a high-speed exit from the main roadway for all exiting traffic,
iii. simplify signing and the decision process,
iv. satisfy driver expectancy by placing the exit in advance of the separation structure,
v. provide uniformity of exit patterns, and
vi. provide decision sight distance for all traffic exiting from the main roadway.

The full cloverleaf interchange, where a weaving section exceeds 1,000 vehicles per hour (vph) is an example where operational efficiency may be improved by the development of single exits and entrances.

The loop ramps of a full cloverleaf interchange create a weaving section adjacent to the outside through lane, and considerable deceleration-acceleration occurs in the through lane. By using collector-distributor roads, as shown in **Plate 9.12**, a single exit is provided, and weaving is transferred to the collector distributor road.

Plate 9.12 Four-Leg Interchange, Cloverleaf with Collector–Distributor Roads

Without a collector-distributor road, the second exit of a cloverleaf interchange occurs beyond the separation structure and, in many cases is hidden behind a crest vertical curve. The single

−exit design places the exit from the main line in advance of the structure and is conducive to a uniform pattern of exits. Where the through roadway overpasses the crossroad in a vertical curve, it may be more difficult to develop full decision sight distance for the loop ramp exit of a conventional cloverleaf interchange. The use of the single-exit design may make it easier to obtain the desired decision sight distance due to the exit occurring on the upgrade.

Some arrangements of partial cloverleaf loop ramps may feature single exits as shown **Figure 9.24F,** and still be inferior because they do not provide any of the desirable purposes previously discussed.

On a full cloverleaf interchange, the single exit is developed by using a collector-distributor road for the full length of the interchange. On certain partial cloverleaf arrangements, the single exit can be developed by elongating the loop ramp in the upstream direction to the point where it diverges from the right-turn movement well in advance of the separation structure. The elongation of the loop ramp may be done with a spiral, simple curve, tangent, or a combination of these.

There are some cases where a single exit does not work as well as two exits such as at high-volume, high-speed directional interchanges. This concern usually occurs at the fork following the single exit from the expressway/motorway, particularly when the traffic volume is great enough to warrant a two-Lane exit and the distance from the exit terminal to the fork is insufficient for weaving and proper signing. There is often some confusion at this second decision point resulting in poor operation and a high crash potential. Because of this it may be advantageous on some directional interchanges to provide two exits on each expressway/motorway leg.

Generally, the provision for single exits is more costly because of the added roadway, longer bridges and in some cases additional separation structures. The overall efficiency of a cloverleaf interchange with collector-distributor roads should be taken into consideration. Where ramp volumes are low and not expected to increase significantly, or where a particular cloverleaf weave does not exceed about 1000 veh/h, it will often be impractical to use collector-distributor roads. These conditions can be expected in rural areas or on low-volume expressway/motorway.

Collector-distributor roads may still be an option if significant future turning volumes are expected or site investigations reveal a definitive need for such a configuration. **Figure 9.40** shows various interchange configurations that are compatible with the concepts of uniform exit patterns and exits in advance of the separation structure.

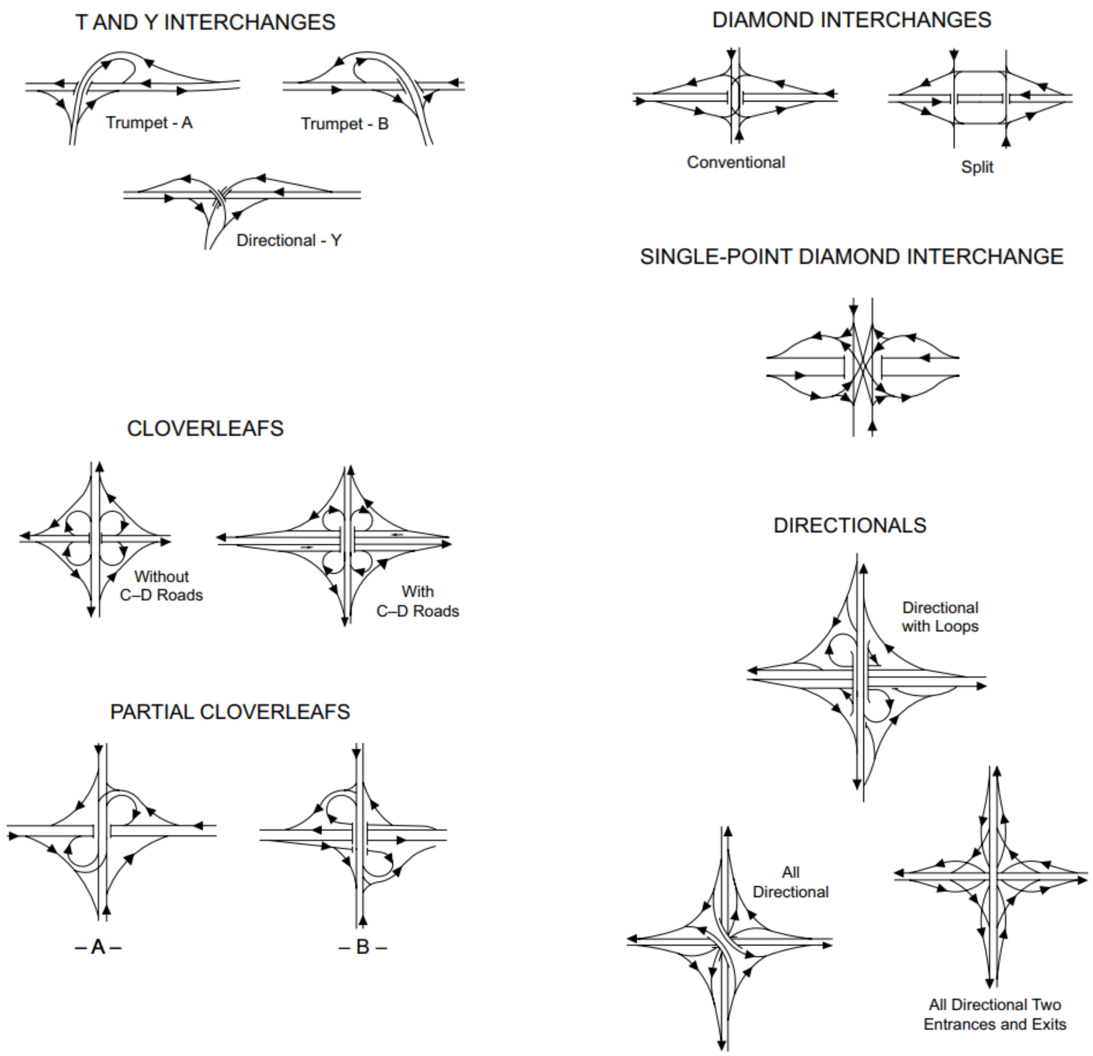


Figure 9.40 Interchange Forms with One and Two Exits

9.4.3.13 WRONG-WAY ENTRY

Wrong-way entry onto expressway/motorway and arterial roads is not a frequent occurrence, but it should be given special consideration at all stages of design in order to discourage wrong-way manoeuvres. Most wrong-way entrances occur at expressway/motorway exit ramps, at at-grade intersections along divided arterial roads and at transitions from undivided to divided highways. Several factors that contribute to wrong-way entrances are related to interchange design. These factors concern the interchange configuration and, more particularly, the crossroad terminal of the exit ramps which are discussed below.

i. Partial interchanges are particularly noteworthy in respect to wrong-way entry. Where provision is not made for one or more of the movements at an interchange, wrong-way entry may occur due to driver confusion.

ii. Exit ramps that connect to two-way frontage roads may also have potential for wrong-way entry. Without channelization on the frontage road, they may appear as open entries. Some of the "scissors" channelization has proved to be confusing, resulting in wrong-way use.

iii. Exit ramps with a sweeping connection to the street (e.g., outer connection, loop, and some diamond ramps) have a low rate of wrong - way entry. However one-way ramps that connect as an unchanneled T-intersection can lead to wrong-way entry.

iv. Partial cloverleaf interchange forms, where the exit and entrance ramps are adjacent to each other, are also susceptible to potential wrong-way entry. Signing, markings, and geometric design features of ramp intersections should be coordinated to provide positive guidance for drivers.

v. Unusual or odd arrangements of exit ramps are confusing and conducive to wrong-way entry. An example is the button-hook or J-shaped ramp that connects to a parallel or diagonal street or frontage road, often well-removed from the interchange structure and other ramps. Another example is a pair of right-turn connections to a lateral or parallel road (frontage road) that is offset from the separation structure.

As shown in **Figure 9.41** and **Figure 9.42** a sharp rectangular intersection is provided at the junction of the left edge of the ramp entering the crossroad and the right edge of the travelled way. The control radius should be tangent to the crossroad centreline, not the edge. This type of design discourages the improper right turn onto the one-way ramp. Where practical ramps should intersect the crossroad at right angles. As shown in **Figure 9.41** and **Figure 9.42** islands can also be used in the terminal areas where ramps intersect the crossroads. The islands provide a means of channelizing the track into proper paths and can be effectively used for sign placements. Design of the islands should take into consideration initial or future signal installations at the ramp terminals.

Roundabout ramp terminal intersections may mitigate or minimize the potential for wrong-way entry due to the channelization provided by the splitter islands separating the entry and exit movements.

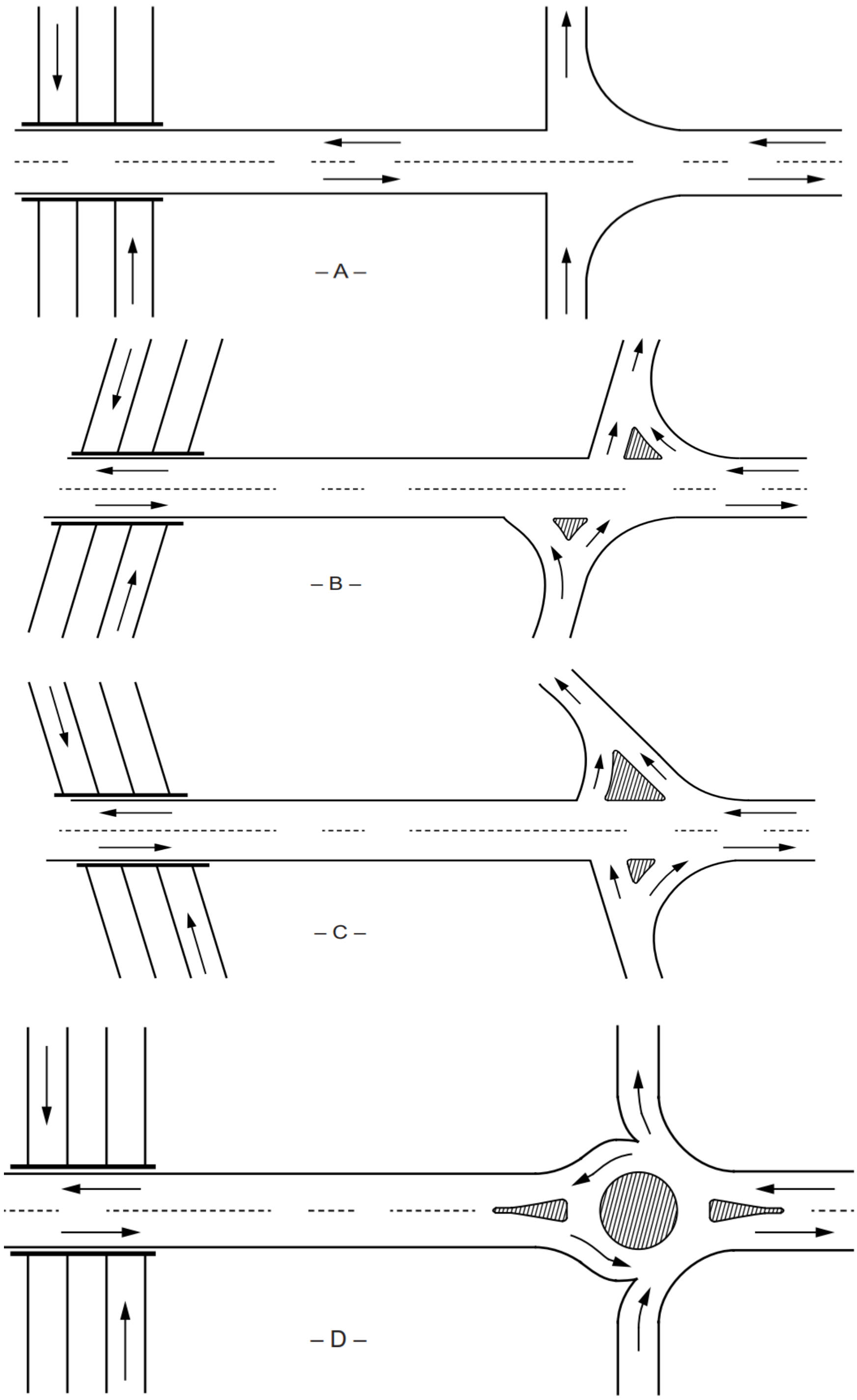


Figure 9.41 Two-Lanes Crossed Designs to Discourage Wrong-Way Entry

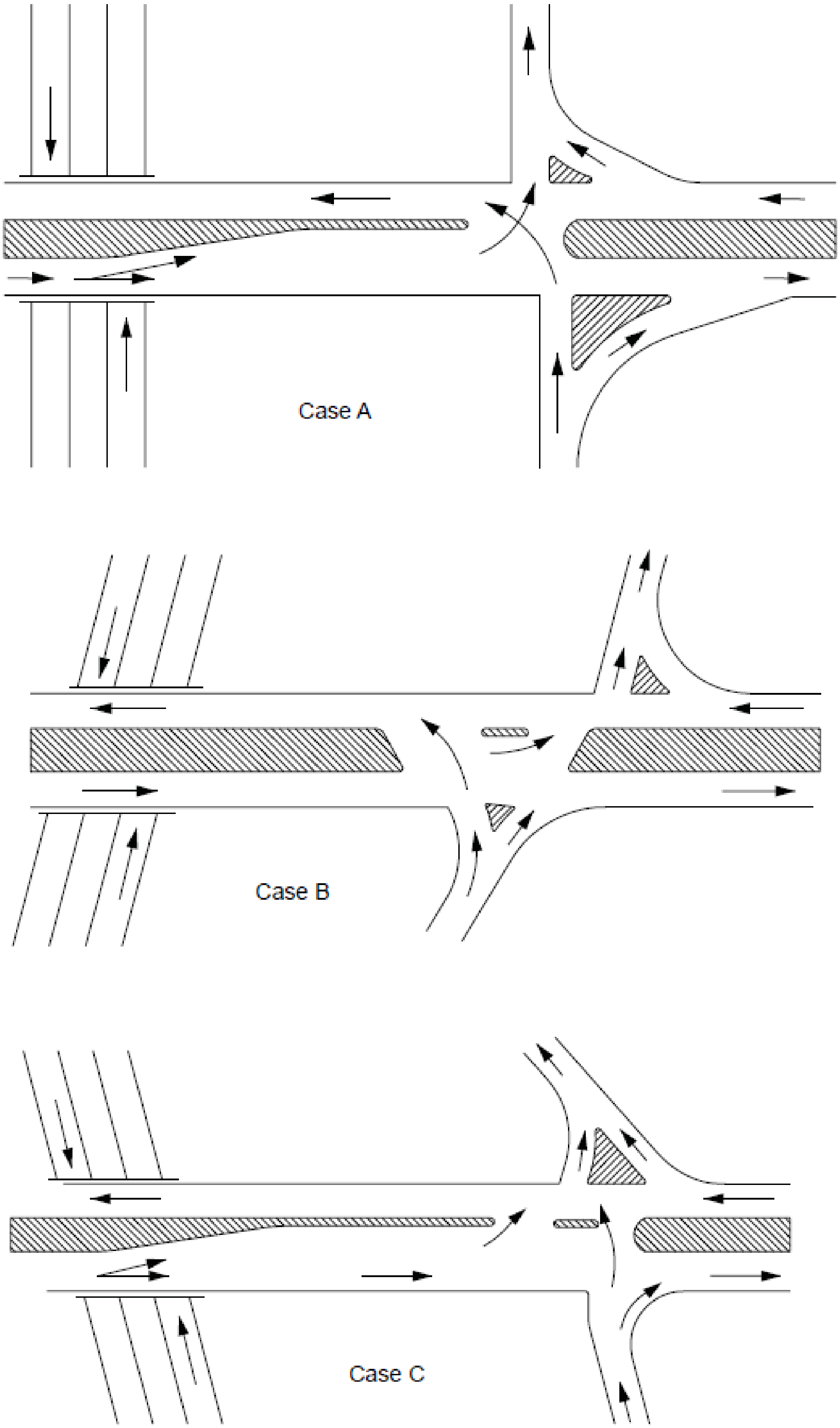


Figure 9.42 Divided Crossroad Designs to Discourage Wrong-way entry

On undivided crossroads a non-traversable median (except at turn points) introduced within the interchange limits helps prevent wrong-way entry on diamond, partial cloverleaf, and full cloverleaf interchanges. Provision of a median as a deterrent to wrong-way movement as illustrated in **Figure 9.42** is a very effective treatment. The median makes the left-turn movement onto the exit ramp terminal very difficult, and a short-radius curve or angular break is provided at the intersection of the left edge of the exit ramp and the crossroad to discourage wrong-way right turns from the crossroad. Where adjacent off- and-on ramps join a minor road, the ramp roadways should be separated. The ramp-crossroad intersection at a diamond interchange should be well removed from any other nearby intersection, such as a frontage road-crossroad intersection. Local road connections within the length of any exit ramp should be avoided. Temporary ramp terminals warrant special attention in layout details to avoid wrong-way entry paths.

Additional design techniques to reduce wrong-way movements are:

i. providing for all movements to and from the expressway/motorway to reduce intentional wrong-way entry,
ii. using conventional easily recognized interchange patterns to reduce driver confusion and hence wrong-way entry, and
iii. narrowing the arterial highway median opening to reduce the probability of left-turn movements onto expressway/motorway off-ramps.

Open sight distances throughout the entire length of the ramp help prevents wrong-way use. Especially important is the overview of the ramp terminal when approaching from the crossroad. The terminus of a beside exit ramp with a crossroad may appear to an unfamiliar driver on the crossroad as an entrance ramp, and wrong-way entry may occur at night when volumes are low and traffic control devices are less effective. Therefore, roadway lighting along the crossroad should be considered to enhance driver recognition of the intended path.

In the design of any interchange, consideration should be given to the likelihood of wrong-way travel and to the practical measures that may be taken in the design and traffic control for preventing or discouraging such usage.

9.4.4 RAMPS

9.4.4.1 TYPES AND EXAMPLES

The term “ramp” includes all types, arrangements, and sizes of turning roadways that connect two or more legs at an interchange. The components of a ramp are a terminal at each leg and a connecting road.

The geometry of the connecting road usually involves some curvature and a grade. Generally, the horizontal and vertical alignment of ramps is based on lower design speeds than the intersecting highways, but in some cases it may be equal. **Figure 9.43** illustrates several types of ramps and their characteristic shapes. Various configurations are used; however, each can be broadly classified as one of the types shown. Each ramp generally is a one-way roadway.

i. **Diagonal ramps** (**Figure 9.43A**) are provided at service interchanges and generally cater for all turning movements. They are almost always one-way but usually have both a left- and right-turning movement at the terminal on the minor intersecting road. A diagonal ramp may be largely tangent or wishbone in shape with a reverse curve. Diamond interchanges generally have four diagonal ramps.

ii. **A loop ramp** may have single turning movements (left or right) or double turning movements (left and right) at either or both ends. **Figure 9.43B** shows the case where there are only single turns made at both ends of the ramp. With this loop pattern, a left-turning movement is made without an at-grade crossing of the opposing through traffic. Instead, drivers making a left-turn travel beyond the highway separation, turn to the right through approximately 270 degrees to enter the other highway. The loop usually involves more indirect travel distance than any other type of ramp.

iii. With a **semidirect connection** (**Figure 9.43C**), the driver exits to the right first, heading away from the intended direction, gradually reversing, and passing around other interchange ramps before entering the other road. This semidirect connection may also be used for right turns, but there is little reason for its use if the conventional diagonal can be provided. A descriptive term frequently associated with this type of ramp is "jug-handle," the obvious plan shape. Travel distance on this ramp is less than that for a comparable loop and more than that for a direct connection.

iv. **Outer connector ramps (Figure 9.43D**) are exclusively for one right-turn movement and provide the most direct left-turn connection between two roadways.

v. **Directional ramps (Figure 9.43E)** that are used at system interchanges are exclusively for one left-turn movement and provide the most direct left-turn connection between two roadways. They must only be used as part of a major fork or branch connection.

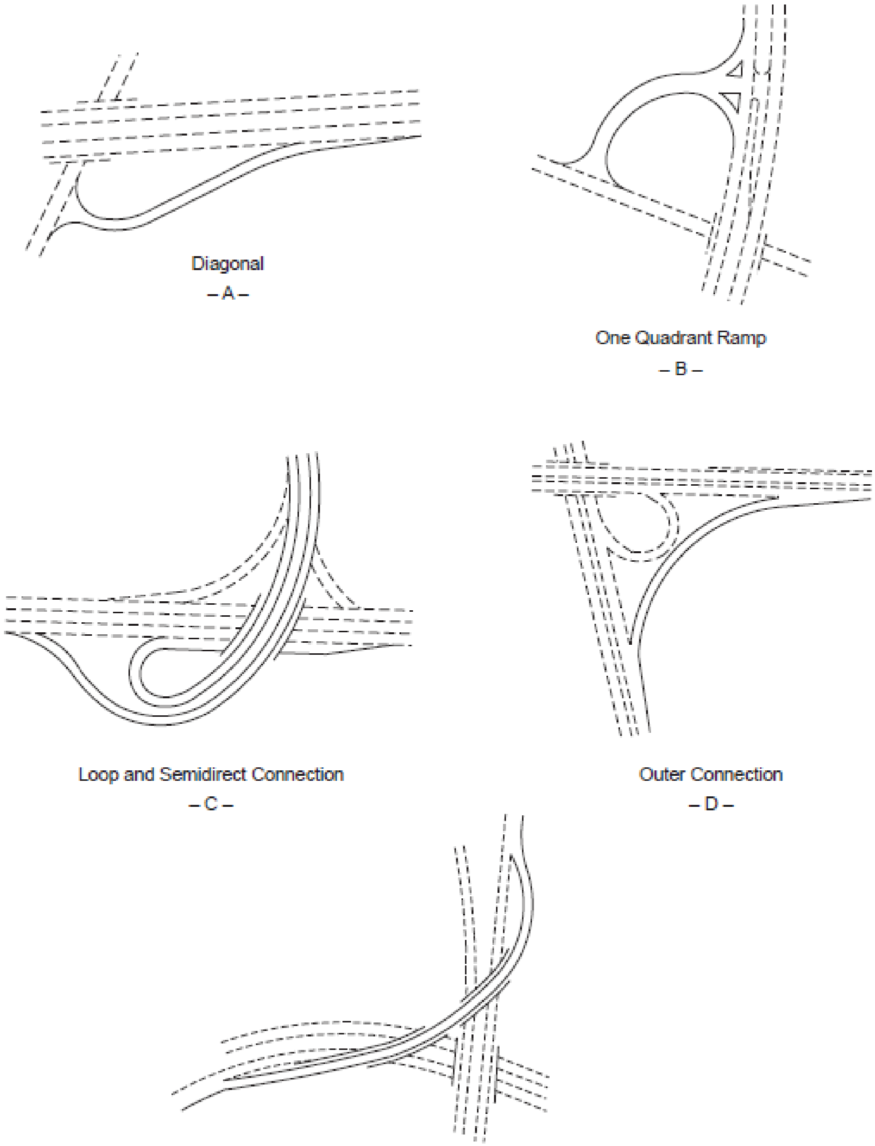


Figure 9.43 General Types of Ramps

9.4.5 INTERCHANGE DESIGN CRITERIA

9.4.5.1 MAIN LINE ALIGNMENT DESIGN STANDARD

For drivers traveling along the main line of a high-speed road, it is desirable to be able to see an interchange from as far away as possible. Moreover, the road must have a structure that enables vehicles to safely and smoothly access the interchange.

Concerning the high-speed road main line alignment elements (horizontal curve radius, vertical gradient, vertical curve radius) around the interchange ramp, as is indicated in **Table 9.3** , design values that are gentler than the desirable minimum values corresponding to each road design speed should be used. shows the absolute minimum values for main line alignment elements.

Table 9.3 Desirable Minimum Values for Main Line (around the Interchange Ramp) Alignment Elements

Design class		A1, B1, C1					A2, B2, C2			
Design speed (km/h)		120	100	80	60	50	80	60	50	40
Minimum horizontal curve radius (m)		2,000	1,500	1,100	500	300	900	450	250	200
Maximum vertical gradient (%)		2.0	2.0	3.0	4.5	5.0	4.0	5.0	5.5	6.0
Minimum vertical curve radius (m)	Crest	45,000	25,000	12,000	6,000	4,000	9,000	4,500	2,500	1,400
	Sag	16,000	12,000	8,000	4,000	3,000	6,000	3,000	2,000	1,400

Table 9.4 Absolute Minimum Values for Main Lane (around the Interchange Ramp) Alignment Elements

Design class		A1, B1, C1					A2, B2, C2			
Design speed (km/h)		120	100	80	60	50	80	60	50	40
Minimum horizontal curve radius (m)		1,500	1,000	700	350	200	500	200	150	100
Maximum vertical gradient (%)		2.0	3.0	4.0	5.5	6.0	5.0	6.0	6.5	7.0
Minimum vertical curve radius (m)	Crest	23,000	15,000	6,000	3,000	2,000	4,500	2,500	1,200	700
	Sag	12,000	8,000	4,000	2,000	1,500	3,000	1,500	1,000	700

9.4.5.2 RAMP ALIGNMENT AND DESIGN SPEED

Generally speaking, both ends of a ramp connect to roads that have high design speed, or one end connects to an intersection that is subject to temporary stopping. Accordingly, since a ramp always entails speed changes rather than uniform speed, the ramp alignment must be designed to smoothly adjust to changes in travel speed.

The alignment of an exit ramp off a high-speed road, should be designed taking into consideration the awareness of the driver with regard to the reduced speed.

Ramp design speeds are indicated for reference according to connecting road design classifications and design speed. The ramp should be designed within the scope prescribed in **Table 9.5** while considering traffic volume, vehicle type composition, terrain and so on. Concerning the Upper and Lower Roads, for example, in the case of an interchange that connects a Design Class A road with a Design Class B Road, the Upper Road is the Design Class A Road, and the Lower Road is the Design Class B Road. In a situation where two roads of the same Design Class intersect, traffic volume shall be used as criteria to determine the Upper Road.

Table 9.5 Ramp Design Speed according to Connected Roads and Design Speed

Design Class		Upper Roads									
		A						B, C			
Lower Roads	Design speed(km/h)	120	100	80	60	50	40	80	60	50	40
A	120	80									
		60									
		50									
		(40)									
	100	80	80								
		60	60								
		50	50								
		(40)	(40)								
	80	80									
		60	60	60							
		50	50	50							
		40	40	40							
	60	60	60	60	60						
		50	50	50	50						
		40	40	40	40						
	50	50	50	50	50	50					
		40	40	40	40	40					
	40	40	40	40	40	40	40				
B, C, D	80	60	60	50	40	40	40	50			
		50	50	40	35	35	35	40			
		40	40	35	30	30	30	35			
	60	50	50	50	40	40	40	50	50		
				40	35	35	35	40	40		
		40	40	35	30	30	30	35	35		
									30		
	50	50	50	50	40	40	40	50	40	40	
				40	35	35	35	40	35	35	
		40	40	35	30	30	30	35	30	30	
	40			40	40	40	40	40	40	40	40
		40	40		35	35	35		35	35	35
				35	30	30	30	35	30	30	30
E, Intersection for Stop, Toll Booth		40	40	40	40	40	40	40	30	30	30
		35	35	35	35	35	35	35			
		30	30	30	30	30	30	30	25	25	25

9.4.5.3 RAMP CROSS-SECTION COMPOSITION

In determining the ramp cross-section composition, it is necessary to consider the following:

i. Main line design speed

ii. Main line cross-section composition

iii. Traffic volume

iv. Vehicle types and composition

v. Ramp design speed

Ramps can either be a 1-lane ramp or a 2-lane ramp. Usually, the 1-direction 1-lane type is adopted, however, 2-lane ramps can be used in 1 direction or 2 directions.

As shown in , the type of ramp is determined by the Design Class of the Upper Road. The cross-section composition based on the type of ramp i.e. Type I, Type II, and Type III is shown in **Table 9.7** while the cross-section details for Type-I, Type-II and Type-III ramps are shown in **Table 9.8, Table 9.9** and **Table 9.10** respectively. The absolute minimum widths are shown in brackets in the tables.

Table 9.6 Ramp Type according to Design Class

Upper Road		Ramp Type
Design Class	A*	Type I
	B*	Type I and II
	C, D and E	Type III

* In urban areas with right-of-way restriction, Type III type can be used.

Table 9.7 Width Composition according to Ramp Type

Ramp Type	Lane Width (m)	Shoulder Width (m)			Total Ramp Width (m)	
		1 direction 1 lane		1 direction 2 lanes, 2 directions 2 lanes	1 direction 1 lane	1 direction 2 lanes, 2 directions 2 lanes
		Left side	Right side	Left and Right sides		
Type I	3.50	1.00	2.50 (1.5*)	0.75	7.00 (6.0*)	8.50
Type II	3.25	0.75	1.50	0.75	5.50	8.00
Type III	3.25	0.75	1.25	0.50	5.25	7.50

***Absolute minimum value**

Table 9.8 Type I Ramp Cross-section Composition (Unit: m α, β: Widening)

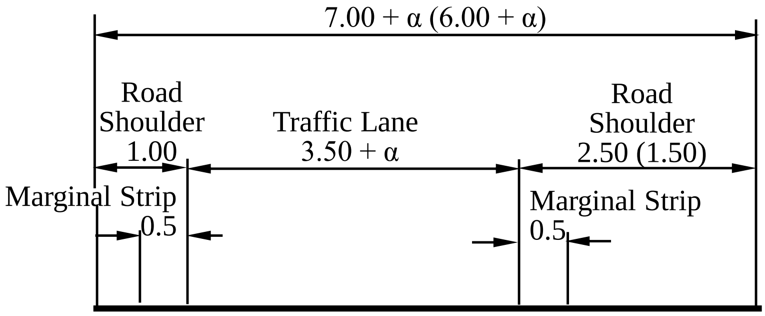
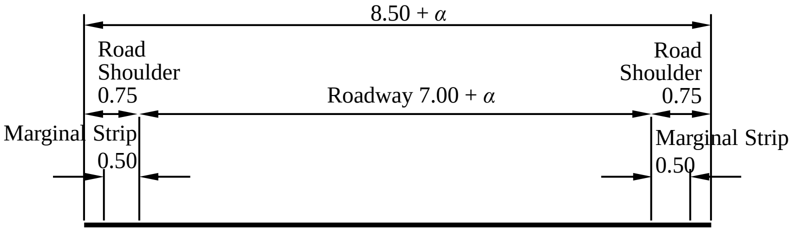
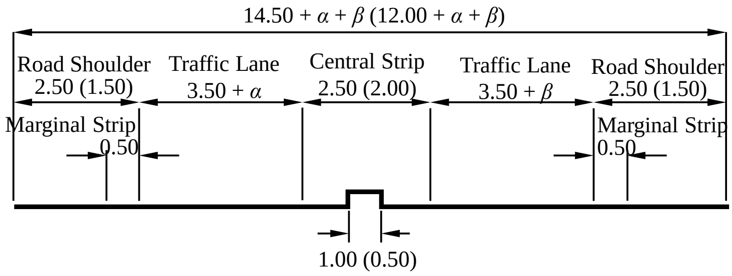
Number of Lanes and Directions	Cross-section Composition
1 direction 1 lane	
1 direction 2 lanes, 2 directions 2 unseparated lanes	
2 directions 2 separated lanes	

Table 9.9 Type II Ramp Cross-section Composition (Unit: m, α, β: Widening)

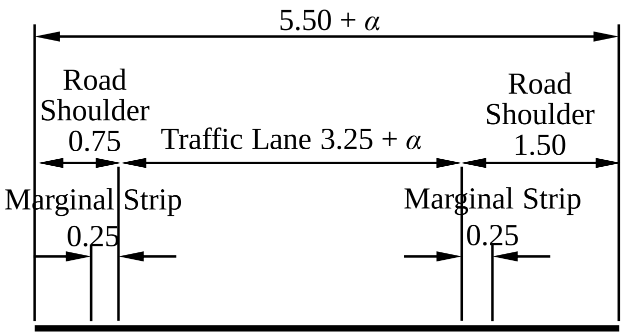
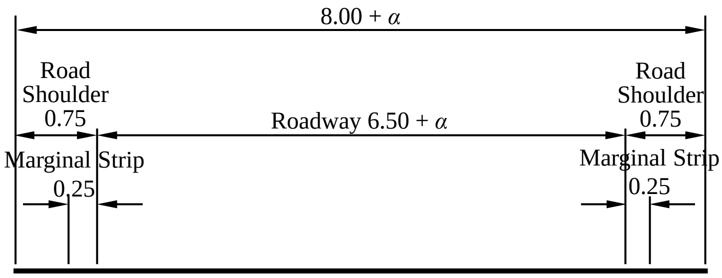
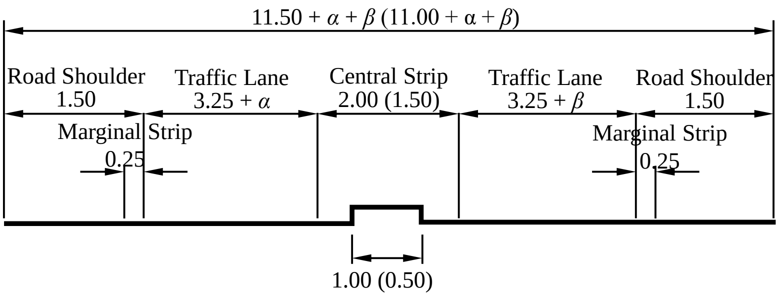
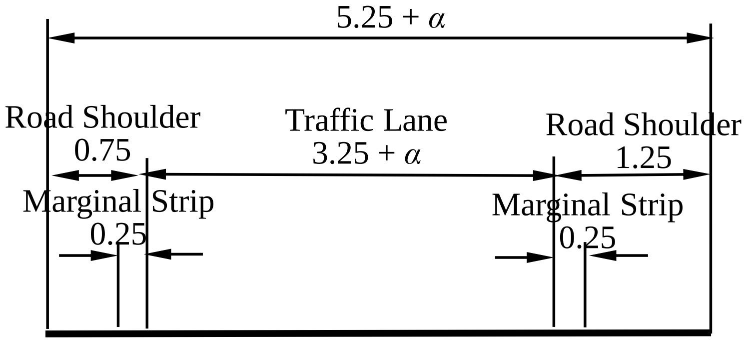
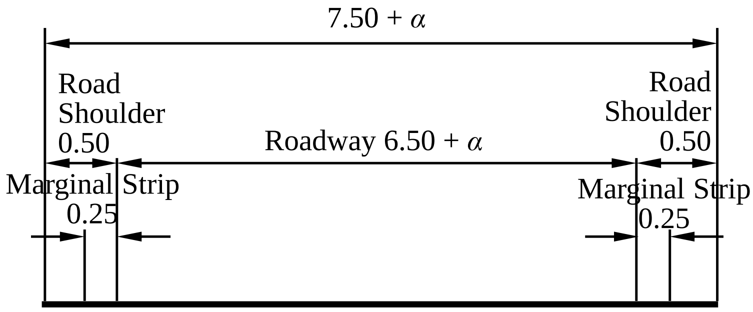
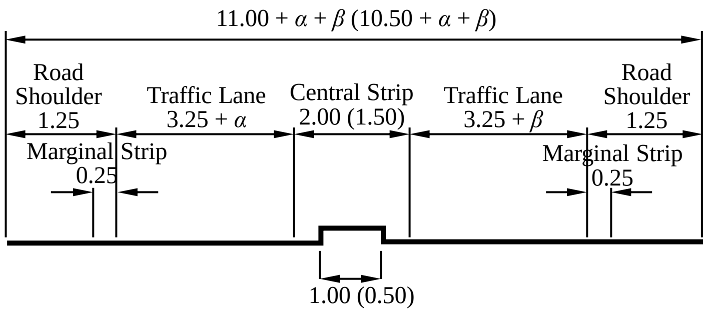
Number of Lanes and Directions	Cross-section Composition
1 direction 1 lane	
1 direction 2 lanes 2 directions 2 unseparated lanes	
2 directions 2 separated lanes	

Table 9.10 Type III Ramp Cross-section Composition (Unit: m, α, β: Amount of widening)

Number of Lanes and Directions	Cross-section Composition
1 direction 1 lane	
1 direction 2 lanes 2 directions 2 unseparated lanes	
2 directions 2 separated lanes	

9.4.5.3.1 RAMP SHOULDER

Design consideration for ramp shoulders are as follows;

i. Shoulders should be provided on ramps and ramp terminals in interchange areas to provide a space that is clear of the travelled way for emergency stopping, to minimize the effect of breakdowns, and to aid drivers who may be confused.

ii. When paved shoulders are provided on ramps, they should have a uniform width for the full length of ramp.

iii. The left and right shoulder widths may be reversed (swapped) if needed to provide additional sight distance.

9.4.5.4 MINIMUM RAMP RADIUS

The minimum ramp curve radius is an important element in deciding the size of an interchange and has a major impact on traffic safety. The minimum curve radius is derived for each design speed based on the maximum superelevation value and the maximum permissible side friction coefficient value. Based on these conditions, as shown in **Table 9.11**, the adopted minimum curve radius is smaller than the absolute minimum values specified according to the ramp design speed. This is because drivers are more alert at entrance to ramps.

The maximum superelevation should be 9% based on the explanation given in **Section 7.2.3.3.1** for general roadway. The superelevation rate with respect to the ramp design speed and curve radius is shown in **Table 9.12.**

Table 9.11 Ramp Minimum Curve Radius

Design speed (km/h)	Minimum curve radius (m)	
	Desirable Minimum	Absolute Minimum
80	280	230
60	140	110
50	90	70
40	50	40
35	40	30
30	30	20
25	20	15

Table 9.12 Ramp Curve Radius and Superelevation

Ramp design speed (km/h)				Superelevation (%)
80	60	50	40,35,30,25	
Curve radius (m)				
230-329	110-179	70-119	15-69	9
330-379	180-219	120-159	70-89	8
380-449	220-269	160-199	90-129	7
450-539	270-329	200-239	130-159	6
540-669	330-419	240-309	160-209	5
670-869	420-559	310-409	210-279	4
870-1,239	560-799	410-589	280-399	3
1,240-3,500	800-2,000	590-1,300	400-800	2

9.4.5.5 RAMP WIDENING

The widening of ramp is designed according to **Table 9.13** through **Table 9.15** based on the ramp type and curve radius.

Table 9.13 Ramp Widening (case of 1 direction 1 lane)

Type I	Type II	Type III	Widening (m)
Curve radius (m)			
		15-20	3.00
15-20	15-20	21-22	2.75
21-22	21-22	23-24	2.50
23-24	23-24	25-27	2.25
25-26	25-27	28-31	2.00
27-28	28-31	32-36	1.75
29-31	32-35	37-43	1.50
32-35	36-43	44-53	1.25
36-41	44-53	54-71	1.00
42-47	54-71	72-103	0.75
48-57	72-99	104-199	0.50
58-71	100-189	200-699	0.25
72-	190-	700-	0.00

Table 9.14 Ramp Widening (case of 1 direction 2 lanes and 2 directions 2 lanes, Design Class A)

Type I	Type II	Type III	Widening (m)
Curve radius (m)			
		15-21	5.00
		21-22	4.50
	15-21		4.25
		22-23	4.00
15-21	21-22	23-24	3.75
	22-23	24-25	3.50
21-22	23-24	25-26	3.25
22-23	24-25	26-27	3.00
23-24	25-26	28-29	2.75
24-25	26-27	30-31	2.50
25-26	27-28	32-33	2.25
26-27	29-30	34-35	2.00
27-28	31-33	36-38	1.75
29-30	33-35	39-42	1.50
31-32	36-38	43-47	1.25
33-35	39-41	48-52	1.00
36-38	42-46	53-59	0.75
39-42	47-51	60-69	0.50
43-46	52-59	70-83	0.25
≥47	≥60	≥84	0.00

Table 9.15 Ramp Widening (case of 1 direction 2 lanes and 2 directions 2 lanes, other than design class A)

Type I	Type II	Type III	Widening (m)
Curve radius (m)			
		15-21	3.50
		21-22	3.25
	15-21	22-23	3.00
	21-22	23-24	2.75
	22-23	24-25	2.50
15-21	23-24	25-26	2.25
21-22	24-25	27-28	2.00
22-23	25-26	29-30	1.75
23-24	27-28	31-33	1.50
25-26	29-30	34-37	1.25
27-28	31-33	38-41	1.00
29-30	34-37	42-47	0.75
31-33	38-41	48-55	0.50
34-37	42-47	56-65	0.25
38-	48-	66-	0.00

9.4.5.6 RAMP TRANSITION CURVE

The transition curve is determined based on the ramp design speed. The standard values of the clothoid curve minimum parameter (A) are provided in **Table 9.16.**

Assuming the curve radius to be R and the transition curve length to be L, the general formula for clothoid curve is given by **Equation 9.1**.

$$R \cdot L = A^2 \tag{9.1}$$

Where,

A: clothoid parameter (m), is given by **Equation 9.2**

$R/3 \leq A \leq R$

$$A = \sqrt{\frac{V^3}{3.6^3 C}} \tag{9.2}$$

Where C is the centrifugal acceleration change rate (m/s^3)

Table 9.17 shows the minimum curve radius above which transition curves can be eliminated using **Equation 9.3**.

$$R = \sqrt[3]{\frac{A^4}{24S}} \tag{9.3}$$

Where S is the shift (m) =0.2m

Table 9.16 Ramp Transition Curve Minimum Parameters (A)

Design speed (km/h)	80	60	50	40	35	30	25
Centrifugal acceleration Change rate, C (m/s^3)	0.60	0.90	1.05	1.15	1.20	1.25	1.30
Minimum parameter, A (m) Calculated	135	71.7	50.5	34.5	27.7	21.5	16.1
Minimum parameter, A (m) Design value	140	70	50	35	30	20	15

Table 9.17 Radii Where There Are No Need For Transition Curve

Design speed (km/h)	80	60	50	40	35	30	25
Minimum curve radius (m) calculated	411	177	111	67	-	-	-
Minimum curve radius (m) Design value	410	180	110	70	70	70	70

9.4.5.7 RAMP SIGHT DISTANCE AND VERTICAL CURVE

The ramp stopping sight distance, as with general roadways in **Table 7.10** of **Section 7.2.6.1** should be regarded as the standard.

The ramp vertical gradient is generally not directly related to design speed as much as compared to general roadways. However, since the vertical gradient should be made gentler as the ramp design speed increases, upper limit values for gradient corresponding to the Design Class and Design Speed of the Upper Road as specified in **Table 9.18** should be adopted. Absolute minimum values can be applied in cases where it is unavoidable due to terrain and other special reasons. However, the maximum absolute value of 10% is a threshold value, and it is desirable to adopt a gentler value as far as possible.

Similarly, concerning the ramp vertical curve, minimum values for the vertical curve radius and vertical curve length are specified according to the ramp design speed as shown in **Table 9.19**.

Table 9.18 Maximum Ramp Vertical Gradient

Design speed (km/h)	Maximum vertical gradient (%)			
	Design Class A & B		Design Class C, D, & E	
	Desirable value	Absolute value	Desirable value	Absolute value
80	4.0	6.0	-	-
60	5.0	7.0	6.0	8.0
50	5.5	7.5	7.0	9.0
40	6.0	8.0	8.0	10.0
35	6.5	8.5	8.5	10.0
30	7.0	9.0	9.0	10.0
25	7.5	9.5	9.5	10.0

Table 9.19 Ramp Minimum Vertical Curve

Design speed (km/h)	80	60	50	40	35	30	25
Crest vertical curve radius (m)	3,000	1,400	800	400	350	250	200
Sag vertical curve radius (m)	1,800	1,000	700	500	350	250	200
Vertical curve length (m)	70	50	40	35	30	25	15

9.4.5.8 RAMP TERMINALS

The terminal of a ramp is that portion adjacent to the through-travelled way, including speed change lanes, tapers, and islands. Ramp terminals may be the at-grade type, as at the crossroad terminal of diamond or partial cloverleaf interchanges, or the free-flow type where ramp traffic merges with or diverges from high-speed through traffic at flat angles. Design elements for the at-grade type are discussed in Chapter 8, and those for the free-flow type are discussed in the following sections.

Terminals are further classified as either single or multilane, according to the number of lanes on the ramp at the terminal, and as either a taper or parallel type, according to the configuration of the speed-change lane.

9.4.5.8.1 LEFT-SIDE ENTRANCES AND EXITS

Left-side entrances and exits are contrary to driver expectancy when intermixed with right-side entrances and exits. Therefore, extreme care should be exercised to avoid left hand entrances and exits in the design of interchanges.

Left-side ramp terminals break up the uniformity of interchange patterns and generally create uncertain operation on through roadways. Left-side entrances and exits are considered satisfactory for collector-distributor roads; however, their use on high-speed, free-flow ramp terminals is not recommended. Because left-side entrances and exits are contrary to driver expectancy, special attention should be given to acceleration/deceleration lengths, signing, and the provision for decision sight distance preceding the approach nose of the exit ramp in order to alert the driver that an unusual situation exists. There should be a clear view of the whole of the exit terminal. If it is not practical to provide decision sight distance because of horizontal or vertical curvature or if relocation of decision points is not practical additional traffic control devices for advance warning of the conditions should be considered.

9.4.5.8.2 TERMINAL LOCATION AND SIGHT DISTANCE

Where diamond ramps and partial cloverleaf arrangements intersect the crossroad at grade, an at-grade intersection is formed. Desirably, this intersection should be located an adequate distance from the separation structure to provide adequate sight distance for all approaches. Sight distance criteria are detailed in **Section 7.2.6**. Drivers prefer and expect to exit in advance of the separation structure. The use of collector–distributor roads and single exits on partial cloverleafs and other types of interchange configurations automatically positions the main-line exit in advance of the separation structure.

Designs that result in an exit concealed behind a crest vertical curve should be avoided, especially on high-speed facilities. Desirably, high-speed entrance ramp terminals should be located on descending grades to aid truck acceleration. Adequate sight distance at entrance terminals should be available so that merging traffic on the ramp can adjust speed to merge into gaps on the main facility. Loop ramps that are located beyond the structure, as in the conventional cloverleaf or in certain arrangements of partial cloverleafs, usually need a parallel deceleration lane. The actual exit from the auxiliary lane is difficult for drivers to locate even when sight distance is not restricted by a vertical curve. Placing the exit in advance of the structure via a single exit alleviates this concern.

9.4.5.8.3 RAMP TERMINAL DESIGN

Profiles of ramp terminals should be designed in association with horizontal curves to avoid sight restrictions that will adversely affect operations. At an exit into a ramp on a descending grade, a horizontal curve ahead should not appear suddenly to a driver. Instead, the initial crest vertical curve should be made longer and sight distance over it should be increased so that the location and direction of the horizontal curve are apparent to the driver sufficiently in advance to provide time for the driver to respond appropriately. At an entrance terminal from a ramp on an ascending grade, the portion of the ramp intended for acceleration and the ramp terminal should closely be parallel to the through-lane profile to permit entering drivers to have a clear view of the through road ahead, to the side, and to the rear.

It is desirable that profiles of highway ramp terminals be designed with a platform on the ramp side of the approach nose or merging end. This platform should be at least 60 m in length and should have a profile that does not greatly differ from that of the adjacent through-traffic lane. A platform area should also be provided at the at-grade terminal of a ramp. The length of this platform should be determined from the type of traffic control and the capacity at the terminal.

9.4.5.8.4 TRAFFIC CONTROL

On major roads, ramps are arranged to facilitate all turning movements by merging or diverging manoeuvres. On minor roads, some of the left-turning movements often are made at grade. The left-turning movements leaving the crossing roadway preferably should have median left-turn lanes. For low-volume crossroads, the left-turning movements from ramps normally should be controlled by stop signs. The right-turning movements from ramps into multilane crossroads should be provided with an acceleration lane or generous taper or should be controlled by stop or yield signs. Ramps approaching stop signs should be nearly perpendicular to the crossroad and be nearly level for storage of several vehicles. Ramp terminals at crossroads can also be controlled by roundabouts.

Traffic signal controls may be needed at ramp terminals on the minor road where there is sufficient volume of through and turning traffic. In such cases, the intersections formed at the terminals should be designed and operated in the same manner as any other traffic-signal-controlled intersection at grade. Signal controls should be avoided on express-type roads and confined to the minor roads on which other intersections are at grade and some of which are signalized. In or near urban areas, signal control is especially appropriate at ramp terminals on roads that cross over or under an expressway/motorway. Here the turning movements usually are sizable, and the cost of right-of-way and improvements is high. As a result, appreciable savings may be realized by the use of diamond ramps with high-type terminals on the expressway/motorway and signalized terminals on the crossroads.

9.4.5.8.5 DISTANCE BETWEEN A FREE-FLOW TERMINAL AND STRUCTURE

The terminal of a ramp should not be near the grade-separation structure. If it is not practical to place the exit terminal in advance of the structure, the exiting terminal on the far side of the structure should be well removed to provide drivers leaving the through lanes some distance after passing the structure to see the exit and begin the exit manoeuvre. Decision sight distance should be provided, where practical. The distance between the structure and the approach nose at the ramp terminal should be sufficient for exiting drivers to leave the through lanes without undue hindrance to through traffic. Such distance also aids drivers who enter from a ramp terminal on the far side of the structure, so they have a clear view well back on the through road behind or to the left. Such drivers may be able to see back along the road beyond the limits of the structure, but as a general rule, the entering driver's view is obstructed by the crest of the profile at an overpass and by the columns, abutments, and approach walls at an underpass.

The conditions for determining the distance between a structure and the far side approach nose are similar to those discussed for speed-change lanes. A minimum distance between the structure and an exit nose of about the same length as a speed-change taper is suggested. Decision sight distances are desirable but are not rigid controls for ramp design. Topographic or right-of-way controls may govern the overall shape of the ramp.

9.4.5.8.6 DISTANCE BETWEEN SUCCESSIVE RAMP TERMINALS

On urban expressway/motorway, two or more ramp terminals are often located in close succession. To provide sufficient weaving length and adequate space for signing, a reasonable distance should be provided between successive ramp terminals. If the ramp terminals are too close to each other, there is a greater possibility that drivers will become confused and make mistakes. To ensure safe and smooth traffic, it is important to secure clearance between the connections so that drivers have less difficulty in making judgments.

According to AASHTO, the total time it takes between a driver seeing a sign, etc. and reacting and the time it takes to change lanes is between 5-10s, and it is necessary to secure no less than the distances shown in **Table 9.20** as a standard. If the distance between the connections cannot be secured due to unavoidable reasons, it is desirable to caution motorists by placing a warning sign beforehand. The distances labelled L in the **Table 9.20** are measured between the painted noses. **Figure 9.44** shows the definition of the ramp spacing dimension.

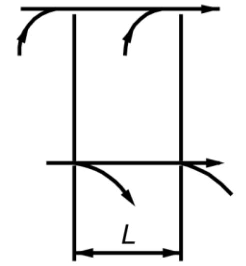
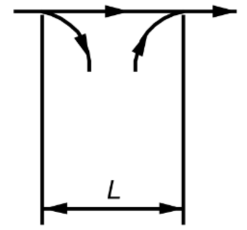
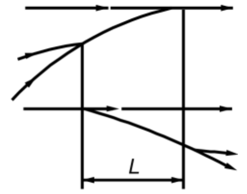
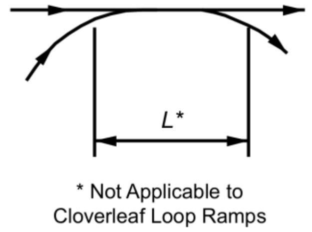
Where an entrance ramp is followed by an exit ramp, the absolute minimum distance between the successive noses is governed by weaving considerations. The spacing policy for EN-EX ramp combinations is not applicable to cloverleaf loop ramps. For these interchanges, the distance between EN-EX ramp noses is primarily dependent on loop ramp radii and roadway and median widths. A recovery lane beyond the nose of the loop ramp exit is desirable.

When the distance between the successive noses is less than 450m, the speed-change lanes should be connected to provide an auxiliary lane. This auxiliary lane improves traffic operation over relatively short sections of the expressway/motorway route and is not considered an addition to the basic number of lanes.

There are two typical scenarios of an EX-EN combination. The shortest dimension 120 to 150m would be that of an exit followed by the entrance for a "buttonhook" or "gullwing" design where the expressway/motorway ramps are serving a local road parallel to the expressway/motorway versus a local road crossing the expressway/motorway as an over or underpass. The second scenario would be when an exit ramp and subsequent entrance ramp are servicing grade separated ramps (ramp braids).

Due to the vertical and horizontal relationships of this configuration, the spacing will generally be greater than the minimum values in **Table 9.20** reflecting a condition where both ramp profiles are changing.

Table 9.20 Distance between Ramp Connections

<table>
<tr><th colspan="2">EN-EN or EX-EX</th><th colspan="2">EX-EN</th><th colspan="2">Turning Roadways</th><th colspan="4">EN-EX (Weaving)</th></tr>
<tr><td colspan="2"></td><td colspan="2"></td><td colspan="2"></td><td colspan="4"></td></tr>
<tr><td rowspan="2">Full Exp/Mway</td><td rowspan="2">CDR</td><td rowspan="2">Full Exp/Mway</td><td rowspan="2">CDR</td><td rowspan="2">System Inter-change</td><td rowspan="2">Service Inter-change</td><td colspan="2">System to Service Interchange</td><td colspan="2">Service to Service Interchange</td></tr>
<tr><td>Full Exp/Mway</td><td>CDR</td><td>Full Exp/Mway</td><td>CDR</td></tr>
<tr><th colspan="10">Minimum Lengths Measured between Successive Ramp Terminals (m)</th></tr>
<tr><td>300</td><td>240</td><td>150</td><td>120</td><td>240</td><td>180</td><td>600</td><td>480</td><td>480</td><td>300</td></tr>
</table>

Notes: EN - Entrance
EX - Exit
Full Exp/Mway - Full expressway/motorway
CDR - Collector distributor road

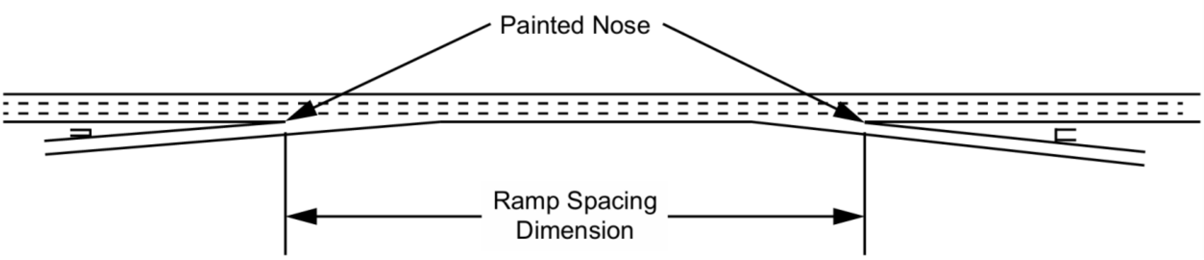


Figure 9.44 Ramp Spacing Dimension

9.4.5.8.7 SPEED-CHANGE LANES

Drivers leaving a highway at an interchange are required to reduce speed as they exit onto a ramp. Drivers entering a highway from a turning roadway accelerate until the desired highway speed is reached. Because the change in speed is usually substantial, provision should be made for acceleration and deceleration to be accomplished on auxiliary lanes to minimize interference with through traffic and to reduce crash potential. Such an auxiliary lane, including tapered areas, may be referred to as a speed-change lane. The terms “speed-change lane,” “deceleration lane,” or “acceleration lane” as used herein apply broadly to the added lane that joins the travelled way of the highway to the turning roadway and do not necessarily imply a definite lane of uniform width. This additional lane is a part of the elongated ramp terminal area.

A speed-change lane should have sufficient length to enable a driver to make the appropriate change in speed between the highway and the turning roadway. Moreover, in the case of an acceleration lane, there should be additional length to permit adjustments in speeds of both through and entering vehicles so that the entering driver can position the vehicle opposite a gap in the through-traffic stream and then manoeuvre into the stream before the acceleration lane ends. This latter consideration also influences both the configuration and length of an acceleration lane.

Two general forms of speed-change lanes are (1) the taper type and (2) the parallel type. The taper type provides a direct entry or exit at a flat angle, whereas the parallel type has an added lane for changing speed. Either type, when properly designed, will operate satisfactorily. However, the parallel type is favoured in certain areas. Furthermore, taper type entrances have been found to encourage merge speeds that are closer to expressway/motorway speeds than parallel type entrances, however; where there are main-line volumes that meet or exceed capacity, parallel type entrances allow additional flexibility to drivers in selecting a merge location.

A. EXIT (DECELERATION LANE)

There are generally two types of terminal exits: the parallel type (**)**, and the taper type, but the taper type (**Figure 9.46**) is more commonly used. Especially in cases where high-speed roads with high design speed and curved road horizontal alignment are involved, as a rule, it is desirable to adopt the taper type. The taper type exit fits the direct path preferred by most drivers, permitting them to follow an easy path within the diverging area. The taper-type exit terminal beginning with an outer edge alignment break usually provides a clear indication of the point of departure from the through lane and has generally been found to operate smoothly on high-volume roads. The divergence angle is usually between 2 and 5 degrees.

As is shown in **Table 9.21**, the standard value from the point where the width of the deceleration lane is secured to the end of the diverging lane is specified according to the main line design speed.

The deceleration lane is usually a single lane, however, in cases of two lanes, the standard value is obtained by multiplying the values in **Table 9.21** by between 1.2-1.5. Where the traffic volume leaving the main line at an exit terminal exceeds the design capacity of a single lane, a two-lane exit terminal should be provided. To satisfy lane-balance needs and not reduce the basic number of through lanes, it is usually appropriate to add an auxiliary lane upstream from the exit. A distance of approximately 450m is recommended to develop the full capacity of a two-lane exit. As with single lane exits, attention should be given to obtaining the appropriate deceleration distance between the exit and first horizontal curve on the ramp. Typical designs for two-lane exit terminals are shown in **Figure 9.47**; the taper is illustrated in **Figure 9.47A** and the parallel type in **Figure 9.47B**.

In cases where the deceleration lane is provided with downhill gradient, as is shown in **Table 9.22,** the design value is obtained by multiplying the deceleration lane length by an adjustment factor.

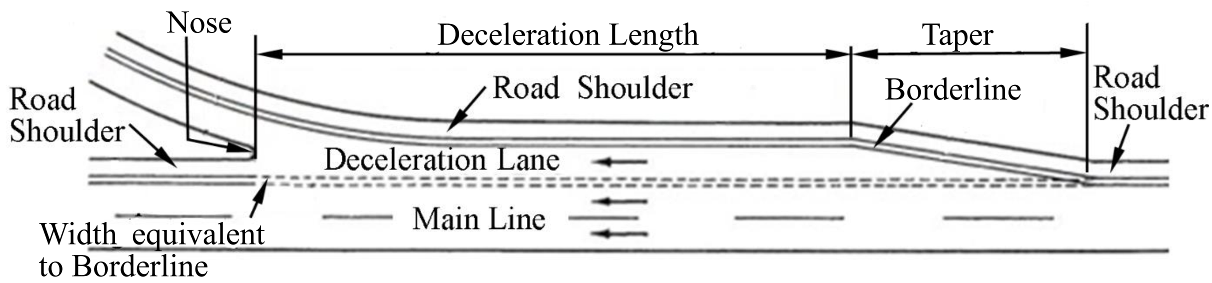
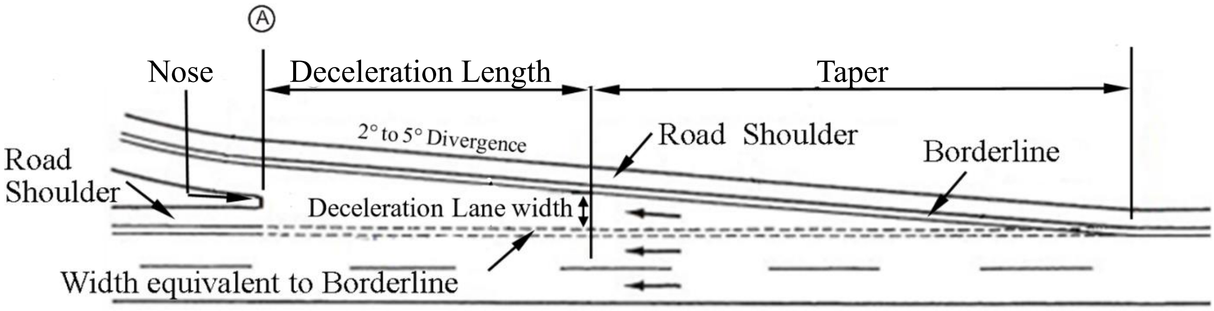


Figure 9.45 Parallel Type Deceleration Lane



Note: Point A controls speed on the ramp. The deceleration lane should not start back on the curvature of the ramp unless the radius equals 300m or more.

Figure 9.46 Taper Type Deceleration Lane

Table 9.21 Ramp Deceleration Lane Length

Main line design speed (km/h)	Rural		Urban	
	Deceleration length (m)	Taper length for Parallel type (m)	Deceleration length (m)	Taper length for Parallel type (m)
120	190	100	110	70
100	110	90	80	60
80	80	70	50	45
60	80	60	30	20
50	60	50	20	20
40	50	40	20	15
30	30	20	20	10
20	20	20	10	10

For taper type, the taper lane length is determined by runoff rate of 1/15 to 1/20. Normally the taper length for taper type deceleration lane is longer than parallel type.

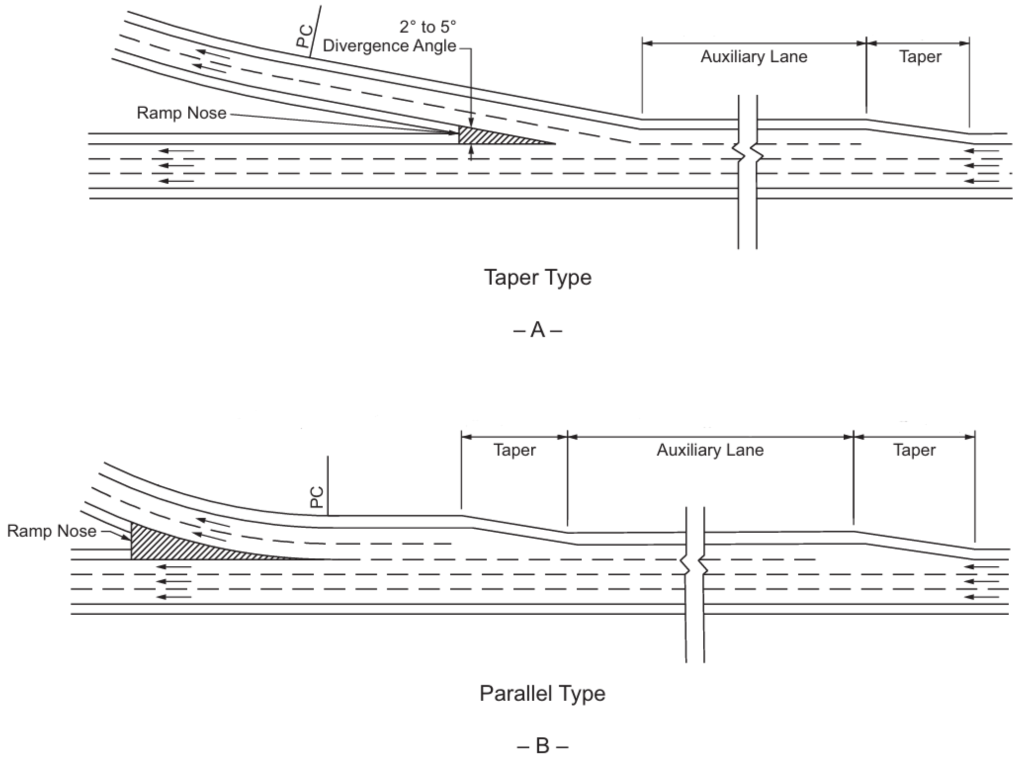


Figure 9.47 Two-Lane Exit Terminals

Table 9.22 Ramp Deceleration Lane Length Adjustment Factor

Average gradient i (%) of main line	0<i≦2	2<i≦3	3<i≦4	4<i
Downhill deceleration lane length adjustment factor	1.00	1.10	1.20	1.30
Uphill deceleration lane length adjustment factor	1.00	0.9		0.8

B. ENTRANCE (ACCELERATION LANE)

The acceleration lane is the section from the end of the merging lane to the end of the tapered lane. Similar to the deceleration lane, the acceleration lane can either be the parallel type or the taper type, but the taper type is not adopted as much as it is for the deceleration lane (**Figure 9.48**). This is because the acceleration lane is longer than the deceleration lane and is mainly used by accelerating vehicles, hence there is no need for a track directly merging into the main lane.

As is shown in **Table 9.23**, the standard value from the end of the branch to the point where the width of the acceleration lane is secured is specified according to the main line design speed.

The acceleration lane is usually a single lane, however, in cases of two-lanes, the standard value is obtained by multiplying the values in **Table 9.23** by between 1.2-1.5. Two-lane entrances are warranted for two situations: either as branch connections or because of capacity needs for the on-ramp. To satisfy lane-balance needs, at least one additional lane should be provided downstream. This addition may be a basic lane, if needed for capacity, or an auxiliary lane that may be reduced 750 to 900 m downstream from the entrance or at the next interchange. In some instances, two additional lanes may be needed because of capacity considerations.

Also, in cases where the acceleration lane has an uphill gradient, as is shown in **Table 9.24**, the standard value is obtained by multiplying the acceleration lane length by an adjustment factor according to the average value of the uphill gradient.

To secure safety in the merging section, it is desirable to place a “merge ahead” sign on the main line before the merge.

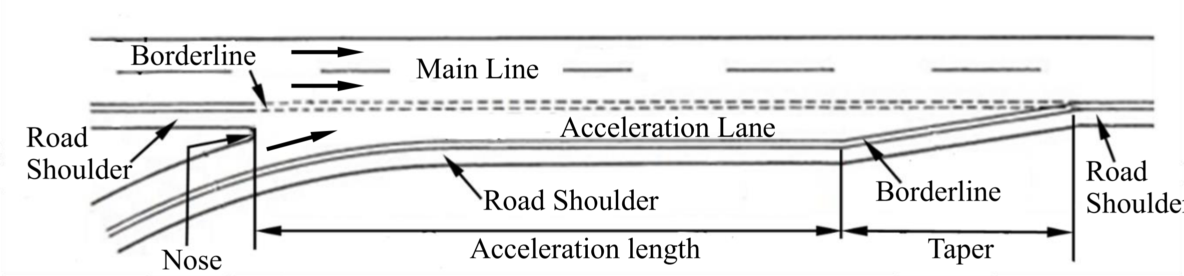


Figure 9.48 Parallel Type Acceleration Lane

Table 9.23 Ramp Acceleration Lane Length

Main line design speed (km/h)	Rural		Urban	
	Acceleration length (m)	Taper length (m)	Acceleration length (m)	Taper length (m)
120	370	100	250	70
100	280	90	190	60
80	140	70	90	50
60	100	60	65	35
50	40	50	40	30
40	30	50	25	25
30	20	20	20	10
20	10	20	10	10

Table 9.24 Ramp Acceleration Lane Length Adjustment factor

Main lane average gradient i (%)	0<i≦2	2<i≦3	3<i≦4	4<i
Uphill acceleration lane length adjustment factor	1.00	1.20	1.30	1.40
Downhill acceleration lane length adjustment factor	1.00	0.7	0.6	0.5

9.4.5.8.8FREE-FLOW TERMINALS ON CURVES

The previous discussion was based on roadways with a tangent alignment. Because the curvature on most expressway/motorway is slight, there is usually no need to make any appreciable adjustments at ramp terminals on curves. However, where the curves on an expressway/motorways are relatively sharp and there are exits and entrances located on these curves, some adjustments in design may be desirable to avoid operational difficulties.

On expressway/motorway having design speeds of 100 km/h or more, the curves are sufficiently gentle so that either the parallel type or the taper type of speed-change lane is suitable. With the parallel type, the design is about the same as that on tangent and the added lane is usually on the same curvature as the main line. With the taper type the dimensions applicable to terminals located on tangent alignment are also suitable for use on curves. A method for developing the alignment of tapered speed-change lanes on curves is illustrated in **Figure 9.49.** On curved sections, the ramp is tapered at the same rate relative to the through-traffic lanes as on tangent sections.

Wherever a part of a tapered speed-change lane falls on curved alignment, it is desirable that the entire length be within the limits of the curve. Where the taper is introduced on tangent alignment just upstream from the beginning of the curve, the outer edge of the taper will appear as a kink at the point of curvature.

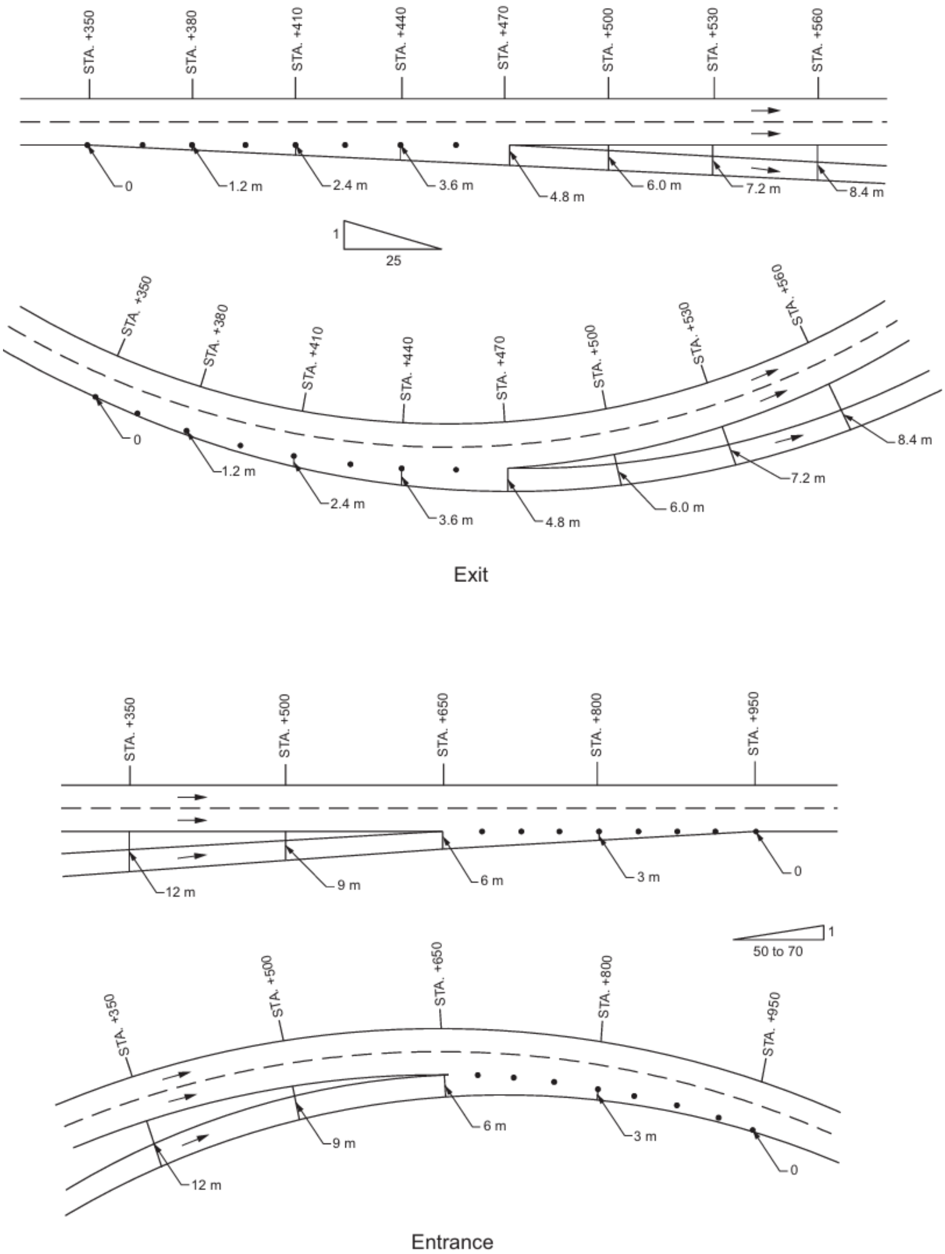


Figure 9.49 Layout of Taper – Type Terminals on Curves

At ramp terminals on relatively sharp curves, such as those that may occur on expressway/motorway having a design speed of 80 km/h, the parallel type of speed-change lane has an advantage over the taper type. At exits the parallel type is less likely to confuse through traffic, and at entrances this type will usually results smoother merging operations. Parallel-type speed-change lanes at ramp terminals on curves are illustrated in **Figure 9.50**.

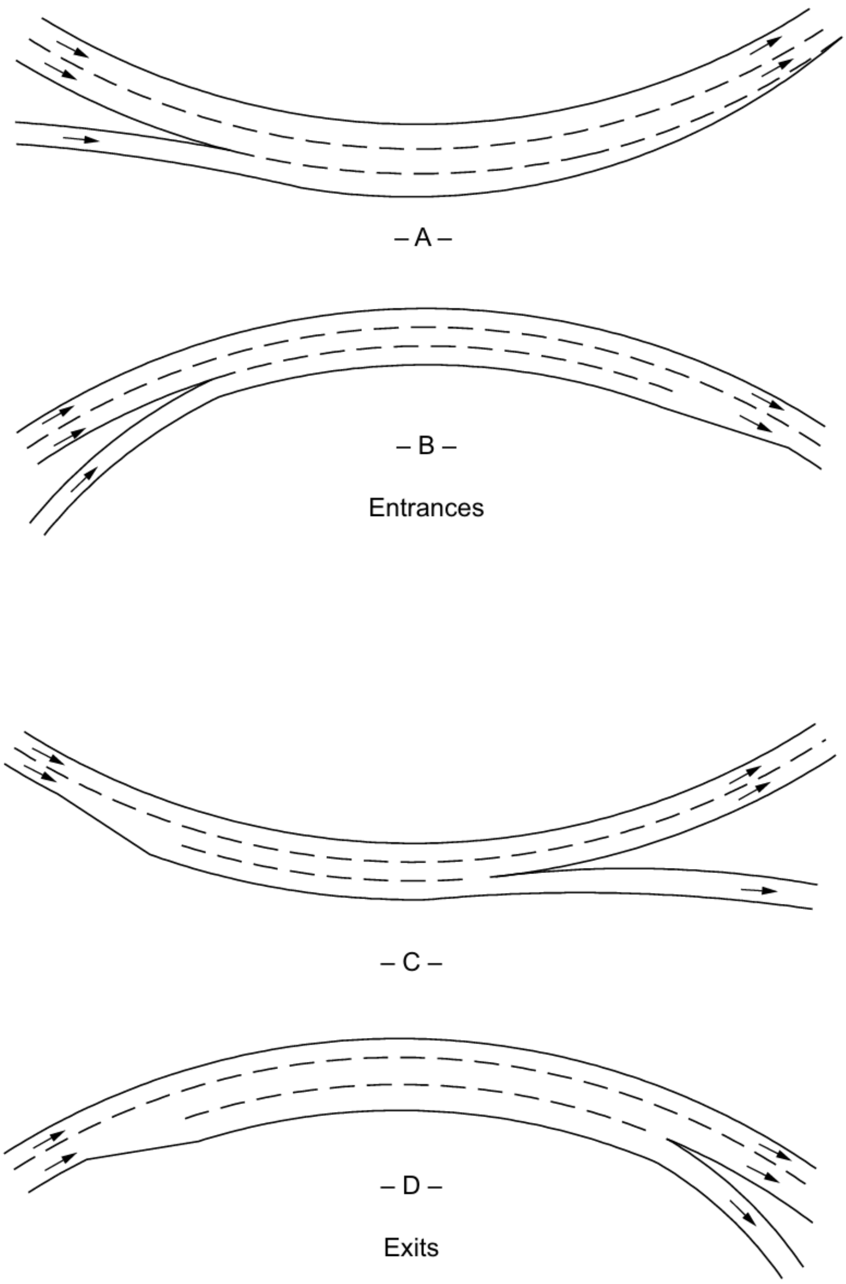


Figure 9.50 Parallel – Type Ramp Terminals on Curves

Entrances on curved sections of highway generally operate better than exits. **Figure 9.50A** and **Figure 9.50B** show entrances with the highway curving to the left and right, respectively. It is important that the approach curve on the ramp has a very long radius as it joins the acceleration lane. This aligns the entering vehicle with the acceleration lane and lessens the chances of motorists entering directly onto the through lanes. The taper at the end of the acceleration lane should be long, preferably about 90m in length. When reverse-curve alignment occurs between the ramp and speed-change lane, an intervening tangent should be used to aid in superelevation transition.

An exit may be particularly troublesome where the highway curves to the left (**Figure 9.50C**) because traffic on the outside lane tends to follow the ramp. Exits on left-turning curves should be avoided, if practical. Caution should be used in positioning a taper-type deceleration lane on the outside of a left-turning main-line curve. The design should provide a definite break in the right edge of the travelled way to provide a visual cue to the through driver to avoid being inadvertently led off the through roadway. To make the deceleration lane more apparent to approaching motorists, the taper should be shorter, preferably no more than 30 m in length. The deceleration lane should begin either upstream or downstream from the Point of Curvature (PC). It should not begin right at the PC, as the deceleration lane appears to be an extension of the tangent, and motorists are more likely to be confused. The ramp proper should begin with a section of tangent or a long-radius curve to permit a long and gradual reversing of the superelevation.

An alternate design, which will usually avoid operational concerns, is to locate the exit terminal a considerable distance upstream from the PC. In this design, a separate and parallel ramp roadway is provided to connect with the ramp proper.

With the highway curving to the right and the exit located on the right (**Figure 9.50D**), there is a tendency for vehicles to exit inadvertently. Again, the taper should be short to provide additional “target” value for the deceleration lane. With this configuration, the superelevation of the deceleration lane is readily achieved by continuing the rate from the travelled way and generally increasing it to the appropriate rate for the ramp curve.

9.4.5.9 GORE AREA

The term “gore" indicates an area downstream from the shoulder intersection points as illustrated in **Figure 9.51.** The physical nose is a point upstream from the gore, having some dimensional width that separates the roadways. The painted nose is a point, having no dimensional width, occurring at the separation of the roadways. The neutral area refers to the triangular area between the painted nose and the gore nose and incorporates the physical nose. The geometric layout of these is an important part of exit ramp terminal design. It is the decision point area that should be clearly seen and understood by approaching drivers. Furthermore, the separating ramp roadway not only should be clearly evident but should also have a geometric shape appropriate for the likely speeds at that point. In a series of interchanges along an expressway/motorway, the gores should be uniform and have the same appearance to the driver.

As a general rule, the width at the gore nose is typically between 6.0 to 9.0 m, including paved shoulders, measured between the travelled way of the main line and that of the ramp. This dimension may be increased if the ramp roadway curves away from the expressway/motorway immediately beyond the gore nose or if speeds in excess of 100 km/h are expected to be common.

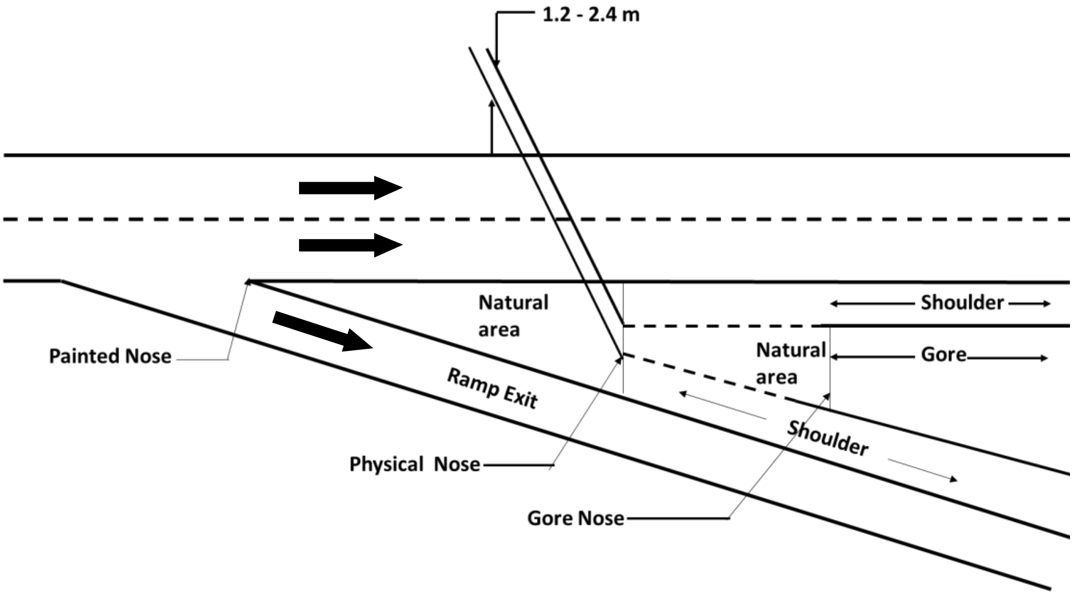


Figure 9.51 Typical Exit Gore Area Characteristics

The entire triangular area, or neutral area, should be striped to delineate the proper paths on each side and to assist the driver in identifying the gore area.

Rumble strips may be placed in the neutral area but should not be located too close to the gore nose because such placement renders them ineffective for warning high-speed vehicles. In all cases, supplemental devices of this type should be placed to provide the driver with ample advance warning to make timely corrections in the vehicle's path.

The rate of traffic crashes in gore areas is typically greater than the rate of run-off-the-road crashes at other locations. For this reason, the gore area, and the unpaved area beyond, should be kept as free of obstructions as practical to provide a clear recovery area. The unpaved area beyond the gore nose should be graded to be as nearly level with the roadways as practical so that vehicles inadvertently entering will not be overturned or abruptly stopped by steep slopes. Heavy sign supports, luminaire supports and roadway structure supports should be kept well out of the graded gore area. In addition, yielding or breakaway supports should be employed for the exit sign, and concrete footings, where used, should be kept flush with the ground level.

Unfortunately, there will be situations where placement of a major obstruction in a gore is unavoidable. Gores that occur at exit ramp terminals on elevated structures are a prime example. Also, there are occasions when locating a bridge pier in a gore cannot be avoided. Guardrails and bridge rails are designed to handle angular impacts but are not effective in handling the kind of near head-on impacts that occur at these gores.

In recognition of the exposed position of fixed objects in gore areas, a considerable effort has been directed toward the development of cushioning or energy-dissipating devices for use in front of such fixed objects. At present, several types of crash cushions are being used. These devices substantially reduce the severity of fixed-object collisions. Thus, adequate space should be provided for the installation of a crash-cushion device whenever a major obstruction is present in a gore on a high-speed highway. Reference may be made to **Section 12.5.7** for details on the installation of crash-cushion devices.

9.4.5.9.1GEOMETRIC STRUCTURE AT NOSE

The geometric structure at nose is designed based on the following considerations:

i. **Speed change (deceleration and acceleration) lane cross-section composition**

As a rule, the cross-section composition of speed change lanes should be the same as the ramp cross-section composition. Widths that correspond to a marginal strip should be secured between the speed change lane and main line carriageway. In cases where the speed change lane and main line are adjacent in parallel and taper type designs, a width that corresponds to a marginal strip should be secured as shown in **Figure 9.52**.

ii. **Minimum curve radius at nose**

The minimum curve radius at the exit ramp nose should be as indicated in **Table 9.25** according to the main line design speed.

iii. **Transition curve**

The parameter of the clothoid curve used around the exit ramp nose should not be less than the absolute minimum values shown in **Table 9.26** according to the main line design speed.

iv. **Vertical curve radius**

The vertical curve radius of the ramp near the nose should not be less than the minimum values shown in **Table 9.25** according to the main line design speed.

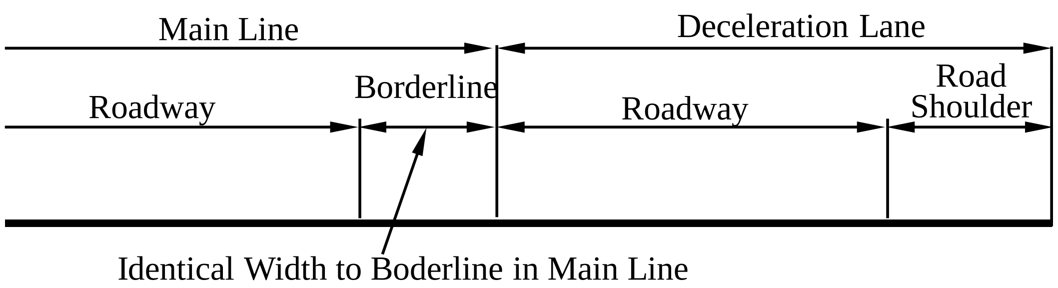


Figure 9.52 Speed Change Lane Cross-section Composition

Table 9.25 Minimum Horizontal Curve Radius at Nose

Main line design speed (km/h)	120	100	80	60
Minimum vertical curve radius at nose (m)	250	200	170	100

Table 9.26 Nose Transition Curve Parameter (A)

Main line design speed (km/h)	120	100	80	60
Absolute minimum value (m)	70	60	50	40
Desirable minimum value (m)	90	70	60	50

Table 9.27 Minimum Nose Vertical Curve Radius & Length

Main line design speed (km/h)		120	100	80	60	50
Crest vertical curve radius (m)	Desirable Minimum	2000	1800	1600	900	700
	Absolute minimum	1400	1000	800	450	350
Sag vertical curve radius (m)	Desirable Minimum	1500	1500	1400	900	700
	Absolute minimum	1000	850	700	450	350
Vertical curve length (m)	Desirable Minimum	70	65	60	40	35
	Absolute minimum	50	45	40	35	30

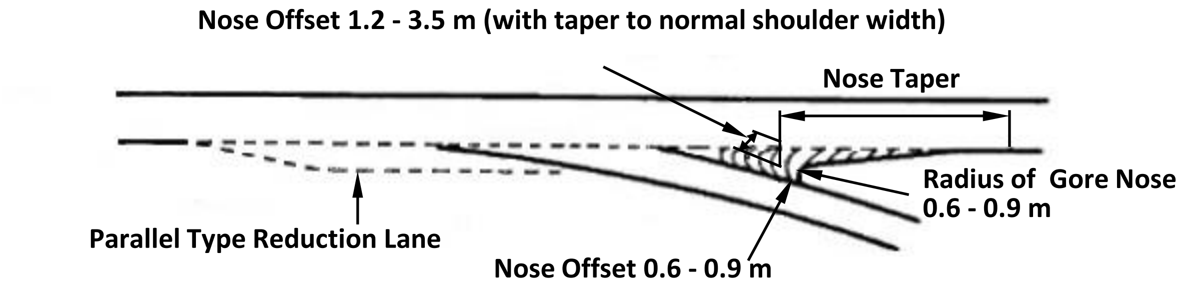
9.4.5.9.2STRUCTURE OF GORES

The gore nose should be designed as follows for the following three types: (1) Exit Gore, (2) Major Fork Gore, (3) Merge Gore.

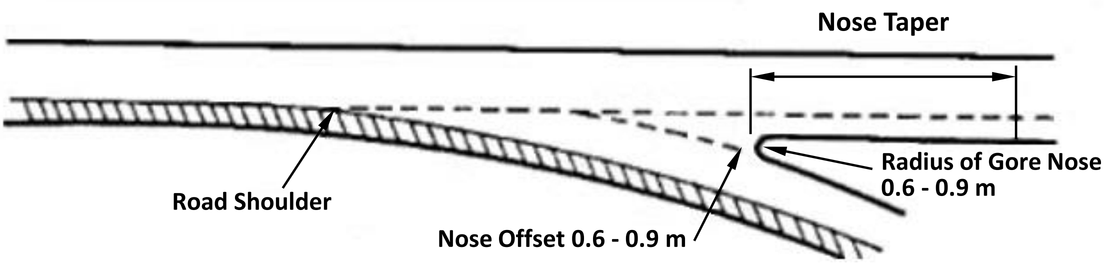
i. **Exit Gore**

In the case of a diverging from main line to ramp, gore nose offset must be provided for the benefit of vehicles that have gone the wrong way. The size should be determined over the scope shown in **Figure 9.53** upon considering the deceleration lane length and shape. If the deceleration lane is parallel (**Figure 9.53 (a)**), the nose offset on the main line side should be the same as the complete parking shoulder width.

On a road that has a shoulder wide enough for parking (**Figure 9.53 (b)**), there is no particular need for an offset because the shoulder performs the function of an offset.



(a) Case of narrow shoulder

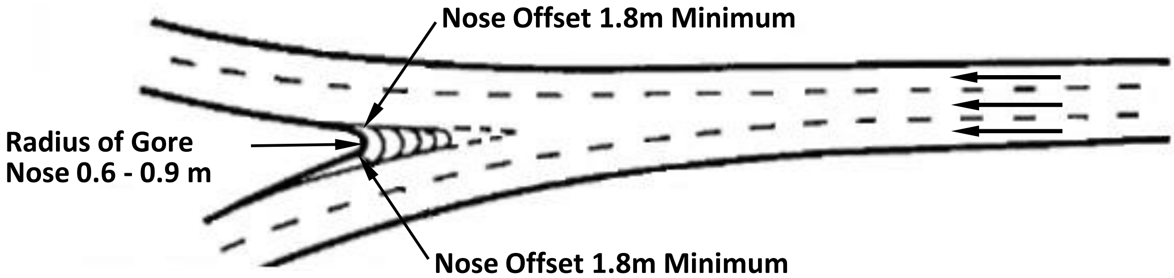


(b) Case of wide shoulder

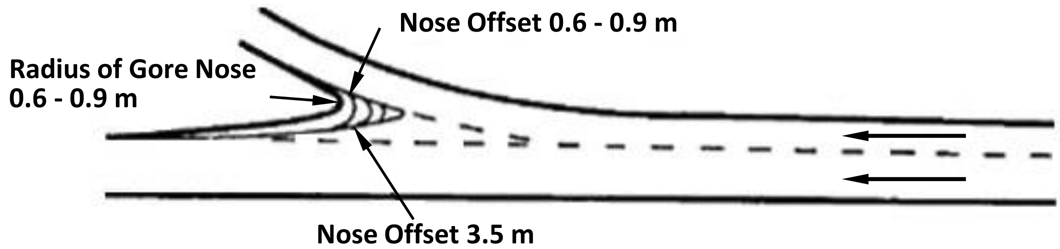
Figure 9.53 Detail of Exit Gore and Nose Offset

ii. Major Fork Gore

In the case where main lines diverge (**Figure 9.54 (a)**), it is desirable to have gore nose of at least half the width of a lane or 1.8m from the edge of the carriageway on both sides. The gore nose runoff should have ample tapered length to allow vehicles that have gone the wrong way to safely return to the carriageway. In cases where the number of lanes is reduced (**Figure 9.54 (b)**), this tapered length should not be less than the values indicated in **Table 9.28** that are obtained in respect to the average travel speed assuming that vehicles take 1 second to move 1 meter to the side. A smooth runoff is provided from the secondary parabola in a straight line. The section that is widened due to the offset should have the same pavement as the main line carriageway to facilitate driving. The nose should be edged with a mountable kerb on the ramp side and main line side for around 10m, and the end should be rounded with a radius of 0.6-0.9m.



(a) Major Fork Gore



(b) Major Fork Gore (in case of reduction of lanes)

Figure 9.54 Detail of Major Fork Gore

Table 9.28 Nose Taper Length

Design speed (km/h)	Nose taper length per 1m of nose offset (m)
120	12
100	11
80	10
60	8
50	7

iii. **Merge Gore:**

There is no need to provide gore nose for merging lanes. The desirable nose end curve radius should be 0.3-0.6m (not as obvious as in the case of a diverging nose) to reduce the inflow angle.

9.4.6 OTHER INTERCHANGE DESIGN FEATURES

9.4.6.1 PEDESTRIAN AND BICYCLE ACCOMMODATION

The accommodation of pedestrians and bicycles through interchanges should be considered early in the development of interchange configurations. High-density land use in the vicinity of an interchange can generate heavy pedestrian movements, resulting in conflicts between vehicles and pedestrians.

The movement of pedestrians and bicycles through interchanges can be enhanced by providing walkways or paths separate from the vehicular traffic. When walkways or paths are provided, they should be placed as far from the roadway as practical and be wide enough to handle the anticipated pedestrian or bicycle volumes. To maximize usage, the walkway or path should provide the most direct route through the interchange with minimal change in vertical alignment. Through complex interchange configurations, the use of informational signing may be appropriate to direct users to appropriate alternate routes.

Where non-motorized users will be crossing an interchange ramp, adequate sight distance should be provided so that drivers can detect the presence of pedestrians and bicyclists and users can perceive gaps in the traffic flow. To provide increased visibility at night, walkways/path ramp crossings should have overhead illumination. Where there are high volumes of pedestrians and bicyclists and insufficient gaps in the traffic flow to allow users to cross the ramp, actuated signals or an overpass/underpass should be considered.

9.4.6.2 RAMP METERING

Ramp metering seeks to regulate the flow of vehicles at expressway/motorway ramps in order to achieve some operational goals where the main-line expressway/motorway is highly congested. These operational goals include:

i. balance expressway/motorway demand and capacity,
ii. maintain optimum expressway/motorway operation by reducing incidents that delay traffic, and/or
iii. reduce crash frequency.

Ramp metering offers the potential to reduce congestion and its direct effects through the optimal use of expressway/motorway capacity. Metering can significantly reduce expressway/motorway crash frequencies by reducing stop and go driving behaviour and smoothing the flow of traffic entering expressway/motorway facilities. Ramp metering can also improve overall system performance by increasing average expressway/motorway throughput and travel speed, and decreasing travel delay.

Metering may be limited to only one ramp or integrated into a series of entrance ramps.

Ramp metering consists of traffic signals installed on entrance ramps in advance of the entrance terminal to control the number of vehicles entering the expressway/motorway. The traffic signals may be pretimed or traffic actuated to release the entering vehicles individually or in small (usually two-vehicle) platoons.

Pretimed metering releases vehicles at regular intervals that have been determined by traffic studies and, usually, simulation modelling. Traffic-actuated metering involves detectors used to measure the traffic conditions on the expressway/motorway main line and ramp. The metering rate is determined through one of a number of algorithms. Traffic-actuated metering can be based solely on local conditions on the ramp and on the expressway/motorway adjacent to the ramp or on conditions throughout the corridor or expressway/motorway system.

Ramp metering to improve merge operations involves detectors on the upstream approach of the expressway/motorway to determine acceptable gaps in the traffic flow. The traffic on the entrance ramp is released to coincide with the gap detected in the traffic on the expressway/motorway.

9.4.6.3 PLANTINGS

Proposed plantings should be selected with regard to their ultimate growth. Improperly located shrubs or trees may decrease horizontal sight distance on curves and seriously interfere with lateral sight distance between adjacent roadways. Even low-lying groundcovers may shorten vertical sight distance on curving ramps.

Trees or shrubs may be used to outline travel paths or to give drivers a sense of an obstruction ahead. For example, the ends of a directional island or approach nose may be planted with low-growing shrubs that will be seen from a considerable distance and direct the driver’s attention to the need for a turn. Shrubs that could cause vehicle damage on impact or obscure signs or warning devices should be avoided.

9.4.6.4 ACCESS IN THE VICINITY OF INTERCHANGES

9.4.6.4.1ACCESS TO THE EXPRESSWAY/MOTORWAY (MAJOR ROAD)

Legal and physical limitation of access along expressways/motorways to interchange locations eliminates deceleration, crossing, turning, parking and driveway conflicts between high-speed and low-speed vehicles, and between pedestrians, animals and vehicles, and thus makes a significant contribution to safe operation of high-speed roads. The same is true with respect to interchanges. Access from private properties may be restricted or restored by providing frontage roads on both sides outside the boundary of the expressway/motorway. Where new subdivisions are proposed beside an expressway/motorway reservation, the developer should provide properties abutting the expressway/motorway with access from roads internal to the subdivision. No private access should be allowed to the main carriageways or ramps of an expressway/motorway. The only exception is privately owned traffic generators such as service centres. Access control on expressway/motorway provides the greatest single benefit to road safety on these high-speed facilities.

9.4.6.4.2 ACCESS TO THE MINOR ROAD

Access control on the minor road in the vicinity of interchanges may be necessary to improve safety and traffic operational efficiency of the interchange and its approaches. Factors to be considered in determining the limits of such access control include:

i. Existing and possible future development in the vicinity of the ramp terminals (parking and driveways).
ii. Separation of intersections between frontage roads or local roads from the minor road ramp terminal so that they do not fall within the deceleration lanes.
iii. Costs of prohibiting abutting access.
iv. Channelisation of the crossroad and ramp terminals.
v. Provision for pedestrians.
vi. Legally declared status of the road.

A major arterial with partial control of access facilitates the flow of through traffic and has characteristics similar to an expressway/motorway but includes some at-grade intersections and some carefully selected and predetermined service facilities. Control of driveways and roadside development is an integral part of access control strategies for roads. Usually, utility services are not permitted along expressway/motorway, so eliminating the hazards of poles and excavation of the road or road reserve, and slow moving or stationary maintenance vehicles.

9.4.6.4.3 SERVICE CENTRES AND REST AREAS

Service centres and rest areas are costly and should be installed on rural or urban expressway/motorway only after preparation of a strategic plan to identify suitable locations. The layout design should be developed in consultation with landscape architects and other relevant specialists. Consultation with the service centre operators is also essential.

Service centres may be located on the expressway/motorway carriageway in the vicinity of an interchange only if the separation is large enough to ensure that no operational problems will result from associated weaving manoeuvres. The investigation should take account of future traffic flows and (as a general rule) it will be difficult to achieve a satisfactory outcome for service centres on urban expressway/motorway.

Where necessary, service centres may be placed in the quadrants of rural spread diamond interchanges (i.e., in the area bounded by the ramp, the major road and the minor road). However, where this occurs:

i. The entry from the expressway/motorway into the service centre should be via a deceleration lane on the left-hand side of the exit ramp situated at least 100 m past the physical exit nose.
ii. Re-entry to the expressway/motorway should desirably be via a ramp that passes under the abutment of the bridge over the expressway/motorway and merges with the expressway/motorway entry ramp.

iii. Under no circumstances should an exit from the service centre be allowed directly onto the expressway/motorway exit ramp as this will not be expected by drivers on the ramp and will lead to safety problems.

iv. Convenient and safe access should be provided for local traffic, either through an additional leg at the ramp terminal with the minor road or a separate intersection (entry/exit) on the minor road.

v. Safe, convenient and secure pedestrian routes and facilities should be provided for local residents from abutting land so that they are not tempted to seek alternative routes across expressway/motorway carriageways or ramps.

Service centres may offer the only 24-hour service available to a local area. Local people may often use the facility for take away food and drinks late at night, so it is important that interchanges and the centre have a high standard of lighting to provide for their protection and to deter crime.

The facilities offered by service centres and rest areas and other aspects relating to the road design and traffic management of service centres and rest areas is covered in **Section 11.5**.

9.5 REFERENCES

1. Ghana Highway Authority Road Design Guide, 1991.
2. Japanese Road Structure Ordinance, April 2021.
3. Explanation and Operation of the Road Structure Ordinance.
4. Geometric Design Manual of Uganda, (2005).
5. A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.
6. Asian Highway Design Standard for Road Safety, 2017.
7. Austroads. 2015. Guide to Road Design Part 1, AGRD01-15, Austroads, Sydney, Australia. Austroads. 2019.
8. South African Geometric Design Guidelines (2003).
9. Geometric Design Manual, Federal Democratic Republic of Ethiopia, Ethiopian Roads Authority, 2013.
10. O'Callaghan, N., M. Bekavac, and M. Mak. Guide to road design part 4C: interchanges. No. AGRD04C-23. 2023.
11. FHWA. Manual on Uniform Traffic Control Devices for Streets and Highways. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2009. Available at http://mutcd.fhwa.dot.gov
12. Fitzpatrick, K., M. A. Brewer, S. Chrysler, N. Wood, B. Kuhn, G. Goodin, C. Fuhs, D.

Ungemah, B. Perez, V. Dewey, N. Thompson, C. Swenson, D. Henderson, and H. Levinson. National Cooperative Highway Research Program Report 835: Guidelines for Implementing Managed Lanes, NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2016.

13. Gluck, J., H. S. Levinson, and V. Stover. National Cooperative Highway Research Program Report 420: Impact of Access Management Techniques. NCHRP, Transportation Research Board, Washington, DC, 1999. Available at http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_420.pdf.
14. ITE. Recommended Design Guidelines to Accommodate Pedestrians and Bicyclists at Interchanges. Institute of Transportation Engineers, Washington, DC, 2014.
15. Jacobson, L., J. Stribiak, L. Nelson, and D. Sallman. Ramp Management and Control Handbook, FHWA-HOP-06-001. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, January 2006. Available at http://www.ops.fhwa.dot.gov/publications/ramp_mgmt_handbook/manual/manual/pdf/rm_ handbook.pdf.
16. Leisch, J. P. Freeway and Interchange Geometric Design Handbook. Institute of Transportation Engineers, Washington, DC, 2005. Available for purchase from http://www.ite.org.
17. National Transportation Safety Board. Wrong Way Driving, Highway Special Investigation Report, NTSB/SIR-12/01. Washington, DC, 2012. Available at https://www.ntsb.gov/safety/safety-studies/Documents/SIR1201.pdf
18. Neudorff, L., J. E. Randall, R. Reiss, and R. Gordon. Freeway Management and Operations Handbook. FHWA-OP-04-003. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, September 2003.
19. Ray, B., J. Schoen, P. Jenior, J. Knudsen, R. J. Porter, J. P. Leisch, J. Mason, R. Roess, and Traffic Research & Analysis, Inc. National Cooperative Highway Research Program Report 687: Guidelines for Ramp and Interchange Spacing. NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2011. Available at http://www.trb.org/Publications/Blurbs/165126.aspx
20. Rodegerdts, L., J. Bansen, C. Tiesler, J. Knudsen, E. Meyers, M. Johnson, M. Moule, B. Persaud, C. Lyon, S. Hallmark, H. Isebrands, R. B. Crown, B. Guichet, and A. O'Brien. National Cooperative Highway Research Program Report 672: Roundabouts: An Informational Guide. NCHRP, Transportation Research Board, Washington, DC, 2010. Available at http://www.trb.org/Publications/Blurbs/164470.aspx
21. Schroeder, B., C. Cunningham, B. Ray, A. Daleiden, P. Jenior, and J. Knudsen. Diverging Diamond Interchange Informational Guide, FHWA-SA-14-067. Federal

Highway Administration, U.S. Department of Transportation, Washington, DC, August 2014.

22. Torbic, D. J, J. M. Hutton, C. D. Bokenkroger, D. W. Harwood, D. K. Gilmore, M. M. Knoshaug, J. J. Ronchetto, M. A. Brewer, K. Fitzpatrick, S. T. Chrysler, and J. Stanley. National Cooperative Highway Research Program Report 730: Design Guidance for Freeway Mainline Ramp Terminals. NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2012.
23. TRB. Transportation Research Circular 430: Interchange Operations on the Local Street Side: State of the Art. Transportation Research Board, National Research Council, Washington, DC, July 1994. Available at http://trid.trb.org/view.aspx?id=408296 23. TRB. Access Management Manual. Transportation Research Board, National Research Council, Washington, DC, 2015.
24. TRB. Highway Capacity Manual: A Guide for Multimodal Mobility Analysis, Sixth Edition. Transportation Research Board, National Research Council, Washington, DC, 2016.
25. Garber, Nicholas & Fontaine, Michael & Eit,. (2023). Final Report Guidelines For Preliminary Selection Of The Optimum Interchange Type For A Specific Location.
26. Standard Details, Road Signs and Markings for Urban and Trunk Road (1991), Ministry of Roads and Highways.

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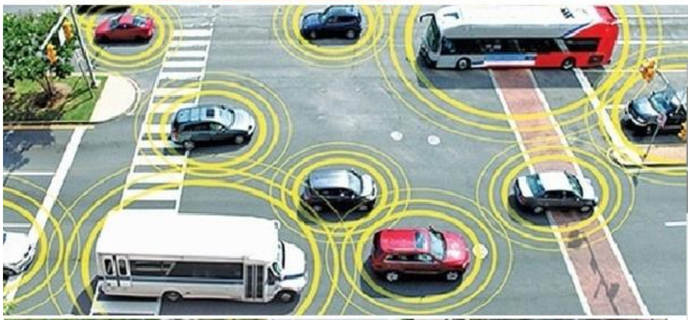
Chapter 10 Traffic Calming Measures

Chapter 11 Accessories to Road

Chapter 12 Road Furniture



GHANA ROAD DESIGN GUIDE 2023

CHAPTER 10

TRAFFIC CALMING MEASURES

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CHAPTER 10 TRAFFIC CALMING MEASURES

10.1. INTRODUCTION

The main role of a high-speed road is to carry long-distance motorised traffic, but the road becomes part of an urban area where it passes through settlements and towns. Road design must therefore take local traffic into account on such sections, most importantly pedestrian traffic.

The free movement and speed of through-traffic has hitherto had priority over safety for vulnerable road users in road design. Speed limit signs are posted but not respected by the road users and the police do not have the capacity to enforce the limits in all settlements. Accident statistics now show the consequences; pedestrians representing the majority of road fatalities and most of them occurring where high-speed roads pass through settlements. International experience has shown that speed calming measures can have positive impact on safety at these locations.

It is the Ministry of Roads and Highway's objective to improve safety for pedestrians in all road projects.

This chapter describes the different traffic calming devices that can be used on roads in Ghana, where they can be used, and how they should be designed.

10.2. GENERAL RECOMMENDATIONS

Several elements have to be considered when designing a traffic calming project. It should always be considered if it is possible to:

i. Minimise the number of intersections / access roads.
ii. Separate turning traffic from through traffic at major intersections.
iii. Make provision for walkways.
iv. Segregate pedestrians from vehicular traffic where feasible, e.g., with overpasses /underpasses or by signalisation.
v. Protect pedestrian crossings with traffic calming measures that are clearly visible to drivers, e.g., by placing pedestrian crossings on table humps or by protecting the crossings with humps, signals, or islands.
vi. Discourage on-street parking.
vii. Avoid direct access to premises from the main road by providing service roads.
viii. Provide safe and adequate parking and stopping spaces clear of the main carriageway.
ix. Provide bus-bays at designated locations.
x. Ensure that new developments are well set back from the main road.

xi. Provide streetlights.

The above elements should be considered and planned in detail before a traffic calming project is implemented. Moreover, a road accident data analysis should be carried out based on Micro Accident Analysis Package (MAAP) or similar software. However, physical measures are required to encourage or compel drivers to slow down to a reasonable speed through populated areas with vulnerable road users.

10.3. LOCATION

Traffic calming devices should be placed so they do not appear unexpectedly in the street scene. Ample stopping sight distance should be ensured to allow drivers to reduce speed sufficiently or, if necessary, to stop (Refer to **Section 7.2.6**). The available distance when approaching the calming measures must be sufficient to enable a driver travelling at operating speed to stop the vehicle safely before hitting a pedestrian in its path.

10.3.1. SIGNS AND MARKINGS

A traffic calming device and its immediate surroundings must be designed so that there is a clear visual difference from the rest of the road. It should be ensured that the carriageway marking and road signs are perceived and observed in due time and, if necessary, pre-warnings should also be provided. For use of signs and road markings refer to “Draft Manual for Signs and Markings, 2007”.

10.3.2. CONSTRUCTION MATERIALS

In this chapter some recommendations for use of materials are given. For further recommendations refer to “Ministry of Roads and Transport (MRT) Standard Specifications for Road and Bridge Works, 2007”.

10.4. TYPES OF TRAFFIC CALMING MEASURES

This chapter recommends seven different traffic calming measures based on best practice to be used on roads in Ghana:

i. Road Humps
ii. Rumble Strips
iii. Jiggle Bars
iv. Raised Islands / Centre Islands
v. Road Narrowing
vi. Town Gates
vii. Pre-warnings

The seven measures can be divided into three groups. These are:

i. Vertical deflection (road humps, rumble strips and jiggle bars)

ii. Horizontal deflection (raised island and narrowing)

iii. Visual deflection (town gates and pre-warnings)

In the following sections, each of the seven main types is briefly described. The application of the main types is shown in **Table 10.1**.

Table 10.1 Application of the main types of the traffic calming measures

Type of Traffic Calming Measures	Design Class			Desired Speed (km/h)		
	A	B & C	D & E	≥ 60	50	≤ 40
Road Hump		x	x		x	x
Rumble Strips		x	x		x	x
Jiggle Bars	x	x	x	x	x	x
Road Islands		x	x		x	x
Road Narrowing		x	x	x	x	x
Town Gates		x	x	x	x	x
Pre-Warnings	x	x	x	x	x	x

10.4.1. ROAD HUMPS

The most effective self-enforcing device to use for speed control is the road hump. Two alternative designs have proved to be most effective. These are the circular hump and trapezoidal hump.

Humps are only allowed on roads with speed limit 50km/h or lower. The length of the road hump, if round topped, shall be 4.0 m and if trapezoidal for use as a pedestrian crossing, it shall be a minimum of 6.0 m, unless otherwise ordered by the Engineer. The road hump shall extend across the full width of the carriageway including the hard shoulders to avoid drivers from by-passing them.

If predominantly traversed by passenger cars, the gradient of the ramps of a trapezoidal road hump (speed table) shall be 1:15 relative to the gradient of the road on which it is built and, if consideration is to be given to heavy goods vehicles or buses, the gradient shall be flattened to 1:40 relative to the gradient of the road.

Humps should always be clearly marked, as illustrated in **Plate 10.1** and **Figure 10.1**, with chequerboard markers on the ramps and pedestrian crossing on top.

Plate 10.1 Trapezoidal hump in asphalt, near Kumasi.

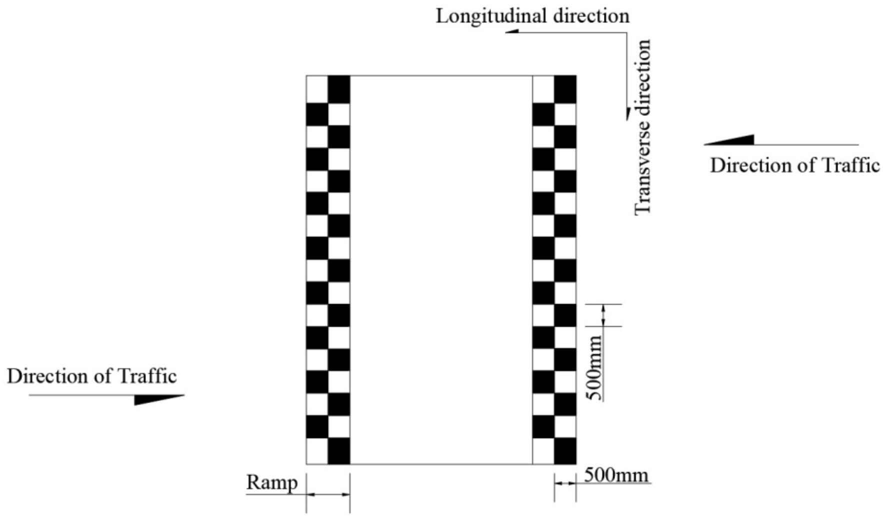


Figure 10.1 Typical layout of checked pattern on road hump

10.4.1.1. DESIGN OF CIRCULAR HUMPS

For circular humps the following radius and chord length for different speed levels as shown in **Table 10.2** should be applied.

Table 10.2 Recommended radii and chord lengths of circular humps

Desired speed (km/h)	Radius (m)	Chord length (m)
50	113	9.5
40	53	6.5
30	20	4.0

The recommended crown height for all circular humps is 100mm above the road surface as shown in **Figure 10.2**, **Figure 10.3** and **Figure 10.4**. Heights less than the assumed 100mm will result in higher speeds than those mentioned. Heights above 100mm may cause damage to vehicles.

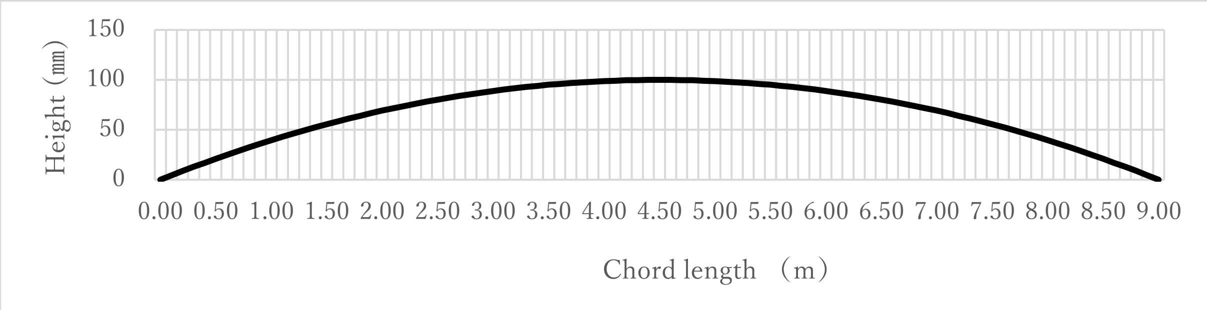


Figure 10.2 Cross section for circular hump designed for 50 km/h

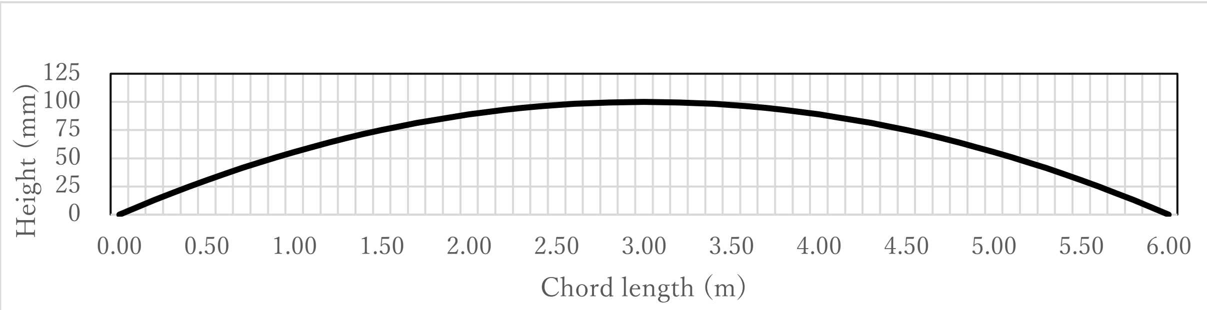


Figure 10.3 Cross section for circular hump designed for 40 km/h

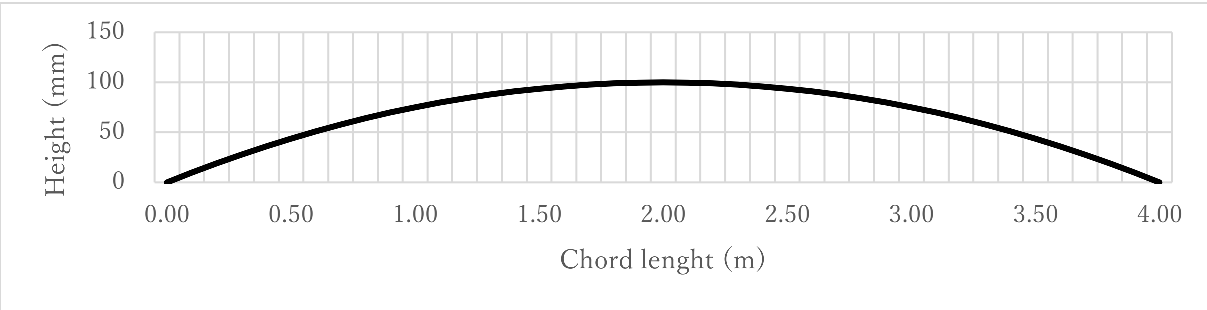


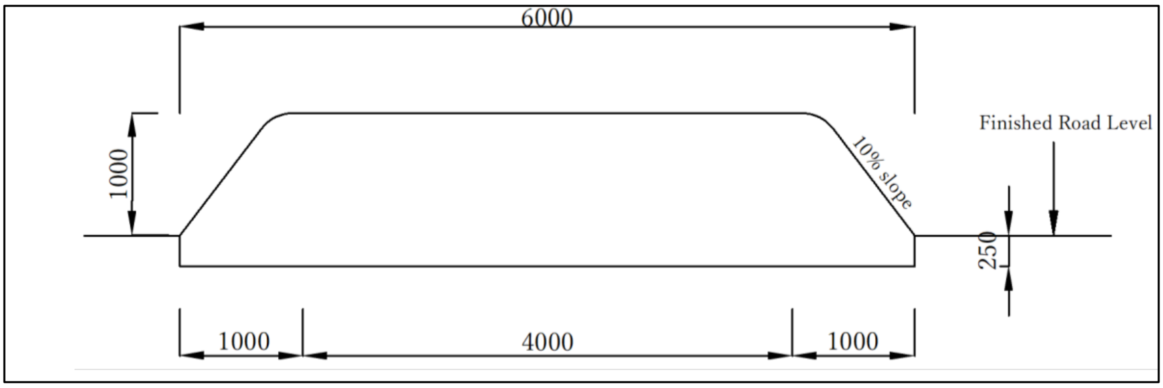
Figure 10.4 Cross section for circular hump designed for 30 km/h

10.4.1.2. DESIGN OF TRAPEZOIDAL HUMPS

The basic requirements to trapezoidal humps are almost the same as for the circular humps but the profile has changed from a circular to a trapezoidal shape, in other words a raised, flat area with two ramps. The following standards for different speed levels should be used as indicated in **Table 10.3**. **Figure 10.6**, **Figure 10.7** and **Figure 10.8** shows examples of trapezoidal humps in asphaltic concrete, lineflex/thermoplastic/epoxy, and reinforced concrete for 30km/h speed limit. For 40km/h and 50 km/h the recommended ramp height in **Table 10.3** should be applied.

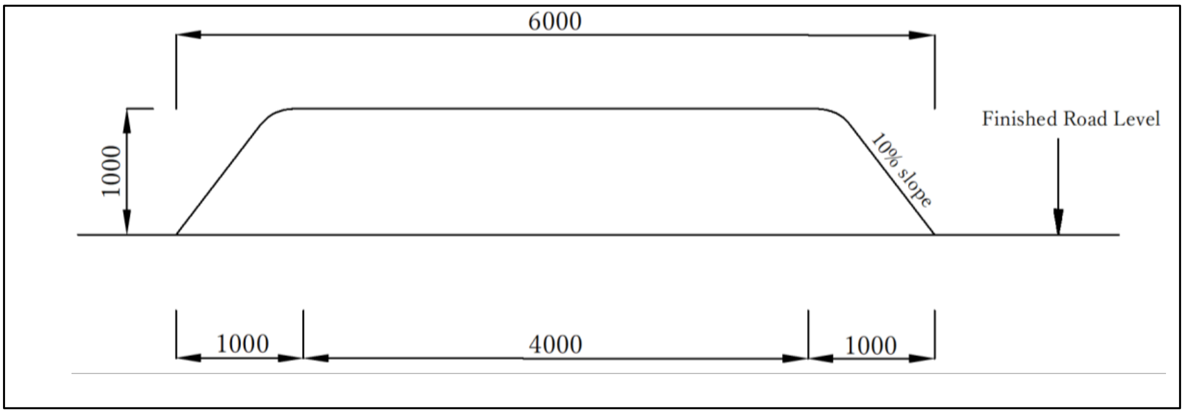
Table 10.3 Recommended design values for trapezoidal humps

Desired speed (km/h)	Length of ramp (m)	Length of hump (m)	Ramp height (m)	Gradient (%)
50	1.0	6.0	0.075	7.5
40	1.0	6.0	0.085	8.5
30	1.0	6.0	0.100	10.0



All dimensions are in mm

Figure 10.5 Cross section for trapezoidal hump (Type-1: Asphaltic Concrete)



All dimensions are in mm

Figure 10.6 Cross section for trapezoidal hump (Type-2: Lineflex/Thermoplastic/Epoxy)

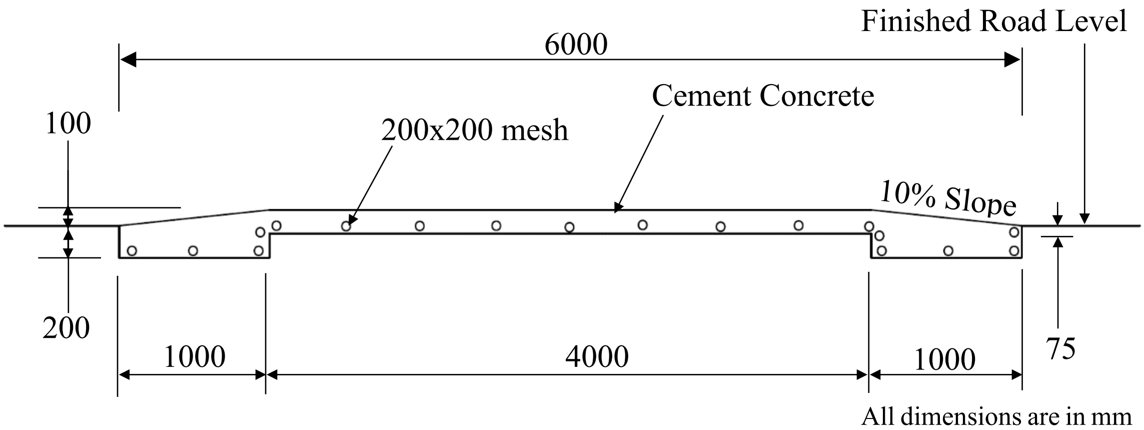
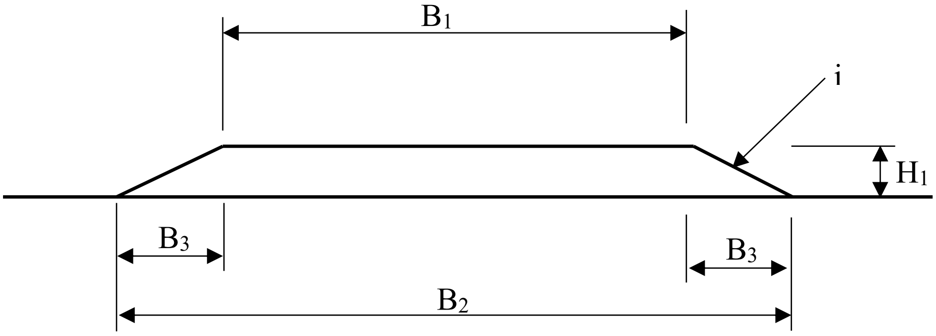


Figure 10.7 Cross section for trapezoidal hump (Type-3: Cement Concrete)



Design Speed (km/h)	B_1 (mm)	B_2 (mm)	B_3 (mm)	H_1 (mm)	i (%)
30	4,000	6,000	1,000	100	10.0
40	4,000	6,000	1,000	85	8.5
50	4,000	6,000	1,000	75	7.5

Figure 10.8 Recommended design of trapezoidal hump

10.4.1.3. APPROACHES TO HUMPS

When approaching road humps consideration should be given to the structural improvement of the pavement to prevent deformation of the pavement structure due to breaking effects and static loading of trucks.

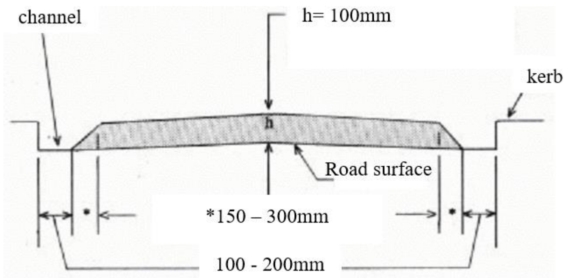
10.4.1.4. RAMP MARKINGS

As an aid to visibility, the road humps are to be demarcated with reflective paint markings, typically in a chevron or chequered pattern, and the provision of edge marker posts. Refer to MRT Standard specification for Road and Bridge Works, 2007, **Section 23.12.4**.

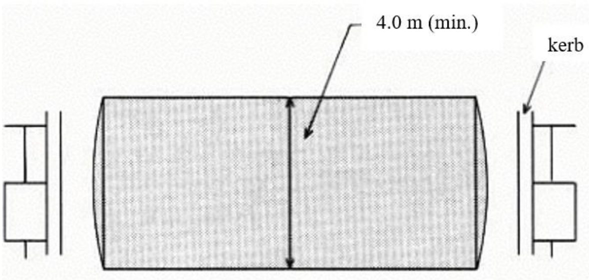
They should only be used in conjunction with other elements which ensure that the road users expect them. Such elements include first and foremost the necessary markings. For instance, road humps should be marked with chequerboard markings as shown in **Plate 10.1**. For further information about signs and markings see the "Draft Manual of Road Signs and Markings, 2007".

10.4.1.5. DRAINAGE

Along kerbed sections a drainage gap of 100 to 200 mm wide shall be left at both ends of the road hump as shown in **Figure 10.9**. Slots should be used where the hump serves as a pedestrian crossing.



Cross section



Plan

Figure 10.9 Example of hump drainage design at kerbed section

Plate 10.2 Trapezoidal Road hump with drainage slot at both ends

10.4.1.6. ROAD HUMPS WITH PEDESTRIAN CROSSING

To protect pedestrians at pedestrian crossings which are not regulated with traffic signals, pedestrian crossing could be placed on top of a trapezoidal hump. The flat–topped road humps are effective in slowing down vehicles sufficiently to enable pedestrian to use the crossing safely. When a pedestrian crossing is placed on top of a road hump, it is still necessary to pre-warn the vehicles about the pedestrian crossing and the road hump. A simple design plan for a pedestrian crossing on top of a trapezoidal hump is shown in **Figure 10.10.**

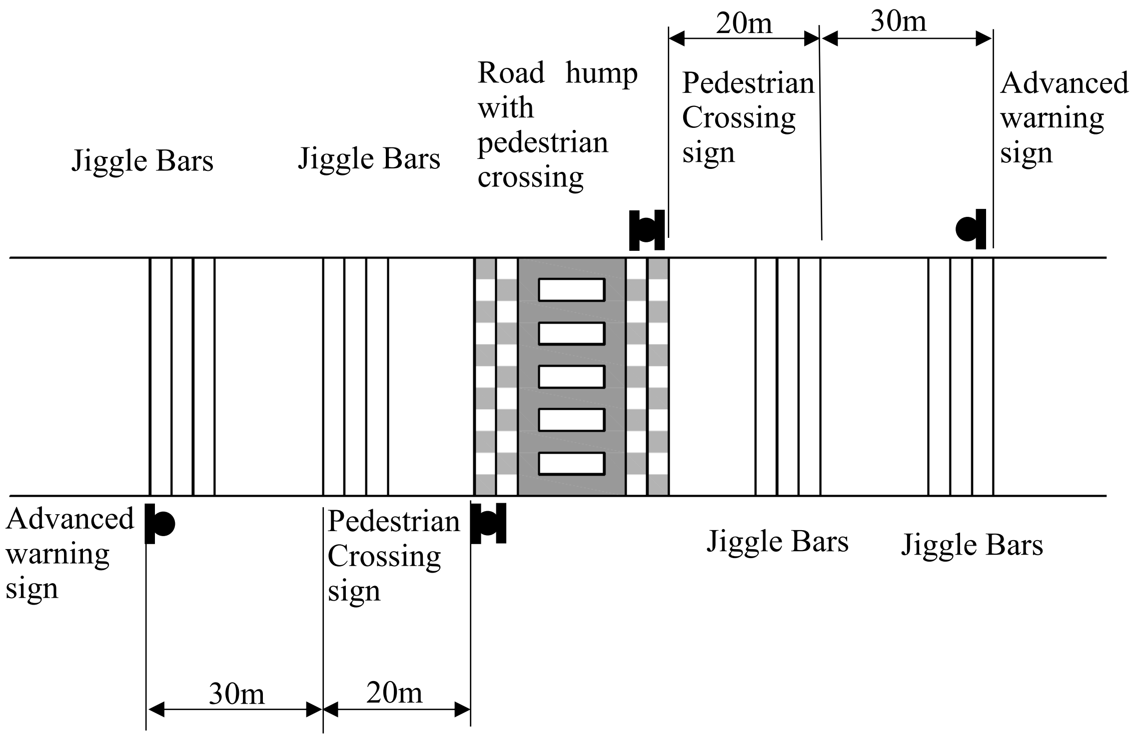


Figure 10.10 Examples of a simple design plan for a Pedestrian crossing on top of a trapezoidal hump

The following principles can be applied:

i. The first set of jiggle bars should be placed 50m before the pedestrian crossing/road hump.

ii. "Road hump" warning sign should be placed 50m before the pedestrian crossing/road hump.

iii. The second set of jiggle bars should be placed 20m before the pedestrian crossing/road hump.

iv. "Pedestrian crossing" information sign should be placed on both sides next to the pedestrian crossing / road hump.

If road humps are used in combination with a pedestrian crossing, the location should be lit at night with streetlights, studs and reflectors as shown in **Plate 10.3**.

Plate 10.3 Pedestrian crossing placed on top a trapezoidal hump

10.4.1.7. DISTANCE BETWEEN HUMPS

If it is necessary to place several humps on a stretch to keep the speed down, it is recommended to place the humps with the distances shown in **Table 10.4**.

Table 10.4 Recommended relationship between desired speed and spacing of humps

Desired Speed (km/h)	Distance between humps (m)
50	250
40	100
30	75

10.4.2. SPEED CUSHION

On roads with bus traffic, speed cushions could be installed instead of circular or trapezoidal humps. Speed cushion causes less interference than ordinary humps to larger vehicles such as buses and trucks but still reduce the speed of cars.

A speed cushion is a form of road hump, occupying part of the traffic lane in which it is installed. The speed cushions are generally located in pairs, arranged transversely across the carriageway. The two humps in a speed cushion are constructed as two circular humps with radii and chord length as shown in **Table 10.5**. The recommended crown height for circular humps in speed cushions is 100mm.

Table 10.5 Recommended radii and chord length for circular humps in speed cushions

Desired Speed (km/h)	Radius (m)	Chord Length (m)
50	113	9.5
40	53	6.5
30	20	4.0

The widths of the circular humps and their placement next to each other are shown in **Figure 10.9** for a speed cushion designed for 40 km/h.

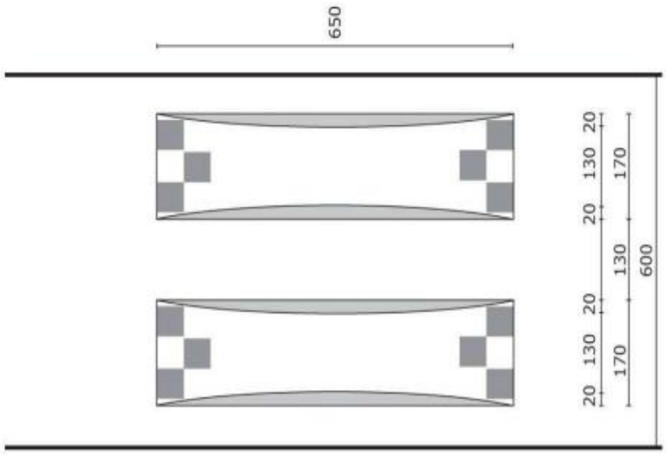


Figure 10.11 layout of speed cushion for 40 km/h

Transverse gaps between the base of a cushion and the kerb, as well as between adjacent cushions, should be 65 - 130 cm. To accommodate cyclists and motorcyclists the cushion should not be located adjacent to a grate inlet. To protect motorcyclist and cyclist not to fall in the cushion, the gap between the circular humps and the road surface should be levelled with a gradient of 1:2.

10.4.3. RUMBLE STRIPS

Rumble strips are transverse treatment designed to cause noise and a bit of discomfort when vehicles cross them. They alert drivers and create an impression of speed. Rumble strips are unlikely to produce a major reduction in speed. However, they are very effective in combination with road humps. Rumble strips can typically be used on the approach to villages, trading areas, dangerous intersections, or road humps.

The rumble strips shall extend across the full width of the carriageway including the hard shoulders to avoid drivers from by-passing them except for kerbed sections where a drainage gap of 100 to 200 mm wide shall be left at both ends of the rumble strips.

Rumble strips should be 15 - 25 mm high and made of thermoplastic, lineflex, asphalt, or concrete as shown in Error! Reference source not found.. They are usually laid in a pattern (

typically 2 or 3 groups of 4 or 5 strips). Sometimes the width of the strips and the spacing (within the groups and between groups) is varied in order to make the "rumple" more noticeable, if the driver does not slow down. The recommended width of rumble strips varies between 30cm and 50cm. The space between the rumble strips varies between 50 and 200 cm.

Plate 10.4 Rumble strips in thermoplastic/line flex

10.4.4. JIGGLE BARS

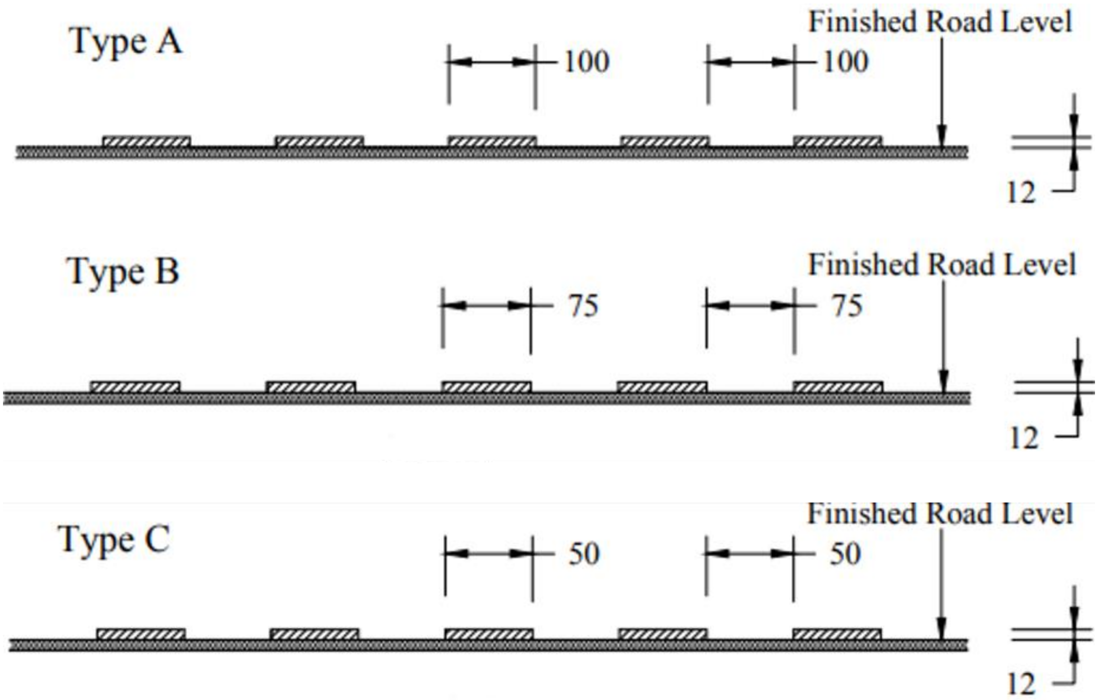
Jiggle bars are transverse treatment designed to cause noise when vehicles go over them. The idea is to create an impression of speed for the driver, who in turn slows down. Jiggle bars do not create a major reduction in speed, but they are useful to make the driver aware of changes - for instance if the driver is approaching a built-up area, pedestrian crossings, road humps or other hazards. Jiggle bars should be 9 - 12mm high**. Plate 10.5** shows an example of jiggle bars.

The bars are made of thermoplastic or lineflex within the following three types:

i. Type A has 5 bars with a width of 100 mm and spacing of 100mm.
ii. Type B has 5 bars with a width of 75 mm and spacing of 75 mm.
iii. Type C has 5 bars with a width of 50 mm and spacing of 50 mm.

Figure 10.12 shows the three types of jiggle bars.

Plate 10.5 Jiggle bars in thermoplastic/line flex



All dimensions are in mm

Figure 10.12 Types of jiggle bars (thermoplastic/lineflex)

10.4.5. RAISED ISLAND/CENTRE ISLAND

Centre island in connection with road staggering/narrowing is a speed reducing device. It also serves for the purpose of separating two-way traffic and to allow pedestrians to cross the road in two stages. The pedestrians can use the island as a refuge when they cross the road.

Centre islands can be installed in connection with approaches to built-up areas to make it clear for drivers that they are approaching a populated area where many pedestrians cross the road. **Plate 10.6** shows an example of centre island.

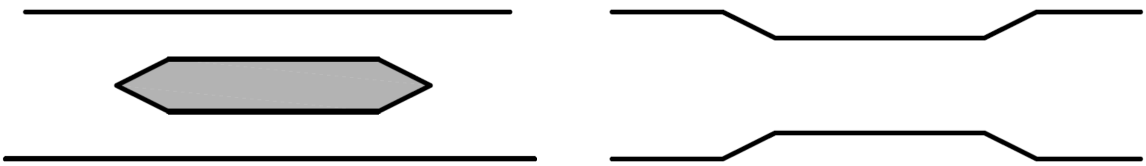
Plate 10.6 Example of a centre island

Centre islands should not be used if the speed limit is above 50km/h. Centre islands should be delimited by clearly recognizable kerbs to discourage drivers from attempting to cross them. It is important to take into account that there is a risk of material damage and injury if a vehicle hits a kerb with a speed above 30km/h. The driver may lose control of the vehicle, either because a tyre explodes at the collision with a sharp edge or because the vehicle bounces off its course on impact. It is therefore advisable to avoid high, sharp kerbs in the design of centre islands if there is a risk of collision at speeds above 30km/h. In such places the kerb should be bevelled with a maximum height of 6 - 8cm.

Special attention should be paid to visibility of centre islands to prevent collisions. The island and the surrounding area should be supported with sufficient signs, delineators, road markings, streetlights etc. to ensure that drivers see the islands in due time. Centre islands must be pre-warned with signs 50m before the island. Centre islands can also be combined with rumble strips and road humps.

10.4.6. ROAD NARROWING

There are basically two ways of narrowing the road. The first is narrowing the road from the centre and the second is narrowing from the roadside. The two types are shown in **Figure 10.13**. In both cases the space available for traffic is reduced, which will encourage drivers to slow down. The drivers get a visual signal that they must reduce speed to negotiate the narrowing safely.



Narrowing the road from the centre   Narrowing from the roadside

Figure 10.13 The two types of road narrowing

10.4.6.1. NARROWING THE ROAD FROM THE CENTRE

A narrowing from the road centre is created with a centre island. The centre island should be constructed with a raised area limited with kerbs or with ghost islands limited with delineators.

a. Centre island with kerbs

Reference is made to **Section 10.4.5** - Raised Islands/Centre Islands.

The minimum width of the central island should be 1m if the central island is not used as a refuge for pedestrians. If the central island is also functioning as a refuge for pedestrians, the minimum width of the central island should be 2m.

Narrowing the carriageway from the centreline by construction of kerbed centre islands is mostly used on 2-lane traffic roads and can be used on roads with desired speeds of 50km/h and less.

b. Ghost island with delineators

Ghost islands can be applied with painted road markings or thermoplastic and should be supported with reboundable delineators to discourage drivers from driving on or crossing the islands as shown in **Plate 10.7**.

The minimum width of a ghost island should be 0.5m.

Ghost islands can normally be used on existing carriageways without extending the cross-section and it can be used at all speed levels. It can for instance be used in smaller settlements where the desired speed level is 70km/h or in semi-urban areas with few crossing pedestrians.

Plate 10.7 Ghost Island with delineators

10.4.6.2. NARROWING FROM THE ROADSIDE

Narrowing from the roadside can be constructed with kerbs or markings. The construction principles are the same as described under **Section 10.4.6.1**.

10.4.6.3. LANE WIDTHS

It is recommended to use the lane widths stated in **Table 10.6** when road narrowing are applied. The recommended widths depend on the speed limit.

Table 10.6 Recommended Lane widths for 2-lane narrowing's for different speed limits

Speed Limit (km/h)	**Lane Width (m)**
60-80	3.50
50	3.25
40	3.00
30	2.75

10.4.7. TOWN GATES

Town gates are normally used on roads to mark the entry to an area with a lower speed limit.

First and foremost, a town gate must function visually by means of signs, centre islands, humps, rumble strips, planting, change of road surface, gantry, lighting etc. In addition, the carriageway can be slightly narrowed.

Town gate signs labelled with town names, posted speed and other features are installed on both sides of the carriageway to encourage slower driving on approaches to cities, villages, and small settlements. They are installed 200m before the first building.

Plate 10.8 shows an example of a town gate on the approach to Apeguso township. The gate consists of double signs indicating the town name and the posted speed. Furthermore, the gate is supported by delineators and a ghost island. The drivers are here clearly warned about the settlement ahead and the need to reduce speed.

Plate 10.8 Example of town gate signs

10.4.8. PRE-WARNING

The purpose of pre-warnings is to warn drivers about a hazard, settlement, or speed limit ahead and ensure that they are aware of the need to slow down. A pre-warning can simply be a warning sign with the relevant information, as shown in **Figure 10.14** and **Figure 10.15**.

Figure 10.14 Pre-warnings for road humps ahead (left), and rumble strips/jiggle bars (right)

Pre-warnings can be supplemented with a plate indicating the distance to the hazard. For example, the pre-warning for 5 road humps ahead can also supplemented with a plate with the text “5 humps ahead”.

Figure 10.15 Example of pre-warning sign

10.5. REFERENCES

1. Ghana Highway Authority Road Design Guide, 1991.
2. Standard Details, Road Signs and Markings for Urban and Trunk Road (1991), Ministry of Roads and Highways.
3. Ministry of Roads and Transport (MRT) Standard Specifications for Road and Bridge Works, 2007.
4. Draft Manual for Signs and Markings, 2007.
5. Japanese Road Structure Ordinance, April 2021.
6. Geometric Design Manual of Uganda, (2005).
7. A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.
8. Asian Highway Design Standard for Road Safety, 2017.

CHAPTER 11

ACCESSORIES TO ROAD

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CHAPTER 11 ACCESSORIES TO ROAD

11.1 INTRODUCTION

This chapter discusses the following accessories to roads:

i. Intelligent Transportation System (ITS)
ii. Traffic Signals
iii. Parking
iv. Rest Areas
v. Heavy Vehicle Inspection Site
vi. Toll Plaza
vii. Emergency Escape Ramps
viii. Road Protection Facilities
ix. Pedestrian Facilities
x. Protective Screening at Overpasses

11.2 INTELLIGENT TRANSPORTATION SYSTEM

11.2.1 INTRODUCTION

For the purpose of this guide, the scope of Intelligent Transportation System **(ITS)** is limited to road transportation. ITS is the integration of advanced communications technologies into the transportation infrastructure and within vehicles to improve transportation safety and mobility and enhance productivity. ITS encompasses a broad range of wireless and wire line communications-based information and electronics technologies.

ITS relies heavily on data collection and analysis. Once the system is built to collect data and analyse it, the results are then used to control, manage, and plan transportation systems. Sensors play an important role in data collection.

ITS contributes to the safe and smooth flow of traffic by providing real-time information on road conditions, weather conditions, road works, warnings etc.

ITS also helps transport planners to achieve the following policy objectives:

- Tackle congestion, pollution, poor accessibility, and even social exclusion.
- Reduce journey times and improve reliability – either in actuality, or simply by changing people’s perceptions.
- Improve the efficiency with which transport systems function.
- In certain circumstances – for example, parking guidance systems – it can help to support economic and retail vitality.

Some examples of ITS application as illustrated in **Figure 11.1** are as follows:

i. Real time information, both for public transport and private road transport, so that users have up-to-the minute information on services, where they are, and on incidents/delays and how to avoid them. On the roads, such information can also improve safety.

ii. The use of geographical information systems (GIS) and relational databases to keep inventories of transport infrastructure in an area (e.g., the condition of the road network) to better manage and prioritise maintenance work.

iii. "Smartcard" ticketing on public transport, to give the passenger the best deal for the bundle of trips that they might be making in a particular period of time, and to provide the operator(s) with detailed information about their passengers' travel habits. The latter information can be useful for apportioning revenue between operators, as well as for service planning.

iv. Detailed route planning information (often in real time) for both public transport and car users.

v. Parking guidance systems, to reduce parking search time.

vi. Public transport information in various formats (e.g., audible) for disabled people.

vii. Traffic signal control, in real time, to improve the efficiency of traffic flow, or to afford priority to particular user groups such as bus passengers, or pedestrians, within a network.

viii. Sophisticated booking and scheduling software can help to maximise vehicle utilisation in a demand responsive transport (DRT) scheme.

The flowchart of ITS system is shown in **Figure 11.2**. Example of ITS at an intersection and for vehicle specification measurement facilities are shown in **Figure 11.3** and **Figure 11.4** respectively.

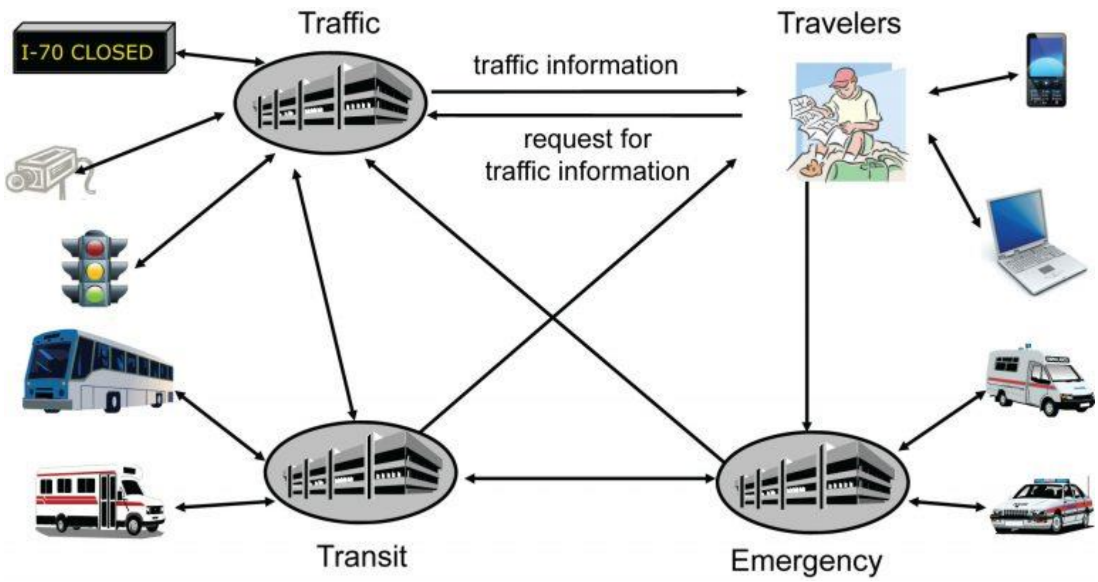


Figure 11.1 Schematic Image of ITS system

Data collection
Hardware: sensors, cameras, GPS
Data type: traffic count, surveillance, speed and time, location, vehicle weight, delays etc

Data Transmission
Rapid and real-time data transmission between the road and Traffic Management Center

Data Analysis
Error rectification, data cleaning, data synthesis and adaptive logical analysis

Data Transmission
Rapid and real-time data transmission between the Traffic Management Center and the traveller

Intelligent Information
Real-time information like travel time, travel speed, delay, accidents on roads, change in route, diversions, work zone conditions etc.
delivered by a wide range of electronic devices like variable message signs, highway advisory radio, internet, SMS, automated cell.

Figure 11.2 Flowchart of ITS system

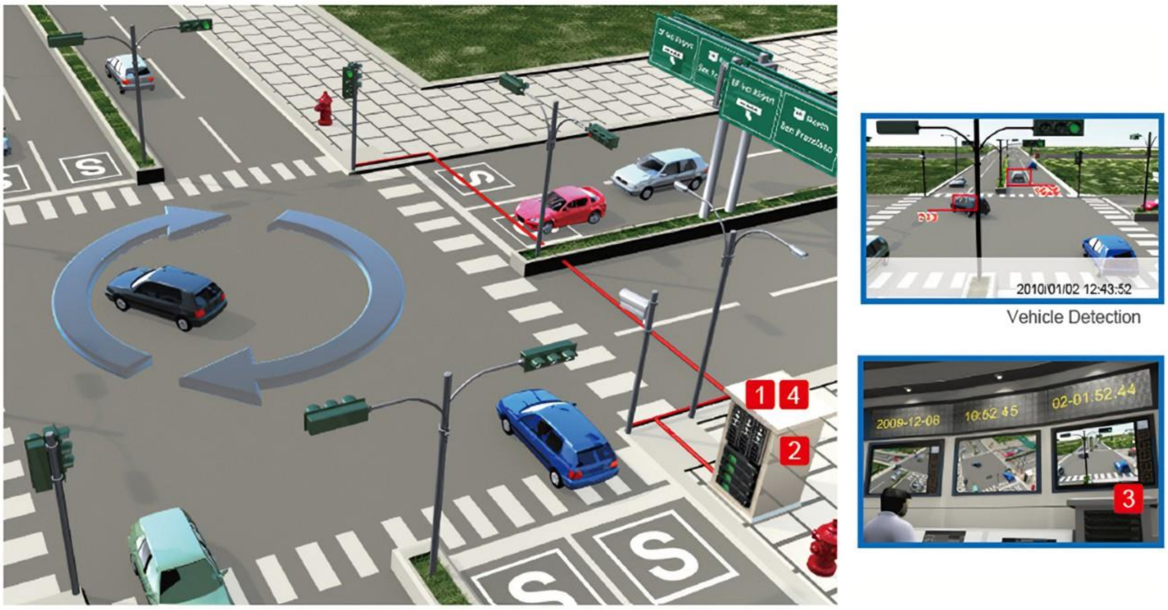
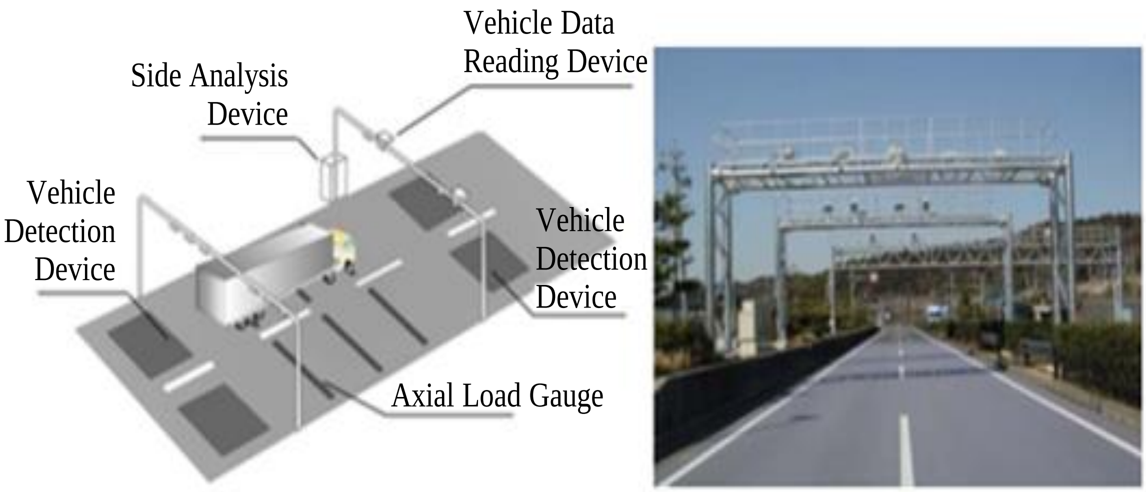


Figure 11.3 Example of ITS at an intersection



(Conceptual Figure) (Photograph)

[Vehicle Weight Automatic Measuring Device]

Figure 11.4 Example of Vehicle Specification Measurement Facilities

11.2.2 ITS APPLICATIONS

One of the major application of ITS is smart traffic management using the following systems:

i. Incident detection

ii. Variable Speed Limits

iii. Ramp Control

iv. Traffic Signal Control
v. Parking Management
vi. Demand Responsive Transport Management
vii. Freight And Fleet Management
viii. Speeding Detection
ix. Vulnerable Road User Systems (E.G., Puffin Crossings)
x. Multimodal Trip Planning
xi. Passenger Information Systems
xii. Route Guidance and Navigation
xiii. Variable Message Signs
xiv. Road User Charging (RUC), Tolling and Access Control
xv. Public Transport Payment

11.2.2.1 INCIDENT DETECTION

ITS can be used to detect when there has been an incident on any transport system, and to communicate this knowledge to a control centre. ITS can, further, be used to put into effect information and/or traffic management strategies in response to certain types of incidents, in order to reduce their impact. For example, an accident may occur on a motorway. This is detected by roadside CCTV cameras and picked up in the control room. Variable message signing (VMS) is then activated to:

i. manage the traffic that is too close to the accident to take another route (by e.g. lane closures, lane control, temporary speed limits); and
ii. the VMS is used to advise traffic further away from the accident to take another route.

11.2.2.2 VARIABLE SPEED LIMITS

Due to the speed flow relationship in traffic, above a certain speed (around 80 km/h on motorways), flow in vehicles per hour past a given point begins to decline – the effect of higher speed is cancelled out by the larger gaps that drivers leave between vehicles. Therefore, at peak periods, it can be effective to lower speed limits to maximise road capacity and also to reduce congestion caused by the over-reaction of drivers to changes in speeds, and the “wave propagation” effect that this has. In order to do this, variable speed limit signing is required together with, if possible, some form of automatic enforcement (e.g. average or point speed cameras). The reduced congestion and speeds have a knock-on benefit on energy consumption.

11.2.2.3 RAMP CONTROL

Ramp control is used at peak periods to regulate the flow of traffic along a slip road (ramp) onto a motorway or other grade-separated road. Sensors on the main road detect traffic density and then the optimum level and spacing of joining traffic is calculated, and its access onto the main road regulated by traffic lights. This should in theory minimise the congesting effect on the main road of the joining traffic.

11.2.2.4 TRAFFIC SIGNAL CONTROL

ITS is used to manage linked and isolated traffic signalled junctions more efficiently, in relation to actual demand on the network in real-time. Inductive loop detectors in the pavement surface detect traffic levels, speeds and queue lengths. They communicate this to a local signal control computer and this in turn if necessary, communicates with a computer controlling the signals for a whole area of a town or city (a "cell") – but communications are kept as local as possible to minimise communication times and costs. The signal controllers compute the most effective cycle times and green times for their signals, but these have to be within user-defined maxima and minima – if the maximum cycle time is 120 seconds, the signal controller cannot override this. The introduction of real-time signal control of this nature – commercial systems in use around the world include Split, Cycle and Offset Optimisation Technique (SCOOT) and Sydney Co-ordinated Adaptive Traffic (SCAT) – typically increases the capacity of a group of linked signals by around 10%. In the short term, at least, such capacity increases should reduce congestion and therefore energy consumption (although these benefits may be eroded if traffic levels increase as a result of the reduced congestion).

Such signal control systems can be adapted to give priority to certain vehicles – most typically, trams and/or buses. Detectors note when a priority vehicle is on approach and (again within user-defined limits) can bring a green signal forward, or delay a red signal, in order that the priority vehicle does not have to wait to get through the junction. Increasingly, the identification of the priority vehicle is by satellite (GPS – geographical positioning system) linked to the public transport operator's control room. This means that priority can be given only to those vehicles that need it (ones running late) whereas, with inductive loop detection, all public transport vehicles are given equal priority, which is less efficient. Emergency vehicles can also enjoy priority, similarly.

Signal control systems can also be linked to real time information for drivers through variable message signs. Thus, when detectors pick up particularly bad congestion at one junction or in one street, they can relay this information (possibly via a human controller) to variable message signs, which can then be used to advise drivers of alternative routes and/or modes (e.g. "City centre congested, use park and ride!"). Smart signal control systems can also be used as a form of access control, or "gating" to certain sensitive areas. For example, one road may be used by a large number of public transport vehicles, or be particularly environmentally sensitive – therefore, queueing traffic may be particularly undesirable in that location. Signal timings can be used to "move the queue" from the sensitive area to a less sensitive one, and then only to permit through the optimum amount of traffic into the sensitive area. This system of "gating" – in real time – is used in Kingston, a suburb of London, UK. The same technique can be linked to pollution monitoring, so signals react in real time to "move" queuing traffic around, such that pollution hotspots do not build up in certain parts of town.

11.2.2.5 PARKING MANAGEMENT

Parking management may seek to achieve the following in relation to parking:

i. Inform drivers about parking opportunities.
ii. Assist in the distribution and management of limited numbers of parking spaces, including their pricing.
iii. Assist in the enforcement of parking.

ITS has a number of possible applications in relation to these objectives. Parking guidance systems have traditionally linked counters (microwave, inductive loop or infrared) at the entrances and exits of off-street car parks (which monitor occupancy and queueing) to variable message signs on key links into and around the town or city centre, in order to advise drivers where they are most likely to find a space, close to their final destination. These systems work as long as the car park operators maintain the car park counters and keep the local authority (which is normally responsible for the signs) updated about any alterations to their car parks, such as a change in the number of spaces, of the entrance points or, indeed, whether the car park has completely closed down! These systems also depend on the car parks having discrete entry and exit points where it is possible to install directional traffic counters, so that it is possible to keep a continuous, accurate measurement of the number of cars in the car park at any one time. This is why, to date, there have been no examples of parking guidance systems that inform drivers about where to park on-street. In addition, parking guidance only merits the investment where the distribution of parked cars between car parks is skewed, so that a few car parks are very popular, with others remaining only part-full. If there is low demand at all car parks, or demand is evenly-distributed at most times of day, then parking guidance may be of less use.

That said, parking guidance systems in Southampton were shown to reduce parking search time by 50%, to 1.1 minutes; and in Valencia, 31% of drivers changed their parking destination in response to parking guidance information.

Parking space management does not have to rely on ITS. However, it can be more efficient and more targeted where ITS is used, normally in order to more accurately relate information about the user to their access to a space. For example, it can be used:

i. In company car parking space management, to allow a member of staff access only to certain car parks, at certain times of day, on certain days of the week. This would also allow that member of staff to be charged on a "pay as you go" basis for parking, depending on how often they park and for how long. This has a much greater impact on travel behaviour than a flat rate monthly or annual charge.

ii. Also in company car parking space management, "parking cash out", where the employee receives a daily payment from the employer, which they can keep if they travel to work by a means other than by car on their own, but which they forfeit if they drive alone.

Enforcement of parking space management is another area where ITS can be very useful. Paper parking permits are relatively easy to copy and so fraud can be a problem. It is much more difficult to copy permits that contain microchips and electronic checking equipment can also decide whether a permit has been obtained or is being used fraudulently than can a parking attendant who is relying on checking paper permits by eye. In a situation where, for example, a business may be allowed to use one permit but switch it between vehicles, electronic enforcement makes it much easier to detect whether a permit has been fraudulently copied to use on more than one vehicle at a time.

Finally, parking payment systems increasingly use ITS. Across Europe it is more and more common to find towns and cities that allow the payment of on-street parking by mobile phone – Tarragona in Spain makes this system available at 112 locations across the city, for example.

The user registers and then sends an SMS to an on-street ticket machine when they wish to park, paying the bill through their bank account later on. In Singapore, where there is an electronic pay as you go road pricing system, it is also possible to use the smartcard used for road pricing to pay for parking and public transport as well.

11.2.2.6 DEMAND RESPONSIVE TRANSPORT MANAGEMENT

Demand Responsive Transport (DRT) is a form of public transport that, instead of operating on fixed routes at fixed times, operates with some level of diversion/flexibility to take users where they want, when they want. From the user perspective, the most flexible form of public transport is the taxi, but it comes with a matching price tag. DRT normally comes some way between a taxi and a conventional bus and, to maximise the flexibility and the efficiency of the service, a sophisticated booking/scheduling system is frequently employed. This has a number of objectives:

i. To ensure the highest possible utilisation of the vehicle and the driver(s).

ii. To keep journeys convenient for the user. For example, since a DRT is by its nature shared, someone already on the vehicle may have to put up with some inconvenience as it diverts off their most direct route to pick up someone else. ITS makes it easy to put constraints into the scheduling system, such as defining a maximum diversion, and maximum journey time, for passengers already on the vehicle.

iii. To allow users to make bookings by a variety of means – not only by phone, but by SMS and internet.

iv. To make sure that no trips are forgotten. With a paper schedule, there is a risk that a driver may miss out a pick-up by mistake or drop people off in the wrong order. With combined scheduling/routeing software and a communications link between driver and the control/booking centre, the router can indicate to the driver where they must go next.

v. To incorporate additional bookings at short notice, once a vehicle is out on the road. If someone calls in needing a trip, scheduling software can quickly calculate whether a vehicle nearby can pick them up and, if it can, this can be communicated to the driver whilst he is en route.

vi. To alert users when a vehicle is close by, so that they can get ready to be picked up. This can reduce dwell times and so increase vehicle utilisation.

vii. To store user details (address, most common trips, disabilities etc.) in a database, to simplify and speed up the booking procedure.

11.2.2.7 FREIGHT AND FLEET MANAGEMENT

Fleet management is an immensely important activity for any organisation that has even a small fleet of vehicles. It is therefore relevant to even small local authorities, as well as to bus operators (which may or may not be owned by the public sector). Fleet management is used to ensure that a fleet of vehicles is utilised to maximum efficiency. It depends on each vehicle being able to communicate its location, journey purpose and state (e.g., running normally, malfunctioning) to a central control room. This is normally done using satellite and radio technology, although certain bus only Automatic Vehicle Location (AVL) systems use roadside beacons – clearly these are suitable only if you are locating vehicles within a limited geographical area (e.g., a single city).

By monitoring how vehicles are used, fleet managers can:

i. Schedule and re-schedule vehicles more efficiently.

ii. Assess the need for more or fewer vehicles to carry out a set number of tasks (e.g., deliveries) – since it is possible to see the average time taken and how far different drivers deviate from the average.

iii. Assess individual driver behaviour e.g., the time taken to carry out a delivery; fuel consumption in relation to driving style.

iv. Manage services in real time. If for example a controller of a bus operation finds, from AVL, that all the buses on one route (line) in one direction are running late, he can use radio control to stop one or more of the buses before they reach the route terminus, and put them back into service in the opposite direction, to ensure that large gaps in service do not develop. Bus operators who have implemented AVL for fleet management reasons have realised fleet efficiency savings of around 9% (GOTIC, 2002).

11.2.2.8 SPEEDING DETECTION

Speeding is a major contributory factor to road accidents, and it increases both the risk of an accident occurring, and the severity of that accident. The National Road Safety Strategy IV (NRSS IV) has been prepared to provide the broad framework for road safety management in Ghana and it coincides with second UN Decade of Action for Road Safety 2021-2030, with the ambitious target of preventing at least 50% of road traffic deaths and injuries by 2030.

ITS can make a major contribution to the achievement of such targets, as follows:

i. Point speed cameras. These measure the speed of a vehicle at a short point on the road, such as at an accident blackspot, using radar detection, and conventional camera film (which is not always installed, so the camera is not effective 100% of the time). Vehicles exceeding the speed limit are sent a fine. A study of 38 UK sites where speed cameras were introduced between 2000 and 2004 found that, at these sites, "Both casualties and deaths were down – after allowing for the long term trend, but without allowing for selection effects (such as regression-to-mean) there was a 22% reduction in Personal Injury Collisions (PICs) at sites after cameras were introduced. Overall, 42% fewer people were killed or seriously injured. At camera sites, there was also a reduction of over 100 fatalities per annum (32% fewer). There were 1,745 fewer people killed or seriously injured and 4,230 fewer personal injury collisions per annum in 2004. There was an association between reductions in speed and reductions in PICs." (http://www.dft.gov.uk/stellent/groups/dft_rdsafety/documents/downloadable/dft_rdsafety_ 610816.pdf - page 4.)
ii. Average speed cameras. Installed over a stretch of road, these are linked to numberplate recognition systems that calculate the average speed of a car over that stretch. Similar enforcement to point cameras, but they use digital technology, so they are "on" all the time.
iii. Signs that alert drivers to their speed, but without any enforcement. For example, on the entry to a town, vehicles exceeding the urban speed limit will be detected by the sign which will flash a message "Slow Down – 50 km/h Speed Limit". These have been shown to reduce speeds by 2 to 20 km/h at a range of sites (Winnett and Wheeler, 2002).
iv. Intelligent Speed Adaptation (ISA) uses satellite GPS technology to indicate to a vehicle its own location relative to speed limits. "Active" ISA then introduces automatic control to the vehicle's engine and braking system so that the driver cannot exceed the speed limit. There are trials of ISA underway in the UK, the Netherlands and Sweden. Evaluation of the UK trial indicates that mandatory active ISA could produce (given a

1998 vehicle fleet) annual fuel savings of 2.3 billion litres of petrol and 1.4 billion of diesel in the UK alone (Carsten and Tate, 2005).

The link between speeding detection and energy use is twofold: firstly, above a certain point, vehicles travelling at higher speeds consume more energy; and, secondly, a reduction in accidents will tend to make the use of "slow" modes (walking, cycling) more attractive, thus contributing to modal shift. This has been observed in many cities where rates of cycle use have increased – a "virtuous circle" of safer roads and more cyclists leads to even safer roads for cyclists, and so more cyclists feel tempted to give cycling a try.

11.2.2.9 VULNERABLE ROAD USER SYSTEMS (E.G., PUFFIN CROSSINGS)

"Vulnerable" users include children, the elderly and the physically challenged. They are all disproportionately represented in road accidents, especially as pedestrians. In addition, because physically challenged persons have reduced mobility and/or sensory perception, a conventional transport system may not fully meet their needs. ITS can be used to adapt our transport systems to make them easier and safer to use for these groups of people. For example:

i. Pedestrian crossings that detect, using cameras, how long people are taking to cross the road, and giving them more time if they need it.
ii. Audible public transport information for blind/partially sighted people. In some cases, this can be provided only when needed – blind and partially sighted people are given a pocket-sized device that activates a bus stop or other information point to give them audible information when they need it.
iii. Auditory location finder for blind/partially sighted people.
iv. Real-time next stop indicators (visual and audible) on buses and trains.

11.2.2.10 MULTIMODAL TRIP PLANNING

Many public authorities are concerned to provide ever better (public) transport information in order to encourage a mode shift from car to public transport in order to achieve other wider objectives. Multimodal trip planning services – available on the phone, internet, WAP and/or SMS – may do this. The user is able select their origin and desired destination, and the interface then produces a variety of trip options on a variety of modes of transport.

11.2.2.11 PASSENGER INFORMATION SYSTEMS

When linked to AVL (see above), ITS has the ability to provide Real Time Information to Public Transport Passengers (RTPI) through a variety of media such as at-stop displays, SMS messaging and the internet. A schematic representation of a typical system is shown **Figure 11.5**. Theoretically, these systems should lead to some energy saving by promoting modal shift from car to bus.

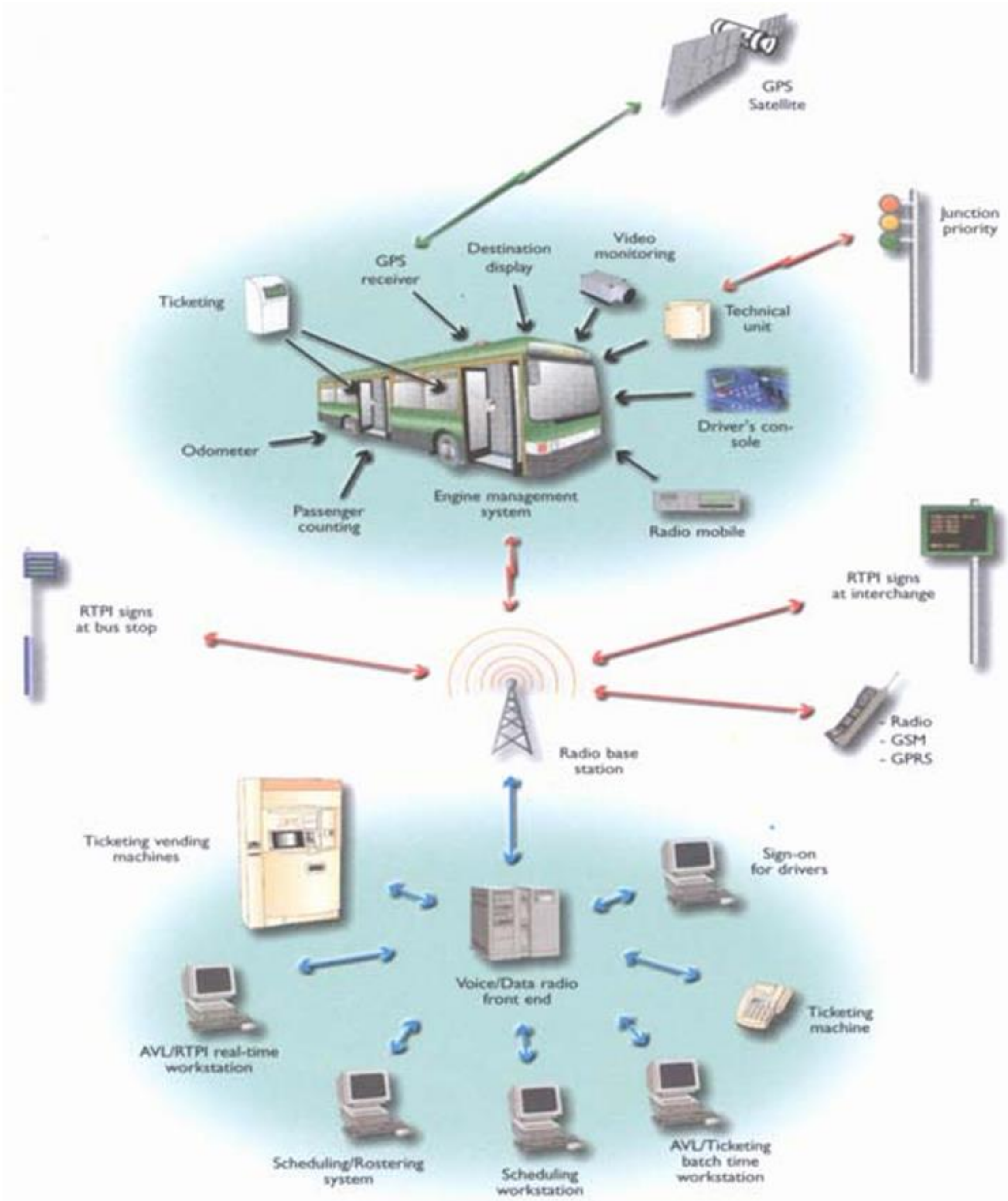


Figure 11.5 Schematic representation of RTPI system

11.2.2.12 ROUTE GUIDANCE AND NAVIGATION

Global Positioning Satellite (GPS), as used in fleet management and AVL (**Figure 11.5**) can also be used to provide drivers with very detailed route guidance information. This is increasingly commonly fitted to cars as standard. It obviously has a beneficial effect in terms of energy consumption as it can reduce the number of kilometres driven (when navigational errors are made in manual navigation) and can be used in tandem with congestion information to give drivers advice on how to choose routes to avoid incidents.

11.2.2.13 VARIABLE MESSAGE SIGNS

Variable Message Signs (VMS) are a form of ITS that provides information to travellers. As its name suggests, the message on a VMS can change in real time. This is usually achieved by means of a display that can be programmed from a control centre remote from the sign. The advantage of VMS is, of course, that the sign can change to reflect prevailing conditions. They are used predominantly to advise drivers on main roads of difficult driving conditions (e.g. "Bridge closed to high vehicles due to high winds"), to give notice of incidents (e.g. "accident at Bunso Junction") and also to advise of diversionary routes that can be taken when an incident has occurred.

11.2.2.14 ROAD USER CHARGING (RUC), TOLLING AND ACCESS CONTROL

Tolls are collected from road users in order to collect all or some of the costs of constructing a road, bridge or tunnel. Road user charging is applied to existing roads as a way of managing demand (congestion) and/or raising money for new infrastructure and services. In any event, a means of collecting the money is required, and of enforcing non-payment. ITS can be an extremely useful tool in so doing. Some examples of the way in which it is used are as follows:

i. The London Congestion Charge (www.cclondon.com) is enforced by automatic number plate recognition (ANPR - cameras photograph number plates and then a computer compares the numberplate with those stored in a database of people who have paid their charge that day). ITS must be used for the numberplate recognition, but also to allow people to register on the database when they pay. They can do this by paying in cash at a pay station (e.g. a petrol station), or by pre-registering and then paying on the internet, by SMS or by phone. Clearly all these systems have to be interlinked and this is a very good example of a complex ITS application. With complexity comes cost; system set-up was around €300 million, and the operating costs are around €3.20 per charge payer.

ii. Many toll bridges, toll motorways and all four congestion charging schemes in cities in Norway use a tag-based system for collecting revenue. Vehicles whose owners register for the scheme carry an electronic tag that is "read" by beacon when the car passes through a charging point. They are then billed monthly according to the number of charged trips they have made. ANPR is used for enforcement.

iii. In Singapore, a congestion charging scheme uses an In-car Unit (IU) as shown in **Figure 11.6**, with which every vehicle must be equipped. The user buys a card rather like a phone card and must insert the card in the IU before driving into a charged area. Charges in Singapore vary by location and by time of day, but a beacon activates the IU when the vehicle drives past a charging point, and the correct amount of money is deducted from the card in the IU. Once again, ANPR is used for enforcement.

Figure 11.6 Singapore Electronic Road Pricing In-vehicle Unit

11.2.2.15 PUBLIC TRANSPORT PAYMENT

ITS facilitates public transport fares payment in several important ways:

i. Electronic ticket machines on buses and trains allow the issue of more specific ticket types, which benefits the passenger. However, they also collect a great deal of management information (e.g., with certain ticket machines it is possible to measure the speed of a bus along a route) which is of use to the operator.
ii. Ticket issuing machines at stations and stops reduce boarding times and time spent waiting for tickets, thus shortening the overall public transport journey.
iii. Smartcards retain information about the individual user (e.g. that they have a concessionary permit) as well as facilitating a wider range of fares options than is possible with paper based tickets. They can also be used to reduce ticketing fraud where this is a problem. They can, therefore, have benefits for the passenger as well as for the operator.

11.2.3 EXAMPLES OF APPLICATION OF INTELLIGENT TRANSPORTATION SYSTEM (ITS)

A. PUBLIC TRANSPORT PAYMENT – OYSTERCARD

The Oystercard is a public transport smartcard used in London, UK. Some Oystercards are used like season tickets (abonnements) but they can also be used as stored value tickets – passengers can charge them up with money (even if they also use their Oyster as a season ticket on another part of the network). It is contactless, using radio frequencies for the card to communicate with the card reader. It is also the world's first public transport ticket to feature price-capping – passengers can use it to pay single fares but, if the cash value of their single fares in a day exceeds the price of a one-day ticket, then they are only charged for that one-day ticket. The same is true of travel over a week. Oystercard also gives passengers a roughly 33% discount compared to buying cash fares. There is therefore a major incentive for passengers to move over to the Oystercard, which is particularly important on buses in London, where traditionally passengers have paid the driver if they want a cash fare, which slows down the buses (at stops) very considerably. The use of a Smartcard also gives operators much better data about how many passengers travelled on which trains and buses, which can (depending on the nature of the financial contract between operator and public transport authority) make the distribution of revenue to operators much more accurate than if there are only paper tickets.

B. PARKING GUIDANCE SYSTEM, AALBORG

Aalborg's parking guidance system is based on real time information of the number of unoccupied parking places in each parking facility of the system. Parking information is displayed on number of Variable Message Signs (VMS) on the main roads leading to the city centre as well as on the circular roads around the city centre. The circular roads distribute car traffic to the parking spaces on the periphery of the city centre.

The parking guidance system embraces 9 parking facilities (7 parking grounds and 2 parking houses) containing around 3000 parking spaces around the city centre. A total of 39 VMS and 8 conventional signs are linked to the system.

C. ACCESS CONTROL, ROME

Rome adopted in 1994 the access limitation to the Limited Traffic Zone (LTZ) of the city centre sectors east of Tiber (area of 4.6km2). In 1998, the payment for a yearly permit to access the area only for specific users was introduced. In October 2001, during the PRoGR€SS demonstration, the electronic full scale Access Control System and flat-fare Road Pricing scheme (ACS+RP) called IRIDE was switched on with the use of 23 entrance gates and a complex control centre located in STA. The automation of the access control system is accomplished through the use of a series of gates that can effectuate, without user intervention, the identification and/or the applicable tariff for vehicle entrance into the restricted area (vehicle-ground beacon). The enforcement is active during the weekdays from 6.30am to 6.00pm and on Saturday from 2.00 to 6.00pm.

The following types of technology infrastructure, based on the technology used for the TELEPASS system:

- TV Camera and infra-red Illuminators
- Microwave Transponder
- On-board Unit with Smart Card

Scale

- Full real pricing scheme-real charging, real users, real revenues
- Area covered by system: 4.6km^2

- Number of charging points: 22+1 entrance gates
- Number of users: 30,000 resident vehicles, 30,000 service vehicles (free access), 50,000 plates for disabled peoples (free access), 29,000 authorised individuals and 8,000 freight delivery vehicles (have to pay for access)
- Number of trips per day: about 70,000.

The effect of the scheme was to lead to a 10% decrease in traffic during the day, a 20% decrease in traffic during the restriction period, a 15% decrease in the morning peak hour (8.30-9.30), 10% increase of two wheels and a 6% increase in public transport use.

D. CITY OF GLASGOW, SCOTLAND

Intelligent Transport System gives regular information to the daily commuters about public buses, timings, seat availability, the current location of the bus, the time taken to reach a particular destination, next location of the bus and the density of passengers inside the bus.

E. SEOUL GOVERNMENT, KOREA

The Seoul government has designed the routes of its new night bus services based on an analysis of night-time mobile phone location data. The city worked with private telecommunication companies to analyse calls made between midnight and 5 am, and matched this data, anonymously and in aggregate, with billing addresses to determine which routes would experience greater demand for overnight services.

F. NEW ORLEANS, USA

The **city of New Orleans** has optimized the locations of its ambulances on standby, based on patterns of emergency calls.

G. SYDNEY METRO, AUSTRALIA

The highly anticipated **Sydney Metro**, Australia's biggest public transport project, will feature a driverless mass-transit system that is anticipated to nearly double the city's existing transit capacity.

H. COPENHAGEN, DEMARK

The Cityringen project in Copenhagen is a very ambitious project. The Cityringen is a driverless metro that will form a new circular line in the centre of the city and consists of two parallel tunnels some 15.5 km long and 17 underground stations, situated an average of 30 metres below street level. The fully automated line is driverless and, once fully operational, will provide a 24-hour transport system that guarantees the mobility of 240,000 passengers a day (or 130m a year).

I. **ETC 2.0: TWO-WAY COMMUNICATION BETWEEN VEHICLES AND THE ROAD, JAPAN**

ETC 2.0, a more advanced ETC system, has become fully operational. The most significant change achieved by the new system is its facilitation of large-volume interactive communication.

The previous-generation system used communication technology only to collect tolls on highways, but ETC 2.0 offers new services using a vehicle's driving history data, such as GPS information, distance travelled, and acceleration and rapid braking data. This data is sent from ETC 2.0 onboard units to roadside antennas (ITS spots) that are installed every 10–15 km on inter-city highways and every 4 km on urban highways.

11.2.4 INFORMATION DISPLAY SYSTEM

The information display board and types of information to be provided are shown in **Table 11.1** and **Table 11.2.**

Table 11.1 Types of Information Display Board

Type of Display Board	Information Display (Output)
Display panel	The display contents are directly inputted into the panel.
Subtitles	Subtitles are written vertically or horizontally, and light is internally generated.
Light transmitting	Subtitles are written vertically or horizontally, and a parabolic reflector behind the subtitles is used to position a parallel line on the subtitles, and light passing through that passes through the transmitting lens on the surface panel to form the characters.
Electric light display	Characters are formed by bulbs installed on the surface. This system can be used with both fixed displays and free displays.
LED	Characters are formed by LED elements installed on the front face. This system can be used with both fixed displays and free displays.

Table 11.2 Types of Information

Type of information	Incident	Display items
Weather	Abnormal weather that may cause road disasters or impede traveling	Weather information concerning heavy rain, fog, etc. Issuance of warnings, cautionary, etc.
Disaster	Road disasters caused by landslide, bridge failure, flooding etc.	Disasters such as rock falls, landslides, inundated roads, road collapse, landslips, etc.
Accident	Accidents that hinder traffic	Traffic accidents
Obstacle on the road	Obstacle on the road that hinders traffic	Disabled vehicle, festivals/floating activities on the road etc.
Road works	Road works that hinder traffic	Under construction/maintenance
Regulatory	Traffic regulations arising from disasters, accidents, etc.	Closed to traffic, closed to heavy vehicles, single-side traffic, alternate traffic
Diversion	Diversions when traffic regulations are in effect	Diversion, no diversion, name of diversion road
Cautionary	Traffic cautions	Travel caution, drive slowly,
Guidance	Route information	Town/City guidance, road guidance
Preliminary	Regulations arising from road works and preliminary notice of the lifting of regulations	Contents and period of road works, etc.

11.3 TRAFFIC SIGNALS

Signalisation is a form of traffic control which provides conflicting movements the right of way by time separation (**Figure 11.7**). While the general objective is to reduce delays and increase capacity of vehicle traffic, the interest of pedestrians and slow vehicles should be given equal weight. Safety should be a primary consideration with adequate allowance for failure on the part of users to comply with rules.

Figure 11.7 Traffic Signals at an Intersection

Signalisation is adopted where traffic volume is high with a significant proportion of crossing or turning traffic. It is also adopted to provide pedestrians and slow vehicles a safer opportunity where traffic volume or speeds are high and there are multiple traffic lanes.

Traffic signals for vehicle traffic consist of three-aspect displays of red, amber and green. Green signal display and sometimes red signal display could be in the form of an arrow pointing upward to the left or the right.

Outside built-up areas, it is generally not advisable to provide isolated signalised intersections where traffic speeds are high, and drivers may not expect signalised control.

The design of traffic signal devices and warrants for their use are provided in the Manual on Uniform Traffic Control Devices (MUTCD) by the Federal Highway Administration (FHWA) of the United States Department of Transportation.

11.3.1 GENERAL REQUIREMENTS

The key principle of signalised intersection is separation and allocation of conflicting traffic streams to different stages of signal display. It may be acceptable to permit offside-turning on a full green signal against conflicting straight ahead traffic, subject to relatively low traffic speeds, simple intersection layout and clear indication of give-way markings.

It may be desirable to restrict certain movements such as offside turning to increase the capacity of a signalised intersection as long as the implications of diversion are fully considered. The geometry of road corners and channelisation layouts should be designed to discourage the banned turning movements.

It is generally necessary to limit the cycle time of traffic signals within 120 seconds and desirably 90 to 100 seconds. Above a cycle time of 120 seconds, almost no extra capacity is gained due to the build-up of long vehicle headway, and delays increase considerably. The inter-green period between the end of the green signal on one stage and the start of the green signal on the next stage should be determined from conflict points specific to the intersection geometry and the paths of traffic and pedestrians. Inter-green values should not be less than 5 seconds and larger values up to 15 seconds may be required:

i. when the distance across the junction is excessive
ii. to improve safety on high-speed roads
iii. where there is insufficient offside-turning traffic to justify provision of a separate stage

The length of the inter-green period will need to be optimised between increased risk of conflicts and excessive delays which could lead to non-compliance by drivers.

It is desirable to provide at least two sets of traffic signals with synchronised displays for each approach arm. The primary signal is located just upstream of the intersection and the secondary signal is located just downstream of the intersection. Pedestrian signals should be provided to give positive indications to pedestrians.

Traffic signals may be mounted on poles or overhead structures in cantilever or gantries. Irrespective of the mounting form, consistent usage should be adopted along a road.

Signal controllers should be provided at vantage position to facilitate manual control or testing. Their position should not result in obscuring of critical visibility.

11.3.1.1 PEDESTRIAN CROSSINGS

Pedestrian signals should be provided at signalised crossings wherever there is reasonable demand for crossing pedestrians. The layout of signalised intersections should, as far as practical, facilitate pedestrians with minimum crossing width, limited number of lanes to cross and reduced traffic speeds, especially where there is still ample reserve capacity for vehicle traffic. A full pedestrian stage may also be considered in these circumstances.

Nearside turn channelizing islands may not be in the interest of pedestrians since they increase the number of steps to cross a road. If they are required, the layout should not induce high traffic speeds through a pedestrian crossing.

It is preferable to allow pedestrians to cross a divided road in one attempt at a signalised intersection. If this is not practical and pedestrians have to wait at the median or refuge island, sufficient space should be provided for pedestrians to wait and pass each other, with a minimum average pedestrian area occupancy of 0.2 m^2/person.

If a staggered pedestrian crossing is provided, pedestrian paths are preferably oriented towards approach traffic. Furthermore, pedestrian signals should be positioned on the upstream side of the crossing for the same reason.

Where pedestrian demand is low at a signalised crossing, consideration should be given to the installation of push buttons for pedestrians to activate the pedestrian signal stage. Push buttons should be located at each side of a pedestrian crossing with additional ones on wide pedestrian refuge islands. They should be positioned to encourage pedestrians looking towards approaching traffic.

Where a straight pedestrian crossing with a limited median or refuge is provided, the overall green period for pedestrians including both green displays and flashing green time should be sufficient to enable pedestrians to cross the full width of the road at normal walking speed. The length of the flashing green time would be determined from the maximum distance from safe refuge to safe refuge. Green displays should be sufficient to clear all waiting pedestrian but not less than 5 seconds.

The adequacy of the overall green period may be checked by the **Equation 11.1**:

$$PC = K \times GTP \times W \tag{11.1}$$

Where,

PC = Pedestrian crossing capacity in pedestrians per hour
GTP = Green time proportion= (Pedestrian green + flashing green time)/Cycle time
W = Width of pedestrian crossing
K = Saturation flow for pedestrians= 1,900 ped/metre/hour

The separation distance between a pedestrian crossing and the stop line should normally be 2m but extension up to 6m could be desirable with a high-speed approach.

There should be adequate green time for a pedestrian to cross at the walking speed of 1.1m/s. Where the crossing is frequently used by elderlies and pedestrians of special needs, and in the case of heavy pedestrian usage, it is recommended to reduce the design walking speed to the order of 0.9m/s.

Both straight ahead and offside turning traffic movements should not be permitted over a pedestrian crossing when the pedestrian signal is on green display. It is also strongly advised not to permit traffic making nearside turn on red signals at any time. It is tolerated but not advisable to permit nearside turning into an arm when pedestrian signals are green particularly where turning traffic speeds are high and drivers are not willing to give way.

Where one traffic stream is stopped while another traffic stream is running on the same approach, pedestrians could be tempted to cross in front of waiting traffic. In this case, it is strongly advisable to provide a channelizing island separating the traffic streams with an additional set of traffic signals. Alternatively, green signals are concurrently displayed for both through traffic and turning traffic.

The following additional safety measures could be considered:

i. Traffic calming measures
ii. Staggered crossings with a median or refuge island
iii. Countdown timer
iv. Automatic extension of flashing green signals for pedestrians
v. Audible signals
vi. Red light speed cameras

11.3.1.2 SIGNING AND DELINEATION

Advance warning signs showing "Traffic Signals Ahead" are needed for approaches with inadequate visibility. They are also needed for high-speed approaches and isolated traffic signals.

Traffic lanes may be narrowed down on the approach to a signalised intersection. Lane widths down to 3.0m would be readily acceptable in urban area.

Two or more sets of arrow markings showing the permitted movements should be provided on each traffic lane ahead of the Stop line. To encourage compliance to lane arrow markings, the number of traffic lanes on the downstream side of an intersection should be the same as the number of upstream lanes marked as straight ahead lanes.

Where traffic movements are restricted for a particular set of traffic signals, appropriate traffic signs indicating the mandatory or prohibited movements should be provided.

11.3.2 SIGNAL HEAD TYPES

Types and general locations of signal heads are as follows:

i. Primary: Mounted on the far-right side of each intersection approach. Locating primary signal faces overhead on the far side of the intersection has been shown to provide safer operation by reducing intersection entries late in the yellow interval and by reducing red signal violations, as compared to post-mounting signal faces at the roadside or locating signal faces overhead within the intersection on a diagonally-oriented mast arm or span wire. On approaches with two or more lanes for the through movement, one signal face per through lane, centred over each through lane, has also been shown to provide safer operation (MUTCD, 2009).
ii. Secondary: Mounted on the far-left side of each intersection approach.
iii. Auxiliary: An auxiliary head may be located on the near right side of each intersection approach or in any other position, which may appear necessary due to local conditions.
iv. Pedestrian: Mounted on the far side of the intersection in line with the painted crosswalk

Signal head sizes are to be as indicated in **Table 11.3**.

Table 11.3 Signal Head Sizes

Signal Head Type	Area Classification	Lens Size and Shape
Primary	All Areas	300mm round
Secondary	Rural and Small Urban Areas	200mm round green, yellow and red with 300mm green arrow
	Large Urban Areas	300mm round
Auxiliary	Rural and Small Urban Areas	200mm round green, yellow and red with 300mm green arrow
	Large Urban Areas	300mm round
Pedestrian	All Areas	Combination walk/don't walk indication 425mm x 467mm

11.3.3 SIGNAL HEAD VISORS

A signal head visor shall be in accordance with the MUTCD and should be used to direct the signal indication to the appropriate approaching traffic, especially if conflicting signal faces are readily visible, and to reduce sun phantom which can result when external light enters the lens. A signal head visor should be used with each lens on the signal head face and are made of the same material as the housing. The rear of the signal head visor must have four, slotted mounting tabs for easy attachment and for securing the visor to the signal housing door. The signal head visor mounting method must permit the signal head visor to be rotated and secured at 90^0 for horizontal signal head installations. The signal head visor shall have a minimum length 240mm and a minimum downward tilt of 3.5^0 measured from the centre of the lens. There are three types of signal head visors: cut-away, tunnel, and full-circle visors as shown in **Figure 11.8**.

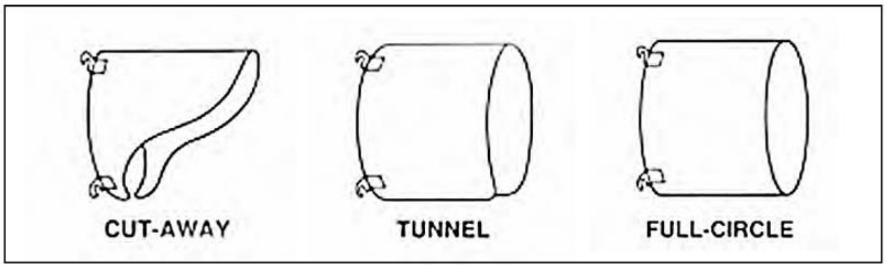


Figure 11.8 Types of Signal Head Visors

Cutaway visors, sometimes referred to as a cap or partial visors, are signal head visors with the bottom cut away. This type of signal head visor reduces water accumulation and does not let birds build nests within the visor. Tunnel visors reduce the signal visibility from other approach directions by providing an almost complete circle around the lens. Tunnel visors look like an inverted "U" that encircles and shields the lens from a minimum 300^0 with the opening at the most bottom of the lens. This type of signal head visor reduces water accumulation and does not let birds build nests within the visor. Full-circle visors are similar to tunnel visors with the exception that it provides a complete circle around the lens. Full-circle visors have a sharp angular beam cut off for signal installations where highly directional beam characteristics are necessary to prevent driver confusion, such as streets intersecting at a very sharp angle of 35^0 or less. Full-circle visors should only be considered when using visibility-limited traffic signal devices. This type of signal head visor has a drawback in that it inherently has possibility of water accumulation and bird nests built that can block the lens.

Cut-away and tunnel visors are normally used, but the decision on which signal head visor type should be determined using engineering judgment on a site-by-site basis. To assist in this determination, first measure the angle between the lines of sight for approaching vehicles as shown in **Figure 11.9(a)**. If the approach bends to a near 90^0 angle as shown in **Figure 11.9(b)**, then use engineering judgment to determine the line of sight angle. Consideration of the line of sight angle should also be given for vehicles at the stop lines as shown for diagonal spans in **Figure 11.9(c)** and for mast arms in **Figure 11.9(d)**.

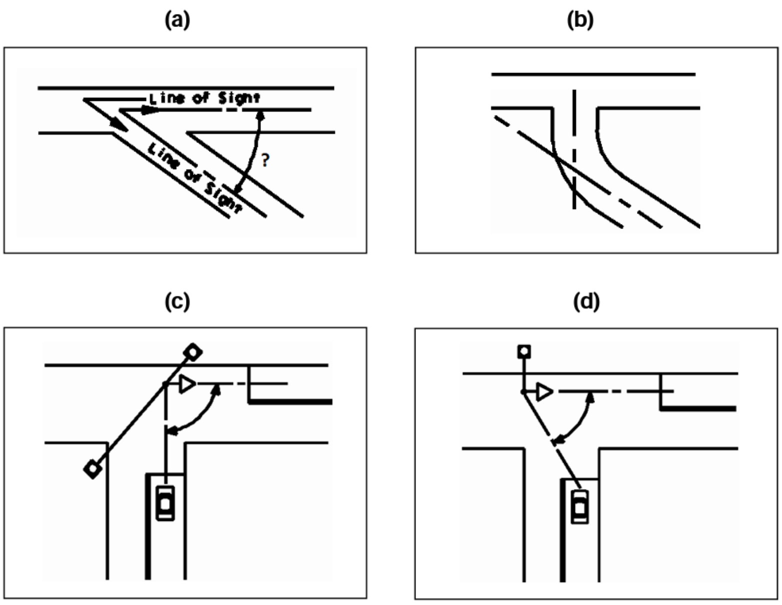
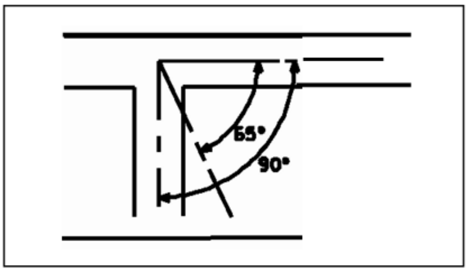
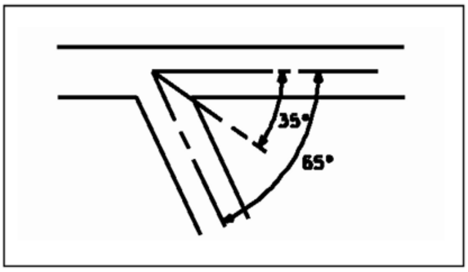


Figure 11.9 Line of Sight Angle Measurements

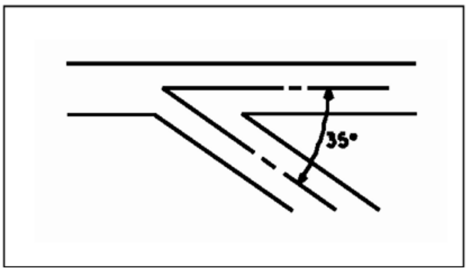
After determining the line of sight angle for approaching vehicles as shown in **Figure 11.9**, the recommended signal head visor type should be determined using **Figure 11.10**.



For roadway approaches with line of sight angles between 65^0 and 90^0 line of sight angles, use cut away or tunnel signal head visors.



For roadway approaches with line of sight angles between 35^0 and 65^0, use tunnel signal head visors or full-circle signal head visors.



For roadway approaches with line of sight angles less than 35^0, use full-circle signal head visors with visibility-limited signal devices.

Figure 11.10 Recommended Signal Head Screening Types

11.3.4 SIGNAL HEAD PLACEMENT

Signals should be mounted on poles, davits, mast arms, or gantries. Mounting heights, as measured to the lowest portion of the signal head, are as follows:

i. Primary (non- supplemental) signals mounted above roadways should be mounted at any height that meets visibility requirements. When practicable, the primary head should be located in such a position that it will be situated within the driver's cone of vision, at a minimum height of 4.5m clear of the pavement and at a distance normally in the range of 12 to 55m from the near side stop line, with a desirable minimum distance of 15m. Standard mounting height is between 5 and 6 m above the roadway.
ii. Secondary and auxiliary (supplemental) signals should be mounted at any height that meets visibility requirements. When the signal is mounted clear of the travelled roadway the bottom of the signal head shall be not less than 2.75m above the walkway however the standard mounting height is 4.9m clear of the walkway. Where a walkway does not exist, these measurements refer to the pavement surface of the centre of the roadway.
iii. The mounting height of pedestrian signals shall be not less than 2.75m.

If a signalized through movement exists on an approach, a minimum of two primary signal faces shall be provided for the through movement. If a signalized through movement does not exist on an approach, a minimum of two primary signal faces shall be provided for the signalized turning movement that is considered to be the major movement from the approach.

The minimum number and location of primary (non-supplemental) signal faces for through traffic should be provided in accordance with **Table 11.4**.

In addition to the primary signal faces, one or more supplemental (secondary and auxiliary) pole-mounted or overhead signal faces should be considered to provide added visibility for approaching traffic that is traveling behind large vehicles. All signal faces should have backplates. A supplemental near-side signal face is required where primary signal faces are located more than 55m beyond the stop line. A supplemental near-side signal face may be beneficial where primary signal faces are located between 46 and 55 m from the stop line. A supplemental near-side or far-side signal face may also be beneficial where approaching speeds are 70km/h or greater.

Table 11.4 Primary Signal Head Placement

STRAIGHT THROUGH LANES		
Number of Lanes	**Number of Primary Heads**	**Placement of Primary Heads**
1	2	Centred over through lane
2	2	Centred over each through lane
3	3	Centred over each through lane
4 or more	4 or more	Centred over each through lane
LEFT TURN LANES		
Left Turn Type	**Primary Head Type**	**Placement of Primary Heads**
Protected/Permissive	Flashing Green Arrow, Steady Yellow Arrow and Steady Green Ball	Centred over left-most through lane
Protected - Single Left Turn Lane	Steady Green Arrow	Centred on the left turn lane, either post mounted in median or overhead arm mounted
Protected - Dual Left Turn Lane	Steady Green Arrows	Centred over each left turn lane, either post mounted in median or overhead arm mounted

11.3.5 POLE PLACEMENT

Signal poles should be placed between 1.5 m and 3 m from the face of kerb or edge of pavement, preferably behind the walkway. Pole arms should be oriented at 90° to the centreline of the road, except where the intersection is skewed. When laying out a skewed intersection, ensure the arms do not block the view of the signal heads.

Other considerations for pole placement are:

i. Clear zone requirements.
ii. Ease of access for pedestrians.
iii. Arm reach to ensure signal head is over lane centres or lane markings as appropriate.
iv. Minimizing the number of poles required.
v. Limiting number of heads on a pole shaft to four.

11.3.6 LEFT TURN PHASING

Left turns at signalized intersections are phased in three different manners as follows:

i. A Permissive left turn has no signal indication other than a green ball, which permits a left turn when opposing traffic is clear.
ii. A Protected left turn presents a continuous green arrow indication while all opposing traffic is held by a red ball. A Protected left turn is always terminated with a yellow ball. A Protected left turn signal head requires the placement of a "left turn signal sign".
iii. A Protected/Permissive left turn presents a flashing green arrow followed by a green ball. During the flashing phase (advanced movement), opposing through traffic is held by a red ball. After the advance has timed out, left turn traffic is presented with a green ball permitting the movement when conflicting traffic is clear. The protected phase of this movement is always terminated with a non-flashing yellow arrow indication.

Protected left turns are typically used in the following circumstances:

i. Permissive left turns are deemed hazardous due to gap judgment difficulty caused by high speed, geometrics or visibility.
ii. More than one left turn lane on the approach.
iii. Lack of sight distance to oncoming vehicle.
iv. High pedestrian volumes.
v. High accident experience.
vi. Left turn phase is in a lead-lag operation.

Protected/Permissive left turns are appropriate in cases where:

i. Peak hour left turn traffic volumes justify the movement.
ii. Left turn delays are a concern.
iii. Accident experience dictates.

Care should be taken when considering a left turn phase, as it could cause delays at the intersection by increasing the total cycle length.

11.3.7 ADVANCED WARNING FLASHERS

Advanced warning flashers should be used where sight distance to an intersection is less than optimal, or where design speed of the road is sufficiently high to justify warning motorists of signal status.

11.3.8 AUDIBLE PEDESTRIAN SIGNALS

Where required, use audible pedestrian signals to assist visually impaired pedestrians. The audible signal is interconnected with the Walk signal, and produces a "cuckoo" or "peep" sound, depending on the direction of crossing. The cuckoo sound is used for north-south crossings and the peep is used for east-west crossing. Where the streets are not oriented north-south and east-west, maintain consistency with adjacent signals.

11.3.9 VISIBILITY REQUIREMENTS

The primary visibility requirements for signalised intersections are:

i. Visibility of traffic signals
ii. Visibility of pedestrian and slow vehicle crossings
iii. Intervisibility at the intersection
iv. Control of undesirable visibility

11.3.9.1 VISIBILITY OF TRAFFIC SIGNALS AND CROSSING AREAS

Signal visibility distance is defined as the distance in advance of the stop line from which a signal must be continuously visible for approach speeds varying between 40 and 80 km/h. For speeds exceeding 80 km/h, the minimum visibility distance must equal or exceed the minimum stopping sight distance.

The two primary signal faces required as a minimum for each approach should be continuously visible to traffic approaching the traffic control signal, from a point at least the minimum sight distance provided in **Table 11.5** in advance of and measured to the stop line. This range of continuous visibility should be provided unless precluded by a physical obstruction or unless another signalized location is within this range.

Table 11.5 Visibility of Traffic Signals

85th percentile Speed (km/h)	Minimum Visibility Distance (m)	Desirable Visibility (m)	Add for % Downgrade (m)		Subtract for % Upgrade (m)	
			5%	10%	5%	10%
40	65	100	3	6	3	5
50	85	125	5	9	3	6
60	110	160	7	16	5	9
70	135	195	11	23	8	13
80	165	235	15	37	11	20

Traffic signals should be readily visible by vehicles waiting at the stop line. This is achieved by providing:

i. a single set of signals after the intersection
ii. a set of secondary signals after the intersection

These signals should be orthogonal or else within 30 degrees of the sightline of approach drivers. A backing board in black of dark colour and light colour border should be provided around traffic signals if they are viewed against the sky or a distracting background.

Adequate visibility is required for both the pedestrian crossing and waiting area on the walkway. Further guidance is given under the topic of pedestrian crossings.

11.3.9.2 CONE OF VISION

Visibility of a signal head is influenced by three factors:

i. Vertical, horizontal, and longitudinal position of the signal head;
ii. Height of driver's eye; and
iii. Windshield area.

Lateral vision is considered to be excellent with 5° of either side of the centreline of the eye position (10° cone) and adequate within 20° (40° cone). Horizontal signal position should therefore be as follows:

i. Primary heads within the 10° cone; and
ii. Secondary heads within the 40° cone.

Vertical vision is limited by the top of the windshield. Signal heads should be placed within a 15° vertical sight line. Overhead signals should be located a minimum of 15 m beyond the stop line. Refer to MUTCD for additional detail.

11.3.9.3 HIGH VEHICLES

Drivers of vehicles following high vehicles must be able to see at least one signal head upon reaching the dilemma point. The dilemma point is defined as the location where a driver seeing the signal indication change from green to yellow must decide either to bring the vehicle to a safe stop or proceed through and clear the intersection prior to the start of the conflicting green. Factors to consider in assessing signal head visibility are road geometry, design speed, spacing between vehicles, and horizontal and vertical signal head locations.

11.3.9.4 ENVIRONMENTAL

Signal heads need to stand out from the surroundings in order to prevent confusion due to distractions. Primary signal heads must have backboards. Backboards are optional for secondary and auxiliary heads. Backboards must be yellow with a reflective surface. A yellow reflective tape border on the backboard can increase signal visibility.

11.3.9.5 FLASH RATES

The effectiveness of flashing signals is influenced by flash rates.

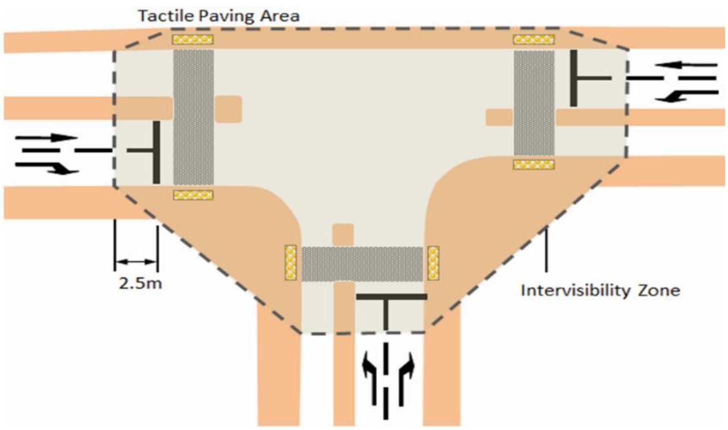
Recommended rates are:

1. Red and amber balls: 50 to 60 flashes/minute
2. Arrows: 100 to 120 flashes/minute

The ON and OFF periods should be equal.

11.3.9.6 INTERVISIBILITY

Visibility should be provided between drivers located 2.5m ahead of a Stop line and a vehicle located on a conflicting traffic stream and waiting areas of pedestrian crossings (**Figure 11.11**).



Observer	Eye-height	Target	Object Height
Driver	1.05m – 2.0m	Vehicle	0.6m – 2.0m

Figure 11.11 Intervisibility zone at signalised intersections

11.3.9.7 CONTROL OF UNDESIRABLE VISIBILITY

Misinterpretation of traffic signals could lead to severe collisions. For this reason, traffic signals for one traffic stream should not be visible to another traffic stream.

Where two independent sets of traffic signals along a route are in the proximity, drivers at the upstream signals could be misguided by the downstream signals. The problem may be alleviated by synchronizing the two sets of traffic signals and installing hoods or louvres over the signal aspects.

It is also necessary to check that drivers are unlikely to follow traffic signals for conflicting arms by mistake. This is particularly the case for skewed intersections.

Lane control signals or variable message signs showing green or red colour arrows should not be installed immediately downstream of a signalised intersection to avoid misleading drivers.

11.3.10 SIGNALISED INTERSECTIONS ON HIGH-SPEED ROADS

Signalised intersections with approach speeds exceeding 70km/h require special attention. Traffic signals should not be adopted if traffic speeds on any approach exceed 100 km/h.

It is always desirable to reduce traffic speeds on the approach and through a signalised intersection. Outside built-up areas, traffic speeds preferably do not exceed 70km/h. In built-up areas, traffic speeds should be limited to 50km/h and desirably even lower where there are many pedestrians or slow vehicles. It may be desirable to reduce speed limits locally around an intersection.

Additional care is needed to reduce traffic speeds on the approach at the end of a free-flow divided road and for signalised intersection with a steep downhill approach.

Additionally, the following measures should be considered for signalised intersections on high-speed roads:

i. Adoption of high intensity signal aspects of large size, in the order of 300mm diameter
ii. Duplicate primary signals
iii. Provision of backing boards on signals
iv. Use of advance warning signs on all approaches
v. Use or addition of overhead traffic signals to complement pole mounted signals

Typical Layouts of High speed or major signalised intersections are illustrated in **Figure 11.12**.

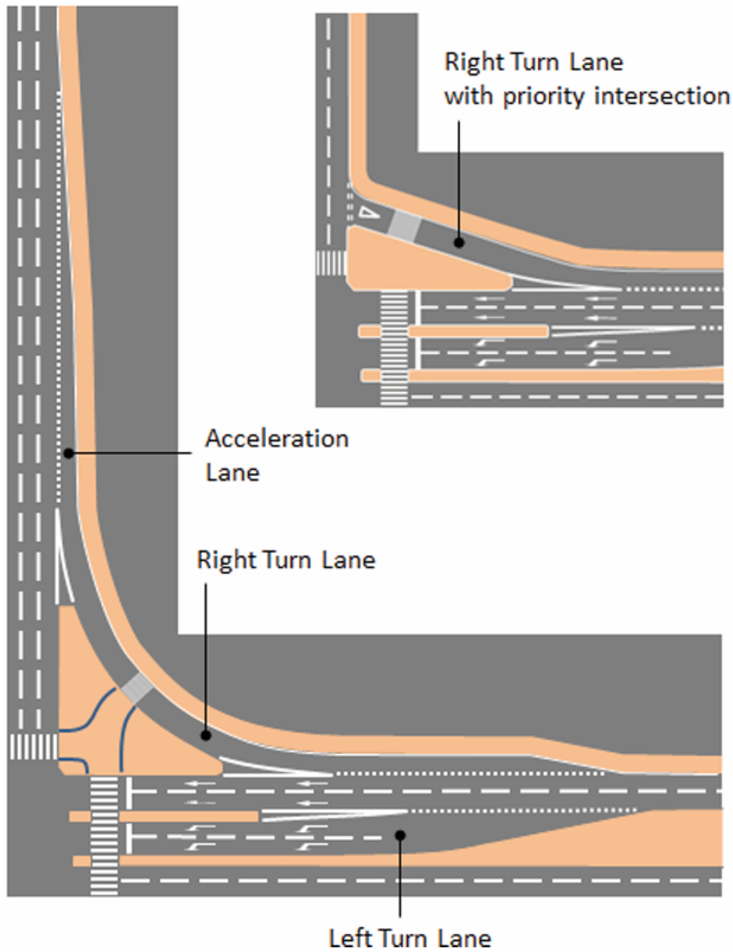


Figure 11.12 High Speed/Major Signalised Intersections

11.3.10.1 OFFSIDE-TURNING MOVEMENTS

It is generally not advisable for offside turning traffic proceeding with a green signal to give way to opposing through traffic. This is particularly the case for traffic speeds exceeding 50km/h and the opposing direction consists of multiple through traffic lanes. Separate signals should be provided for offside turning traffic to proceed when all through traffic on the opposing direction has stopped.

On high-speed roads and where offside turning traffic exceeds 300 veh/hour, one or more offside turn lanes segregated from straight ahead traffic lanes should be provided with a separate set of traffic signals.

11.3.10.2 NEARSIDE-TURNING MOVEMENTS

Segregated nearside turn lanes should be considered if there is a significant volume of nearside turning traffic. A free-flow right-turn lane with an acceleration lane will provide a larger capacity.

The alternative layout without an acceleration lane is more favourable for the provision of pedestrian or slow vehicle crossings. It is also appropriate where nearside-turning traffic joins a road of lower hierarchy.

To facilitate pedestrians crossing a nearside turn lane, a speed table may be considered in conjunction with other traffic calming measures.

11.3.10.3 ADDITIONAL MEASURES

Signalised intersections with increased risk due to high approach speeds on long steep downhill grade may require additional measures including:

i. Warning signs
ii. Narrowing of traffic lanes
iii. High friction surfacing
iv. Transverse rumble strips

Measures based on intelligent transportation system techniques include:

i. Speed related signal changes including extension or delay in signal onset
ii. Countdown timer for traffic and/or pedestrians and slow vehicles
iii. Red light speed cameras
iv. Light-emitting warning signs
v. Vehicle activated signs
vi. Variable message signs

11.3.11 MODES OF TRAFFIC SIGNAL OPERATION AND THEIR USE

Traffic signals operate in either pre-timed or actuated mode or some combination of the two. Pre-timed control consists of a series of intervals that are fixed in duration. Collectively, the preset green, yellow, and red intervals result in a deterministic sequence and fixed cycle length for the intersection. In contrast to pre-timed control, actuated control consists of intervals that are called and extended in response to vehicle detectors. Detection is used to provide information about traffic demand to the controller. The duration of each phase is determined by detector input and corresponding controller parameters. Actuated control can be characterized as fully-actuated or semi-actuated, depending on the number of traffic movements that are detected. **Table 11.6** summarizes the general attributes of each mode of operation to aid in the determination of the most appropriate type of traffic signal control for an intersection. The attributes of the various modes of operation are discussed in additional detail in the following subsections.

Table 11.6 Relationship between Intersection Operation and Control Type

	Type of Operation	**Isolated**	**Coordinated**	**Semi-Actuated**	**Key Benefit**
Pre-timed	**Fixed Cycle Length**	**Conditions Where Applicable**	**Example Application**	**Fully-Actuated**	**Coordinated**
	Yes	Yes	No	No	No
Actuated	Where detection is not available	Where traffic is consistent, closely spaced intersections, and where cross road is consistent	Where defaulting to one movement is desirable, major road is posted <60km/h and cross road carries light traffic demand	Where detection is provided on all approaches, isolated locations where posted speed is >60km/h	Arterial where traffic is heavy and adjacent intersections are nearby
	Work zones	Where traffic is consistent, closely spaced intersections, and where cross street is consistent	Highway operations	Locations without nearby signals; rural, high speed locations; intersection of two arterials	Suburban arterial
	Temporary application keeps signals operational	Where traffic is consistent, closely spaced intersections, and where cross street is consistent	Lower cost for highway maintenance	Responsive to changing traffic patterns, efficient allocation of green time, reduced delay and improved safety	Lower arterial delay, potential reduction in delay for the system, depending on the settings

11.3.11.1 PRE-TIMED CONTROL

Pre-timed control is ideally suited to closely spaced intersections where traffic volumes and patterns are consistent on a daily or day-of-week basis. Such conditions are often found in downtown areas.

They are also better suited to intersections where three or fewer phases are needed. Pre-timed control has several advantages. For example, it can be used to provide efficient coordination with adjacent pre-timed signals, since both the start and end of green are predictable. Also, it does not require detectors, thus making its operation immune to problems associated with detector failure. Finally, it requires a minimum amount of training to set up and maintain. However, pre-timed control cannot compensate for unplanned fluctuations in traffic flows, and it tends to be inefficient at isolated intersections were traffic arrivals are random.

11.3.11.2 SEMI-ACTUATED CONTROL

Semi-actuated control uses detection only for some of the movements at an intersection, typically the minor movements. The phases associated with the major-road through movements are operated as "non-actuated." That is, these phases are not provided detection information. In this type of operation, the controller is programmed to dwell in the non-actuated phases and, thereby, sustain a green indication for the highest flow movements (normally the major road through movement). Minor movement phases are serviced after a call (demand) for their service is received.

Semi-actuated control is most suitable for application at intersections that are part of a coordinated arterial road system. Semi-actuated control may also be suitable for isolated intersections with a low-speed major road and lighter crossroad volume.

Semi-actuated control has several advantages. Its primary advantage is that it can be used effectively in a coordinated signal system. Also, relative to pre-timed control, it reduces the delay incurred by the major-road through movements (i.e., the movements associated with the non-actuated phases) during periods of light traffic. Finally, it does not require detectors for the major-road through movement phases and hence, its operation is not compromised by the failure of these detectors.

The major disadvantage of semi-actuated operation is that continuous demand on the phases associated with one or more minor movements can cause excessive delay to the major road through movements if the maximum green and passage time parameters are not appropriately set. Alternatively, because the major street has no detection and is thus guaranteed a minimum green time regardless of the presence of traffic, there can be unnecessary delay to the minor movement traffic in off-peak hours. Another drawback is that detectors must be used on the minor approaches, thus requiring installation and ongoing maintenance.

11.3.11.3 FULLY-ACTUATED CONTROL

Fully-actuated control refers to intersections for which all phases are actuated and hence, it requires detection for all traffic movements. Fully-actuated control is ideally suited to isolated intersections where the traffic demands and patterns vary widely during the course of the day. The controllers can be programmed in coordinated signal systems to operate in a fully-actuated mode during low-volume periods where the system is operating in a "free" (or non-coordinated) mode. Free operations mode may be an intersection operating with “Recall” or “Fixed Time.” Fully-actuated control can also improve performance at intersections with lower volumes that are located at the boundary of a coordinated system and do not impact progression of the system.

There are several advantages of fully-actuated control. First, it reduces delay relative to pre-timed control by being highly responsive to traffic demand and to changes in traffic pattern. In addition, detection information allows the cycle time to be efficiently allocated on a cycle-by-cycle basis. Finally, it allows phases to be skipped if there is no call for service, thereby allowing the controller to reallocate the unused time to a subsequent phase. The fully-actuated control requires initial and maintenance costs.

11.4 PARKING

Generally, parking can be defined as the act of stopping a vehicle and leaving it in the one location for a period of time, whether or not anyone remains in the vehicle.

Parking forms a necessary component of the transport system. It is needed to allow for the safe storage of vehicles while they are not in use and enables drivers to undertake their intended activity at their destination. It forms an interface between the road network and other land uses.

Parking is provided either:

i. at the kerbside in the form of on-street parking or
ii. in dedicated facilities outside the carriageway(s) of a street, i.e., off-street.

Parking is not a cause but rather an effect, where demand is generated by:

i. land use type and intensity,
ii. spatial distribution and availability of supply and choice.

Off-street parking is the preferred option for the mass storage of vehicles, with on-street parking mainly used in city and town centres to service road users with the need for high levels of access including public transport, commercial vehicles, physically challenged persons and emergency services.

Places where parking is not generally allowed includes but is not restricted to:

i. on, or on the approaches to, a walkway, ramp, intersection or pedestrian crossing

ii. between any parked vehicle and the centre of the road
iii. in front of an access
iv. within 20m of a fire hydrant
v. in a “No Stopping” area
vi. in a “No Parking” area (except briefly to pick up or put down goods or passengers).

    “No Parking” signs should indicate the period and range for which no parking is allowed.

vii. in parking bays for physically challenged persons unless displaying an appropriate parking permit.
viii. On any bridge or other elevated structure on a highway or within a highway tunnel.

11.4.1 PRINCIPLES OF PARKING MANAGEMENT

The type of parking strategy to employ varies based on the overall parking supply and demand needs of a given area. **Table 11.7** presents different factors that determine a user’s choice of parking location and facility.

Table 11.7 Parking facility choices

Decision factor	On-street facilities	Off-street facilities
Location	On-street parking, if available, is dispersed geographically throughout an area and may be closer or further from any single use depending on availability.	Off-street parking is concentrated in a single facility and may or may not be public or dedicated to one use.
Convenience	If parking is widely available, users will likely be able to park close to their destination. In situations where parking is in high demand and street spaces are not readily available, street parking may be perceived as inconvenient.	Dedicated parking attached to a single use may not be open to the general public. Parking in a structure may be perceived as inconvenient.
Visibility and information	Since on-street parking is dispersed, users can easily assess parking options without altering driving path but may move around multiple blocks looking for parking. Time restrictions are not always readily visible while driving.	Users may be unfamiliar with the price, time restrictions or public nature of a structure or lot and, without visible signage, may be reluctant to turn into the lot or structure.
Safety	Areas with good pedestrian lighting and lots of activity have fewer safety concerns associated with on-street parking. Some users, however, may not feel comfortable with parallel parking on busy streets. Others may not feel comfortable parking in areas that feel unsafe or have less desirable uses.	Underground garages and large or poorly lit structures can be perceived as unsafe by users. If so, these facilities may only be used if other parking is unavailable. If a structure is well designed and patrolled, it may be perceived as safer than on-street parking.

Source: Based on City of Denver (2010), page 10.

11.4.1.1 KEY PRINCIPLES

The following presents a set of key parking management 'principles' (City of Denver 2010). This set of parking principles describe the nature of parking universally and provides a base understanding of parking operations (**Figure 11.13**).

Figure 11.13 Key parking management principles

11.4.1.1.1 PRINCIPLE 1: PARKING SUPPLY AND DEMAND

Parking supply refers to the total number of spaces available for use. Parking in a given area is supplied through many types of facilities that are owned, managed and used differently.

Parking is typically categorised into on-street and off-street parking. These two categories differ in several important ways. Off-street parking falls into four categories:

i. Government owned off-street public parking
ii. Government owned off-street private parking
iii. Privately-owned off-street public parking
iv. Privately owned off-street parking that is dedicated to a specific use.

There are critical differences between on-street and off-street parking when viewed from an administrative or management perspective. The supply of on-street parking is relatively fixed and a city's ability to expand that supply is constrained to changes that can be made through street reconfiguration or reconfiguration of line marking. Conversely, off-street parking supply can be expanded more readily through construction of new facilities including at-grade (surface or open air) and, deck, multi-storey or underground facilities.

Parking 'demand' refers to the amount of parking that is needed at a specific time and place. Demand is influenced by:

i. vehicle ownership
ii. the popularity of an area
iii. the nature of the surrounding uses

iv. availability of alternative forms of transportation, and

v. other external factors like fuel costs.

Demand rates typically fluctuate and differ on a daily, weekly, seasonal or even annual basis. The parking characteristics of an area are directly related to the nature of these cycles. For example, demand in an office precinct will peak during the day on weekdays but demand at restaurants and theatres may peak on weekends or evenings.

11.4.1.1.2 PRINCIPLE 2: OCCUPANCY OR UTILISATION

Parking occupancy is one of the central concepts in parking management. Whether in reference to on-street parking or to an off-street car park, parking occupancy describes the percentage of spaces that are occupied at any given time. Parking occupancy rates, also called ‘utilisation’, reflect the relationship between parking supply and demand. A low occupancy rate in an area means that there are many spaces that are empty or unused. While this may be convenient for drivers traveling to that destination, lower occupancy rates can also mean that oversupplies of parking or inappropriate parking prices exist in the area. By contrast, an area, or precinct that has a very high level of occupancy could mean that the available parking supply is limited and needs management to accommodate a certain level of demand.

Ideally, the occupancy of parking facilities should be high enough to ensure that they are occupied at a level that justifies the supply but not so high that it is unreasonably difficult to find a space. Eighty-five per cent occupancy at times of peak demand means that approximately one parking space in every seven should be vacant. When peak parking occupancy (the average of the four highest hours of demand in a day) is regularly above 85%, a change to the parking management approach may be necessary. This 85% benchmark is a recognised best practice approach to the management of on-street parking. It means that the parking resource is well used but people can still easily find a space, thus reducing customer frustration and congestion. Generally, parking is considered ‘at capacity’ when available spaces are 85% occupied at times of peak demand (Shoup 2005).

11.4.1.1.3 PRINCIPLE 3: DURATION AND TURNOVER

Parking duration refers to the length of time a vehicle occupies a space. Parking turnover (or churn) describes how frequently a parking space becomes available or ‘turns over’ during an hour. The rate at which spaces become available is important since it describes the number of opportunities different users will have to occupy a space. For example, a vehicle belonging to a shop employee could either occupy a parking space in front of that shop for a full eight hours (providing access for one person) or it could turn over every 60 minutes and provide convenient access for eight different customers.

Ideally, both on-street and off-street parking should be managed so that they can accommodate a range of different stay durations based on the needs of the surrounding land uses. A popular retail or commercial area, e.g., requires conveniently located parking spaces that are regulated for ‘short term use’ – anywhere between 30 minutes and two hours. Parking around entertainment or restaurant districts may require parking durations that are longer than two hours.

11.4.1.1.4 PRINCIPLE 4: ENFORCEMENT

Compliance with parking regulations is an important component of the parking system. Parking rules and restrictions are put in place to support parking goals such as turnover or access. The success of parking management strategies are often tied to the level of compliance. Parking infringements or fines are issued to encourage compliance with rules and to maintain the intent of the parking management philosophies in place within a given area. While enforcement is often necessary to ensure that rules and restrictions are observed, there are significant resource implications associated from both a labour and equipment standpoint. A clear definition of existing resources and implications are an important consideration when selecting a management tool or designing a parking management program for an area. If parking management strategies and complementing enforcement are designed well, it increases the likelihood that the desired goals of an area are achieved (e.g. increased turnover, access, utilisation). Under these conditions, parking can truly function as an asset and meet the diverse needs of various stakeholders so that it is easier for those user groups to access the area.

11.4.1.1.5 PRINCIPLE 5: PARTNERSHIP

A city’s strategic planning documents set a clear vision for the future. However, the management of day-to-day parking operations within a diverse land use and transportation system is a more complicated endeavour. To achieve success, partnerships with external stakeholders are an imperative component of any parking management program. Partnerships with those who are impacted most by parking policies can help ensure that strategies are reasonable and are tailored to achieve specific desired outcomes. Extensive stakeholder input and buy-in is needed to effectively understand the implications or potential effects of new policies. Input is necessary from a broad cross-section of stakeholders including business associations, property and business owners, residential organisations, managers of private car parks and other interested individual citizens or groups.

11.4.1.1.6 EDUCATION AND INFORMATION

The long-term impacts of the increasing demand for the limited supply of parking creates cost implications and also social, environmental and economic impacts that confirm that current parking trends are unsustainable. Demand satisfaction is unsustainable and there are many more strategic benefits to charging for parking than simply raising revenue.

Extensive and continuous education of the city’s stakeholders on the benefits of alternative transport other than private motor vehicles will have long-term positive benefits for a city and the health of people. Parking demand and supply issues and options should be regularly published by the Municipal/Metropolitan Assembly in its website, brochures, advertisements and other publications. The following rationale should be reinforced in all education and responses to parking issues:

i. Drivers cannot expect long-term free parking close to their destination.
ii. There are environmental aesthetic and financial costs associated with unlimited supply of parking.
iii. Improved compliance has benefits for all stakeholders.
iv. The user pay principle is fair and applies to most services and products as well as to every other cost associated with owning and using a motor vehicle.
v. Better parking control and management will benefit all stakeholders.

Future parking information should therefore include not only the underlying rationale but also parking user information about the different options available, simpler and easy-to-understand signage, easy-to-use equipment and incentives for parkers to investigate and use off-street parking in preference to on-street parking.

11.4.1.2 METHODS OF DETERMINING SUPPLY

There are two methods most commonly used to determine the appropriate supply of parking for a particular development or area demand. These methods are:

i. determining supply based on generic parking provision standards or
ii. relating supply to an actual estimate of demand based on field observations and/or other survey data.

The setting of a parking supply ceiling based on what is considered to constitute the appropriate sustainable level of provision for an area is a variation on the first method.

The estimation of parking demand using either parking standards or survey results, and the subsequent provision of parking capacity to address that demand, may not necessarily always be appropriate to the complex task of engineering efficient, safe, and effective land use transport systems. Practitioners should seek to establish what constitutes an appropriate supply of parking for an area and should then seek to balance the demands for that parking through appropriate management mechanisms.

11.4.1.2.1 MINIMUM CAR PARKING PROVISION STANDARD

Table 11.8 provides minimum off-street car parking standard for various land use in Ghana.

Table 11.8 Minimum Car Parking Requirements

Land use	Minimum Parking Requirements
Residential A, B, & C Zone	1 per dwelling(minimum)
Residential D	1 per 2 dwelling units
Residential E	2 per 5 accommodation units
Commercial offices	1 per $50m^2$ gross floor area
Commercial Government offices (CBD)	1 per $200m^2$ gross floor area
Shops	1 per $40m^2$ retail space
Hotels outside CBD	1 per room of accommodation
Service Industry	1 per $100m^2$ floor area
General Industry	1 lorry space per $50m^2$
Public buildings & places of Assembly	1 per $40m^2$ or 1 per 20 seats, or beds whichever is greater
Recreation facilities	1 per $10m^2$ floor area
Educational establishment	1 per 4 members of staff
Restaurants and bars	1 per $20m^2$ floor area
Any other development not listed	As specified by planning Authority

Residential A - is intended to remain as an area for low density residential development, with a housing density of 10-15 dwellings per hectare.

Residential B - is intended to provide for a variety of residential uses at net site density of 16-30 dwellings per hectare.

Residential C - is intended for intensive residential development with housing density >30 dwellings per hectare

Residential D - is intended for intensive multi-storey residential accommodation in the form of flats or apartments.

Residential E - is intended to be used for a community dwelling such as barracks, residential hostels, institutional housing for the physically challenged, boarding and guest houses containing more than 6 rooms.

Source: Zoning guidelines, Town and Country Planning Department (2011)

11.4.1.3 PAY PARKING

11.4.1.3.1 GENERAL PRINCIPLES

As pay parking generally results in reductions in car use and traffic congestion among other environmental benefits, it is one of the essential transport measures necessary to ensure the long-term viability of commercial centres.

Pay parking increases equity by charging users (user pay) for their parking costs and by reducing the parking costs imposed on non-drivers. Paying directly rather than indirectly benefits consumers because it reduces parking and traffic problems and allows individuals to decide how much parking to purchase giving them an opportunity to save money. Drivers may use a space as long as they want, as long as they are prepared to pay for it.

11.4.1.3.2 PAY PARKING OBJECTIVES

Objectives for pay parking should be considered in order to determine how fees will be structured:

i. For traffic management – peak period fees should be high enough to encourage a shift in travel modes or times.
ii. For parking management – fees during peak demand periods and at the most convenient locations should be high enough to generate a maximum 85% occupancy rate. If prices are too low, parking becomes saturated causing motorists to cruise around in search of a space. The target is to ensure that at times of peak demand, 15% of spaces (one in seven) are available.

11.4.1.3.3 ON-STREET PARKING MANAGEMENT

On-street parking plays an important role in the effective functioning of commercial centres and access to residential areas. Many businesses rely on on-street parking to provide access for their customers and meet their loading requirements. On-street parking also caters for specific uses such as dedicated space for taxis and mobility parking for people with impaired mobility.

On-street parking management broadly consists of the following:

i. Unrestricted: where there are no limitations on parking
ii. Time restricted: with a range of time limitations and enforcement used to ensure compliance
iii. Reserved parking: reserved for a certain type of user, such as mobility card holders, or taxis, or for loading zones
iv. Priced parking: with varying rates applying sometimes alongside a time restriction.

11.4.2 PARKING AND THE ENVIRONMENT

When designing parking systems, consideration should be given to the aesthetics of the urban environment, the safety and convenience of users and minimisation of the environmental impacts of parking.

11.4.2.1 URBAN DESIGN CONSIDERATIONS

Large expanses of parked vehicles or paved areas can be quite unattractive and can detract from the character of a street, an area or a development. In order to avoid this situation, surface parking areas should be detached into smaller groups of bays separated by landscaping or other uses or activities especially where parking areas front the street. For this same reason private off-street parking should generally be located at the rear of buildings or in undercroft facilities.

Innovative approaches to parking design should be adopted in order to maintain amenity and encourage the use of parking areas for community activities in addition to parking such as weekend markets, fairs, sporting activities and other entertainment activities.

All parking areas should be paved with consideration given to the appropriate use of surface materials, textures and colours. Paving blocks, coloured concrete and other such material may enhance the look of paved areas. Concrete pebbled sections combined with grass can provide a firm base that will blend in with surrounding grassed areas.

Carefully planned landscaping can be used to soften the appearance of car parks and improve the feel of the immediate environment. The use of architectural features such as solid or intermittent brick walls, earth mounds and fences can also help to blend off-street car parks in with the surrounding environment and promote a more user-friendly atmosphere.

The application of good urban design principles can be very effective in shielding deck and multi-storey parking structures to reduce visual impact and in sensitively integrating parking structures into the urban fabric of the street. The design of parking structures should complement the surrounding built form in terms of scale height and character. Parking structures should also be co-located with other building uses such as retail and office space in order to maintain pedestrian interest and activity at street level. Underground parking facilities where land values are high enough to justify the cost, present opportunities for the construction of buildings or parks above the parking levels.

The following general design principles should be promoted where appropriate:

i. encourage mixed-use of parking areas
ii. require landscaping as a component of any car park design
iii. screen unsightly areas from the public and blend in with the urban fabric of the street
iv. provide space and facilities to accommodate seating, market stalls, outdoor eating or street entertainment as appropriate
v. create continuous active frontages and avoid blank walls, large gaps and expanses of unsightly asphalt
vi. locate access driveways on minor roads away from intersections and with due consideration of the form and nature of other activity in the street: at the same time ensure that as far as possible, access to parking is in a location which is identifiable and attractive to use
vii. use consistent construction materials throughout
viii. prohibit on-site car parking between the walkways and the fronts of buildings

11.4.2.2 ENVIRONMENTAL IMPACTS OF PARKING

Excessive paving increases water pollution and stormwater flooding and can result in a loss of greenspace.

Cars driving around looking for a place to park add to the amount of air pollution. Exhaust fumes trapped within parking structures intensify the problem. Exhaust fans or open structures reduce the levels inside the structures by spreading the pollutants to the outside. However, this still adds to pollution levels.

Car parking clearly does have environmental impacts and appropriate consideration should be given to ways to ameliorate these impacts in the design of any facility. Some of the design features of an environmentally sensitive parking design are as follows:

i. Reduce the amount of land paved for parking. This can be done through strategies such as cluster development.
ii. Use on-site stormwater storage and percolation with the use of natural wetland for filtering.
iii. Maximise greenspace through strategies such as preserving trees, streams and shorelines.
iv. Use paving materials that allow water to percolate and grass to grow.

An example of environmentally sensitive paving materials is shown in **Plate 11.1**.

Plate 11.1 Example of environmentally sensitive paving material design

11.4.3 ON-STREET PARKING

A roadway network should be designed and developed to provide for the efficient movement of vehicles operating on the system. Although the movement of vehicles is the primary function of a roadway network, segments of the network may, as a result of land use, also provide on-street parking.

In the design of expressways/motorways and access-controlled facilities, as well as on most arterials, collectors, and local streets in rural areas, stopping or parking should be permitted only in emergencies.

On-street parking generally decreases through-traffic capacity, impedes traffic flow, and increases crash potential. Where the primary service of an arterial is the movement of vehicles, it may be desirable to prohibit parking on arterial roads in urban areas and arterial highway sections in rural areas. However, within urban areas and in rural communities located on arterial highway routes, on-street parking should be considered in order to accommodate existing and developing land uses. Often, adequate off-street parking facilities are not available. Therefore, the designer should consider on-street parking so that the proposed road or highway improvement will be compatible with the land use. Wherever on-street parking is provided, accessible on-street parking must be included.

In a road with high place function, parking can make a positive contribution by providing a buffer between moving vehicles and pedestrians, and by narrowing the available carriageway to reduce traffic speeds. Parking can also be located as a buffer between traffic lanes and a cycle path. However, the benefits of such parking should be considered in the context of the overall streetscape design, including pedestrian access, street planting, furniture and visual character. Space for opening of car doors, for vehicle overhang of kerb, for poles and other street furniture, is required in the Street Furniture Zone.

On-street parking for cars generally comprises the following:

i. parallel kerbside parking
ii. angle kerbside parking
iii. centre-of-road parking.

Facilities are also provided for trucks (e.g., loading zones), motorcycles, buses, taxis, bicycles and other special users.

Consideration needs to be given to the following factors when locating on-street parking:

- provision of adequate end clearances to intersections and driveways
  - regulatory ‘no-stopping’ distance at an intersection
  - preservation of adequate intersection sight distances

  - prohibition of parking to accommodate queues on the approach to intersections
  - provision of right turn lanes at intersections
  - pedestrian crossings, bus stops, railway level crossings, fire hydrants, road bridges
- preservation of safe and convenient pedestrian access
  - wheel stops to prevent angle parked vehicles intruding on narrow walkways (less than 2 m wide)
- protection of through traffic
  - kerbside parking around a right-hand curve with limited sight distance across the curve
  - parking just beyond a crest
  - a parking area which starts just beyond a roadway narrowing or lane reduction
  - parking on the left-hand side of a one-way roadway
  - any other location where a parking zone protrudes an unexpectedly large distance into a roadway, or where parking manoeuvres may encroach into a high-speed traffic lane
- unsafe parking locations
  - on the inside of sharp curves
  - within a T-junction
  - on islands and reservations including the central island of a roundabout
- provision of disabled parking and access to destination, e.g., kerb ramps.

11.4.3.1 PARALLEL PARKING

Parallel kerbside parking in the direction of traffic flow is the most common form of on-street parking. It has the following advantages:

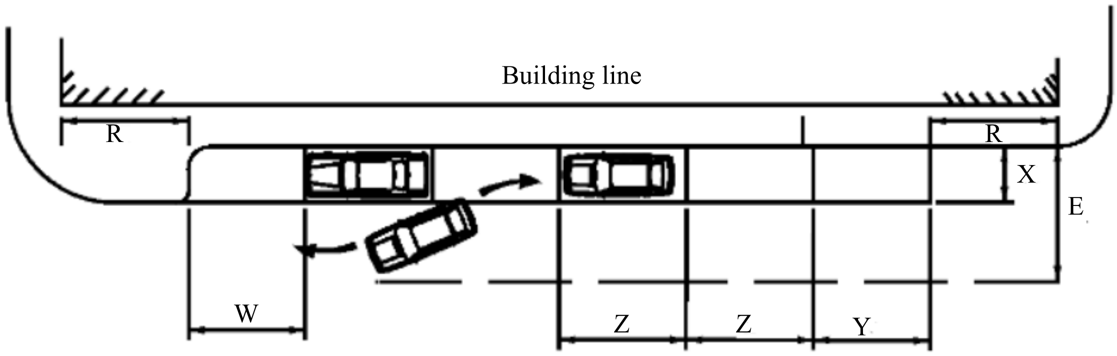
i. Road crashes associated with parking manoeuvres are minimised compared to angle parking.
ii. It requires less lane width than angle parking.

Entering and leaving parking spaces from a through traffic lane introduces slow-moving and reversing movements that may conflict with the traffic flow. These movements would need to be able to be identified by other drivers so they can take appropriate actions. Hence, parallel parking is best suited to roads with lower speed limits. Where the speed limit is 60 km/h or less, there should desirably be 0.5 m clearance from the nearest moving traffic lane. This clearance should be increased by 1.0 m for each 10 km/h by which traffic speeds exceed 60 km/h, up to a maximum of 3.0 m.

In areas of high demand and turnover, parking bays should be marked in accordance with the MRH's Standard Details, Road Signs and Markings for Urban and Trunk Road (1991) and GHA Road Signs – Final Draft (2007). Parking bays should be designed to provide sufficient space for reversing manoeuvres.

Plate 11.2 is an example of on-street parallel parking. **Figure 11.14** shows a layout and dimensions for parallel parking spaces.

Plate 11.2 An example of on-street parallel parking in Kumasi with faded marking



Space Usage	X (m)	Y (m)	Z (m)	W (m)
Cars and Light commercial vehicle (LCV) Normal conditions	2.3	5.4 min.	6.0 to 6.7	6.3 or length Z whichever is greater
Cars and LCV Restricted roadway	2.1			
Trucks and Buses	2.6			

R = The regulatory or approved no-stopping distance
For dimension E refer to **Figure 11.16**
Nose out parking bays slope in opposite direction

Figure 11.14 Layouts for parallel parking spaces

11.4.3.2 ANGLE PARKING

Angle parking involves parking at angles other than 0 degrees (parallel parking). It has the following advantages:

i. It can accommodate up to twice as many vehicles per unit length of kerb as parallel parking. This difference is a function of the angle used where low angles of 30 degrees and less give little advantage and the maximum advantage occurs where 90 degrees is used.
ii. The parking manoeuvre is easier for angle parking than parallel parking especially for small angles.

Disadvantages of angle parking include:

i. More roadway width is required for angle parking bays and associated parking manoeuvres. This requirement may present a problem for commercial vehicle parking as the increased length of those vehicles may encroach into traffic lanes. Hence adequate parallel parking space should be provided for those vehicles.
ii. All angle parking presents a greater hazard to road users than parallel parking. This situation is mainly due to the fact that parking at an angle always requires reversing which causes bottleneck effects in the moving traffic and may lead to collisions

directly involving the reversing vehicle.

iii. There can be sight/visibility issues and increased conflict with pedestrians crossing midblock.

The decision whether to use angle parking should be based on consideration of:

i. width of road
ii. traffic volume
iii. type of traffic
iv. traffic speed characteristics
v. vehicle dimensions
vi. expected turnover
vii. land use served
viii. functional road classification.

It is recommended that angle parking be avoided on higher speed (> 50 km/h) roads. However, if angle parking is provided for, it should be used in conjunction with other protective measures such as indented (embayed) parking and manoeuvring space to lessen its negative effects.

Long vehicles are usually unable to make use of angle parking spaces. In commercial areas, for example, adequate parallel loading spaces should also be provided to cater for long vehicles and commercial vehicles.

Wheel stops may be required to control encroachment onto pedestrian paths by excess kerb overhang, where the walkway is 2 m or less in width.

Figure 11.16 indicates the minimum widths between the separation line or median, and the kerb, for parking angles 30^0, 45^0, 60^0 and 90^0 respectively that should be available before parking is permitted.

Angle parking may be either:

i. Advance (‘front-in’ or advance-reverse) or
ii. Reverse (‘reverse-in’ or reverse-advance).

Any town or city applying angle parking should be consistent in adopting one form or the other.

Figure 11.15 and **Table 11.10** shows the relevant dimensions required when setting out angle parking. **Plate 11.3** is an example of on-street angle parking.

Plate 11.3 An example of on-street angle parking (Advance parking) in Kumasi

11.4.3.2.1 ADVANCE VERSUS REVERSE ANGLE PARKING

Reversing out of advance angle parking bays involves some of the vehicle protruding into the adjacent road space before the driver can see oncoming vehicles. This affects safety and also interferes with traffic movement. Unless there is a manoeuvre space between the parking bays and the traffic lanes, the obstructed sight lines when reversing out are particularly dangerous for the less visible and more vulnerable bicycle traffic, making reverse angle parking preferable on designated bicycle routes or in areas with significant bicycle use.

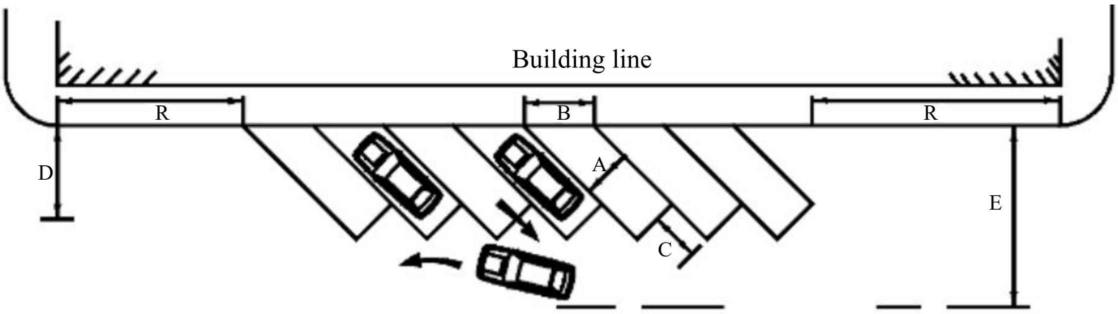
Reversing into reverse angle parking bays may reduce many of the problems associated with advance parking. However, it creates a traffic hazard as the vehicle stops in the moving traffic stream prior to reversing into a parking bay and the nose swings into the adjacent through traffic lane at the start of the back-in manoeuvre. Reverse angle parking may also create excessive walkway obstruction from the rear overhang and will produce exhaust fumes on the walkway.

Relative merits of advance versus reverse parking are presented in **Table 11.9.**

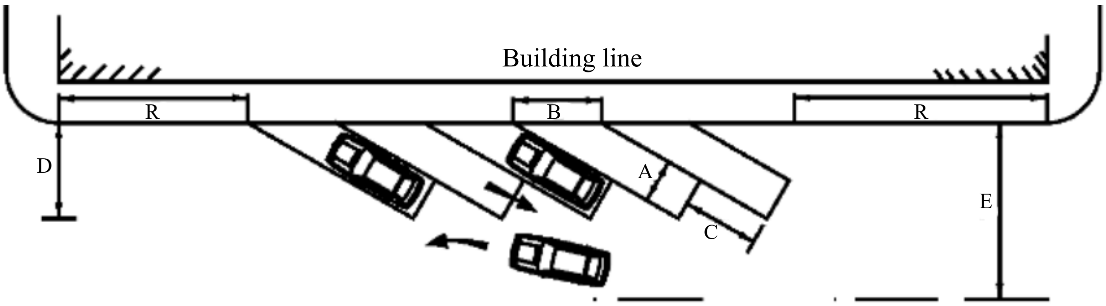
Table 11.9 Relative merits of advance versus reverse parking

Issue	Advance parking situation	Reverse parking situation	Preferred option
Exhaust emissions	Exhaust facing away from walkway.	Vehicle's exhaust directed onto pedestrian walkway (causing discomfort and staining of footway paving) and into open doors of shops in retail precincts.	Advance parking
Loading/unloading vehicles	Boot/rear hatch faces away from the footpath exposing the motorist/shopper to moving traffic.	Boot/rear hatch faces towards the footpath allowing for safer loading/unloading.	Reverse parking
Judgement in a reversing manoeuvre	Reversing occurs into a space relatively free of fixed obstructions provided the motorist is able to observe approaching traffic or the approaching traffic poses no significant hazard).	Reversing occurs into a limited and obstructed space.	Advance parking
Motorist confusion	Vacant spaces are clearly visible, and a motorist is able to slow down and move directly into a parking space in a single movement causing little confusion or delay to the following motorists.	It is more difficult to observe vacant spaces and a motorist needs to actually pass the parking space in order to reverse into it potentially confusing a following motorist who may also wish to park in the same space.	Advance parking
Disruption to passing traffic when reversing	Motorist reversing out from the parking bay can select a time when passing traffic will not be disrupted.	Stationary motorist about to reverse into the parking bay tends to disrupt passing traffic by trapping a vehicle behind.	Advance parking
Traffic and cyclist safety	Motorist leaving an advance parking space must reverse approximately 1 m or more before gaining a clear view of approaching traffic and cyclists. This is aggravated by increasing numbers of large 4WDs and vans.	Motorist about to drive forward from a reverse parking space has a relatively good view of approaching traffic and cyclists without moving forward significantly.	Reverse parking

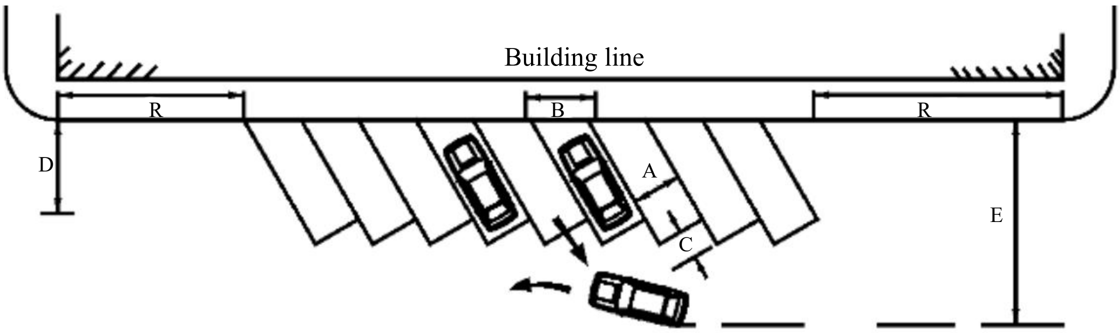
Issue	Advance parking situation	Reverse parking situation	Preferred option
Impact with kerb obstructions	Motorist can more easily view high kerbs and footpath obstructions whilst moving in the normal forward motion into the parking space.	Motorist reversing into the parking space cannot easily view the obstructions and the rear overhang is generally greater than the front overhang which results in greater footpath intrusion.	Advance parking
Pedestrian safety	Motorist reverses into a vehicle-based environment.	Motorist reverses into a pedestrian environment. Vehicle projections e.g., tow bars, bicycle racks etc. may also pose an additional hazard for pedestrians.	Advance parking



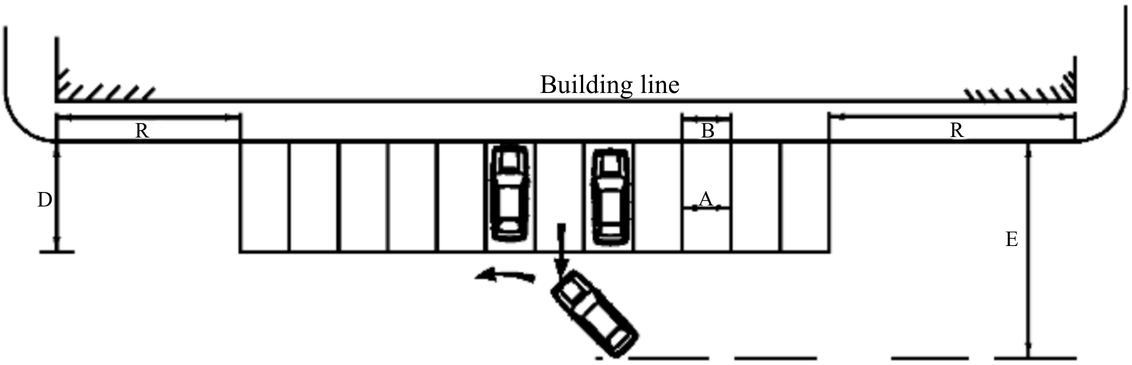
A. 30^0 Angle Parking



B. 45^0 Angle Parking



C. 60^0 Angle Parking



D. 90^0 Angle Parking

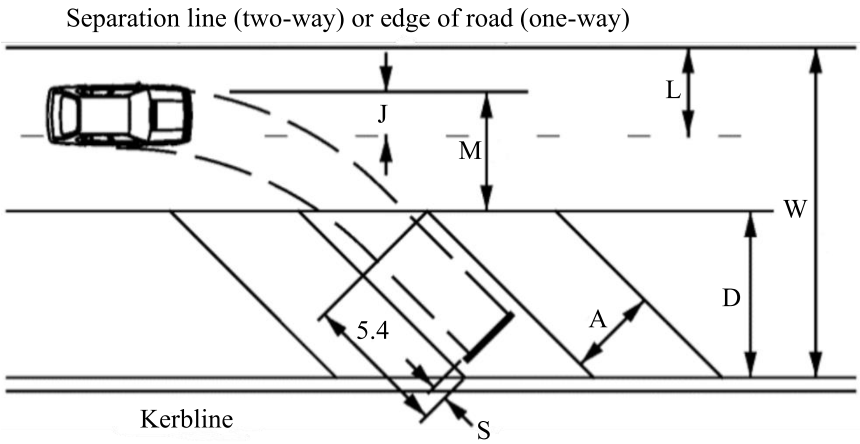
Figure 11.15 Layouts for angle parking spaces

Table 11.10 Minimum dimensions for angle parking spaces

Angle (^{0})	Use Category	A (m)	B (m)	C (m)	D_1 (m)	D_2 (m)	D_3 (m)
30	Low	2.1	4.2	3.6	4.4	4.1	4.5
	Medium	2.3	4.6	4	4.4	4.1	4.7
	High	2.5	5	4.3	4.4	4.1	4.9
	Disabled	3.2	6.4	5.5	4.4	4.1	5.4
45	Low	2.4	3.4	2.4	5.2	4.8	5.5
	Medium	2.5	3.5	2.5	5.2	4.8	5.6
	High	2.6	3.7	2.6	5.2	4.8	5.7
	Disabled	3.2	4.5	32	5.2	4.8	6.1
60	Low	2.4	2.8	1.4	5.7	5.1	5.9
	Medium	2.5	2.9	1.4	5.7	5.1	6.0
	High	2.6	3	1.5	5.7	5.1	6.0
	Disabled	3.2	3.7	1.8	5.7	5.1	6.3
90	Low	2.4	2.4	-	5.4	4.8	5.4
	Medium	2.5	2.5	-	5.4	4.8	5.4
	High	2.6	2.6	-	5.4	4.8	5.4
	Disabled	3.2	3.2	-	5.4	4.8	5.4

Notes:

R = The regulatory or approved no-stopping distance
D_1 = Where parking is to a wall or high kerb not allowing any overhang
D_2 = Where parking is to a low kerb which allows 600mm overhang
D_3 = Where parking is controlled by wheel stops installed at right angles to the direction of parking
LCV = Light commercial vehicle
For dimension E refer to **Figure 11.16**
Nose out parking bays slope in opposite direction



Parking Criteria						One Lane		Two Lanes	
Angle (^{0})	**A (m)**	**D* (m)**	**M (m)**	**D+M (m)**	**D+M-J (m)**	**L (m)**	**W (m)**	**L (m)**	**W (m)**
0	2.1-3.2	2.3	3.0	5.3	2.8	3.5	6.3	7.0	9.8
30	2.1-3.2	4.1-5.4	2.9-3.1	7.0-8.5	5.1-5.3	3.5	8.6-8.8	6.5	12.1-12.3
45	2.4-3.2	4.8-6.1	3.5-3.9	8.3-10.0	6.7-6.9	3.5	10.2-10.4	6.5	13.7-13.9
60	2.4-3.2	5.1-6.3	4.3-4.9	9.4-11.2	7.8-8.3	3.5	11.3-11.8	6.5	14.8-15.3
90	2.4-3.2	4.8-5.4	5.4-6.2	10.2-11.6	8.3-9.1	3.5	11.8-12.6	6.5	15.3-16.1
One way traffic volume (veh/h) in lanes adjacent to parking						0 - 800		800 - 1600	
Note *The smaller values of D provide for kerb overhang									

Where,

A	=	Space width
D	=	Effective space encroachment
M	=	Manoeuvre space
J	=	Allowable encroachment into adjacent lanes: Assume 2.5m generally, reduced encroachment may be considered where traffic speeds are greater than 60km/h
L	=	Width for traffic lanes
W	=	Total width required from kerbline to separation line or median
S	=	Wheel stop distance: 0.6m nose in, 0.9m nose out
E	=	D+M

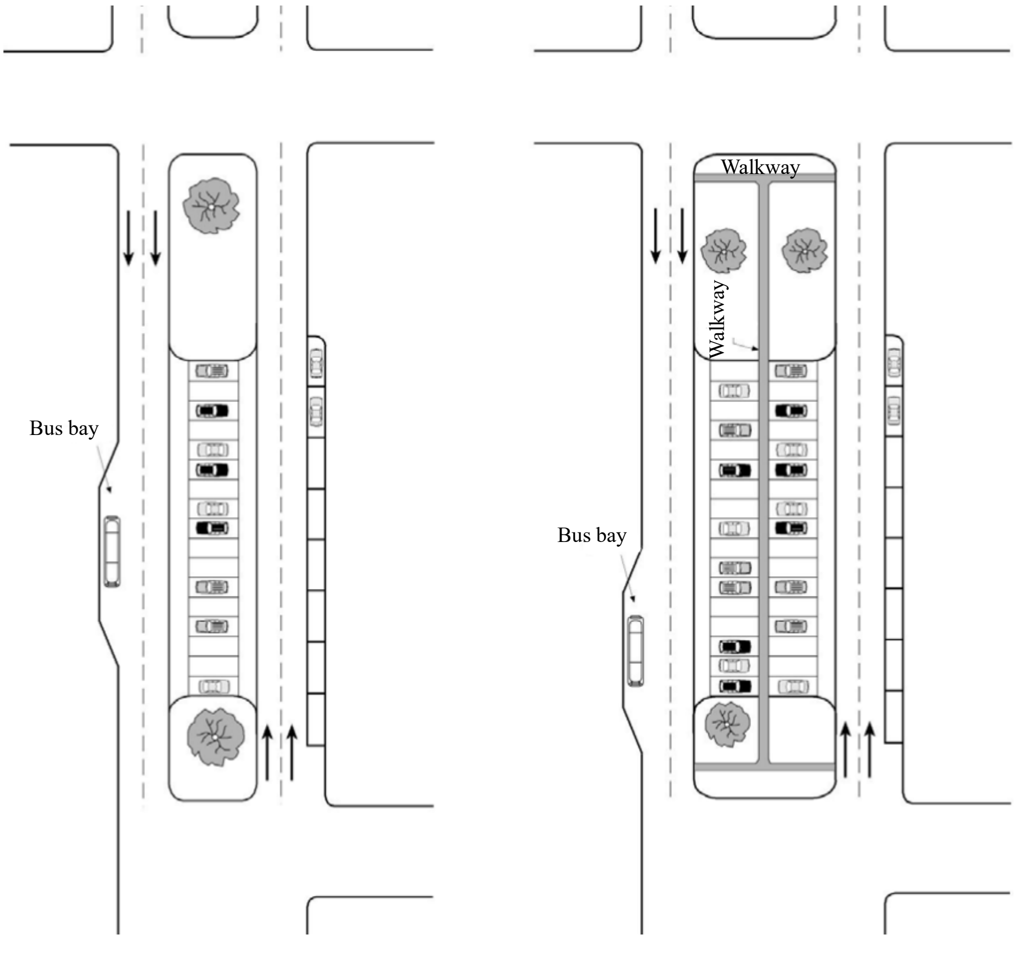
Figure 11.16 Minimum width for on-street parking (Adapted from Austroads)

11.4.3.3 CENTRE-OF-ROAD PARKING

Centre-of-road parking should only be considered on lower speed non-arterial roads (speed limit ≤ 50 km/h) with little through traffic unless the parking area can be well-separated from the through-traffic flow and access points can be well-designed and kept to a minimum number of locations. The provision of centre-of-road parking separates opposing traffic flows and provides a continuous refuge for pedestrians, but it does generate additional pedestrian movements across the road. Centre-of-road parking is often arranged as angle parking in two rows separated by a walkway/median. In some situations, centre-of-road parking must be accessed via designated entrance and exit points. Where this requirement does not apply and the characteristics of the road are suitable e.g., a low volume or speed environment, centre-of-road parking can be designed to be accessed directly from the roadway providing that sufficient manoeuvring clearance is provided between the parking spaces and the traffic lanes (see **Figure 11.17**).

The combination of kerbside parking and centre-of-road parking provides a large number of parking bays per unit length of road. It is rarely possible to combine angle kerbside parking with centre-of-road parking because of the large roadway width required.

If time limits are introduced, a combination of centre-of-road parking with kerbside parking provides an opportunity for different time limits to be used. Short-term parking can be provided at the kerbside while longer-term parking can be assigned to the centre-of-road parking.



(a) Single row parking **(b) Double row parking**

Figure 11.17 Typical centre-of-road angled parking layouts

Table 11.11 gives a guide to the minimum roadway width, related to traffic volume, which should be available before centre-of-road parking is permitted. For traffic volumes greater than those shown in **Table 11.11**, there are no general criteria that can be applied, so a traffic engineering assessment should be made of the conditions in every instance. Centre of road parking is typically provided such that vehicles drive into the bay, then proceed in a forward direction as part of the unparking manoeuvre.

Where overall roadway widths are sufficient to allow centre-of-road parking within a wide median, a parking area isolated from through traffic, is to be preferred. Such a facility can be designed in a similar way to an off-street car park.

Table 11.11 Minimum roadway width for centre of road parking

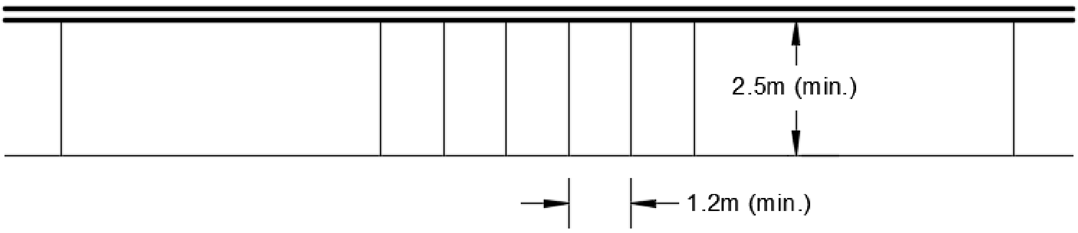
One-way flow, vehicles per hour	Minimum roadway width (m)
Up to 400	23
401–800	29

Source: Austroads

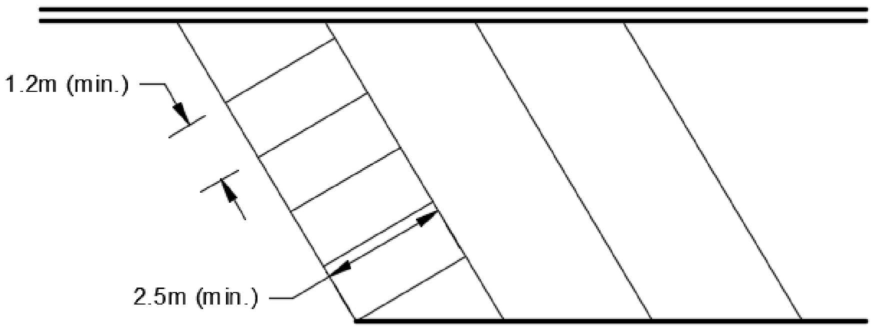
11.4.3.4 PARKING FOR MOTORCYCLES

Motorcycle parking zones are normally provided in groups according to demand. Conversion of parking spaces as illustrated in **Figure 11.18** can provide the required facilities. Use of irregular spaces and undersize remnants should also be considered.

Where cars are found to occupy motorcycle spaces, installation of kerbing may be required. The minimum size of a motorcycle space is 2.5 m x 1.2 m.



A. Parallel parking zone



B. Angle parking zone
(Only car spaces at the ends can be converted, and then only if roadway is not too steep)

Figure 11.18 Conversion of a car parking space to motorcycle spaces

11.4.3.5 PARKING FOR PHYSICALLY CHALLENGED PERSONS

In any parking zone, it is desirable to set aside a number of parking spaces for physically challenged persons. Such spaces should be in angle parking zones, as adequate provision for persons with physical challenges at kerbside parallel parking spaces, particularly the provision of wheelchair access, can be difficult. Where available parking is largely parallel, it is usually more practicable to provide special side street or off-street parking areas, which include disabled parking spaces.

Clear signposting to these areas from the main street must be provided.

11.4.4 OFF-STREET PARKING

Streets are multi-modal public places that are central to accessibility. To this end wherever possible, off-street parking should be provided for the majority of parked vehicles and on-street parking should be used to mainly support those road users with needs for high levels of access, such as public transport couriers and service vehicles, physically challenged persons and emergency services.

Off-street parking may be provided within the road reservation or in private or public areas outside the road reservation.

11.4.4.1 TYPES OF OFF-STREET PARKING

A. Private Lots

Private lots are off-street parking spaces where you can park if you are a member or associated with the attached real estate or business in some way. While you can pay a required fee to access private lots in some places, it is not always an option; it depends on the location and parking purposes. These private areas are under the control of a private agency or individual and are often seen in places like colleges and residential areas.

B. Public Parking Lots

Public parking lots are any publicly owned, operated or maintained parking lot or parking facility. Though many are locally governed, some are privately owned and used only for their customers. These are very common near bars, shops, and restaurants.

C. Parking Garages

Like private lots, these tend to come with a fee for parking during a particular period of time. When talking about parking garages, there are several different types:

- Single-level garages
- Multi-level garages
- Underground garages

D. Automated parking garages

This involves driving your vehicle on the parking system, and the system automatically moving your car to an available space.

11.4.4.2 DESIGN OF OFF-STREET PARKING WITHIN THE ROAD RESERVATION

Off-street parking within major road reservations usually involves parking within a:

i. service road (technically on-street parking but off-street parking standards may be appropriate)

i. railway commuter parking area adjacent to the road

ii. small area with formal spaces

iii. rest area, service centre or truck parking area (usually rural roads)

iv. park and ride facility.

Park and ride facilities and service centres may be developed as part of new urban expressway/motorway projects on land within the road reservation or acquired as part of the project.

Access may be via ramps on the main carriageways or an interchange and in either case will be an integral part of the road design.

Under no circumstances should parking areas be provided within traffic islands or the central island of roundabouts. Guidance on the planning and design principles for rest areas, service centres and truck parking bays and their associated parking layouts is provided in **Section 11.5**.

Safety of the major road traffic is a major consideration and the access from and to the main road must be designed appropriately. Safety of the operation within the facility is also a prime concern and the layout and sizing of the elements must reflect this.

The design of off-street parking within the road reservation should ensure that:

i. appropriate access is provided from and to the road
ii. adequate length of entrance is provided between parking bays and the road to prevent queuing of entering vehicles back onto the road
iii. there are adequate numbers of spaces for the range of vehicles expected to use the facility
iv. where necessary, different types of vehicles (e.g. truck, car and car plus caravan) are provided with appropriate spaces and are separated
v. conflicts between vehicle and pedestrian movements within the car park are eliminated or managed
vi. pedestrian access facilities are well located and designed
vii. the design of the car park does not compromise the provision of appropriate geometric standards for the carriageway/s of the frontage road (i.e. major access roads)
viii. the drainage design prevents any flow of parking area run-off onto the road
ix. traffic movements within the parking area do not cause confusion to motorists using the road.

11.4.4.3 OFF-STREET PARKING PRINCIPLES

Parking principles that are considered in the design of off-street parking areas are as follows:

i. Vehicular traffic flows should be in a continuous, one-way direction. Avoid two-way traffic and the use of dead-end parking lot arrangements.
ii. Whenever possible, the car parking area should be designed as a separate area from the parking area used for trucks, buses, car-trailers and recreational vehicles.
iii. The geometric design of the parking areas should avoid small acute angle kerbed areas and excessive small protrusions or islands into the parking area
iv. The design of the parking areas must be functional, simple, and attractive and provide safe vehicular movements which will not test the driver's ability.

v. The gradient of the parking areas should be relatively level but still sloped or crowned to permit surface drainage flows directed toward inlet structures.
vi. Parking stalls must be indicated with pavement markings to delineate the designated parking pattern and other pavement markings such as directional arrows, crosswalk indicators, etc. should be considered.
vii. If parking stalls are too narrow, the drivers will often ignore stripe demarcation lines and overlap into adjacent stalls. Drivers often do not park precisely in the centre of the designated stall and do not always pull-in to the full depth allowance of the stall. Therefore, where space permits, it is always best to avoid minimum stall dimensions.

Maximum allowable grade for the accessible parking stalls is 1V:50H (2.00%). Parking spaces that are not level may deny use to physically challenged persons since vehicle doors are more difficult to operate on slopes where wheelchairs tend to roll away.

11.4.4.4 DESIGN VEHICLE

In the planning and design of parking spaces, the design vehicle should be selected based on the following considerations:

i. The vehicle type that accounts for a high share when the demand for parking peaks should be the design vehicle.
ii. The design vehicle should not be too large to avoid wastage of parking spaces and to encourage orderly parking.
iii. The design vehicle should not consider future changes in vehicle dimensions. Road markings in parking areas, etc. can be easily revised according to future changes

11.4.4.5 ESTABLISHING THE GEOMETRIC STRUCTURE OF A PARKING LOT

Like road design, the design of off-street parking areas is an iterative process. Many parking facilities within road reservations are relatively small in area and hence design options may be limited. However, designers may be required to develop larger parking areas as part of a road design.

The design of an off-street parking area requires a designer to:

i. establish the range of acceptable access locations
ii. adopt trial locations for access
iii. prepare a preliminary functional design for the accesses taking into account the outcomes of traffic analysis and all required design considerations
iv. adopt a suitable parking module layout
v. design the access roadways and parking areas based on the modules, taking into account the need to accommodate other related facilities (e.g. toilets and picnic areas

in the case of rest areas, refuelling facilities and restaurants in the case of service centres, bus stops and waiting areas in the case of park and ride facilities)

vi. wherever necessary, repeat the process utilising different parking modules and adjusted access locations to provide a suitable option, or options for consideration and approval.

11.4.4.6 PARKING FACILITY LAYOUT

The layouts of off-street parking areas may be comprised of a number of elements, depending on the size of the area, namely:

i. access driveway
ii. parking module
iii. circulation roadways
iv. parking aisles
v. parking spaces/stalls
vi. raised islands to define parking spaces
vii. raised walkways
viii. ramps

An example of an off-street car park illustrating appropriate treatment of car parking elements is shown in **Figure 11.19**.

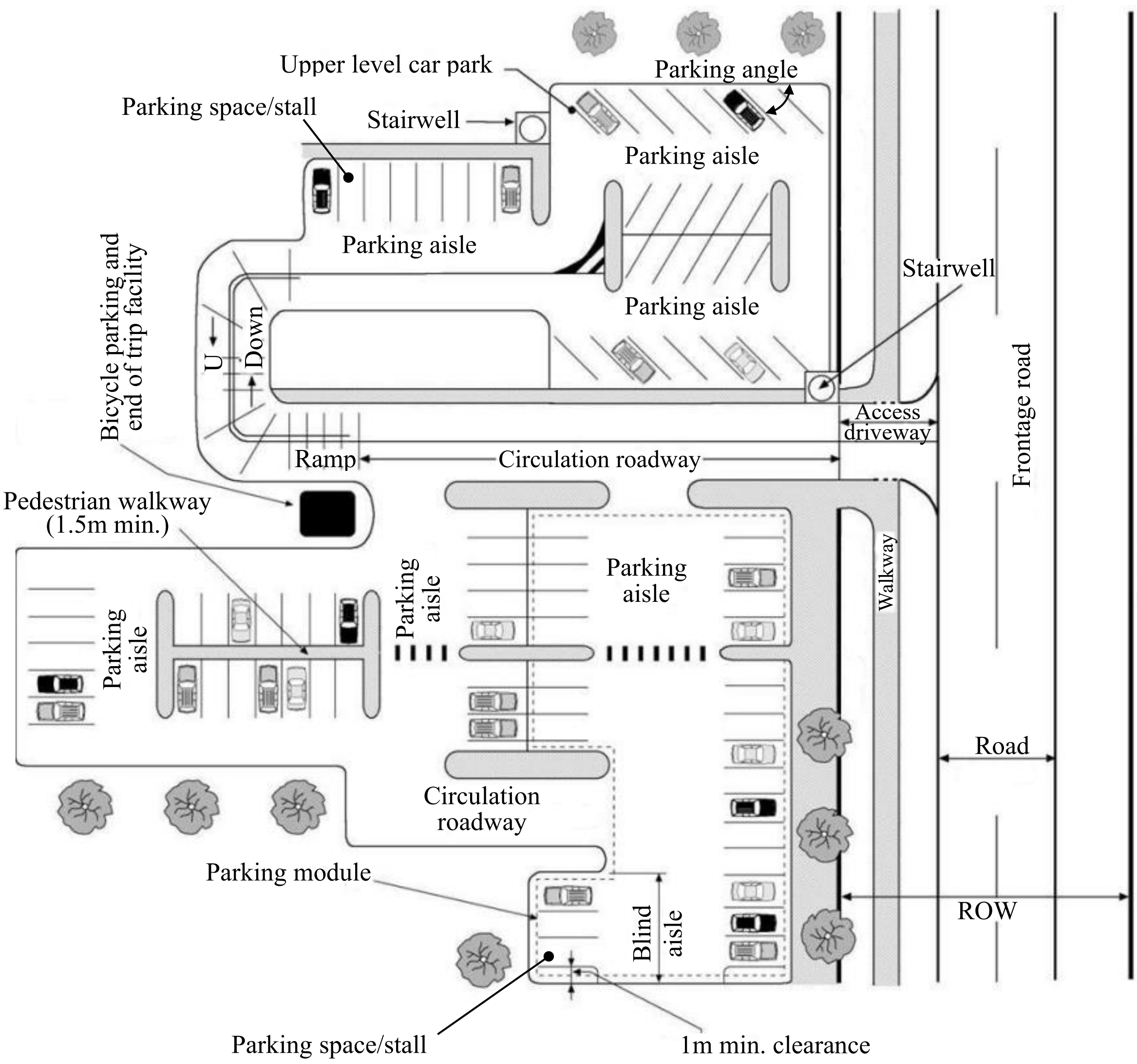


Figure 11.19 Example of an off-street car park (Adopted from Austroads)

11.4.4.7 PRELIMINARY DESIGN CONSIDERATIONS

11.4.4.7.1 DESIGN COORDINATION

The layout design of an off-street car park shall consider the entire facility, including parking modules, circulation roadways, access driveways and, if necessary, frontage road access, as an integrated and co-ordinated design. Provision for traffic within a parking facility shall take into account the following:

i. The need for traffic to move to and from the frontage road with minimum disruption to through traffic and maximum pedestrian safety.

ii. Provision of adequate capacity in circulation roadways and parking aisles to handle peak period movements.

iii. Arrangement of internal roadways to avoid, as far as practicable, conflicts between intersecting streams of circulating traffic.

iv. Provision of minimum length travel paths between entry/exit points and parking spaces.

v. Safe treatment of points of conflict with pedestrians and other road users.

vi. Provision of parking spaces and accessible pedestrian paths for physically challenged persons.

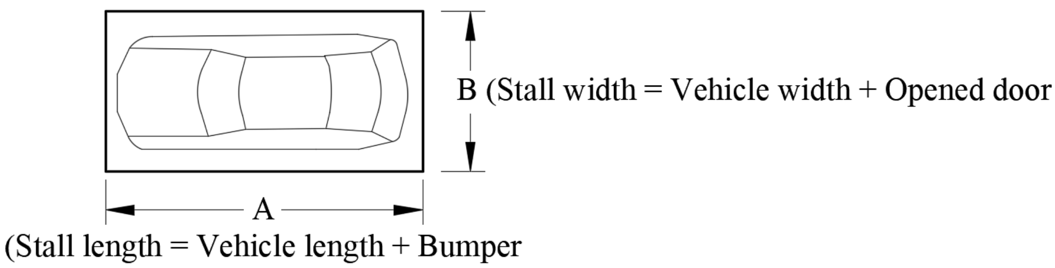
11.4.4.8 SIZE OF PARKING SPACE

The design of parking spaces/stalls takes into consideration the clearance of a vehicle to other vehicles (including size of opened door for boarding and alighting of passengers) and fences.

Table 11.12 shows the recommended minimum dimensions of parking spaces for various design vehicles.

Table 11.12 Size of parking space based on design vehicle

Design Vehicle	Symbol	Stall length, A (Vehicle length clearance) (m)	Stall width, B (Vehicle width clearance) (m)
Small Vehicle	S-5	5.0 = 4.70 + 0.3	2.5 = 1.95 + 0.55
Medium Vehicle	M-6	6.5 = 6.0 + 0.5	2.55 = 2.0 + 0.55
Large Vehicle Type 1	L-9	10.0 = 9.1 + 0.9	3.15 = 2.4 + 0.75
Large Vehicle Type 2	L-12	13.0 = 12.0 + 1.0	3.25 = 2.50 + 0.75
Trailer Type 1	T-17	17.5 = 16.5 + 1.0	3.50 = 2.50 + 1.00
Trailer Type 2	T-21	22.5 = 21.0 + 1.5	3.60 = 2.60 + 1.00



11.4.4.9 METHODS OF PARKING

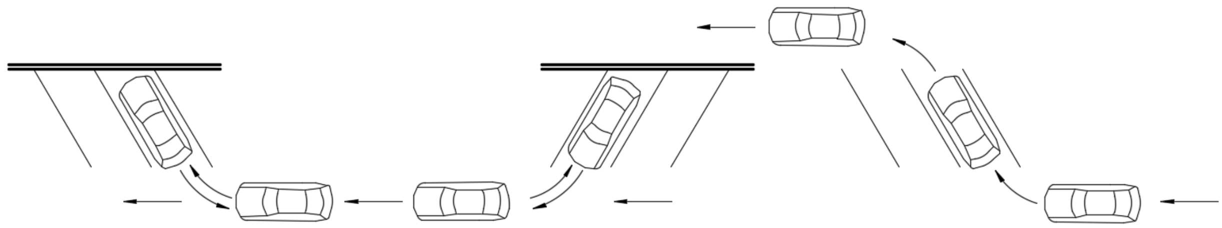
There are three (3) methods of off-street parking. These are

i. Advance – Reverse (Advance parking)

ii. Reverse – Advance (Reverse parking)

iii. Advance - Advance

Advance parking is easier to do, however, leaving the parking space takes more time and can be riskier if the circulating roadway has poor visibility. Reverse parking takes more time for parking, however, leaving is easier. The parking method should be decided upon considering the features of the place where the vehicle parking space is provided. **Figure** 11.20**6** shows the methods of parking.



(a) Advance - Reverse (b) Reverse - Advance (c) Advance - Advance

Figure 11.20 Methods of parking

11.4.4.10 PARKING ANGLE

Parking angles used in off-street car parks are as follows:

i. **90^0 angle parking**

Parking aisles for 90^0 parking is designed for two-way movement even though one-way movement may need to be imposed in some instances. 90^0 parking will in most cases be found to be the most efficient use of space in a large area.

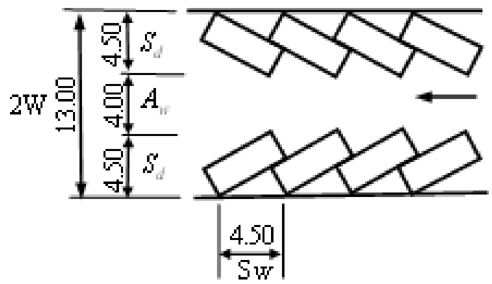
ii. **30^0, 45^0 or 60^0 angle parking**

Where space is limited or does not lend itself to 90^0 parking, 30^0, 45^0 or 60^0 parking may be used instead. Aisles serving such spaces shall be one-way (except where parallel parking is allowed on one side) with forward entry into the spaces only. Such arrangements can have advantages for high turnover parking provided drivers are discouraged from entering aisles the wrong way and reversing into parking spaces.

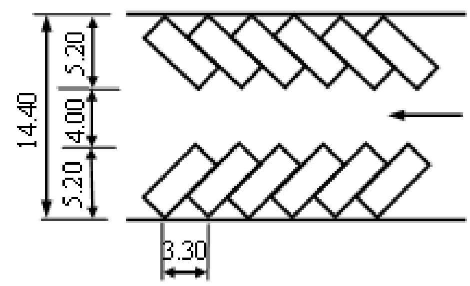
iii. **Parallel parking**

Parallel parking is a method of parking a vehicle in line with other parked cars. Cars parked in parallel are in one line, parallel to the kerb, with the front bumper of each car facing the back bumper of the adjacent one. Parallel parking requires driving the car in reverse gear into the parking space.

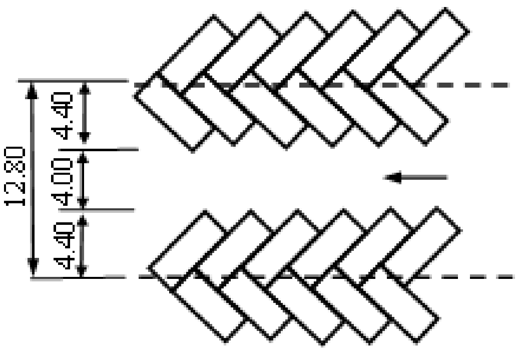
Standard parking arrangement are shown in **Figure 11.21(a)-(n).**



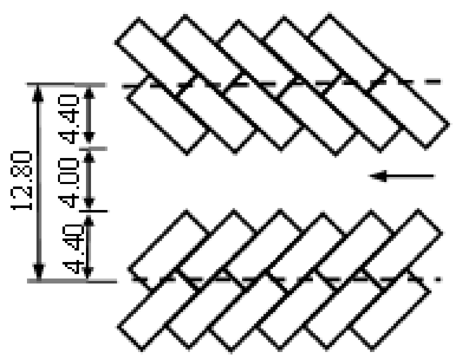
(a) 30^0 Advance – Reverse



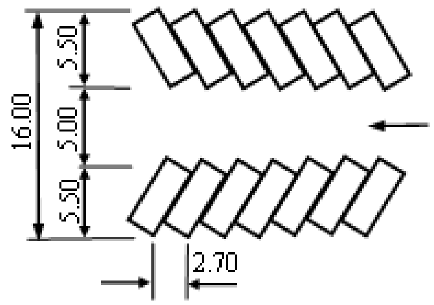
(b) 45^0 Advance – Reverse



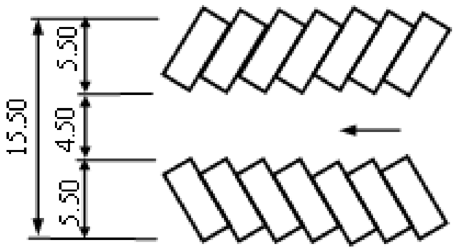
(c) 45^0 Cross Advance – Reverse (Type A)



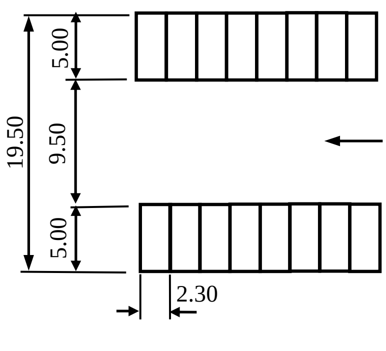
(d) 45^0 Cross Advance – Reverse (Type B)



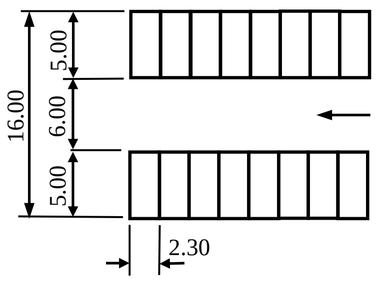
(e) 60^0Advance - Reverse



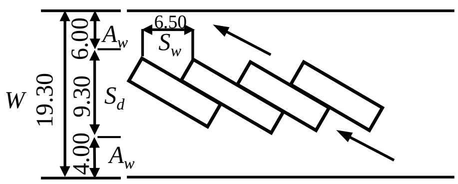
(f) 60^0 Reverse – Advance



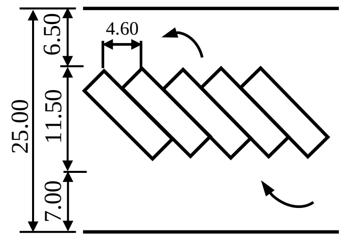
(g) 90^0 Advance – Reverse



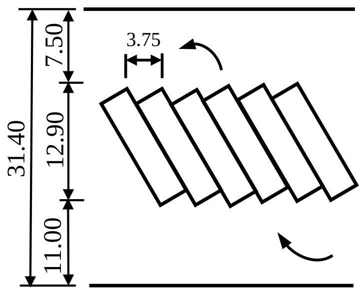
(h) 90^0 Reverse – Advance



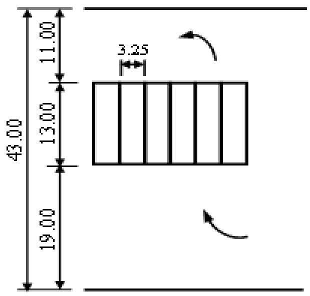
(i) 30° Advance - Advance.



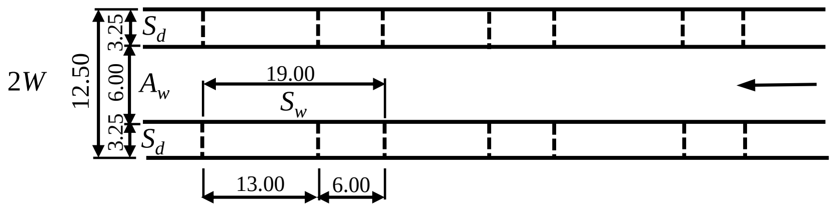
(j) 45° Advance - Advance.



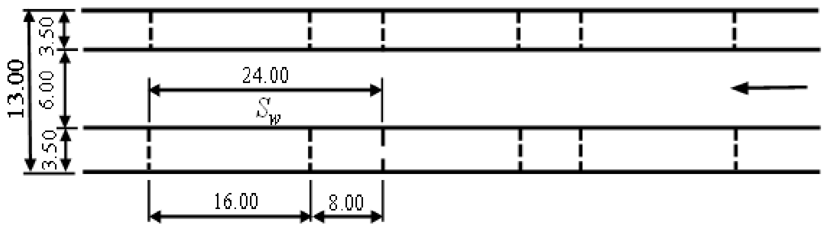
(k) 60° Advance - Advance.



(l) 90° Advance - Advance.



(m) Parallel Reverse – Advance (L-9, L-12. Refer to **Table 11.12**).



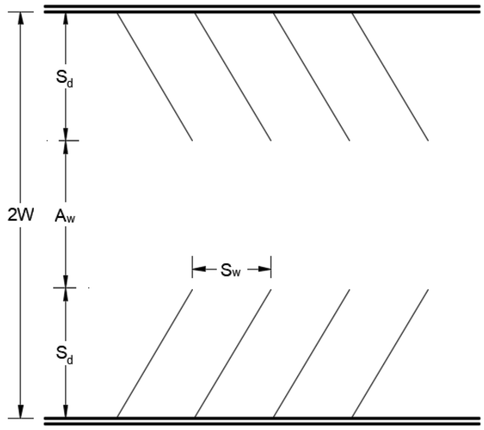
(n) Parallel Reverse – Advance (T-17, T-21. Refer to **Table 11.12**).

Figure 11.21 Standard parking arrangement (Unit; m)

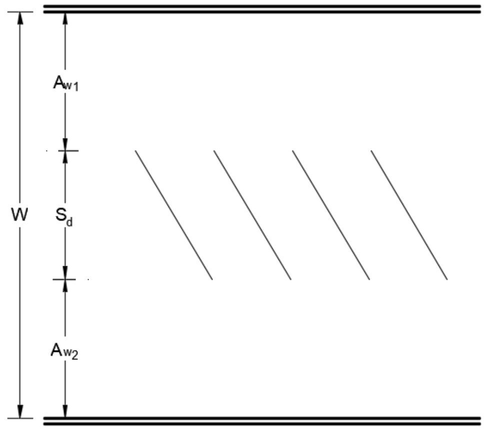
11.4.4.11 DIMENSIONS OF PARKING MODULE

The dimensions of a parking module depend on the angle of parking, method of parking, design vehicle and accessibility. **Figure 11.22** and **Table 11.13** show the dimensions of a parking module.

If a parking aisle exceeds 100 m in length, (i.e., more than about 40 × 90^0 parking spaces on either side) traffic control devices such as speed humps should be placed along the parking aisle to control vehicle speeds. Where vehicle negotiation of such devices may lead to structural damage, compliance with this requirement may be waived.



(a) Advance-Reverse and Reverse-Advance



(b) Advance-Advance

Advance-Reverse and Reverse-Advance

$$W = A_w/2 + S_d \tag{11.1}$$

$$A = W \times S_w \tag{11.2}$$

Advance – Advance

$$W = A_{w1} + A_{w2} + S_d \tag{11.3}$$

$$A = W \times S_w \tag{11.4}$$

Where,

- W: Parking length for a vehicle (m)
- A: Parking area required for a vehicle
- A_w: Width of circulating roadway
- S_d: Transverse length of parking space
- S_w: Horizontal width of parking space

Figure 11.22 Dimension of parking module

Table 11.13 Dimension of parking module

Angle of parking	Method of parking	A_w (m)	S_d (m)	S_w (m)	W (m)	A (m^2)	Standard parking arrangement (Figure 11.21)
Small and Medium Vehicles							
30°	Advance -Reverse	4.00	4.50	4.50	6.50	29.3	(a)
45°	Advance -Reverse	4.00	5.10	3.20	7.10	22.8	(b)
45° Cross	Advance -Reverse	4.00	4.30	3.20	6.30	20.2	(c) (d)
60°	Advance -Reverse	5.00	5.45	2.60	7.95	20.7	(e)
	Reverse-Advance	4.50	5.45	2.60	7.70	20.0	(f)
90°	Advance -Reverse	9.50	5.00	2.25	9.75	21.9	(g)
	Reverse-Advance	6.00	5.00	2.25	8.00	18.0	(h)
Large Vehicle							
30°	Advance - Advance	Aw_1=4.00 Aw_2=6.00	9.30	6.50	19.30	125.5	(i)
45°	Advance - Advance	Aw_1=7.00 Aw_2=6.50	11.50	4.60	25.00	115.0	(j)
60°	Advance - Advance	Aw_1=11.00 Aw_2=7.50	12.90	3.75	31.40	117.8	(k)
90°	Advance - Advance	Aw_1=19.00 Aw_2=11.00	13.00	3.25	43.00	139.8	(l)
Parallel	Reverse-Advance	6.00	3.25	19.00	6.25	118.8	(m)
Truck							
Parallel	Reverse-Advance	6.00	3.50	25.00	6.50	162.5	(n)
Angle	(Refer to **Figure 11.23** and **Table 11.14** for angle parking)						

11.4.4.12 TRUCK PARKING

i. The two main types of parking slots used in truck parking areas are Reverse-Advance (Straight Back-In (SBI) slots) and Advance-Advance (Herringbone Drive-Through (HDT) slots).

ii. Truck parking stalls should be designed for Advance-Advance operation at either a 30°, 45° or 60° angle to the traffic lane.

iii. Stalls at a 30° angle are generally more satisfactory in relation to Advance-Advance vehicle movements. However, they also require a longer overall length of parking area.

iv. In certain situations, parallel park stalls design should be considered in order to increase the capacity of the parking area and safety of the vehicle occupants.

v. Parking stall dimensions should be designed to accommodate the T-21 semi-trailer units.

Refer to **Figure 11.23** and **Table 11.14** for recommended parking dimensions for a T-21 vehicle.

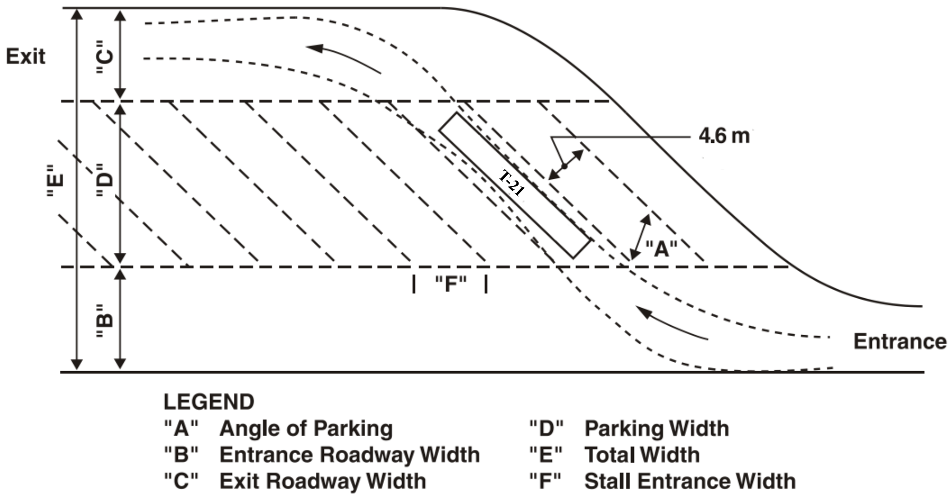


Figure 11.23 Dimensions for truck parking spaces (design vehicle T-21)

Table 11.14 Dimensions for truck parking spaces (design vehicle T-21)

Angle Of Parking (Degrees) "A"	Entrance Roadway Width (m) "B"	Exit Roadway Width (m) "C"	Parking Width (m) "D"	Total Width Parking Area (m) "E"	Stall Entrance Width (m) "F"
30	6.7	6.7	14.0	27.4	9.20
45	9.1	9.1	18.3	36.5	6.51
60	12.2	10.7	21.3	44.2	5.31

11.4.4.13 PHYSICAL CONTROLS

The need for the following physical controls shall be considered:

i. **Kerbs**—on one or more sides of a parking space to protect pedestrian walkways, landscaped areas, and any other non-trafficable areas generally at or just above pavement level, from encroachment. Vehicles may be allowed to park overhanging a kerb at the rear of a parking space, provided that:

(a) the kerb is not more than 150 mm high;

(b) the area up to 1.2 m behind the kerb does not slope up from the kerb; and

(c) the walkway behind the kerb would not be obstructed.

If overhang cannot be tolerated, wheel stops shall be provided. Kerbs in vulnerable locations may require additional devices such as bollards to make them visible

to car drivers.

ii. **Barriers—**to contain vehicles at the edges of platforms or decks, or to prevent encroachment onto pedestrian facilities.

Barriers shall be constructed to prevent vehicles from running over the edge of a raised platform or deck of a multi-storey car park including the perimeter of all decks above ground level. They are required wherever the drop from the edge of the deck to a lower level exceeds 600 mm. At drops between 150 mm and 600 mm, wheel stops shall be provided.

iii. **Wheel stops—**to limit the travel of vehicles when manoeuvring into a parking space as shown in **Plate 11.4**. If used, they shall meet the requirements given below:

1. Typical uses of wheel stops are as follows:
   (a) Control of kerb overhang that may be inconvenient or hazardous to pedestrians.
   (b) Inhibiting contact with an end barrier or high kerb.
   (c) Inhibiting encroachment into an opposing parking space.
2. Wheel stops should be avoided in any situation where they may be in the path of pedestrians moving to or from parked vehicles or crossing a car park for any other purpose. Wheel stops shall be between 90 and 100 mm in height, and 1650 ±50 mm in width.

Plate 11.4 Wheel stop

iv. **Other protective devices**—to prevent damage to structural elements or other unwanted vehicle encroachment.

Physical controls shall not obstruct accessible travel paths for physically challenged persons.

All kerbs, wheel stops, low barriers and other obstructions that could be a tripping hazard to pedestrians shall be surfaced in a colour contrasting with their surroundings.

11.4.4.14 CIRCULATION ROADWAY AND RAMP GRADES

Limiting requirements for grades on circulation roadways and ramps shall be as follows:

(a) Straight ramps: public car parks - as follows:

i. Longer than 20 m - 1 in 6 (16.7%) maximum.

ii. Up to 20 m long - 1 in 5 (20%) maximum. Grade change transitions will usually be required (see Item (d)). The allowable 20 m maximum length shall include any parts of grade change transitions at each end that exceed 1 in 6 (16.7%).

iii. A stepped ramp comprising a series of lengths each exceeding 1 in 6 (16.7%) grade shall have each two lengths separated by a grade not more than of 1 in 8 (12½%) and at least 10 m long.

(b) Straight ramps: private or residential car parks - as follows:

i. Longer than 20 m - 1 in 5 (20%) maximum.

ii. Up to 20 m long -1 in 4 (25%) maximum. The allowable 20 m maximum length shall include any parts of grade change transitions at each end that exceed 1 in 5 (20%).

iii. A stepped ramp comprising a series of lengths each exceeding 1 in 5 (20%) grade shall have each two lengths separated by a grade of not more than 1 in 8 (12½%) and at least 10 m long.

Grade change transitions will be required in both cases where grades are at or near the maximum, see Item (e).

(c) Curved ramps—as for straight ramps, except that the grade shall be measured along the inside edge.

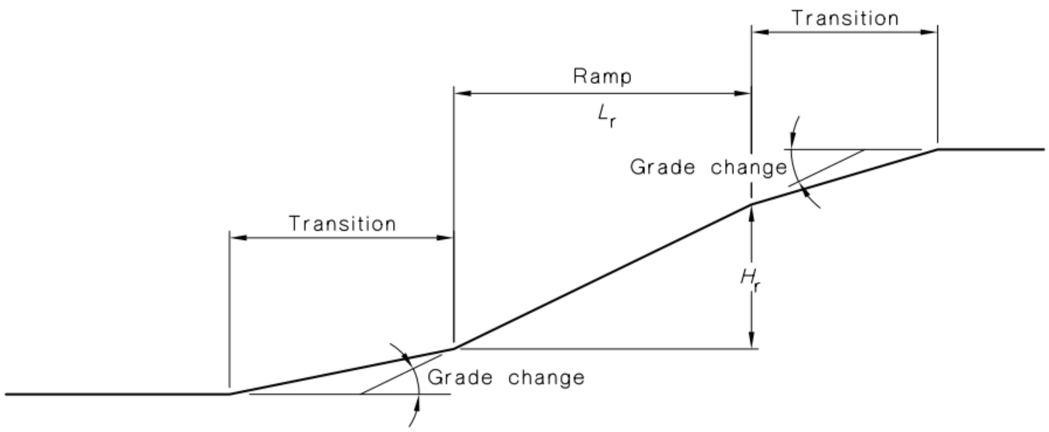
(d) Changes of grade - To prevent vehicles scraping or bottoming, changes in grade in excess of:

i. 12.5 percent algebraically (1 in 8) for summit grade changes; or

ii. 15 percent algebraically (1 in 6.7) for sag grade changes;

require introduction of a grade transition between the main grade lines as illustrated in **Figure 11.24**.

(e) Grade transitions—Transitions of 2.0 m in length will usually be sufficient to correct bottoming or scraping at grade changes up to 18 percent. They may be in the form of a simple chord with grade calculated as half the algebraic sum of the two adjacent grades, as illustrated, but for vehicle occupant comfort may be constructed as short vertical curves.



L_r = length of ramp, in metres
H_r = height of ramp, in metres

Ramp grade = $\frac{H_r \times 100}{L_r}$ percent

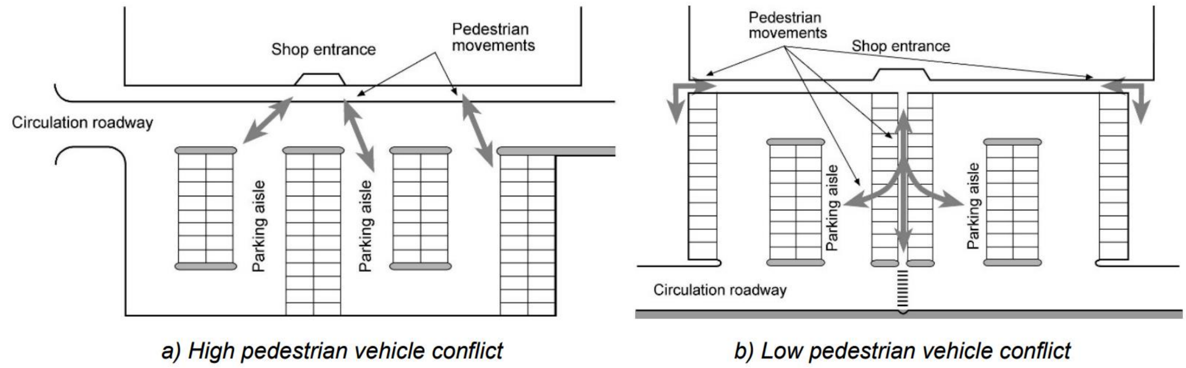
The grade change is computed by subtracting one grade from the adjacent grade, both expressed as percentages and taking account of algebraic sign which, for a given direction of travel, is either uphill—positive or downhill—negative.

Figure 11.24 Changes of grade on ramps

11.4.4.15 PEDESTRIAN TREATMENTS

The following principles should be considered in parking facility planning to allow the safe interaction of vehicles and pedestrians:

i. Pedestrian entrances and exits should be separate from vehicular ones and pedestrian movements should be restricted on circulation roadways since these involve faster moving traffic (**Figure 11.25**).
ii. Safe crossing points should be located away from major concentrations of vehicular movement, be positioned at right angles to the traffic flow and be provided at locations with adequate sight distance. Pedestrians should be guided to crossing points using appropriate signs and pavement markings.
iii. Raised pedestrian walkways can be used in large car parks to separate rows of cars and to provide favourable walking conditions where high pedestrian flows are expected.
iv. Parking layouts should be planned so that the aisles are parallel with the desired pedestrian routes rather than across them. Otherwise, frequent pedestrian links through the rows of parking will be required.
v. A tactile distinction should be made between pedestrian areas and vehicular areas, in order that people with visual impairment can distinguish between the two. The provision of raised areas, walkway areas and tactile paving at all dropped kerbs should achieve this.



Source: ARRB Group

Figure 11.25 Pedestrian conflicts with different parking layouts

Figure 11.25 gives examples of good and poor design of a pedestrian facility. In the example illustrating bad practice (a) although a marked pedestrian crossing has been provided for pedestrians across the main circulating aisle outside of the building it has been constructed in such a way that it terminates at a raised island that does not have a sealed smooth surface or a kerb ramp. Anyone who is in a wheelchair or pushing a pram or other wheeled device would find it extremely difficult to negotiate up and over or around this island. Good practice (b) is to provide easily accessible connective pathways along the main pedestrian desire lines through the car park.

11.4.4.16 ACCESSIBLE PARKING FOR PHYSICALLY CHALLENGED PERSONS

i. Location

Accessible parking spaces for physically challenged persons should be located on the shortest accessible route of travel from the adjoining parking area to an accessible entrance. In parking areas that serve several facilities, accessible parking should be located on the shortest accessible route of travel to an accessible pedestrian entrance of the parking area. If facilities have multiple accessible entrances, accessible parking spaces should be dispersed and located closest to the accessible entrances.

ii. Space, sizing, access, aisles and slope

Accessible parking spaces for physically challenged persons should be at least 2.5m wide (refer to **Figure 11.26**). A 1.5 m wide marked "No Parking" loading area (access aisle) shall be provided adjacent to all accessible parking spaces. Two accessible parking spaces may share a loading area. When only one accessible parking space is required, the location of the loading space shall be on the passenger side. Provision should be made for van accessible space(s) with a loading area of at least 2.5m wide. Parked vehicle overhangs should not reduce the clear width of an accessible route. Parking spaces and access aisles should be level with surface slopes not exceeding 1:50 or 2% in all directions. Accessible parking spaces are required to be on the shortest accessible route of travel from parking to the building entrance(s).

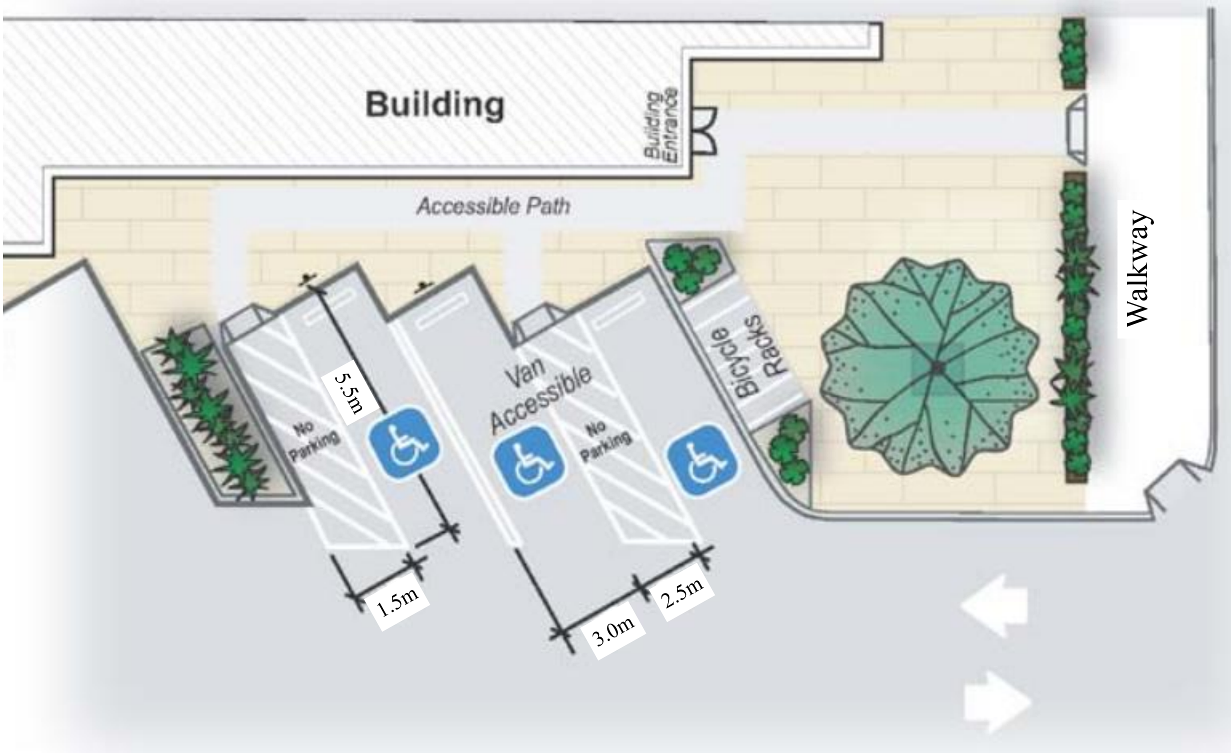


Figure 11.26 Standard accessible parking stall

iii. Signage

Accessible parking spaces should be designated as reserved by a sign showing the symbol of accessibility (see **Figure 11.26**). Such signs should be located so they cannot be obscured by a vehicle parked in the space.

11.4.4.17 LANDSCAPING

The most attractive parking areas are well landscaped. Trees are the most valuable additions to parking areas, whether planted in kerbed islands or on perimeters. They provide shade, visually reduce the mass of open pavement, and mitigate heat gain.

Landscaping should be an integral part of the car parking area and consider the following:

i. Design of landscaping should be integrated with pedestrian, car parking and all other items pertaining to the function of the car park.
ii. Elements including but not limited to vehicle overhang, ability to access car doors, access to pedestrian paths, provision of lighting and signage should not impact upon landscaping or give cause to reduce landscaping.
iii. Landscaping should not obstruct pedestrian or vehicle view lines.
iv. Soft landscaping (ground covers and trees) should be provided along pedestrian priority pathways, throughout the car park and areas where screening is considered required.

v. Trees should be provided with sufficient space to promote healthy growth and protection as shown in **Figure 11.27**. **Plate 11.5** and **Plate 11.6** show narrow landscaping area.

vi. Soft landscaping should be provided with passive irrigation and adequate drainage.

vii. A minimum of 15% of the car parking area should be landscaped.

viii. A target of 35% shaded area should be provided by trees.

ix. Tree species should have a clear singular trunk form.

x. Groundcovers should be hardy and grow to a mature maximum height of 500mm, except where vehicle overhang occurs. In this case appropriate groundcover needs to be used.

xi. Garden beds without trees should be a minimum of 400mm in width.

xii. Shrubs are generally not accepted and should only be used where considered appropriate, i.e., screening walls or fences.

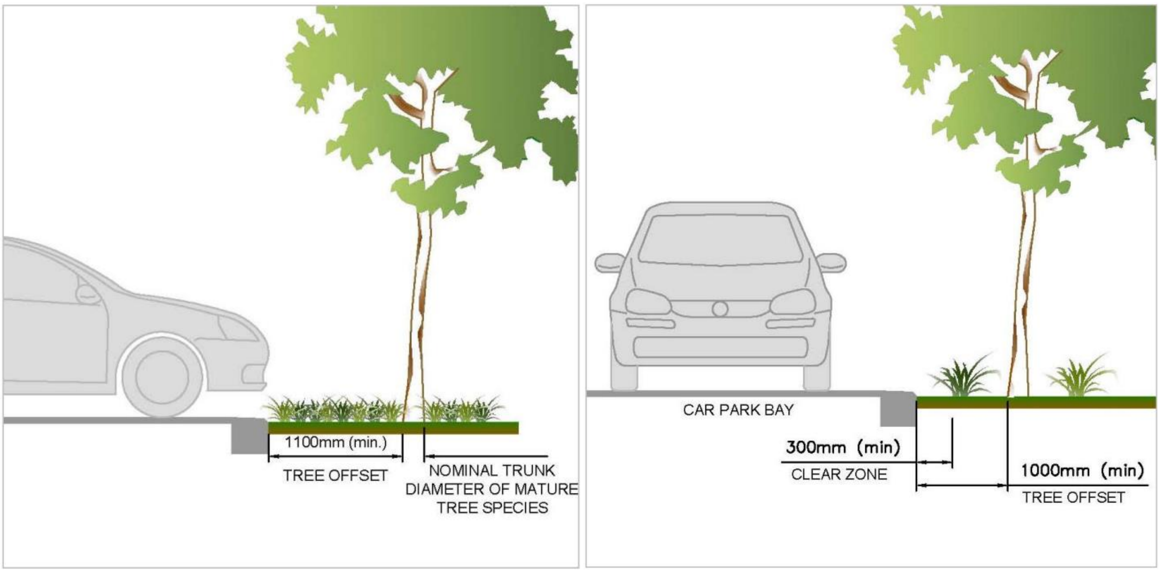


Figure 11.27 Minimum vehicle clearances to trees and planting

Plate 11.5 Tree planting area too small and inappropriate for traffic safety

Plate 11.6 Narrow landscaping area does not allow for vehicle overhang

11.4.4.18 PAVING

For pavement standards for parking, refer to GHA Pavement Deign Guide.

11.4.4.19 SIGNS AND MARKING

Suitable signs should be used to direct all car park operations. Signs and pavement markings are required in car parks for the following reasons:

i. to warn against hazards to personal safety or potential damage to vehicles
ii. to control traffic conflicts, traffic movement and driver behaviour (e.g. speed)
iii. to direct and inform drivers entering and circulating within the car park about vehicular entry points, exits and parking areas
iv. to direct pedestrians to amenities such as elevators, stairs, buildings, routes to fire escapes etc. and to identify parking modules to help people find their parked vehicle easily.

Signs should be clearly visible, easy to read and simple to follow and not used excessively as this may lead to confusion.

Signs displaying the international access symbol should be provided at each accessible parking space. The signs should be displayed on fixed mountings in an area where they are not hidden from view. Pavement marking symbols may be used to supplement signs.

The location of each parking space and direction of traffic flow should be identified by surface markings and should be maintained so as to be readily visible at all times. In general, yellow markings tend to stand out better than white from the background parking surface. White paint on concrete also tends to fade with time, making it difficult to distinguish the striping. All pavement striping should be 10 cm in width. Markings may either be painted or cold applied marking tape.

11.4.4.20 DRAINAGE

Parking areas must be properly sloped and drained to take care of runoff. Apply the following minimums:

i. Ideal slope for all parking area pavements is 2%
ii. Longitudinal pavement slope should be between 0.4% - 5%
iii. Pavement cross slope should be between 1% - 3%
iv. Storm water should be collected on the perimeter of parking areas with a minimum of 2% slope along concrete curb and gutter

11.4.4.21 LIGHTING

Lighted parking areas are an important consideration for facilities that expect early morning, late afternoon, or night time use. All parking areas should be safely illuminated. Intersections with major pedestrian routes and at parking area entrances and exits are especially important when choosing light fixtures and locations. The designer should provide lighting for parking areas that meet the minimums of 0.3- 0.6 m-candles and 10-20 lux. Lighting should not disturb nearby residential areas. Energy efficient lighting should be used. Light pole and tree locations should be planned holistically to ensure one does not adversely impact upon the other, that each are provided with adequate space to perform their function and not give cause to reduce either item.

Lighting standards or poles vary from 6-9 m tall and should be located in islands or on parking area perimeters. Poles that are not protected by a kerb or other structure should be constructed with a concrete base at least 1mhigh or be buffered by concrete filled pipes or bollards.

11.4.4.22 BICYCLE AND MOTOR BIKES PARKING SPACE

Since bicycles and motorbikes generally employ a kickstand for support when parked, a rigid surface such as concrete should be provided to ensure stability while minimizing potential pavement damage. Some standard parking dimensions for bicycles and motorbikes are:

i. Motorbike Parking space width 1.2m min. (1.5m desirable)
ii. Motorbike Parking space length 2.5m
iii. Bicycle Parking space width 0.7m
iv. Bicycle Parking space length 1.8m

Examples of Motor and bicycle parking spaces are shown in **Plate 11.7**.

Plate 11.7 Example of Motor and bicycle parking spaces

11.5 REST AREAS

A rest area, also known as rest stop or service area, is a roadside area with parking facilities separated from the roadway, provided for travellers to stop and rest for short periods. The area may provide drinking water, restrooms, tables and benches, telephones, information displays, and other facilities for travellers.

The provision of rest areas for motorists and heavy vehicle drivers is recognized as an integral part of a holistic approach to the management of driver fatigue on highways. Fatigue is considered a major contributory factor to road accidents nationwide. All drivers are vulnerable to fatigue.

11.5.1 PRINCIPLES SUPPORTING REST OPPORTUNITIES

The following overarching principles should be considered whenever assessing the need for rest areas on the road network.

i. Drivers require regular opportunities to stop and rest.
ii. Rest opportunities should be provided at intervals not exceeding 100km.
iii. Rest opportunities should be supplemented by stopping bays.
iv. Rest areas are provided for fatigue management purposes.
v. Rest areas should be all-weather accessible.
vi. Rest areas should support emergency management operations.

11.5.2 TYPES OF REST AREAS

The functional requirements of rest areas differ according to location, vehicle mix, and desired user type. Traffic volumes at each location will determine capacity requirements, however rest areas can generally be categorised as follows:

i. **Type A**

Sites providing extensive facilities supporting all potential motorist types. These sites are generally ‘mid-block’ and do not conflict with commercial or civic sites within the area.

ii. **Type B**

Sites focused on providing an appropriate number of parking bays with facilities intended to cater for short to medium term rest periods in support of achieving rest during journeys. These sites do not conflict with commercial or civic sites and provide a standard level of fatigue-related facilities on the highway.

iii. **Type C**

Sites providing locations with an adequate number of parking bays at which motorists can safely stop away from the roadway to rest. Facilities may be minimal, potentially including only hardstand areas, bins, and shade. These sites are provided where fatigue-related facilities are required without the need to provide a greater level of facilities. These sites may include those that are adjacent to commercial or civic facilities, or support roads with low vehicle numbers for the appropriate vehicle type.

iv. **Type D**

Sites adjacent to roads with high percentage of truck traffic solely dedicated for truck parking to enable operators take fatigue breaks and address operational needs. Truck parking bays are generally located on both sides of the road in a left-right staggered manner at a minimum distance of 1km apart. Facilities may be minimal, potentially including only hardstand areas, bins, and shade.

To supplement formal rest areas, stopping bays and commercial rest opportunities should be provided in-between rest areas.

i. **Stopping bays**

Stopping bays in each direction should be provided in-between each rest opportunity. Provision of an increased number of bays is highly desirable, particularly on higher volume roads or those that carry significant amounts of freight or tourist traffic.

Stopping bays are intended for stops of no more than 15-30 minutes unless otherwise signed, and provide regular opportunities for drivers to stop, check their vehicle or loads, complete work diaries if required, or take a short break to refresh before continuing their journey.

As a minimum, these bays should cater for 1-2 of the largest vehicle types operating on that road, and should be located at appropriate, safe locations in accordance with traffic patterns on the route.

Safe separation from traffic should be provided, preferably with at least 1 metre of line-marking as a minimum, along with adequate acceleration and deceleration tapers. Facilities are not strictly required however the provision of bins is a desired minimum where possible. On higher volume routes, consideration should also be given to sheltered tables and chairs where they can be safely deployed.

Use of enforcement bays may be appropriate as informal stopping bays, however it should be remembered that these sites will be unavailable when in use by Transport Inspectors or Police when conducting traffic operations.

ii. **Commercial rest area**

On highways, commercial enterprises, local communities or community groups may provide access to locations or facilities that are appropriate for use in support of driver fatigue management.

Sites such as fuel/service stations, fruit stalls, bakeries, tourist information centres, chop bars, restaurants etc., provide a mixture of parking areas, access to fuel and food, and often access to facilities such as washrooms and other services for customers. These sites can often be utilised, particularly by travellers, for fatigue breaks providing drivers the opportunity to ‘stretch their legs and access amenities as required.

11.5.3 STANDARDS FOR REST AREAS

All sites adjacent to the roadside must comply with design and safety standards. Within these requirements, rest areas should provide:

i. Adequate parking for the types and number of vehicles using the area.
ii. As a minimum, rest areas must provide five (5) parking bays per vehicle type accessing the site.
iii. Reasonably quiet and restful environment that is separated, and preferably screened, from the road.
iv. Sealed or paved surface where possible.
v. Safe access for the entrance and exit of the site, with adequate room for manoeuvring safely within the site.
vi. Signs and markings in advance and at the site to inform and guide drivers.
vii. Sheltered tables and benches, rubbish bins and shade (natural or artificial).
viii. Facilities such as lighting, water supply and washrooms.
ix. Additional facilities such as eatery facilities, scenic outlooks, tourist information boards, etc., can also be provided as appropriate.
x. Provision of access for persons with disabilities (PWD’s) to the site and facilities.
xi. If the site is dual-access, clear separation between motorists and heavy vehicles to reduce vehicle interaction.

In addition, if the site caters for trucks, it should also provide:

i. Separation between short and long term parking areas to allow drivers on ‘long breaks’ to sleep without disturbance.
ii. Possible separation between types of heavy vehicles such as those carrying livestock or hazardous goods.
iii. Nose-to-tail parking for heavy vehicles to allow for effective rest with reduced near-cabin noise issues.
iv. Significant shade
v. Sufficient parking bays and manoeuvring room designed for the largest combination permitted use of the route.

Table 11.15 defines the desired standards and facilities for each rest area type.

Table 11.15 Desired standards and facilities for rest areas

Standard/facility	Type A*	Type B*	Type C*	Type D
Capacity (For largest vehicle permitted on route)	Large: >20 bays (>10,000 AADT) Medium: 10-20 bays (1,000–10,000 AADT) Small 5-10 bays (<1,000 AADT)			Large: >15 bays (>1,000 HV AADT) Medium:10 – 15 (500–1,000 HV AADT) Small 5-10 bays (<500 HV AADT)
All-weather seal	Yes	Yes	Gravel	Yes
Separation for Vehicle types	Yes	Desirable	Where possible	No
Separation for long/short term visitors	Yes	Desirable	No	No
Bins	Yes	Yes	Yes	Yes
Natural shade/trees (where available)	Yes	Yes	Yes	Yes
Table/Chairs	Yes	Yes	Yes	Yes
Shelters/artificial shades	Yes	Yes	Yes	Yes
Washrooms	Yes	Yes	Yes	Yes
Lighting	Yes	Yes	Yes	Yes
Separation from road	Well separated and screened with vegetation mounding barrier, etc	Separated and screened where possible	Separated (as a minimum by line marking)	Separated and screened where possible
On-road signage	Yes	Yes	Yes	Yes
Eatery	Yes	Yes	Where Possible	Yes
Information Centre	Yes	Yes	No	No
ATM	Yes	Yes	No	No
Police Post	Yes	Yes	No	No
Security Arrangement	Yes	Yes	Yes	Yes
Lodging facility	Yes	Desirable	No	Yes
Towing Services	Yes	No	No	Desirable

* Where trucks are expected, use the capacity for Type D to estimate the number of bays.

11.5.4 PLANNING

The following major planning parameters should be considered by designers as they lay out a rest area system:

i. **Regional overview**

It is essential that the existing or proposed rest areas on neighbouring roads be considered. Contact and coordination with key stakeholders will ensure that the service provided to motorists along the highway will be consistent and evenly spaced.

ii. **Spacing intervals**

Normal rest area spacing intervals should not be more than 100km. Exceptions to this distance would be where traffic volumes are abnormally high or low.

iii. **Site selection**

Rest area planning should be done concurrently with initial roadway alignment studies. By selecting the best available site, the designer will make the process of site design much easier.

11.5.5 DESIGN

The aim of this section is to promote good practices for rest area design and upgrade activities. It is not intended to provide detailed design or technical information that may apply to specific sites, but rather present considerations that should be addressed during the design and construction of rest areas, and examples of a range of options that may be of use. Every rest area will have a particular set of requirements defined by its location, road type and usage, and many other local requirements.

The design factors that need to be addressed in the development of a rest area are:

i. Rest areas shall be serviceable for all year round (i.e. not to be placed in areas susceptible to flooding, landslide etc.).
ii. Rest Areas at the bottom of a hill or halfway up a hill or on a horizontal curve are generally inappropriate.
iii. Straight sections prior to downgrades, but with good sight distance are preferred. This will enable all vehicles, in particular heavy vehicles, improve egress when leaving or re-entering traffic flow.
iv. Design vehicle
v. Topography (grade, accessibility).
vi. Environmental and social impacts.
vii. Economics (cost of land).

11.5.5.1 REST AREA OFFSET AND SITE SCREENING

A landscape buffer zone is essential to separate the road reservation from the rest area and provide a more restful space. 10m is a desirable minimum width for this zone, however, may not be achievable in situations with limited corridor space. Nevertheless, to maximize safety, the rest area should not be hidden from view. To provide a perception of security, it should be laid out so it can be seen from the road. Ground cover combined with clear trunk trees can help provide both views and a feeling of separation.

11.5.5.2 GEOMETRIC DESIGN

i. **Rest Area Layout**

   a. Rest Area shall be designed to accommodate swept path of the design vehicle that will be using the Rest Area.

   b. The carriageway through the parking area shall be of sufficient width for manoeuvring of design vehicles when parking and leaving.

   c. The parking area shall be of adequate size and arrangement to accommodate the anticipated number of design vehicles.

   d. Parking spaces, whether parallel or angular shall meet the minimum standards specified under **Section 11.4**.

   e. It is recommended to provide separate parking spaces for passenger and heavy vehicles where possible.

   f. Turn around facility for heavy vehicle rest area to avoid fire, flooding or similar hazards ahead could be considered based on regional experience. Where turn around facility is required, design vehicle turning template for left turn out at rest area exit needs to be checked.

   g. When designing for pedestrian/vehicle interactions, the following should be considered:

- Parking areas should be located immediately adjacent to facilities.
- Access roadways should not be located between facilities and car parking areas.
- Roadways designed for vehicle acceleration or deceleration between the highway and the rest areas should not intersect with a location or path a pedestrian is likely to utilise.
- Clear lines of sight, particularly around facilities and pedestrian access points, should be achieved.
- At very large or busy facilities it may be necessary to implement formal pedestrian facilities (marked crossings, etc.) in accordance with appropriate design and safety standards.

ii. **Design Speed**

The design speed for vehicle travel within the rest area should be considered in conjunction with the type of facility and the location within the facility. Rest area should be designed to ensure that potential conflicts between vehicles and pedestrians is minimised and that any necessary interaction occurs at a very low speed. The recommended design speed within a rest area should be 20 – 30 km/h.

iii. **Alignment of Main Line**

Design values of main line alignment elements (horizontal curve radius, vertical gradient, vertical curve radius) around the Rest Area should be gentler than the standard value (Refer to **Section 9.4.5.1**).

iv. **Grade**

Flat grade (0.4 – 5.0 %) is preferred, specifically for heavy vehicles.

v. **Sight Distance**

Safe sight distance for the entry/exit points should be provided (Refer to **Section 7.2.6**).

vi. **Acceleration and deceleration lanes**

Adequate acceleration and deceleration lanes should be provided at the exit and entrance of rest areas. Refer to **Section 8.7.3** on design of acceleration and deceleration lanes.

vii. **Turning radii**

Access into and out of the rest area should consider the turning radii of motorists and the largest type of heavy vehicle to be accessing the site.

viii. **Sealed Access**

Access into the rest area should be sealed to enable safe entry and exit. On unsealed roads, access points should conform to the conditions of the roadway.

ix. **Drainage**

The drainage standard for stopping places shall be the same as for the adjacent road. If the site is located adjacent to sensitive water bodies or there are environmental considerations, stormwater pollutant traps (gross pollutant traps and/or oil and fine sediment traps) should be considered as part of the design.

x. **Signs & Markings**

Sufficient advance and at-location rest area signs supplemented with road line marking should be provided to enable drivers with adequate opportunity to decide to use a rest area and act accordingly in a safe manner. Sign postings and markings of all rest areas should be in accordance with MRH Standard Details, Road Signs & Markings for Urban and Trunk Roads.

11.6 HEAVY VEHICLE INTERCEPTION SITE

This section provides technical guidance for design requirements of Heavy Vehicle Interception Sites (HVISs) which are designed to provide a safe area outside the road carriageway for the following purposes:

i. weighing and inspecting heavy vehicles
ii. inspecting other vehicles
iii. checking the vehicle is being operated in accordance with various legislation (for example, dangerous goods, load security and fatigue)
iv. undertaking other enforcement activities conducted by police, customs, forestry commission or other authorised officials

Although the following sub sections focuses on weigh station, the principles can be applied to the design of other HVIS.

11.6.1 WEIGH STATION

A weigh station is a Heavy Vehicle Interception Site (HVIS) along a highway to inspect vehicular weights and safety compliance criteria, and to provide a source of data for planning and research.

11.6.1.1 TYPES AND CHARACTERISTICS OF WEIGHBRIDGES

There is a wide array of weighbridge types and related methods of weighing that can be used for overload control purposes. In general, there are two types of weighbridges and two methods of weighing as follows:

i. Types of weighbridges: Fixed versus mobile scales
ii. Methods of weighing: Static versus dynamic

Table 11.16 shows the types of vehicle-weighing systems that can be used for vehicle weighing purposes.

Table 11.16 Types of Weighing Systems

Type of Weighing System	Vehicle Element Weighed
Static– fixed	Total weight (GCM)
Static or dynamic: low speed – fixed	Single, tandem. or tridem axle
Static or dynamic: low speed – mobile	Single, tandem, or tridem axle

Although various types of weighing systems may be used for vehicle weighing purposes, they exhibit varying characteristics, and a careful choice must be made in relation to the main purpose of weighing the vehicle. These characteristics are summarized in **Table 11.17**.

Table 11.17 Weighbridge Characteristics

Method of weighing \ Type of weighbridge		Fixed Weighbridges	Mobile Weighbridges
		i. Easy to operate ii. Minimum personnel iii. Cargo off-loading iv. High installation costs v. Limited placement	i. Wide coverage ii. Difficult site selection iii. High operating costs iv. Equipment easily damaged v. Police cooperation vi. Traffic disruption
Static	i. More precision ii. Accepted for legal enforcement iii. Slower (esp. single axle scales)	i. Easiest to operate ii. Highest level of precision iii. Can weigh and register axle units	i. Lowest investment ii. Optimal for enforcement
Dynamic (WIM)	i. Rapid monitoring ii. Lower precision iii. Generally, not acceptable for enforcement	i. Fast for monitoring ii. Requires large installation iii. Requires careful direction of vehicles	i. Minimum disruption of commercial traffic ii. Lowest accuracy iii. Excellent for statistical monitoring

11.6.1.1.1 FIXED/STATIC WEIGHING SYSTEMS

A. Multi-deck

Where the traffic volumes warrant it (typically > 500 vpd) a multi-deck weighbridge (also known as split-deck weighbridge) consisting of four individual decks with lengths typically of 3 m, 6 m, 7 m, and 6 m, respectively, giving an overall length of 22 m, with a width of 3.2 m should be provided. Each deck must be capable of weighing a maximum mass of 40,000 kg, giving a total weighing capacity of 160,000 kg. A standard requirement at all weighbridges should be a roof over the scale. This will improve the working conditions and will make it possible to do weighing in all weather conditions. **Plate 11.8** shows a typical 3.2m x 22m multi-deck weighbridge.

Plate 11.8 Typical 3.2 m × 22 m Multi-deck Weighbridge

When configured correctly, multi-deck weighbridges can individually display the weights of all axle groups of both the truck and trailers. Digital weight indicators are assigned to each separate axle group weight to be displayed as illustrated in **Plate 11.9**. A summing indicator is used to display the total vehicle mass on the multi-deck weighbridge and then relay all deck weights to a PC if required. External remote displays can also be connected to display the axle group weights back to the truck driver (**Plate 11.10**).

Plate 11.9 Vehicle Control Centre

Plate 11.10 Digital Display of Actual and Permissible Axle Group and GCM Masses

Some of the benefits of multi-deck scales are as follows:

i. static weighing which results in very accurate measurement (<1% error) of individual axles and axle unit configurations as well as GCM;

ii. level tolerances on the approach slabs are not normally a problem as the whole vehicle is weighed at once;

iii. it is relatively very quick to weigh a vehicle;

iv. short verification tests can easily be done without test weights (measure any axle or axle unit on each of the weighbridge decks and the results should be consistent); and

v. it is more difficult to "manipulate" the weighing process, as the whole vehicle is weighed in one go (with an axle unit scale it is easy to weigh only part of an axle unit or to weigh one unit twice and skip an overloaded unit).

B. Axle unit scales

Where commercial traffic volumes (typically < 500 vpd) do not warrant the use of a more expensive multi-deck weighbridge, an axle unit scale can offer a cost-effective choice of weighing system. These scales are typically 3.2 m × 4 m and comprise a single deck (see **Plate 11.11**) which can be connected to a digital weight indicator and are capable of weighing a maximum mass of about 40,000 kg. A digital summing indicator can then be used to display the combined weight of the individual axles and axle groups to give the GCM.

Plate 11.11 Typical 3.2 m × 4 m Axle Unit Scale

Some of the benefits of the axle unit scale are as follows:

i. they can weigh any axle unit of a truck (i.e., single, tandem, or tridem unit), not as quickly as a multi-deck scale, but much more quickly than a single axle scale;
ii. level tolerances on the approach slabs no longer have to be as accurate as for the single axle scale but still need to meet minimum requirements;
iii. verification testing is relatively simple (limited staking of test weights); and
iv. it is far quicker to weigh multi-axle vehicles than using a single axle scale but not as quick as using a multi-deck scale.

C. Single axle scales

These may be described as the “first generation” scales that were used commonly in many countries. They typically comprise a single, 3.2 m × 1 m deck placed centrally within a 40 m concrete slab with a recess to accommodate the scale (see **Plate 11.12**). The scale can be connected to a computer linked to a digital reader and printer for producing weighbridge slips indicating the various weights of the axles and axle groups.

Plate 11.12 3.2 m × 1 m Single Axle Scale

Although relatively cheaper to install than multi-deck or axle unit scales, single axle scales have a number of drawbacks including:

i. the sites have to be constructed to very precise level requirements which are not easily met;
ii. weighing of multi axles is cumbersome and time consuming, especially for articulated or truck-trailer vehicles when up to seven or eight separate axles must be weighed;
iii. verification of the scales is difficult due to the difficulty of fitting the test weights onto the small deck.

The risk of weight transfer during the weighing will be the determining factor in levels of accuracy, irrespective of the inherent accuracy of the weighbridge.

11.6.1.1.2 PORTABLE SCALES

A. Static and dynamic

Portable (mobile) scales – either statically or dynamically operated – are normally used for screening purposes. These portable scales can be set up next to any road where there is a suitable surface and an area to pull off and weigh trucks. These scales should not be used for law enforcement purposes but are sufficiently accurate to identify vehicles that are probably overloaded with a high degree of confidence. Due to the fairly high accuracy of the portable scales, screening can take place at considerable distances from the weighbridge, as the chance of diverting vehicles that are legally loaded to the weighbridge is slim. These portable weighing devices are considerably cheaper than static scales, are relatively light, can be set up very rapidly and measure individual wheels, axles, axle units and vehicle/combination mass.

Examples of a static device and a dynamic device, a Vehicle Load Monitor weigh-in-motion (WIM) scale are shown in **Plate 11.13** and **Plate 11.14**.

Plate 11.13 Portable Static Weighing Device

Plate 11.14 Portable Dynamic WIM Device

B. Fixed/dynamic

Weigh-in-motion (WIM): A WIM system is a device that measures the dynamic axle mass of a moving vehicle to estimate the corresponding static axle mass. These systems are designed to capture and record axle weights and gross vehicle weights as vehicles drive over a measurement site at normal traffic speeds. Overhead variable message signs are used to redirect legally loaded vehicles back onto the highway while vehicles suspected of being overloaded are directed to an adjacent lane for accurate weighing on a static scale. Thus, the total number of vehicles to be weighed should be considerably less and a smaller facility may then be adequate.

WIM systems fall into two broad groups as follows:

i. High speed (HSWIM) – vehicle travel > 15 km/h
ii. Low speed (LSWIM) – vehicle speed ≤ 15 km/h

WIMS have traditionally been used for screening rather than enforcement purposes at or near static weighbridges. However, the emergence of a new generation of single-axle weighing fixed WIMS allows vehicles to be weighed at slow speed (typically < 5 km/h) and with sufficient weighing accuracy (< 1%) for enforcement purposes.

Types of WIMS: The most widely accepted and utilized WIM devices are described as follows:

i. **Piezoelectric sensor**: The sensor is embedded in the pavement and produces a charge that is equivalent to the deformation induced by the tyre loads on the pavement's surface. It is common to install two inductive loops and two piezoelectric sensors in each monitored lane. A properly installed and calibrated Piezoelectric WIM system can provide gross vehicle weights that are within 15% of the actual vehicle weight for 95% of the measured trucks.
ii. **Bending Plate**. The bending scale consists of two steel platforms that are typically 0.6 × 2 m, adjacently placed to cover a 3.65 m lane. The plates are instrumented with strain gages, which measures tyre load induced plate strains. The measured strains are then analysed to determine the tyre load. A properly installed and calibrated bending plate WIM system can provide gross vehicle weights that are within 10% of the actual vehicle weight for 95% of the measured trucks. **Plate 11.15** shows a typical bending plate high speed WIM device.

Plate 11.15 Typical High Speed WIM Device (Bending Plate)

iii. **Single Load Cell**. This device consists of two 3 × 3 m platforms placed adjacently to cover the 3.65 m monitored lane. A single hydraulic load cell is installed at the centre of each platform to measure the tyre load induced forces that are then transformed into tyre loads. A properly installed and calibrated single load cell WIM system can provide gross vehicle weights that are within 6% of the actual vehicle weight for 95% of the measured trucks.

11.6.1.1.3 ACCURACY OF WEIGHING SYSTEMS

The accuracy of weighing systems is primarily influenced by the following factors:

i. The error of the scale: This is the difference between the indication and the load placed on the platform of the weighing device. It is affected by such factors as temperature, eccentric load, tilted condition, repeatability, creep, and span stability.
ii. External factors: These are the influences which make a wheel or axle load lower or higher that it would be under perfect conditions. The perfect condition is: absolutely level site, all suspensions of the vehicle in an average, frictionless position, no braking, no vehicle oscillation. These external factors have nothing to do with the scale accuracy.

The accuracy of the weighing system will depend on the type of system used and the weighing method adopted.

The most accurate method of weighing is by the use of a multi-deck, static scale which is not affected by external factors which are produced by unfavourable characteristics of the vehicle and weighing site. In contrast, the least accurate method of static weighing is by the use of a single axle scale which weighs one axle at a time and for which the difference in height between the approach slab and the weighing platform is a critical factor.

11.6.1.2 SELECTION OF WEIGHBRIDGES

The selection of a weighbridge is largely determined by the purpose that it will serve. In this regard, the expected heavy vehicle traffic on a route is the most important determinant. The type of weighbridge scale and other equipment, the size of the buildings, parking areas, queuing space, the number of staff required, etc. are all determined by the current and future traffic. Even secondary design parameters, such as office space, furniture or office equipment, size of water and sewerage handling facilities, number of telephones, etc. are determined by the heavy vehicle traffic.

A weighbridge facility that is inadequate to cope with traffic on a busy route is ineffective and a waste of resources whereas a facility that is too large on a quiet road could be regarded as a white elephant. Similarly, a weighbridge kept open 24 hrs when there is hardly any night time traffic would be wasteful. **Table 11.18** provides guidance on the type of weighbridge that is most suited for handling various heavy vehicle traffic volumes on different road classes.

Table 11.18 Type of weighbridge in relation to design class

Weighbridge type	Design Class
Multi deck scale (both sides)	A
Multi deck scale (four decks)	B & C
Axle unit scale and single axle scale	D

Weighbridges on Class A and B roads are relatively costly due to the required equipment and site facilities, such as type of scale, the use of pre-screening WIMs, the size of parking and stacking facilities and the type office accommodation required to handle larger numbers of heavy vehicles and staff. These weighbridges will have the greatest impact on heavy vehicle overloading and thus justify larger capital and operational expenditure.

Weighbridges on Class C or D roads require smaller facilities and less equipment. Their value in terms of the potential impact on overload control has to be offset against the projected lifetime cost. Benefits not always directly measurable are difficult to quantify compared with the costs. There are however other benefits such as improved road safety and fairer competition in the freight industry that should be considered before deciding whether or not to construct a weighbridge facility on a Class C or D road.

11.6.1.3 LOCATION AND NUMBER OF WEIGHBRIDGES ON THE ROAD NETWORK

In order to ensure that the available resources for overload control are utilized in a cost-effective manner, it is important to adopt an appropriate strategy for deciding on the location and number of weighbridges that should be deployed along the road network. At one extreme, a strategy which seeks to eradicate overloading by locating numerous weighbridges along as many routes as possible will be extremely costly and un-cost effective.

In terms of deciding on an optimum number and location of weighbridges, the law of diminishing returns is very important to acknowledge (i.e., for every weighbridge added after a certain number, every additional investment has a smaller return until the return on that investment does not warrant any further investment.). In this regard, the addition of a new a weighbridge on the road network will only be economically viable if the capital, maintenance and operational costs are less than the savings in pavement damage due to overloading. The economic viability analysis should be conducted over the lifetime of the weighbridge network which requires the costs and benefits be converted to Net Present Values (NPVs).

Strategic matters that influence the location of a weighbridge include proximity to a port-of-entry (border post or a port) or generators of heavy vehicle traffic, such as industrial areas and whether the location is such that escape routes are minimized and that the greatest impact on reducing overloading can be achieved (i.e., where heavy vehicle traffic volumes are the highest and/or the extent of overloading is the highest. The influence of the strategic matters on the location of a weighbridge should be evaluated after the economic viability of the location has been established.

11.6.1.3.1 PERMANENT FACILITIES

Permanent truck weighing facilities have permanent scales and may have buildings. When these facilities are in operation, trucks are required stop. However, when Weigh-In-Motion (WIM) and Commercial Vehicle Information Systems and Networks (CVISN) capabilities have been installed, the driver may be notified to continue without stopping. The notification to continue may be through the use of signs or transponders.

The exact location of a truck weighing facility is generally controlled by topography, highway alignment, and geometrics. It is also desirable to select a site where adequate right of way is already available. Select the most economical site to minimize site preparation, expense, and impact on the environment. Water, electricity availability, and sewage treatment and disposal are other considerations for site selection. Additionally, use the following criteria:

i. Locate the facility such that its operation will not hinder the operation of the highway or other related features such as intersections and interchanges.
ii. To the extent feasible, eliminate options for truck traffic to bypass the weigh site.
iii. Base the site selection on the type and volume of trucks using the route.
iv. Evaluate the operational and safety performance of proposed weigh sites on expressway/motorway.

11.6.1.3.2 PORTABLE FACILITIES

Portable truck weighing facilities have no permanent scales or buildings. When these facilities are in operation, they operate in the same manner as permanent facilities.

Design portable truck weighing facilities located on two-lane and multilane roadways to best fit the existing conditions.

Locate the weighing facility such that its operation will not hinder the operation of the highway or other related features such as intersections.

11.6.1.3.3 SHOULDER SITES

Shoulder sites are used to pull a truck over for inspection and weighing with portable scales (see **Figure 11.28**). Design shoulder sites to best fit the existing conditions. Locate the weighing facility so that its operation will not hinder the operation of the highway or other related features such as intersections.

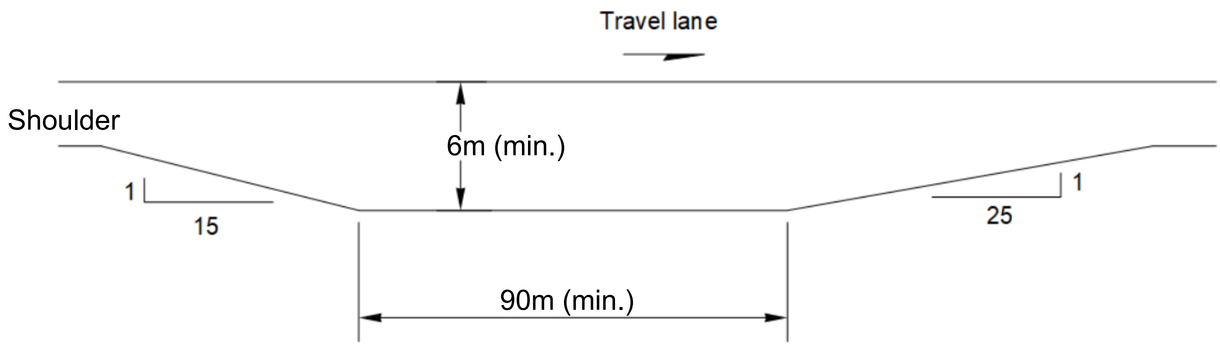


Figure 11.28 Typical layout of a Shoulder Site

11.6.1.4 WEIGHBRIDGE FACILITY LAYOUT AND INSTALLATION

11.6.1.4.1 WEIGHBRIDGE LAYOUT

The layout of weighbridges can vary considerably depending on a variety of factors including:

i. Purpose of the facility
ii. Prosecution of overloaded heavy vehicles
iii. Screening heavy vehicles only
iv. Screening and prosecution of heavy vehicles
v. Volume of heavy vehicles to be weighed

The types of facility that can be provided in relation to the factors indicated above include:

i. Full Traffic Control Centre (FTCC)
ii. Type 1 Traffic Control Centre (TCC 1)
iii. Type 5 Traffic Control Centre (TCC 5)
iv. Lay-by Control Centre (LCC)

A. Full Traffic Control Centre (FTCC)

As the name implies, a FTCC includes a full range of facilities to efficiently and effectively undertake an overload control process at minimum disruption to relatively large volumes of heavy vehicle traffic. Such a facility would normally operate on both sides of the road and would typically include within its operational system the following:

i. A high-speed weigh-in-motion (HSWIM) screening device in the main traffic lane
ii. A low-speed weigh-in-motion (LSWIM) screening device to confirm vehicles suspected to be overloaded as indicated by the HSWIM
iii. A static platform scale for accurately weighing axle and axle unit loads and total vehicle or combination mass for prosecution purposes

The typical layout and capacity of a FTCC for undertaking various aspects of the overload control process is given in Figure **11.29** and **Table 11.19** respectively.

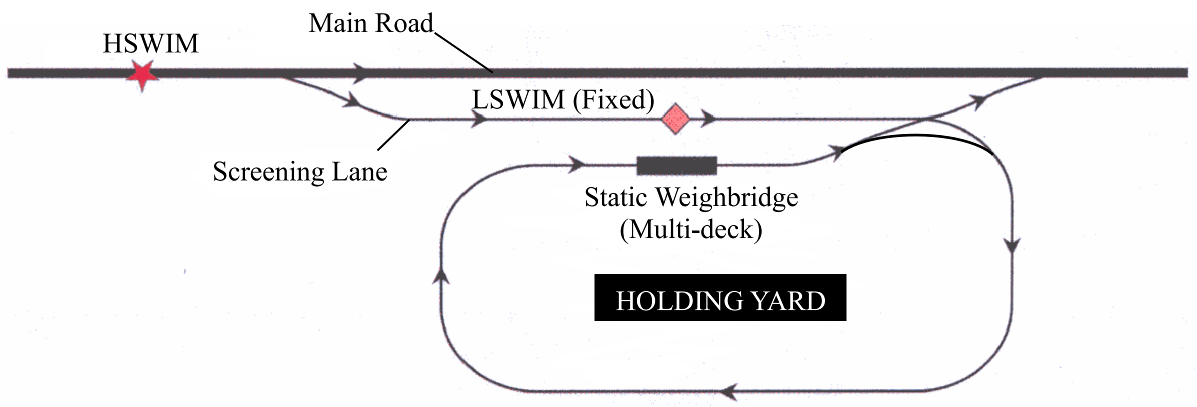


Figure 11.29 Typical Layout of a FTCC Facility

Table 11.19 Capacity characteristic of a FTCC facility

Activity	Typical Capacity
Screening capacity (veh/h)	200
Weighing capacity (veh/h)	50
Prosecution capacity (veh/h)	10
Max system Average Daily Truck Traffic (ADTT)	2,000

Source: Mikros Systems, South Africa

B. Type 1 Traffic Control Center (TCC 1)

A TCC 1 is essentially the same as a FTCC except that it operates on only one side of the road and the HSWIM in the main road is located on an internal screening lane. The drawback of this system is that any vehicles travelling in one direction that are identified as overloaded by the HSWIM must cross over the opposing traffic stream to be weighed. Thus, this type of facility is ideally suited for use where access across the road is provided by an interchange or where traffic flows are not so high as to frustrate the passage of vehicles across the road to the weighbridge.

The capacity of a TCC 1 is very similar to that of an FTCC (see **Table 11.19**). This type of facility is less costly to operate than an FTCC as only one team is required to control the station.

A typical layout is shown in **Figure 11.30**.

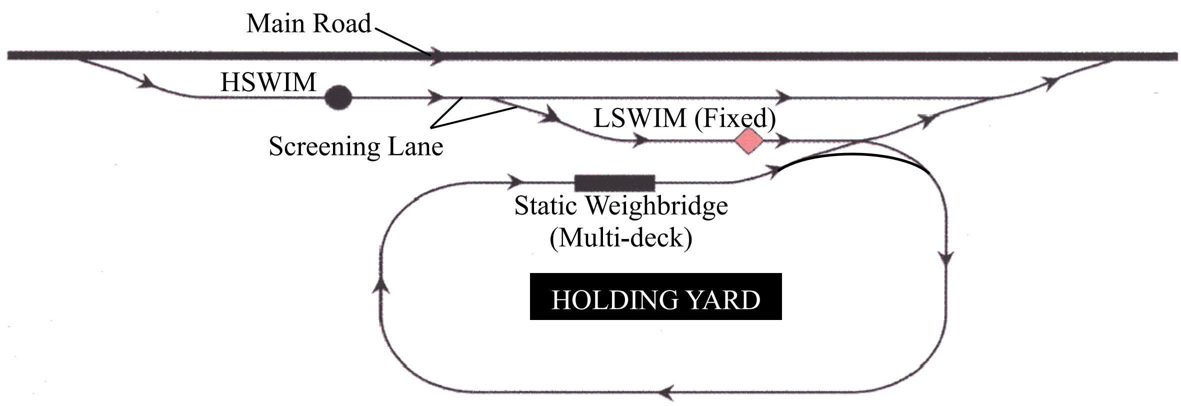


Figure 11.30 Typical Layout of a TCC 1 Facility

C. Type 5 Traffic Control Centre (TCC 5)

A type 5 TCC has fewer control facilities than either a FTCC or TCC 1 in that it does not have in-lane traffic screening but requires all heavy vehicles to leave the main carriageway and cross over a LSWIM. In this layout arrangement (see **Figure 11.31**) legally loaded vehicles can immediately continue with their journey, but overloaded vehicles must proceed to the static weighbridge for weighing and prosecution.

The capacity of a TCC 5 for undertaking various aspects of the overload control process is given in **Table 11.20**.

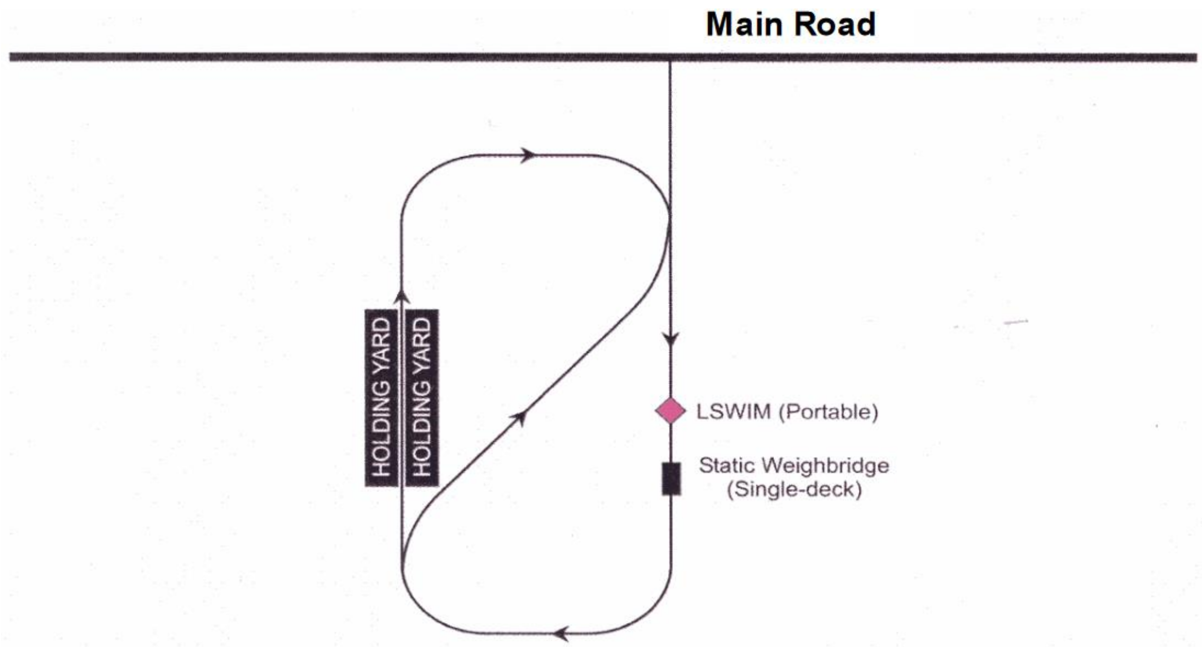


Figure 11.31 Typical Layout of a TCC 5 Facility

Table 11.20 Capacity characteristic of a TCC 5 facility

Activity	Typical Capacity
Screening capacity (veh/h)	40
Weighing capacity (veh/h)	15
Prosecution capacity (veh/h)	5
Max system Average Daily Truck Traffic (ADTT)	400

Source: Mikros Systems, South Africa

D. Layby Control Center (LCC)

A LCC facility essentially consists of a road lay-by at which either a static or mobile weighbridge is installed (see **Figure 11.32**). The facility comprises a suitably constructed level concrete platform adjacent to the road where the weighbridge is installed (or in the case of a mobile vehicle scale – with provision for easy installation of such a scale. The installed weighbridge may be operated in conjunction with a HSWIM as a screening device.

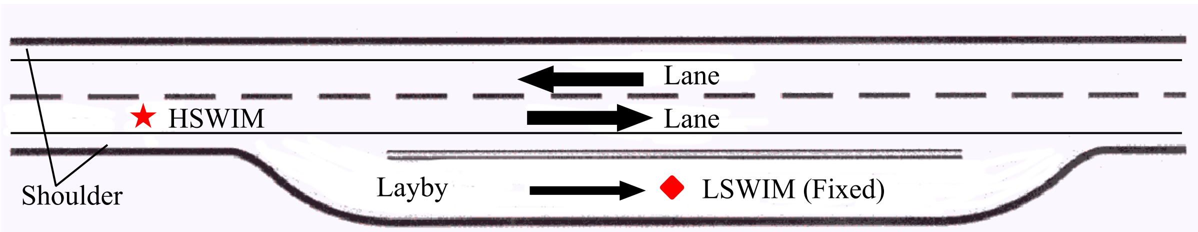


Figure 11.32 Atypical layout of layby with HSWIM screening device

11.6.1.5 WEIGHBRIDGE INSTALLATION

It is critically important that the weighbridge manufacturer's specification for installation of the weighbridge is rigidly adhered to during construction.

Scales can be installed either in a shallow pit, a deep pit or above ground. Most scale manufacturers offer any of the three options.

i. **Above ground installations** are advantageous because they are easy to keep clean, easier to maintain and cheaper to install. Potential problems due to drainage are eliminated. However, they are vulnerable to damage – either maliciously or accidentally – and are therefore not favoured. They also require up and down ramps to get to the scale and make operations more difficult.

ii. **Shallow pit installations** are only recommendable in places where drainage of the scale pit would be problematic. Keeping the scale pit clean is very difficult and as a result is neglected.

ix. **Deep pit installations** – deep enough to allow a technician to stand upright below the scale deck – are favoured because the sensitive mechanisms are protected, maintenance is easier than for the shallow pit and the scale is constructed at ground level. If not properly designed, drainage can be a problem.

11.6.1.6 GEOMETRIC DESIGN REQUIREMENTS

A. Design vehicle

The designer shall determine the design vehicle which should be the largest Multi-Combination Vehicle (MCV) currently permitted and envisaged to operate on the adjacent highway.

The design for the site will also need to consider oversize and over-mass (OSOM) vehicles along the route (typical OSOM length and width limits are 35m and 5.5m respectively although vehicles may exceed these dimensions with an approved permit).

B. Exit requirements

An exit ramp or deceleration lane is required to enable vehicles to safely diverge from the through-lane/s and enter the weigh site without impeding on the through-traffic. Sufficient deceleration length is required, to allow the design heavy vehicle to safely decelerate and stop, prior to any queued vehicles within the weigh site. Heavy vehicles require longer distances than cars do to come to a complete stop and, therefore, deceleration lengths need to be based on design heavy vehicle deceleration rates. The design of exit and entry ramps must be in accordance with **Section 9.4.5.8**.

C. Entry requirements

An entry ramp or acceleration lane is required to enable vehicles to safely exit the weigh station and re-enter the traffic stream on the through-road. Heavy vehicles require very long acceleration distances, which may not be practical to accommodate for a weigh station. Acceleration lanes should allow the design heavy vehicle to accelerate to a speed no less than 20 km/h below the mean free speed of the through-traffic. In the absence of local data, the mean free speed can be assumed to be equal to the sign posted speed limit. The design of the entry ramp must be in accordance with Chapter 9. Acceleration lengths must be in accordance with Chapter 8, using acceleration rates appropriate for the design heavy vehicle.

When the shoulders are designed to be used for deceleration and acceleration lanes, the minimum width is 3.65m with full pavement depth for the deceleration/ acceleration lane lengths.

D. Separation zone

The separation zone is the area located between the road edge line and weigh station. It includes the shoulder of the adjacent road and enough additional width to achieve a safety buffer between the weigh station and adjacent traffic lane. Depending on the traffic volume and site risk assessment, the separation zone shall be fitted with a barrier system and/or a traffic island (painted or raised). The separation zone needs to maintain the shoulder width requirements for the functional classification of the road that the weigh station is being constructed on.

An absolute minimum buffer width of 10m should be created between the weigh station facility and the mainline pavement. A wider separation is desirable.

E. Alignment of Main Line

Design values of main line alignment elements (horizontal curve radius, vertical gradient, vertical curve radius) around the weigh station should be gentler than the standard value (Refer to **Section 9.4.5.1**).

F. Grade and crossfall

Flat grade (0.4 - 5.0 %) is preferred. Longitudinal grade over the entire screening/weighing/inspection areas should be 0%. Refer to **Table 6.4** for values of crossfall over the weighing area and inspection area (depends on materials used).

G. Sight Distance

Safe sight distance for the entry/exit points should be provided (Refer to **Section 7.2.6**).

H. Storage Length for Scale

There should be sufficient space to queue trucks waiting for the scale without backing up onto the mainline. This distance will be based on the number of trucks on the mainline, length of trucks, expected hours of operation, and time required for actual weighing. For design considerations, the design vehicle can be assumed to be the T-21.

I. Violation Storage (holding yard)

A space should be provided to store trucks that are either overweight or which have failed the safety inspection. These areas should be designed to accommodate the T-21 design vehicle.

J. Traffic Control Devices

Adequate signing and pavement markings should be provided prior to and at the truck weigh station. These traffic control devices should be designed and placed in accordance with the MRH's Standard Details, Road Signs and Markings for Urban and Trunk Road (1991) and GHA Road Signs – Final Draft (2007).

K. Lighting

Section 12.8 provides information on lighting design.

L. Inspection Building

An inspection building should be designed for year-round use with sufficient space for computer operations, a service counter for permit issuances, and an emergency shower facility for hazardous-material removal. The inspection building should be in accordance with all local building codes.

M. Landscaping

The weigh station should be designed to minimize the effect on existing vegetation. The designer should also ensure that any new or existing plants will not affect the driver's sight distance to the weigh station or any critical point within the weigh station.

11.6.1.7 TECHNICAL SPECIFICATION FOR AXLE LOAD WEIGHING EQUIPMENT

A. FULL SIZE WEIGHBRIDGE DECKING/STRUCTURE

- The structure/decking must be made of high Ultimate Tensile Strength (UTS) Steel
- Length: 25m (5 Section Modules, 5m x 1.7m)
- Width: 3.4m
- Capacity: 150t
- Decking Plate: 12mm thick Checker Plate
- Suitable for 10 No. Load Cells mounting.

B. LOAD CELLS

- Rated capacity: 30ton.
- OIML R 60 Max n° intervals: Max 6000
- Rated output:2.0±0.002mv/v.
- Combined error：±0.03% F.S
- Non-repeatability：±0.02% F.S
- Non-linearity：±0.02% F.S

- Hysteresis error：±0.02% F.S
- Zero balance: ±1% F.S
- Creep error(30minutes): ±0.02% F.S
- Temperature effect on zero: ±0.02% F.S
- Temperature effect on output: ±0.02% F.S
- Maximum safe overload；150% OF R.C
- Ultimate safe overload；300% OF R.C
- Safe sideload: 10 %
- Accuracy: Plus, or Minus 0.01%
- Environment protection: IP68
- Input impedance: 750±10Ω
- Output impedance: 702±3Ω.
- Temperature: -40°C to +80° C 703±5Ω(cable≥25m)
- Transmission protocol: RS485
- Material: Structure in AISI 304 Stainless Steel
- 8 Strain Gauges High Precision - Repeatability – Reproducibility
- Heavy-Duty Protection At 45°Cone
- Mechanical Protection Cover
- Insulation Disc for Lightning Protection
- Overvoltage and Interferences Protection
- Copper Braided Earthing Cable
- Watertight IP68/IP69k Stainless Steel Connector

C. JUNCTION BOX

- Power Supply: 10-18vdc
- Maximum number of Load cell: 16
- Maximum number of divisions: 6000 for individual field tools, 3x3000 for multi-field tools, CE approval
- Transmission: RS485 cable, wireless
- Container: Stainless steel 280x200x120
- Protection rating: IP68 Structure in AISI 304 Stainless Steel

D. CONTROLLER

- 500 MHz Processor
- 128 MB Ram

- 128 MB Flash
- 10.4' Colour display
- 12 VDC power supply, 110-230vac with external power supply
- CE approved up to 6000 divisions-multi-division and multi-range.
- Versions (3x3000 or 2 x 4000)
- OIML Certification
- NTEP approved.
- Slots for optional cards - serial cards, I/O card, - field bus card, - audio card, sensors interface card - 3rd and 4th weighing machine.
- Connection (optional) - 2 off RS232/422/485 serial ports for connecting printers, repeaters, PCs etc.
- Compact Flash expansion slot
- SD card expansion slot
- Ethernet 10/100 output
- 12 VDC/5A power supply
- 2 off 24 VAC/DC Input/Outputs
- USB ports
- Earthing
- Weighing Machine 2
- Weighing Machine 1
- Conversion speed up to 100 readings per second
- Digital Load cell diagnostic management system
- Correction software for eccentric loads
- Axle by Axle weighing
- Group Axle weighing
- NTEP certification
- CE approved (Directive 2014/31/EU)

E. CAMERA

- Image sensor: 1/2.8” progressive scan RGB CMOS
- Varifocal, 10.9-29 mm, F1.7-1.7
- Horizontal field of view 29˚-11˚
- Vertical field of view 17˚-6.5˚
- Varifocal, Remote focus and zoom, P-Iris control, IR corrected.

- Day and night: Automatically removable infrared-cut filter, Minimum illumination
- Colour: 0.07 lux, at 50 IRE F1.7
- B/W: 0.01 lux, at 50 IRE F1.7, 0 lux with IR illumination on
- Shutter speed: 1/66500 s to 2 s
- Memory: 1024 MB RAM, 512 MB Flash
- Computer capabilities: Machine learning processing unit (MLPU)
- Video compression: H.264 (MPEG-4 Part 10/AVC) Baseline, Main and High Profiles H.265 (MPEG-H Part 2/HEVC) Main Profile, Motion JPEG
- Resolution: 1920x1080 to 160x90, Frame rate With Forensic WDR: Up to 25/30 fps (50/60 Hz) in all resolutions
- No WDR: Up to 50/60 fps (50/60 Hz) in all resolutions, Video streaming Multiple, individually configurable streams inH.264, H.265 and Motion JPEG, H.264 and H.265, Controllable frame rate and bandwidth
- VBR/ABR/MBR H.264/H.265, Video streaming indicator
- Power: Power over Ethernet IEEE 802.3af/802.3at Type 1 Class 3
- Typical: 8.5 W, max 12.95 W, 12–28 V DC, typical 7.8 W, max 12.95 W
- Connectors: Shielded RJ10BASE-T/100BASE-TX/1000BASE-T.
- 3.5 mm mic/line in, Terminal block for 1 alarm input and 1 output.
- (12 V DC output, max. load 25 mA IR illumination, nm IR LEDs
- Range of reach 80 m (262 ft) or more, depending on the scene, Data streaming Event data
- Operating conditions: -40 °C to 60 °C (-40 °F to 140 °F)
- Maximum temperature according to NEMA
- TS2 (2.2.7): 74 °C (165 °F)
- Humidity 10–100% RH (condensing)
- Storage conditions: -40 °C to 65 °C (-40 °F to 149 °F)
- Humidity 5-95% RH (non-condensing)
- Audio streaming Audio in, simplex, two-way audio via, edge-to-edge technology
- Audio encoding 24bit LPCM, AAC-LC 8/16/32/48 kHz, G.711
- PCM 8 kHz, G.726
- ADPCM 8 kHz, Opus 8/16/48 kHz
- Configurable bit rate
- Audio input/output

- External microphone input or line input, digital audio input, network speaker pairing.
- Analytics Object classes: humans, vehicles, Trigger conditions: line crossing, object in area
- Up to 10 scenarios Metadata visualized with color-coded bounding boxes.
- Polygons include/exclude areas, Perspective.
- Configuration, ONVIF Motion Alarm event

F. HEIGHT SENSOR

- Maximal Detection Range: Up to 30 m
- Range Resolution: 10 cm
- Range accuracy: ± 30cm (1σ)
- Beam divergence: 2 x 10 mRad
- Pulse Repetition Frequency (PRF): 250 Hz
- Output Interface Relay: (RS485)
- Wavelength: 905nm ± 10nm
- Operating Ambient Temperature: - 30°C to + 65°C
- Supply Voltage: 12 VDC ± 1 @ 150mA
- Electrical interface connector: D38999/24WA35PN
- Enclosure rating: IP-67

G. EXTERNAL DISPLAY

- Voltage： AC220V；50Hz
- Power: 10 Watt
- Display: 6 bits of high brightness LED
- Operating Temp： －20℃ to 60℃.
- Input signal： 20mA electric current loop, RS232 signal.
- Baud rate： 600, 1200, 2400, 4800, 9600, 19200bps
- Protection: IP67
- Character: 5 inch high
- Size: 750mm x 240mm x 63mm
- Advertise: 200 characters

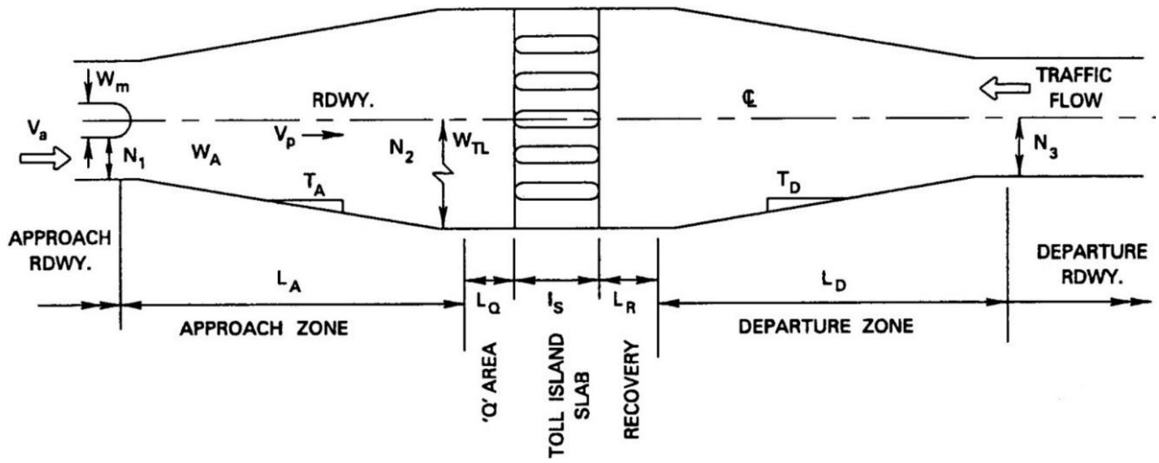
11.7 TOLL PLAZA

11.7.1 INTRODUCTION

A toll plaza (sometimes called toll gates, toll booths or toll barriers) is defined as the area where tolls are collected. This area starts where the approach roadway pavement widens, continues through the toll barrier or collection point, and ends where the pavement returns to the normal roadway cross section. Where no pavement transitions exist, the physical limits of a toll plaza are defined by the beginning of the approach to the toll island, ending beyond the physical terminus of the toll island.

Toll gates are provided on toll roads to collect tolls from users. The layout, size and toll collection system of toll gates must be appropriate according to the planned traffic volume.

Figure 11.33 illustrates a typical toll plaza, defining various areas or elements that constitute a plaza. These areas typically include the approach (transition) zone, the queue area, the toll island or barrier, a recovery area, and the departure (transition) zone. In addition, the plaza encompasses all advance toll plaza—related signing and transitional lighting.



V_a: Roadway approach
V_p Plaza speed (Posted)
N_1 Number of approach travel lanes
N_2 Number of toll lanes (under normal conditions in approach direction)
N_3 Number of departure travel lanes
W_m Width of highway median (if divided)
W_A Width of approach travel lanes including shoulders
W_{TL} Total width of typical toll lanes (N_2)
T_A Approach taper
T_D Departure taper
L_A Length of approach (transition) zone
L_Q Length of tangent queuing area /approach measured between transition zone and toll island slab
I_s Length of Toll Island Slab
L_R Length of Departure (Slab)/Recovery Zone
L_D Length of Total Departure (Transition) Zone

Figure 11.33 Toll Plaza terminology

11.7.2 CONFIGURATIONS OF TOLL PLAZAS

Several basic toll plaza configurations have evolved over the years. The configurations are largely determined by traffic demand, the type of toll system, methods of toll collection, the toll rate schedule, and the physical and environmental constraints of the site. **Figure 11.34** depicts some common toll plaza configurations: a two-way barrier, a split barrier, and express Electronic Toll Collection (ETC)/High-Occupancy Vehicle (HOV) lanes with conventional off-line toll plazas.

11.7.3 TOLL PLAZA CATEGORIES

In general, toll plazas fall into two categories:

i. Toll gates which are installed on mainline and,
ii. Toll gates which are installed at entrances/exits slips or ramp.

Both can be designed to handle one-way or two-way toll collection. The mainline plaza is a toll lane or series of toll lanes running perpendicular to the travelled roadway. The selection of toll plaza configurations depends on the toll system that is adopted.

Both mainline and ramp plazas can have a split design (i.e., two individual plazas, each serving different directions of travel, usually to reduce the right-of-way required or to fit available space). However, this design eliminates the possibility of using the centremost lanes to collect tolls in either direction, a practice referred to as reversible-lane operation. This practice is frequently used where there are directional traffic peaks at a conventional two-way plaza.

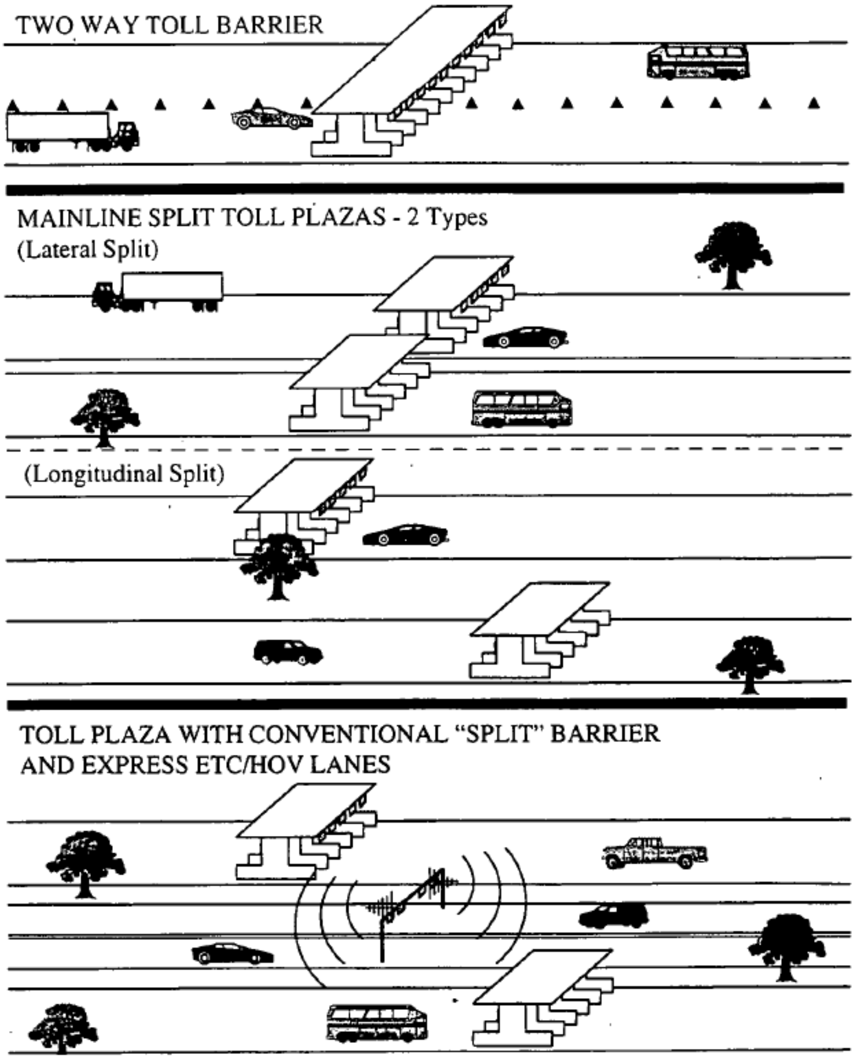


Figure 11.34 Typical toll plaza configuration

11.7.4 BASIC TOLL PLAZA ELEMENTS

Whether designing a toll facility with a single toll booth serving both directions of travel or a multilane main line plaza, certain basic elements are common to many conventional toll plazas:

i. A toll collection point (typically with a booth or automatic coin/ticket machine in each lane),
ii. Toll islands, and
iii. A canopy or protective overhang.

11.7.5 FACTORS AFFECTING TOLL PLAZA DESIGN

The design of a toll plaza is determined by:

i. traffic demand
ii. customers
iii. types of toll systems

iv. methods of toll collection
v. toll rate schedules, and
vi. plaza location.

These factors dramatically affect the configuration, size, and layout of the plaza. In addition, operation and maintenance of the plaza and its staffing are functions of the collection methods and toll system choice. These factors help define spatial needs for plaza support facilities. Aside from quantifiable factors, a number of functional issues dictate toll plaza design:

i. Capacity options,
ii. Environmental issues,
iii. Enforcement,
iv. Security,
v. Accessibility, and
vi. Configuration of payment methods.

11.7.5.1 TRAFFIC DEMAND AND THE CUSTOMER

When designing a toll plaza, it is essential to know not only peak traffic demand, including vehicle mix, but also trip purpose and nearby land use, particularly where special traffic generators are nearby. These facilities tend to generate unusual peak traffic demands. A profile of daily, weekend, and holiday users should be developed. The traffic profile should include hourly volumes, frequency of use, and traffic classification. This information is vital in determining methods of collection, lane configurations for payment, and, ultimately, the number of toll lanes needed. If an hourly profile is not feasible, peak hour or peak traffic demand estimates for the design year—typically projected for 20 years from the Start of design—can be used.

11.7.5.2 TYPES OF TOLL SYSTEMS

There are primarily two types of toll systems: open and closed.

i. **Open Toll System**

   An open toll system is one in which not all patrons are charged a toll. Typically, such a system is used in an urban area where local traffic is permitted to travel free, while through traffic is charged. In such a system, the toll plaza generally is located at the edge of the urban area, where a majority of long-distance travellers are committed to the facility, with a minimum likelihood of switching to a parallel free route, or at the busiest section of the toll way.

ii. **Closed Toll System**

   In a closed toll system, patrons typically pay their share of the facility's operating and maintenance costs, plus debt service based on distance of travel on the facility and type of vehicle. Thus, there are no "free-rides."

a. A closed ticket system

In a closed ticket toll system, plazas are located at all entry and exit points, with the patron receiving a ticket upon entering the system. Upon exiting, the patron surrenders the ticket to the collector and is charged a prescribed fee from the point of entry to the point of exit, based on the vehicle's classification and distance travelled.

b. A closed Cash system

With a closed cash system, the premise is similar except that a cash toll typically is paid upon exiting and at strategically placed mainline plazas along the toll road. This toll also is based on vehicle class and density as well as the average vehicle distance travelled.

A bridge or tunnel facility toll plaza usually is defined as a closed cash toll system because all users are charged a toll either in both directions or one way. In the one-way configuration, the toll usually is twice the two-way charge because the patron usually has little flexibility in selecting an alternate free route.

The major differences between the open and closed systems are operating cost and the initial investment in toll plazas. The closed system, whether ticket or cash, has more collection points and more collectors than an open system.

11.7.5.3 METHODS OF TOLL COLLECTION

There are three methods of toll collection:

i. **Manual collection**

Manual collection requires a toll collector or attendant. Based on vehicle classification as defined by the facility's toll schedule, and usually classified by the collector, a cash toll is received by the collector.

ii. **Automatic toll collection**

Automatic toll collection is based on the use of automatic coin machines (ACMs). These accept both coins and tokens issued by the operating agency. Depending on the toll rate, the use of automated coin or token collection instead of manual collection reduces transaction and processing time as well as the operating cost.

iii. **Electronic toll collection**

Electronic Toll Collection (ETC) is a system that automatically identifies a vehicle equipped with a valid encoded data tag or transponder as it moves through a toll lane or checkpoint. The ETC system then posts a debit or charge to a patron’s account, without the patron having to stop to pay the toll. ETC increases the lane throughput because vehicles need not stop to pay the toll. An example is shown in **Plate 11.16** and **Plate 11.17**.

Plate 11.16 Example of Electric Toll Collection System

Plate 11.17 Example of Electric Toll Barrier

11.7.5.4 TOLL RATES AND SCHEDULE

Toll rates posted in a toll schedule are based on many considerations, including:

i. the potential for traffic diverting to free roadways in the travel corridor
ii. cost of the project
iii. type of patrons
iv. operation and maintenance costs
v. reserve requirements, and

vi. debt service coverage on bond principal and interest.

The toll rates and schedule, in turn, dictate the methods of collection and, when compared with various peak traffic demands, determine the number of toll lanes to be provided.

For open and closed cash systems, toll rates usually are based on vehicle class and the number of axles. For audit purposes, classification equipment is placed in the toll lane to measure axle counts and determine vehicle separation to ensure that the proper toll rates are charged.

11.7.5.5 PLAZA LOCATION

The location of a toll plaza is determined by the type of toll system. In a closed ticket system, plazas are located at each point of entry and exit. In a closed cash system, plazas are located at ramp entry and exit points and along the mainline, and a main line plaza is used at bridge and tunnel facilities. In an open system, main line and ramp plazas are strategically located primarily to intercept through traffic and are placed where a majority of this traffic is least likely to divert to alternative free routes.

The plaza should be accessible to and from the toll facility's mainline plaza or from a local road adjacent to the mainline or a ramp plaza. This will facilitate access by personnel and reduce their round-trip travel. Moreover, it is preferable to locate a plaza where it has easy access to public utility connections to provide improved system integrity and to facilitate construction.

The plaza should be located away from residential areas and other sensitive air and noise receptors and where lighting spillover may be adversely received.

The majority of facilities are located on a tangent segment of roadway or on a gentle curve with adequate sight distance for the roadway design speed.

The selection of a site involves a number of design decisions and revenue considerations. These include the following:

i. Available rights-of-way
ii. Topography
iii. Environmental concerns and impacts
iv. Feasibility of potential abatement measures
v. Number of toll lanes and methods of toll collection
vi. Space for potential reversible lane operation and roadway transitions
vii. Support facilities such as a plaza administration or utility building and parking for employees.

11.7.6 CALCULATION OF NUMBER OF TOLL PLAZA LANES

The standard number of toll gate lanes is obtained from **Table 11.21** based on the traffic volume of entry, average service time and level of service (average number of waiting vehicles). In cases where outbound and inbound carriageways (entrance and exit) are separated, the number of lanes should be designed in respect to the peak traffic volume of each direction.

The traffic volume, average service time, and service standard should be based on the following standards:

i. **Standard hour traffic volume**

   The design hourly volume (DHV) that should generally be used in design is the 30th highest hourly volume of the year, abbreviated as 30 HV, which is typically about 8-12 and 12-18 percent of the ADT on urban and rural roads respectively. Refer to **Section 5.5** for computation of DHV.

ii. **Service time**

   In the case of manual toll collection facility, the service time is 6 seconds at the entrance and 14 seconds at the exit, while in the case of a uniform toll system, the number of lanes is calculated assuming a standard time of 8 seconds. However, if actual times are expected to be very different from these values, another average service time should be used.

iii. **Level of service**

   The level of service is referred to as the average number of waiting vehicles. The desirable and absolute number of waiting vehicle(s) are one (1) and three (3) respectively. The absolute value may be adopted due to terrain or other reasons provided that traffic is not hindered.

iv. **Throughput**

   It is the number of vehicles passing through the toll plaza over a short period of time, usually 1 hour. It is determined by the passing speed, time of vehicles, density of traffic flow, etc. It is evident that the longer vehicles wait at the toll plaza, the lower the throughput. Because we need to consider the construction cost of the toll plaza, when the waiting time is directly proportional to the queue length, the throughput may be determined by the queue length, and the longer the queue length is, the longer vehicles will wait and the lower the throughput will be. Vehicles mainly queue at two points. The first is at the arrival section. The more the tollbooths, the shorter each queue and the larger the throughput will be. The second point is at the departure area. The more the tollbooths, the more vehicles enter into the exit area, the longer the queue and the waiting time will be, hence the smaller the throughput will be. If the exit queue area cannot accommodate the queue, it will cause the traffic jam in entrance area, so as to reduce the throughput. Therefore, too few tollbooths may result in the increase of queue length in entrance area, while too many tollbooths may lead to the increase of queue length in exit area, both of which will reduce the throughput. The throughput is also dependent on the toll collection method as shown in **Table 11.22**.

Table 11.21 Relationship between Number of Lanes, Service Time, Average Number of Waiting Vehicles, and throughput (pcu/h)

Service time (s)		6		8		10		14		18		20	
Average no. of waiting vehicles (q/s)		1.0	3.0	1.0	3.0	1.0	3.0	1.0	3.0	1.0	3.0	1.0	3.0
		Throughput (pcu/h)											
Number of lane(s)	1	300	450	230	340	180	270	130	190	100	150	90	140
	2	850	1040	640	780	510	620	360	440	280	350	250	310
	3	1420	1630	1070	1230	850	980	610	700	480	550	430	490
	4	2000	2230	1500	1670	1200	1340	860	960	670	740	600	670
	5	2590	2830	1940	2120	1550	1700	1110	1210	860	940	780	850
	6	3180	3430	2380	2570	1910	2060	1360	1470	1060	1140	950	1030
	7	3770	4020	2830	3020	2260	2410	1620	1720	1260	1340	1130	1210
	8	4360	4630	3270	3470	2620	2780	1870	1980	1450	1540	1310	1390
	9	4960	5220	3720	3920	2980	3130	2130	2240	1650	1740	1490	1570
	10	5560	5820	4170	4370	3330	3490	2380	2490	1850	1940	1670	1750
	11	6150	6420	4610	4820	3690	3850	2640	2750	2050	2140	1850	1930
	12	6740	7020	5050	5270	4040	4210	2890	3010	2250	2340	2020	2110
	13	7340	7620	5510	5720	4400	4570	3150	3270	2450	2540	2200	2290
	14	7940	8220	5954	6170	4760	4930	3400	3520	2650	2740	2380	2470
	15	8530	8820	6400	6620	5120	5290	3660	3780	2840	2940	2560	2650

Table 11.22 Toll Lane Throughput

Method	Explanation	Car Throughput (vph)	HGV Throughput (vph)
Electronic Toll Collection (ETC)	Transponders, contact-less reading of bar code stickers/proximity cards, Tags (Low speed automatic -vehicles reduce speed, barriers lifts when transponder/card/tad is read)	450 - 900	300 - 500
Card payment	Credit, Debit or Charge Cards (Vehicles stop- barrier lifts when card is passed through reader and has been verified - receipt may be given) Note: Throughput will reduce if driver is required to enter a PIN to verify the transaction	200 -350	150 - 250
Coin Bin	Cash machines/Coin baskets (Vehicle stop-barrier lifts when cash has been verified -change and receipts may be given)	300 - 500	200 - 350
Manual	Card/cash/voucher/token (Vehicle stop-barrier opened by attendance, change and receipts may be given)	250 - 500	200 - 350
Note: 900 vph =4 seconds per transaction 450 vph =8 seconds per transaction 300 vph =12 seconds per transaction 200 vph =18 seconds per transaction			

11.7.7 FORMULATION OF TOLL GATES FACILITY PLAN

11.7.7.1 STANDARD DESIGN PERIOD FOR TOLL GATES FACILITIES

The number of toll gate facilities required in the first year, should be the necessary number of facilities in respect to the design period indicated in **Table 11.23**. A stagewise development should be considered for toll facility because requirement may change in accordance with the increase of traffic volume by road network development. The difference of standard design period should reflect the stagewise development. However, if the planned toll road is very simple or investment risk is taken by investors, it may be considered to design in full scale from the first year. 20 years can be applied as typical case.

Table 11.23 Standard Design Period for Toll Facilities

Toll Facility	Standard design period (years)	
	Stage wise development case	Typical case
Right Of Way (ROW) for toll plaza	15	20
Toll barrier plaza layout	10	20
Toll island and pavement	10	20
Toll gate building	10	20
Toll booth and toll collection facility	5	20

11.7.8 TOLL PLAZA DESIGN STANDARDS

11.7.8.1 COMPOSITION OF LANES AND BOOTHS

Figure 11.35 shows the standard cross section of a Toll plaza.

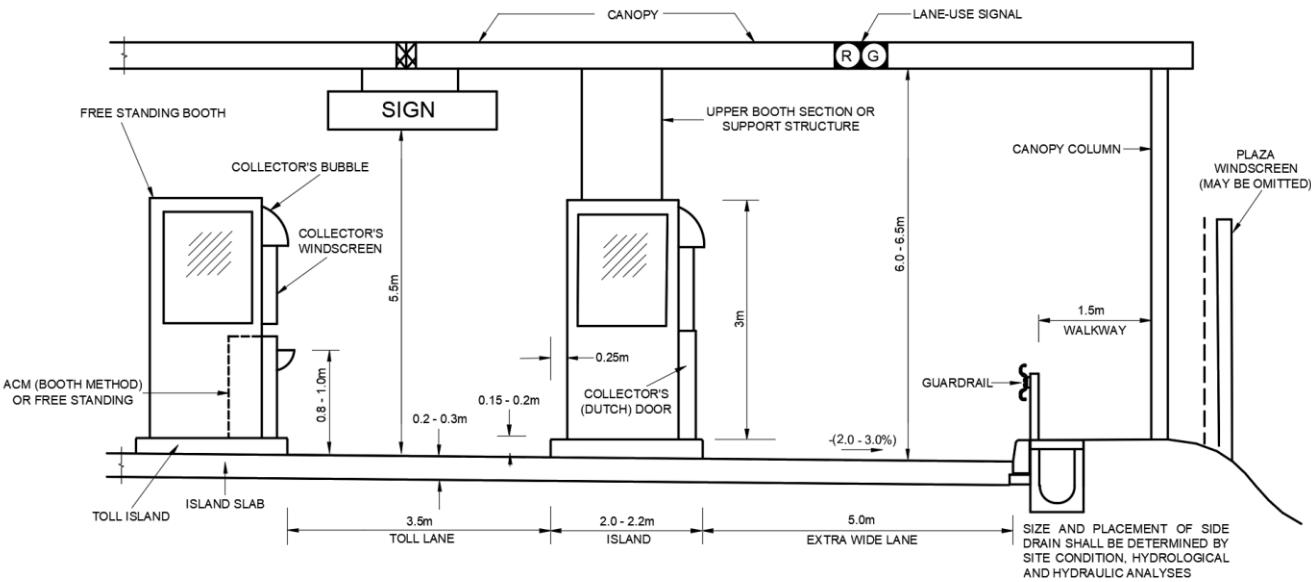


Figure 11.35 Typical Cross Section of a Toll Plaza

A typical Toll Plaza has the following minimum design standards:

i. Lane width inside the toll gate should be 3.50m.

ii. Even if the necessary number of lanes in one (1) direction is one (1) lane, a minimum of two (2) lanes should be adopted, and an island and booth should be established on the left side. In that case a lane width of 5.0m should be provided for the outer lanes to cater for the passage of exceptional vehicles, particularly wide vehicles and those carrying dangerous goods.

iii. Dimensions of a toll island should be as indicated in **Table 11.24**.

Table 11.24 Toll Island Dimensions

Length (nose to nose) (m)	Width (m)	Kerb height (m)
35.0	2.0 - 2.2	0.15 - 0.20

iv. The building clearance of toll collection house should be as indicated in **Figure 11.35**.

v. The minimum construction clearance should be 5.5m.

vi. Minimum level of service (LOS) of C. In heavily congested sections, conditions may necessitate acceptance of lower LOS.

vii. Absolute minimum length of the queuing area (L_Q) is 30m, with 85m being the desirable minimum. The desirable length is based on 11 passenger cars in a toll lane; the length must be adjusted to accommodate the truck percentage.

viii. The departure recovery zone (L_R) should be equal to at least 60m and preferably 90m, a length expected to allow sufficient driver re-orientation, acceleration, and initial merge distance after exiting the plaza.

ix. Full access control.

The toll office should be located on the exit side of the toll gate unless the entrance side is more advantageous in terms of terrain, connecting road, site utilization and other conditions. Consideration should be given to the provision of the toll and administration offices in the same building.

11.7.8.2 GEOMETRIC REQUIREMENTS

i. A straight horizontal alignment is desirable for the toll plaza area. However, in cases where it is unavoidable, in the case of a main line toll plaza, this should be based on **Section 9.4.5.1**. Minimum radius in the case of interchange toll plaza should be 200m.

ii. The vertical curve radius of the toll plaza area, in the case of a mainline toll barrier should be based on **Section 9.4.5.1** and should be 800m or more in the case of an interchange toll barrier or 700m or more in special cases.

iii. Vertical gradient in the toll plaza area should not be greater than 2.0%. In special cases a maximum gradient of 3.0% is recommended. This should be a minimum of 50m before and after the toll gate centreline, however, in the case of a main line toll barrier with a design speed of 80km/h or more, it should be 100m before and after the toll gate centreline.

iv. As a standard, the toll plaza should have a cement concrete pavement with a crossfall of 2%.

v. On a main line toll barrier, the distance from the toll gate centreline to the nose of the central median must be sufficient so as to safely accommodate turning vehicles using the reversible lane(s).

vi. On an interchange toll barrier, the distance from the toll gate centreline to the ramp branch point (gore nose) should not be less than 75m as shown in **Figure 11.36**.

vii. The approach and departure tapers (L_A and L_D) should range between 1:8 and 1:17, with 1:10 being desirable.

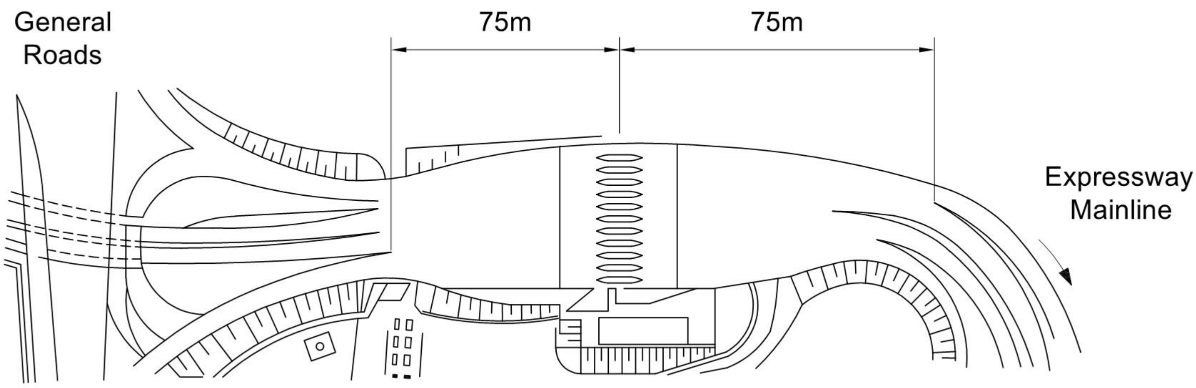


Figure 11.36 Distance from Toll Gate to Ramp Gore Nose

11.7.8.3 TRAFFIC SIGNALS

Traffic signals should be considered for use in each toll lane to instruct drivers to stop, so as to pay the toll, and to allow them to proceed once payment has been verified. In such cases, approved signal heads that carry only red and green signals should be used.

In addition to traffic signals, rising-arm barriers may also be used to prevent vehicles from proceeding before payment has been verified.

If both traffic signals and barriers are used, control of the two systems should be linked so that they cannot give conflicting messages to drivers. Each system should also be capable of operating independently, but only at times when the other system is out of use.

The optimum position and height of the traffic signals should be determined in conjunction with the other toll island equipment and vehicle types using the lane, when categorisation is used. Where necessary, approval should be sought for proposed departures from regulations.

Hazard warning beacons should be provided on the approach end on toll islands for use during periods of poor visibility, particularly when traffic volumes are low. The use, frequency, intensity and optical characteristics of such lights should take account of the potential impact such lights may have on queuing drivers. Examples of hazard warning beacons are given in **Plate 11.18**.

Plate 11.18 Hazard Warning Beacons Fixed to Concrete Bollard

11.7.8.4 TRAFFIC SIGNS

Toll plazas and their approaches require a greater density of signing than usual. The density of signing increases where the toll plaza incorporates dedicated lanes for specific vehicle types, e.g., Motorcycles or Abnormal loads. Signs are required to give advice, information and instructions to motorists.

Traffic signs that may be erected on public roads are prescribed in the MRH's Standard Details, Road Signs and Markings for Urban and Trunk Road (1991) and GHA Road Signs – Final Draft (2007).

Non-prescribed variable message signs will be required above each toll lane to indicate the payment options that are permitted in each open toll lane and to indicate which lanes are open to traffic.

Signs may be considered in two categories: advance signs and toll plaza signs. The advance signs should be located to allow motorists who are unable or unwilling to pay the toll the opportunity to avoid the tolled facility and to take an alternative route. It may be necessary to provide advance signs on other nearby routes as well as on the main route approaches.

Toll plazas require high levels of driver concentration. Advance signing is needed to prepare drivers and provide them with tolling information. This may include some of the following:

peak period and off-peak period start and finish times, vehicle category, the applicable toll level and the accepted payment methods.

Advance signing is necessary to indicate the accepted methods of toll payment. Drivers entering the toll plaza area should be prepared to approach a correct toll lane at a reasonable speed. Clear, consistent and distinctive signing, using a combination of symbols, colours and legends should be used on the advance signs and repeated over the toll lanes with the aim of avoiding late or excessive lane-changing by vehicles.

From the downstream end of the Departure Zone, for the distance along the route in the direction of travel, as shown in **Table 11.25**, there should be no signs on the nearside verge and central reserve or on portal or cantilever gantries.

Table 11.25 Distance from end of departure zone for which there should be no signs

Number of lanes on mainline	Distance (m)
2	380
3	580
4	580

For further guidance and advice on the correct use of traffic signs and traffic signals and control equipment and their maintenance see **Section 12.4** and Manual on Uniform Traffic Control Devices (MUTCD) by the Federal Highway Administration (FHWA) of the United States Department of Transportation.

11.7.8.5 ROAD MARKINGS, STUDS AND TRAFFIC CONES

The road markings help in channelizing the movement of vehicles in a toll plaza. These comprise transverse, longitudinal, diagonal, symbols, and text markings.

Lane lines help to ensure that available carriageway space is used to its maximum capacity. In helping vehicles to maintain a consistent lateral position, they also offer safety benefits and should be used wherever practicable. In the approach zone, lane lines are used to assist drivers in lining themselves up with their chosen booth: see **Plate 11.19**.

Plate 11.19 Lane Markings Within Approach Zone

Chevron markings should be used at the approach to and departure from toll booth islands. They are intended to aid the separation, and merging, of streams of traffic.

'Keep Clear' markings may be considered for use in areas approaching the toll booth in order to ensure that vehicles only enter the area when the vehicle in front has been processed.

Markings should minimize the effects of late lane changes in advance of the toll plaza. Traffic cones may be used temporarily to assist traffic flows, e.g., if one lane becomes blocked or inoperable.

Further measures may also be considered within, and in advance of, the toll plaza area to improve driver awareness and to promote appropriate behaviour.

11.7.8.6 LIGHTING

Toll plaza lighting needs due consideration because user should be able to spot the existence of a toll plaza from a distance at night. The specifications for different types of lighting provided at a toll plaza are listed below:

i. Highway Lighting: Lighting in 100m length on both sides of toll plaza shall be provided to enhance safety and to make drivers conscious of approaching a toll gate.
ii. Canopy Lighting: Higher level of illumination shall be provided at the tollgate and toll booth locations.
iii. Interior lighting: It should be 200-300 Lux as per IS:3646 (Part-II)
iv. High Mast Lighting: IS:1944 (Part I & II) recommends 30 Lux of average illumination on road surface. 30m height of mast is considered suitable.

Refer to **Section 12.8** for details.

11.7.8.7 CONTROL OF VEHICLE SPEED

The control of vehicle speed is essential for the safe operation of a toll plaza. All vehicles should expect, and be prepared, to stop at a toll plaza; this may be within the toll lane or at a queue in advance of the toll lane. The mechanism for achieving this will depend upon the speed of traffic on the approach road.

A risk assessment should be undertaken to establish whether the maximum permissible speed of vehicles within the Approach Zone should be restricted to a level below the permitted speed on the approach road. The designer should seek to ensure that traffic is required to decelerate steadily as it approaches the toll lanes.

On high-speed roads, e.g., motorways, consideration may be given to the introduction of a 'buffer zone' speed limit (i.e. an intermediate speed limit) on a suitable length of the approach road. Types of traffic calming measures are discussed in **Chapter 10**.

11.7.8.8 DRAINAGE

The standard minimum crossfall is 2.5% and the minimum longitudinal gradient recommended is 0.4% to allow for water flow along the roadside edge channel.

For wide carriageways the most direct drainage flow paths are realised when the longitudinal gradient is zero. Therefore, low longitudinal gradients can be acceptable, provided that standard crossfalls are maintained and a continuous edge drainage system is provided.

In setting the vertical and horizontal geometry of a plaza, water should not be allowed to accumulate in the expanses of the approach and departure zones nor in the toll lanes.

Within the transition zones of the toll plaza, the use of high performance and heavy duty longitudinal linear drainage channel systems, in combination with transverse drains need to be considered. It is likely that these drains will require more regular maintenance, but they are likely to provide the most suitable system.

Where a longitudinal drainage system is to be used, the line of the system should coincide with the line of an actual, or theoretical, lane divider, i.e. the line of the drainage channel will appear, to the motorist, as a lane marking (**See Plate 11.19**). The designer should also take into account the effect of longitudinal drainage systems on powered two-wheel vehicles.

Maintenance of the drainage system may, at times, render certain toll lanes unavailable. Care should be taken in determining the layout so that maintenance can be carried out without significantly affecting off peak capacity. See **Section 13.4** for details on pavement drainage.

11.7.8.9 PAVEMENT

Pavements do not usually fail suddenly but gradually deteriorate in service. In the case of the surface course, loss of skid resistance can be equivalent to failure.

Concrete should be considered as the pavement material for the length of the toll island for reasons stated below:

i. to prevent pavement rutting caused by high flows of slow, Heavy Goods Vehicle (HGV) movements using the same, narrowly defined wheel track through a toll lane constraint;
ii. to reduce damage caused by the possible discharge of oils, fuel, and grease;
iii. to prevent surface layer undulation created by vehicle braking and acceleration impact, particularly that of HGVs and Public Service Vehicles (PSVs);
iv. to ensure the integrity of in-lane toll equipment such as loops; and
v. to facilitate a simpler and effective maintenance regime (washing).

Run-on concrete slabs should also be considered for areas constructed on either side of the toll lane and toll island slab. A run-on slab acts as a structural element seated on the island slab at one end and the adjacent pavement construction at the other end to provide a transition between the approach and departure zone pavements.

11.7.8.10 CANOPY

The canopy is a roof structure built over the toll booths and toll lanes. The benefits of a canopy are that:

i. it defines the location of the toll plaza for motorists.
ii. it serves as a mounting frame for signs, variable message signs (VMS), lighting, lane signals and ETC antenna.
iii. it provides support for overhead signs and for security and safety devices.
iv. it provides a route for a variety of services.
v. it provides the motorists and toll attendants with shelter from the elements whilst paying.

The canopy and its attached signs and equipment should be of adequate height to comply with the headroom standard set out in **Figure 11.35**.

On highways which are designated high load routes, at least one lane may need to be sited without the benefit of canopy cover.

A canopy should be designed as a structure over a highway and will be subject to technical approval by the Road Agency. The designer should establish and agree with the Technical Approval Authority (TAA) the appropriate design parameters for the form, location and use of the proposed canopy and its supporting structure. Primary design considerations to observe include:

i. determining the functions to be fulfilled by the canopy.

ii. aesthetic consideration and architectural requirements.
iii. canopy and all attached signs and equipment to be of adequate height to comply with the headroom standard set out in **Figure 11.35**.
iv. prevailing wind direction and path of the sun.
v. canopy support structure impact design loading.
vi. canopy impact design loading.

11.7.8.11 TOLL BOOTHS

In order to provide an efficient, comfortable and safe working environment for staff in toll booths, the design of the internal layout of the booth should be based on sound ergonomic principles.

A health and safety evaluation of the proposed facilities should be undertaken to ensure compliance with the necessary legislation and workplace regulations.

Toll booths should be equipped to deal with minor vehicle fires.

General points to consider are noted below:

i. size
ii. construction
iii. access
iv. lighting
v. staff welfare
vi. ventilation
vii. air quality monitors
viii. safety
ix. security
x. signing
xi. uninterrupted power supply
xii. other amenities; and
xiii. environmental issues

Proprietary booths are available that address many of the above issues. Care should be taken if a bespoke design is to be developed.

11.7.8.12 TOLL LANE ACCESS TUNNEL

Toll booth access tunnels should be given consideration as they are able to provide several operational benefits as well as increased safety for toll staff. The benefits of a tunnel are that it provides:

i. staff access to the toll booths via a stairwell. (This may be limited to every second or third toll island).
ii. a route for toll booth services with access for maintenance.
iii. a location for ACM coin vaults where they can be safely handled.
iv. a secure route for toll revenue from the booth to the administration building.

11.7.8.13 PROTECTION DEVICES

Provisions for protective safety devices on the approach to toll islands need careful consideration. Such devices need to be sufficiently robust to ensure that an errant vehicle is prevented from impacting the toll booth and the canopy supporting structure but also need to effectively absorb and cushion the impact. Impact absorbing barriers (refer to **Chapter 12.5**) should be placed in front of any significant concrete or steel bollards on the approach to a toll island unless a specific risk assessment supports their omission. Example of protective devices are shown in **Plate 11.20**.

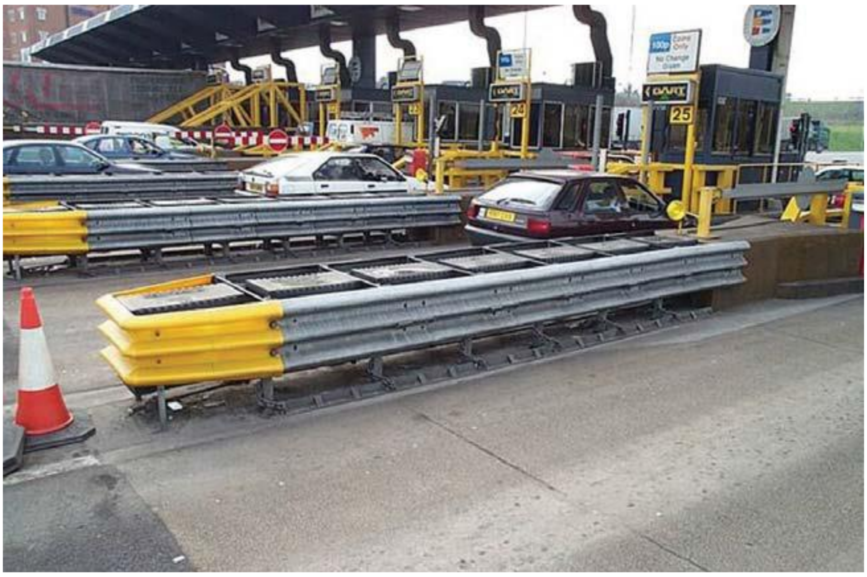


Plate 11.20 Impact attenuator installed at approach of toll island

11.7.8.14 PROTECTION OF THE TOLL ATTENDANT

Protection of the toll attendant is improved by ensuring that vehicles are lined-up and channelled through the toll lane. High kerbs and lateral barriers along the toll islands should ensure that vehicles are kept on-line. The approach end of the toll island should include a substantial concrete or steel bollard capable of stopping an errant vehicle or deflecting such a vehicle along the toll lane. Protection measures for such bollards may be required.

Further measures should be provided on the approach to the toll booth to protect it from any load that may be protruding from the side of a vehicle. Such measures may also be used to protect vulnerable elements of the toll lane equipment.

Examples of the foregoing forms of protection are given in **Plate 11.20** and **Plate 11.21**.

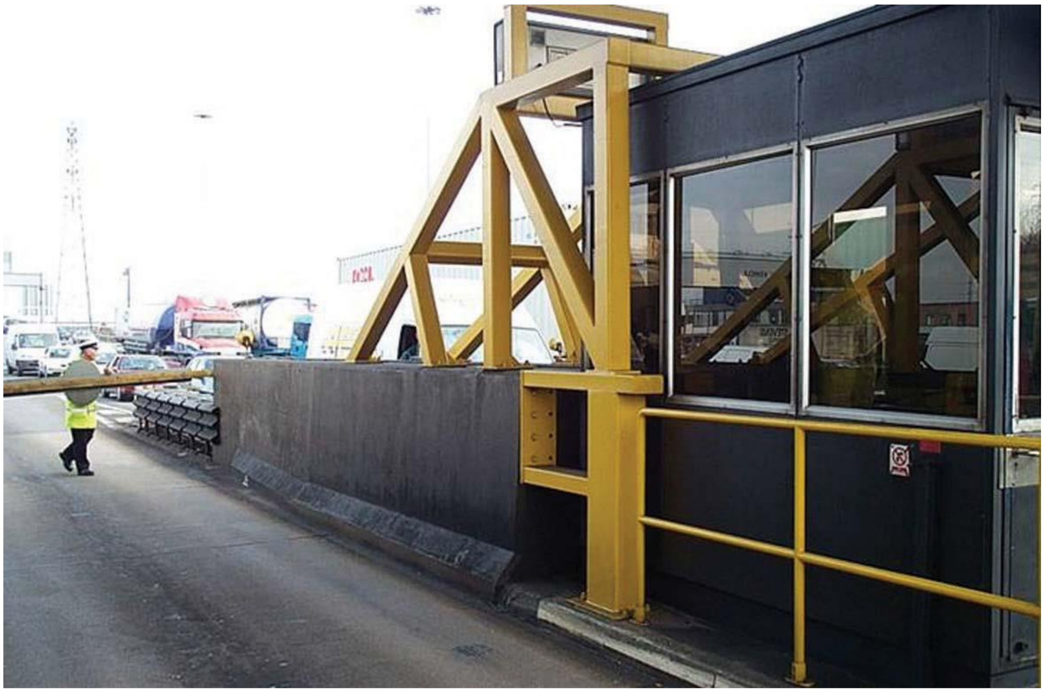


Plate 11.21 Protection of Toll attendant

A safe method of access should be provided for toll collection staff to get to and from the toll booths. The Designer may consider providing a designated at grade walk route to the toll booths. This would be expected to be located at the traffic exit side of the booths and may incorporate walkway signals linked to the barrier position for the lane being crossed. Toll lane access tunnels would reduce the number of lanes to be crossed.

11.7.8.15 PROTECTION OF VEHICLE OCCUPANTS

Devices used to protect toll attendants will be a hazard to the occupants of any vehicle that hits them head-on. They should be designed to minimise this hazard.

Signs and road markings should give clear and adequate warning to drivers of which toll lane they need to use before they arrive at the toll booths.

Impact absorbing barriers should be placed in front of any significant concrete or steel bollards on the approach to a toll island unless a specific risk assessment supports their omission.

11.7.8.16 TOLL COLLECTION AND TOLL LANE EQUIPMENT

The equipment is installed either in or adjacent to the toll booth. The type of toll lane equipment will depend on the facility's configuration and chosen methods of payment. Many, or all, of the following will be included:

- computer equipment
- attendant’s terminal
- receipt printer
- CCTV monitor
- intercom/loudspeakers
- magnetic card reader
- bar code reader
- automatic coin machine (coin basket)
- change machine
- cash transfer equipment
- vehicle detector loops
- treadle/axle counter
- entrance and/or exit automatic toll barriers
- height sensor
- weight in motion detector
- traffic signals (usually only red and green aspects)
- CCTV cameras
- variable massage signs

11.7.8.17 ADDITIONAL INFRASTRUCTURE REQUIREMENTS

Each plaza will require a toll administration building with traffic monitoring and secure cash handling capabilities as well as staff welfare facilities.

In situations where the available land plot is limited, consideration may be given to incorporating administration facilities as part of the toll plaza canopy design.

The highway facility that is being tolled, e.g., a bridge or tunnel, may also require ‘off-carriageway’ infrastructure for maintenance and operation of that facility. The designer should establish whether such facilities can be combined or whether the toll administration facilities are to be kept separate.

The highway facility being tolled may require control over the movements of abnormal and dangerous loads. Provision may be required, within the toll plaza area, for such vehicles to be able to park whilst awaiting an escort or permission to proceed.

A section of tolled highway may include a service area. In such circumstances, consideration should be given to incorporating the toll plaza with the service area.

Consideration should be given to providing alternative access arrangements from the toll administration buildings to the existing local road network.

11.8 EMERGENCY ESCAPE RAMPS

11.8.1 GENERAL

Where long, descending grades exist or where topographic and location controls indicate a need for such grades on new alignment, the design and construction of an emergency escape ramp at an appropriate location is desirable to provide a location for out-of-control vehicles, particularly trucks, to slow and stop away from the main traffic stream. Out-of-control vehicles are generally the result of a driver losing braking ability either through overheating of the break due to mechanical failure or failure to downshift at the appropriate time. Considerable experience with ramps constructed on existing highways has led to the design and installation of effective ramps that save lives and reduce property damage. Reports and evaluations of existing ramps indicate that they provide acceptable deceleration rates and afford good driver control of the vehicle on the ramp.

Rolling resistance is a general term used to describe the resistance to motion at the area of contact between a vehicle's tires and the roadway surface and is only applicable when a vehicle is in motion. It is influenced by the type and displacement characteristics of the surfacing material of the roadway. Each surfacing material has a coefficient, expressed in kg/1000 kg of gross vehicle weight (GVM), which determines the amount of rolling resistance of a vehicle. **Table 11.26** indicates the values for rolling resistance for various roadway surfacing materials.

Table 11.26 Rolling Resistance of Roadway Surfacing Materials

Surfacing Material	Rolling Resistance (kg/1000 kg GVM)	Equivalent Grade (%)[a]
Portland Cement Concrete	10	1.0
Asphalt Concrete	12	1.2
Gravel, Compacted	15	1.5
Earth, Sandy, loose	37	3.7
Crashed Aggregate, Loose	50	5.0
Gravel Loose	100	10.0
Pea gravel	150	15.0

[a] Rolling resistance expressed as equivalent gradient.

11.8.1.1 NEED AND LOCATION FOR EMERGENCY ESCAPE RAMPS

Each grade has its own unique characteristics. Highway alignment, gradient, length, and descent speed contribute to the potential for out-of-control vehicles. For existing highways, operational concerns on a downgrade will often be reported by law enforcement officials, truck drivers, or the general public. A field review of a specific grade may reveal damaged guardrail, gouged pavement surfaces, or spilled oil indicating locations where drivers of heavy vehicles had difficulty negotiating a downgrade. For existing facilities, an escape ramp should be provided as soon as a need is established. Crash experience (or, for new facilities, crash experience on similar facilities) and truck operations on the grade combined with engineering judgment are frequently used to determine the need for a truck escape ramp. Often the impact of a potential runaway truck on adjacent activities or population centres will provide sufficient reason to construct an escape ramp.

Unnecessary escape ramps should be avoided. For example, a second escape ramp should not be needed just beyond the curve that created the need for the initial ramp.

While there are no universal guidelines available for new and existing facilities, a variety of factors should be considered in selecting the specific site for an escape ramp. Each location presents a different array of design needs. Factors that should be considered include:

i. topography,
ii. length and percent of grade,
iii. potential speed,
iv. economics,
v. environmental impact, and
vi. crash experience.

Ramps should be located to intercept the greatest number of runaway vehicles, such as at the bottom of the grade and at intermediate points along the grade where an out-of-control vehicle could cause a catastrophic crash.

A technique for new and existing facilities available for use in analysing operations on a grade, in addition to crash analysis, is the Grade Severity Rating System. The system uses a predetermined brake temperature limit (260°C) to establish a safe descent speed for the grade. It also can be used to determine expected brake temperatures at 0.8km intervals along the downgrade. The location where brake temperatures exceed the limit indicates the point that brake failures can occur, leading to potential runaways.

Escape ramps generally may be built at any practical location where the main road alignment is tangent. They should be built in advance of horizontal curves that cannot be negotiated safely by an out-of-control vehicle without rolling over and in advance of populated areas. Escape ramps should exit to the right of the roadway. On divided multilane highways, where a left exit may appear to be the only practical location, difficulties may be expected by the refusal of vehicles in the left lane to yield to out-of-control vehicles attempting to change lanes.

Although crashes involving runaway trucks can occur at various sites along a grade, locations having multiple crashes should be analysed in detail. Analysis of crash data pertinent to a prospective escape ramp site should include evaluation of the section of highway immediately uphill, including the amount of curvature traversed and distance to and radius of the adjacent curve.

An integral part of the evaluation should be the determination of the maximum speed that an out-of-control vehicle could attain at the proposed site. This highest obtainable speed can then be used as the minimum design speed for the ramp. The 130 to 140 km/h entering speed, recommended for design, is intended to represent an extreme condition and therefore should not be used as the basis for selecting locations of escape ramps. Although the variables involved make it impractical to establish a maximum truck speed warrant for location of escape ramps, it is evident that anticipated speeds should be below the range used for design. The principal factor in determining the need for an emergency escape ramp should be the safety of the other traffic on the roadway, the driver of the out-of-control vehicle, and the residents along and at the bottom of the grade. An escape ramp, or ramps if the conditions indicate the need for more than one, should be located wherever grades are of a steepness and length that present a substantial risk of runaway trucks and topographic conditions will permit construction.

11.8.2 TYPES OF EMERGENCY ESCAPE RAMPS

Emergency escape ramps have been c1assified in a variety of ways. Three broad categories used to classify ramps are:

i. Gravity
ii. Sandpile
iii. arrester bed

Within these broad categories， four basic emergency escape ramp designs predominate. These designs are the sandpile and three types of arrester beds, classified by grade of the arrester beds, descending grade， horizontal grade, and ascending grade. These four types are illustrated in **Figure 11.37**.

The gravity ramp has a paved or densely compacted aggregate surface， relying primarily on gravitational forces to slow and stop the runaway. Rolling-resistance forces contribute little to assist in stopping the vehicle. Gravity ramps are usually long， steep， and are constrained by topographic controls and costs. While a gravity ramp stops forward motion， the paved surface cannot prevent the vehicle from rolling back down the ramp grade and jackknifing without a positive capture mechanism. Therefore， the gravity ramp is the least desirable of the escape ramp types.

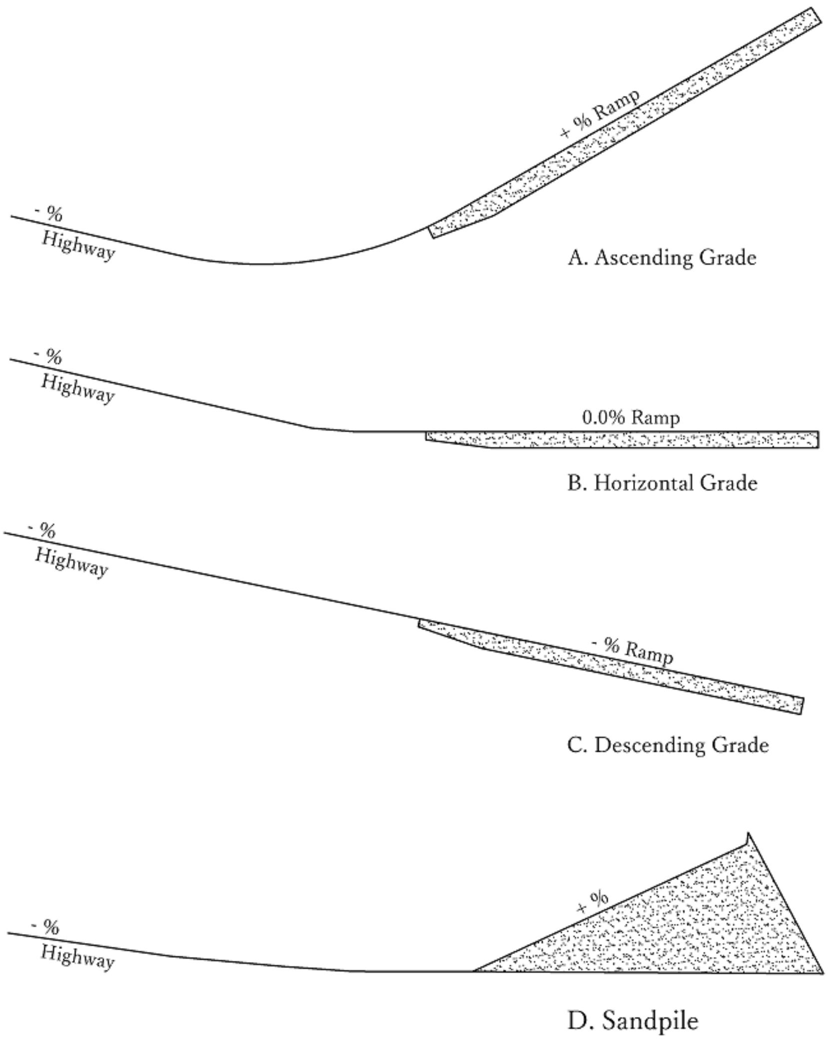


Figure 11.37 Basic Types of Emergency Escape Ramps

Sandpiles, composed of loose, dry sand dumped at the ramp site, are usually no more than 120m in length. The influence of gravity is dependent on the slope of the surface. The increase in rolling resistance is supplied by loose sand. Deceleration characteristics of sandpiles are usually severe and the sand can be affected by weather. Because of the deceleration characteristics, the sandpile is less desirable than the arrester bed. However, at locations where inadequate space exists for another type of ramp, the sandpile may be appropriate because of its compact dimensions.

Descending-grade arrester-bed escape ramps are constructed parallel and adjacent to the through lanes of the highway. These ramps use loose aggregate in an arrester bed to increase rolling resistance to slow the vehicle. The gradient resistance acts in the direction of vehicle movement. As a result, the descending-grade ramps can be rather lengthy because the gravitational effect is not acting to help reduce the speed of the vehicle. The ramp should have a clear, obvious return path to the highway so drivers who doubt the effectiveness of the ramp will feel they will be able to return to the highway at a reduced speed.

Where the topography can accommodate, a horizontal-grade arrester-bed escape ramp is another option. Constructed on an essentially flat gradient, the horizontal-grade ramp relies on the increased rolling resistance from the loose aggregate in an arrester bed to slow and stop the out-of-control vehicle, since the effect of gravity is minimal. This type of ramp is longer than the ascending-grade arrester bed.

The most commonly used escape ramp is the ascending-grade arrester bed. Ramp installations of this type use gradient resistance to advantage, supplementing the effects of the aggregate in the arrester bed, and generally, reducing the length of ramp needed to stop the vehicle. The loose material in the arresting bed increases the rolling resistance, as in the other types of ramps, while the gradient resistance acts in a downgrade direction, opposite to the direction of vehicle movement. The loose bedding material also serves to hold the vehicle in place on the ramp grade after it has come to a safe stop.

Each of the ramp types is applicable to a particular situation where an emergency escape ramp is desirable and should be compatible with established location and topographic controls at possible sites. The procedures used for analysis of truck escape ramps are essentially the same for each of the categories or types identified. The rolling-resistance factor for the surfacing material used in determining the length needed to slow and stop the runaway truck safely is the difference in the procedures.

11.8.3 DESIGN CONSIDERATIONS

The combination of the above extremal resistance and numerous internal resistance forces not discussed acts to limit the maximum speed of an out-of-control vehicle. Speeds in excess of 130 to 140 km/h will rarely, if ever, be attained. Therefore, an escape ramp should be designed for a minimum entering speed of 130 km/h, with a 140 km/h design speed being preferred. Several formulas and software programs have been developed to determine the runaway speed at any point on the grade. These methods can be used to establish a design speed for specific grades and horizontal alignments.

The design and construction of effective escape ramps involve a number of considerations as follows:

i. To safely stop an out-of-control vehicle, the length of the ramp should be sufficient to dissipate the kinetic energy of the moving vehicle.
ii. The alignment of the escape ramp should be tangent or on very flat curvature to minimize the driver’s difficulty in controlling the vehicle.
iii. The width of the ramp should be adequate to accommodate more than one vehicle because it is not uncommon for two or more vehicles to have need of the escape ramp within a short time. A minimum width of 8 m may be all that is practical in some areas,

though greater widths are preferred. Desirably, a width of 9 to 12 m would more adequately accommodate two or more out-of-control vehicles. Ramp widths less than indicated above have been used successfully in some countries where it was determined that a wider width was unreasonably costly or not needed. Widths of ramps in use range from 3.6 to 12 m.

iv. The surfacing material used in the arrester bed should be clean, not easily compacted, and have a high coefficient of rolling resistance. When aggregate is used, it should be rounded, uncrushed, predominantly a single size, and as free from fine-size material as practical. Such material will maximize the percentage of voids, thereby providing optimum drainage and minimizing interlocking and compaction. A material with a low shear strength is desirable to permit penetration of the tires. The durability of the aggregate should be evaluated using an appropriate crush test.

v. Arrester beds should be constructed with a minimum aggregate depth of 1m. Contamination of the bed material can reduce the effectiveness of the arrester bed by creating a hard surface layer up to 300mm thick at the bottom of the bed. Therefore, an aggregate depth up to 100 mm is recommended. As the vehicle enters the arrester bed, the wheels of the vehicle displace the surface, sinking into the bed material, thus increasing the rolling resistance. To assist in decelerating the vehicle smoothly, the depth of the bed should be tapered from a minimum of 75 mm at the entry point to the full depth of aggregate in the initial 30 to 60 m of the bed.

vi. A positive means of draining the arrester bed should be provided to help protect the bed from flooding and avoid contamination of the arrester bed material. This can be accomplished by grading the base to drain, intercepting water prior to entering the bed, underdrain systems with transverse outlets, or edge drains. Geotextiles or paving can be used between the subbase and the bed materials to prevent infiltration of fine materials that may trap water. Where toxic contamination from diesel fuel or other material spillage is a concern, the base of the arrester bed may be paved with concrete and holding tanks to retain the spilled contaminants.

vii. The entrance to the ramp should be designed so that a vehicle traveling at a high rate of speed can enter safely. As much sight distance as practical should be provided preceding the ramp so that a driver can enter safely. The full length of the ramp should be visible to the driver. The angle of departure for the ramp should be small, usually 5 degrees or less. An auxiliary lane may be appropriate to assist the driver to prepare to enter the escape ramp. The main roadway surface should be extended to a point at or beyond the exit gore so that both front wheels of the out-of-control vehicle will enter the arrester bed simultaneously; this also provides preparation time for the driver before actual deceleration begins. The arrester bed should be offset laterally from the through lanes by an amount sufficient to preclude loose material being thrown onto the through lanes.

viii. Access to the ramp should be clearly indicated by exit signing to allow the driver of an out-of-control vehicle time to react, to minimize the possibility of missing the ramp. Advance signing is needed to inform drivers of the existence of an escape ramp and to prepare drivers well in advance of the decision point so that they will have enough time to decide whether or not to use the escape ramp. Regulatory signs near the entrance should be used to discourage other motorists from entering, stopping, or parking at or on the ramp. The path of the ramp should be delineated to define ramp edges and provide night-time direction; for more information, see **Section 12.2**. Illumination of the approach and ramp is desirable.

ix. The characteristic that makes a truck escape ramp effective also makes it difficult to retrieve a vehicle captured by the ramp. A service road located adjacent to the arrester bed is needed so tow trucks and maintenance vehicles can use it without becoming trapped in the bedding material. The width of this service road should be at least 3 m. Preferably this service road should be paved but may be surfaced with gravel. The road should be designed in such a way that the driver of an out-of-control vehicle will not mistake the service road for the arrester bed.

x. Anchors, usually located adjacent to the arrester bed at 50 to 100 m intervals, are needed to secure a tow truck when removing a vehicle from the arrester bed. One anchor should be located about 30m in advance of the bed to assist the wrecker in returning a captured vehicle to a surfaced roadway. The local tow-truck operators can be very helpful in properly locating the anchors.

As a vehicle rolls upgrade, it loses momentum and will eventually stop because of the effect of gravity. To determine the distance needed to bring the vehicle to a stop with consideration of the rolling resistance and gradient resistance, **Equation 11.5** may be used.

$$L = \frac{V^2}{254 \times (R \pm G)} \tag{11.5}$$

Where,

L= Length of arrester bed (m)

V= Entering velocity (km/h)

R= Rolling resistance, expressed as equivalent percent gradient divided by 100

G= Percent grade divided by 100

For example, assume that topographic conditions at a site selected for an emergency escape ramp limit the ramp to an upgrade of 10 percent (G = +0.10). The arrester bed is to be constructed with loose gravel for an entering speed of 140 km/h. Using **Table 11.26**, R is determined to be 0.10. The length of the arrester bed should be determined using **Equation 11.5**. For this example, the length of the arrester bed is about 400m.

When an arrester bed is constructed using more than one grade along its length, as shown in **Figure 11.38**, the speed loss occurring on each of the grades as the vehicle traverses the bed should be determined using **Equation 11.6**.

$$V_f^2 = V_i^2 - 254 \times L \times (R \pm G) \qquad 11.6$$

Where,

V_f= Speed at end of grade (km/h)
V_i= Entering speed at beginning of grade (km/h)
L= Length of grade (m)
R= Rolling resistance, expressed as equivalent present gradient divided by 100
G= Percent grade divided by 100

The final speed for one section of the ramp is subtracted from the entering speed to determine a new entering speed for the next section of the ramp and the calculation repeated at each change in grade on the ramp until sufficient length is provided to reduce the speed of the out-of-control vehicle to zero.

Figure 11.38 and **Plate 11.22** shows a plan and profile of an emergency escape ramp with typical appurtenance and an example of emergency ramp respectively.

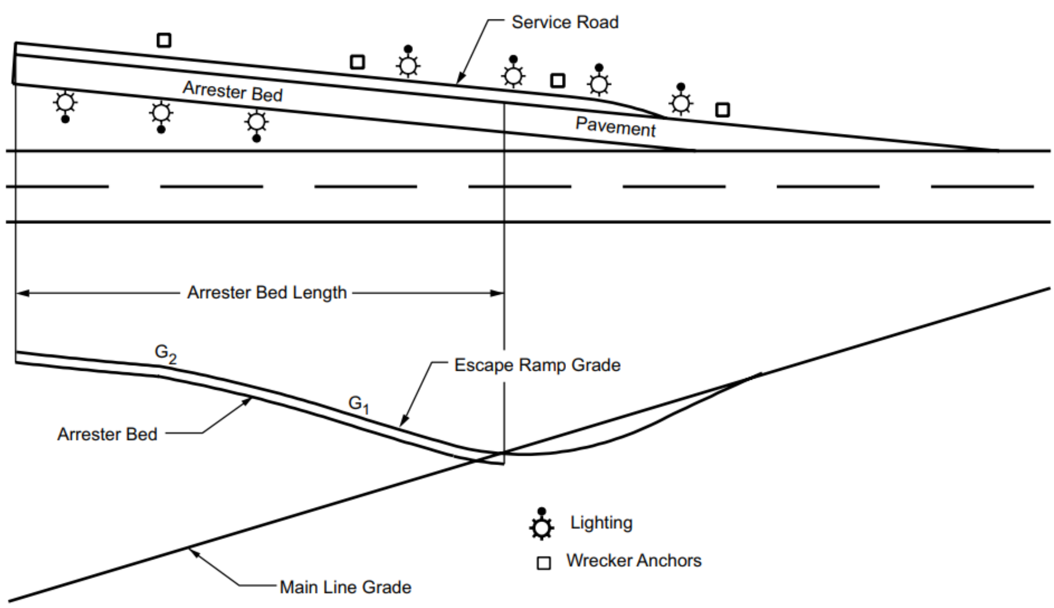


Figure 11.38 Typical Emergency Escape Ramp

Plate 11.22 Example of Emergency Ramp

Where the only practical location for an escape ramp will not provide sufficient length and grade to completely stop an out-of-control vehicle, it should be supplemented with an acceptable positive attenuation device.

Where a full-length ramp is to be provided with full deceleration capability for the design speed, a "last-chance" device should be considered when the consequences of leaving the end of the ramp are serious.

Any ramp-end treatment should be designed with care so that its advantages outweigh its disadvantages. The risk to others as the result of an out-of-control truck overrunning the end of an escape ramp may be more important than the harm to the driver or cargo of the truck. The abrupt deceleration of an out-of-control truck may cause shifting of the load, shearing of the fifth wheel, or jackknifing, all with potentially harmful occurrences to the driver and cargo.

Mounds of bedding material between 0.6 and 1.5 m high with 1V:1.5H slopes (i.e., slopes that change in elevation by one unit of length for each 1 to 1.5 units of horizontal distance) have been used at the end of ramps in several instances as the "last-chance" device. An escape ramp can be constructed with an array of crash cushions installed to prevent an out-of-control vehicle from leaving the end of the ramp. Furthermore, at the end of a hard-surfaced gravity ramp, a gravel bed or an attenuator array may sufficiently immobilize a brakeless runaway vehicle to keep it from rolling backward and jackknifing. Where barrels are used, the barrels should be filled with the same material as used in the arrester bed, so that any finer material does not result in contamination of the bed and reduction of the expected rolling resistance.

11.9 ROAD PROTECTION FACILITIES

11.9.1 ROCKFALL PREVENTION FACILITIES

Rockfall prevention facilities are installed to protect road users and road facilities from rockfall disasters. Such facilities are divided into rockfall prevention and affixing facilities and rockfall protection facilities.

i. Rockfall prevention and affixing facilities entail removing loose rocks and boulders.
ii. Rockfall protection facilities entail protecting against rocks that fall down from slopes.

Rockfalls occur for various reasons and multiple factors involved in many cases. Therefore, when planning rockfall prevention facilities, it is necessary to select the work methods and layout upon fully considering experience, local conditions, the structural function thresholds of rockfall prevention facilities. When planning facilities, rather than taking isolated measures, it is often better to combine a number of approaches.

11.9.1.1 ROCKFALL PREVENTION AND AFFIXING FACILITIES

Rockfall prevention works entail directly dealing with the sources of rockfall occurrence to stop rockfalls from happening, and they mainly comprise “cutting and removal work”, “foot protection”, “adhesion”, “anchor works”, and “wire rope slinging”. The suitable methods should be selected according to the rockfall conditions, slope, and terrain conditions.

i. “Wire rope slinging”, “anchor works”: These methods entail directly suppressing rock slippage at the tops of slopes.
ii. “Foot protection”, “adhesion”: These methods entail consolidating and supporting foundations and bedrocks to ensure that rocks do not slip and roll.
iii. “Cutting and removal work”: This method entails directly removing rocks.

Also, preventive works should be employed in response to individual rockfalls and include such methods as “spray coating” and “grating crib works”. These methods are intended to prevent slope failure including rockfalls and to stabilize slopes.

11.9.1.2 ROCKFALL PROTECTION FACILITIES

In rockfall protection works, protection facilities as illustrated by **Plate 11.23** and **Plate 11.24**, should be installed on road boundaries, mid-slopes, etc. to capture and stop falling rocks or lead them to safety when rocks fall from upper slopes. Rockfall protection works mainly comprise “rockfall protection fences”, “rockfall protection netting”, “rockfall protection retaining walls”, “rock shed works” and so on.

Plate 11.23 Grating Crib Works and Rock Bolt Works

Plate 11.24 Rockfall Protection Fence

11.9.2 BREAKWATER FACILITIES, EROSION CONTROL FACILITIES, ETC.

When constructing a road along a coastline, wave-dissipating blocks, retaining walls, etc. should be installed where necessary according to the terrain, weather conditions among others to prevent erosion and scouring caused by waves. Also, to prevent buildup of sand, erosion control facilities should be installed where necessary. **Figure 11.39** and **Figure 11.40** show examples of breakwater and erosion control facilities respectively.

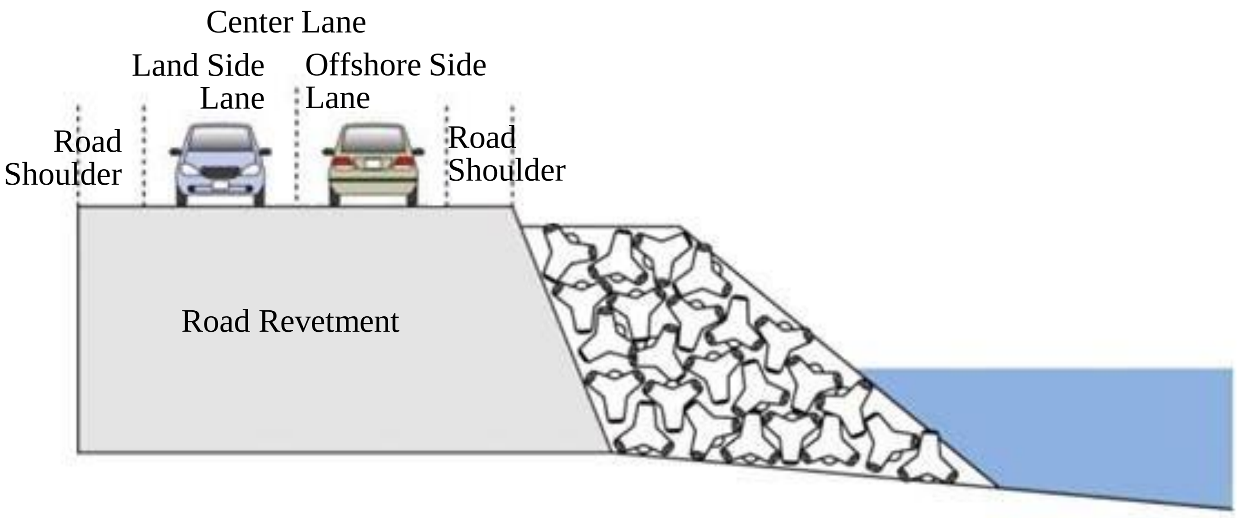


Figure 11.39 Example of Breakwater Facilities (Wave Absorbing Revetment)

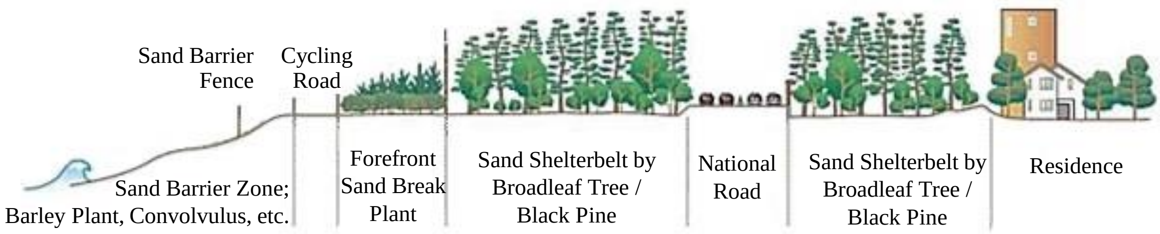


Figure 11.40 Examples of Erosion Control Facilities (Erosion Control Forest)

11.10 PEDESTRIAN FACILITIES

11.10.1 WALKWAYS

Walkways are an integral part of town roads and are sometimes also provided in rural areas. However, the potential for collisions with pedestrians is higher in many rural areas due to the higher speeds and general absence of lighting. The limited data available suggest that walkways in rural areas are effective in reducing pedestrian collisions.

Walkways near or along the highway in the rural and suburban contexts are more often needed at points of development that generate pedestrian concentrations, such as residential areas, schools, businesses, and industrial plants. When suburban residential areas are developed, initial roadway facilities are needed for the community to function, but the construction of walkways is sometimes deferred. However, if pedestrian activity is anticipated, walkways should be included as part of the initial construction.

Pedestrian facilities such as walkways must be designed to be accessible to and usable by the physically challenged.

Recommendable design standard are:

i. The cross slope on walkways should not exceed 2 percent.
ii. Walkway cross section width should comply with values in **Section 6.11**
iii. Where shoulders are intended for use by pedestrians, the shoulder must be accessible to and usable by the physically challenged.
iv. Continuity of walkway should be maintained.
v. Continuity of kerb height should be maintained on the approaches to and over structures or higher speed roadways on structures.
vi. A barrier-type rail of adequate height may be used to separate the walkway and the travelled way.

11.10.1.1 AT-GRADE PEDESTRIAN CROSSING

Pedestrian crossings are provided to improve road safety for pedestrians when crossing a road. Factors to take into account for the provision of pedestrian crossing facilities include:

i. the volume of pedestrians crossing the road
ii. the speed of the traffic
iii. the width of the road
iv. whether there are a lot of children crossing; and
v. whether there are significant numbers of physically challenged pedestrians.

There are many things that can be done to help pedestrians cross the road, including:

i. Formal crossings;
   - uncontrolled (zebra) crossings; and
   - controlled (signal-controlled) crossings.
ii. Humped pedestrian crossings.
iii. Build-outs
iv. Refuge islands
v. Medians
vi. Pedestrian bridges (see next sub-section)
vii. Underpasses (see next sub-section)

The uncontrolled (zebra) and signal-controlled crossings should be used where there are high volumes of pedestrians trying to cross wide and / or busy roads. The safety benefits of zebra crossings depend on the discipline amongst drivers. Where discipline exists and all drivers stopped for pedestrians, this can lead to severe congestion at busier crossings. Signal-controlled crossings, though more expensive to install and maintain, are likely to perform better.

Various requirements have to be met when placing pedestrian crossing facilities. They should be located well away from conflict points at uncontrolled road junctions. Refer to **Section 8.10.1** for details on pedestrian crossings at intersections. Pedestrian fences may be provided at the intersections to ensure the pedestrians cross the road at the specified locations.

Crossings should not be placed adjacent to laybys. If this is unavoidable, laybys should always be beyond the crossing.

Zebra crossings can be made to work better if the crossing is marked on top of a trapezoidal hump. The hump forces approaching vehicles to slow down, and this gives the pedestrian a chance to step onto the crossing and claim priority.

Crossing the road can be made a lot safer by means of simple, informal measures, such as build-outs, refuge islands and medians. **Plate 11.25** shows the use of build-outs.

Plate 11.25 Pedestrian Build-out

Build-outs are useful on wide roads where there is roadside parking, because they extend the walkway further into the road thus improving inter-visibility between pedestrians and drivers.

Refuge islands in the centre of the road enable pedestrians to cross the road in two stages, which makes it much easier and safer. However, kerbed refuge islands are at risk of being hit by speeding vehicles, so they must be well signed – with "Keep right" signs. Alternatively, refuges can be created out of road markings and rumble strips provided within them. Refuge islands should be at least 1.5m wide (preferably 2.0m). The designer should make sure that there is sufficient width of carriageway left to enable traffic to flow freely. A width of 3.0 – 3.5 m is normally enough for one lane of traffic. When considering the provision of crossing facilities at a site, the inter-visibility between pedestrians and drivers should always be checked. It shall be at least equal to the stopping sight distance as detailed in **Section 7.2.6**.

Criteria for provision of pedestrian crossing are as follows:

i. **Speed limit of 30 km/h**

Normally a zebra crossing is not established at speed limit of 30 km/h. If a crossing is needed for some reason, the 85%-percentile has to be 35 km/h or less. Otherwise a speed-reducing measure should be considered.

ii. **Speed limit of 40 km/h**

Normally a zebra crossing will be considered when more than 20 pedestrians/bicycles cross during the max-hour and ADT is ≥ 2,000.

iii. **Speed limit of 50 km/h**

Normally a zebra crossing will be considered when more than 20 pedestrians/bicycles cross during the max-hour and ADT is ≥ 2,000.

iv. **Speed limit of 60 km/h**

Normally a zebra crossing should not be considered at speed limit of 60 km/h and never if ADT is ≥ 4,000. A grade separated crossing must be considered independent of number of pedestrians/bicycles crossing.

v. **Speed limit of 80 km/h**

Roads with ADT ≤ 4,000 and number of pedestrians ≥ 50, grade separated crossing for pedestrians/ bicycles must be established. With ADT ≥ 4,000, a grade separated crossing must then be established independent of number of pedestrians/bicycles crossing.

11.10.2 GRADE-SEPARATED PEDESTRIAN CROSSING

A grade-separated pedestrian facility allows pedestrians to cross at either over or under the roadway and provides pedestrians with a path for crossing the roadway without vehicle interference. Pedestrian separations should be provided where pedestrian volume, traffic volume, intersection capacity, and other conditions favour their use, although their specific location and design need individual study. **Plate 11.26** shows an example of a pedestrian overpass at a major four-leg intersection. They may be needed to accommodate heavy peak pedestrian movements, such as at central business districts, factories, schools, or athletic fields, in combination with moderate to heavy vehicular traffic or where unusual risk or inconvenience to pedestrians may result. Pedestrian separations, usually overpasses, may be needed at expressways/motorways or major arterial where crossroads are terminated. On many expressways/motorways, road overpasses for crossroads may be limited to three- to five-block intervals. Because this situation imposes an extreme inconvenience on pedestrians who desire to cross the expressways/motorways at the terminated roads, pedestrian separations may be provided.

Where there are frontage (service) roads adjacent to the arterial highway, the pedestrian crossing may be designed to span the entire facility or only the through roadway. Separations of both through roadways and frontage roads may not be justified if the frontage roads carry light and relatively slow-moving traffic; however, in some cases the separation should span the frontage roads as well as shown **Plate 11.27**.

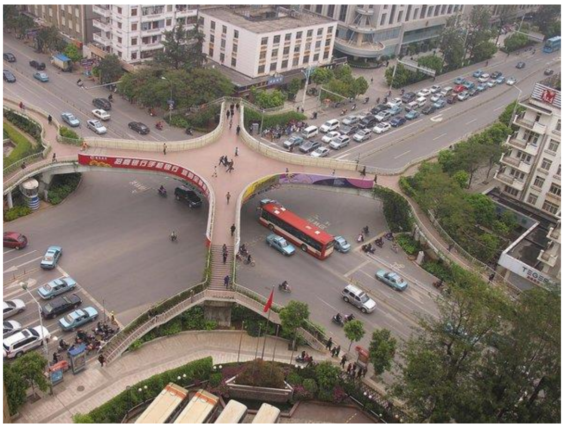


Plate 11.26 Pedestrian overpass at major intersections

Plate 11.27 Pedestrian overpass spanning frontage (service) roads and main road

Pedestrian barriers can be used to try and force pedestrians to use the facility, but local people can destroy them if they feel that the detour is unreasonable. When designing pedestrian bridges and underpasses, the designer should make sure that they are easy to access (special attention given to children, elderly and physically challenged) and as pleasant and safe to use as possible.

Pedestrian fences may be provided at the pedestrian bridges/underpasses to ensure the pedestrians use the facilities. Underpasses should be designed so that the pedestrian can see from one end to the other and can thus choose not to enter if there is a potentially threatening situation.

Generally, pedestrians are more reluctant to use underpasses than overpasses. This reluctance may be minimized by locating the underpass in line with the approach walkway and ramping the walkway gently to permit continuous vision through the underpasses from the walkway.

Good sight lines, wide openings, and lighting are needed to enhance a sense of security. Ventilation may be needed for very long underpass. **Plate 11.28** shows examples of pedestrian underpass.

Plate 11.28 Examples of pedestrian underpass

Pedestrian ramps or elevators should be provided at all pedestrian separation structures. Where desired, a stairway can be provided in addition to the ramp. Elevators should be considered where the length of ramp would result in a difficult path of travel for a person with or without a disability.

Walkways for pedestrian separations should have a minimum width of 2.0m. Greater widths may be needed through tunnels, where overpass screenings create a tunnel effect, and where there are exceptionally high volumes of pedestrian traffic, such as in the Central Business Districts (CBD) of large cities (see **Plate 11.29)** and around sports stadiums or arenas.

Plate 11.29 Example of pedestrian overpass in a Central Business District (CBD)

A serious problem associated with both pedestrian overpasses and highway overpasses with walkways is vandals dropping objects into the path of traffic moving under the structure. The consequences of objects being thrown from bridges can be very serious. In fact, there are frequent reports of fatalities and major injuries caused by this type of vandalism. There is no practical device or method yet devised that can be universally applied to prevent a determined individual from dropping an object from an overpass.

For example, small objects can be dropped through mesh screens. A more effective deterrent is a solid plastic enclosure. However, these are expensive and may be insufferably hot during the day. They also obscure and darken the pedestrian travelled way, which may be conducive to other forms of criminal activity. Any completely enclosed pedestrian overpass has an added problem that children may walk or play on top of the enclosure.

The general need for economy in design and the desire to preserve the clear lines of a structure unencumbered by screens should be carefully balanced against the need to limit the potential for injury to pedestrians and damage to vehicles.

11.10.3 DESIGN STANDARD OF GRADE-SEPARATED PEDESTRIAN CROSSING

11.10.3.1 LAYOUT

Grade-separated pedestrian crossing is used to separate road traffic (motorized and non-motorized) and pedestrian traffic to maintain adequate flow at intersections and crossing points. Based on intersection capacity analysis, the pedestrian traffic may be partially or fully separated from the vehicular traffic.

i. **Partial separation**

At-grade and grade-separated crossings that exist at the same intersection is referred to as partial separation. **Plate 11.30** illustrate an example of partial grade-separated intersection.

Plate 11.30 Example of partial grate-separated intersection

ii. **Full separation**

In full separation, the pedestrian traffic is fully separated from vehicular traffic. This grade-separated crossing is likely to be a landmark. An example of full separated pedestrian crossing is shown in **Plate 11.31**.

Plate 11.31 Example of full separated pedestrian crossing

11.10.3.2 GEOMETRY

Table 11.27 shows recommended values for the design of pedestrian crossing structures.

Table 11.27 Recommended values for the design of pedestrian crossing structures (m)

Item	Stairs	Ramp	Corridor
Cross section width (m)	1.5 (min.)	2.0 (min.)	2.0 (min.)
Gradient (%)	12 (min.) 25 (opt.) 50 (max.)	5% (desirable) 10% (absolute)	0.4
Crossfall (%)	-	-	2
Landing (m)	1.2 (min.)	1.7 (min.)	

Recommended minimum dimensions for underpasses are given in **Table 11.28**.

Table 11.28 Minimum underpass dimensions

Type of underpass	Width (m)	Height (m)
Short (i.e. < 15 m)	2.3	2.5
Long (i.e. > 15 m)	3.3	3.0

The designer shall provide an underpass width based on the anticipated number of pedestrians to use the underpass but should never be less than the values shown in **Table 11.28**.

Ideally there should be a choice for stairs or ramps. Recommended standards for stairs are:

i. Flights of stairs (between landings) should be limited to 12 steps.
ii. Steps should be of equal heights throughout the underpass or pedestrian bridge.
iii. Optimum dimensions for stairs are 300 mm tread (horizontal) and 150 mm rise (vertical)
iv. Handrails (1m above the floor) should be provided on both sides of stairs and ramps central handrails may be advisable where the width of the stairs or ramps exceeds 3m.

The desirable clearance required for pedestrian bridges above the carriageway surface is 5.5m. This headroom must be attained over the full width of the carriageway, including the tip of the cross-fall.

11.11 PROTECTIVE SCREENING AT OVERPASSES

An object or debris thrown, dropped, or dislodged from an overpass structure can cause significant damage and injuries. Protective screening might reduce the number of these incidents; however, that screening will not stop a determined individual. In many cases, increased enforcement also is needed to provide an effective deterrent.

Although the most common protective screening in use is for pedestrian type overpasses, other types of screening such as glare screens are used to protect oncoming traffic on overpasses. Splash or debris screens are used to protect commercial or residential properties that are beneath or adjacent to the structure.

It is not reasonable to establish absolute warrants as to when, where, or what type of barriers or screens should be installed. The general need for economy of design and desire to preserve the clean lines of the structures, unencumbered by screens, must be carefully balanced against the requirement that the highway traveller, overpass pedestrian, and adjoining property be provided maximum protection.

Various types and configurations of screens, usually of a chain-link fence type, can be installed on overpasses in areas where the problem of throwing or dropping objects has been determined to exist.

The simplest design for use on pedestrian overpasses is a vertical fence erected on the bridge railing of the structure. Although this type of design has been effective in keeping children from playing on the railing, the design has proven somewhat ineffective in combating the problem of objects being thrown from the structure. Objects large enough to cause serious damage to passing vehicles still can be thrown over a vertical structure with some degree of accuracy. On pedestrian bridges, a semi-circular enclosure can be placed on top of the two vertical walls to discourage this type of vandalism. This design has further evolved into one with a partially enclosed curved top which is used in the USA and other countries. Objects generally cannot be thrown over the top of a partially enclosed screen with any degree of accuracy.

Care should be taken in the design of chain-link type screens to ensure that the opening at the bottom of the side screens, through which an object can be pushed or dropped, is eliminated or kept to a minimum. Where aesthetics are important, decorative type screening should be used.

Installation of protective screening should be analysed on a case-by-case basis at the following locations:

i. Existing structures where incidents of objects being dropped or thrown from the overpass have occurred and where increased surveillance, warning signs, or apprehension of a few individuals has not effectively alleviated the problem.
ii. An overpass near a school, playground, or other location where it would be expected that the overpass would be frequently used by children not accompanied by adults.
iii. All overpasses in urban areas used exclusively by pedestrians and not easily kept under

surveillance by law enforcement personnel.

iv. Overpasses with walkways where experience on similar structures indicates a need for such screens.

v. Overpasses where private property that is subject to damage, such as buildings or power stations, is located beneath the structure.

Plate 11.32 shows a pedestrian overpass with a protective screening.

Plate 11.32 Pedestrian Overpass (with screen) on Major Highway

11.11.1 LENGTH OF SCREEN

The location of the fence should be determined in consultation with road or railway authorities by considering the actual site condition. The required length as depicted in **Figure 11.41** can be calculated using **Equations 11.7** and **11.8**.

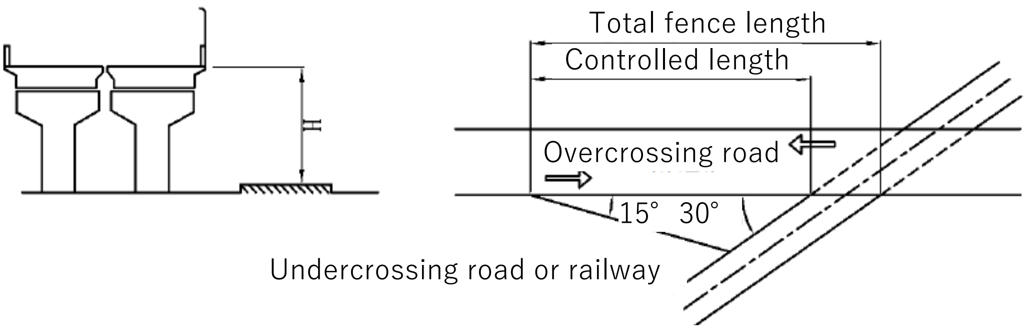


Figure 11.41 Example of controlled length of fence

The controlled length should be provided not to allow obstacles fall into undercrossing roads from approaching roads of the overcrossing bridges.

$$L = V_0 \sqrt{\frac{2(H+3)}{G}\left(\sin 15^o + \frac{\sin 15^o}{\cos\alpha}\right)} \tag{11.7}$$

In case α= 90%

$$L = V_0 \sqrt{\frac{2(H+3)}{G}} \times \cos 15^o \tag{11.8}$$

Where,

L: Controlled length (m)

$V_{0:}$ moving speed of obstacles (m/s). Normally 14m/s (52km/h) can be adopted.

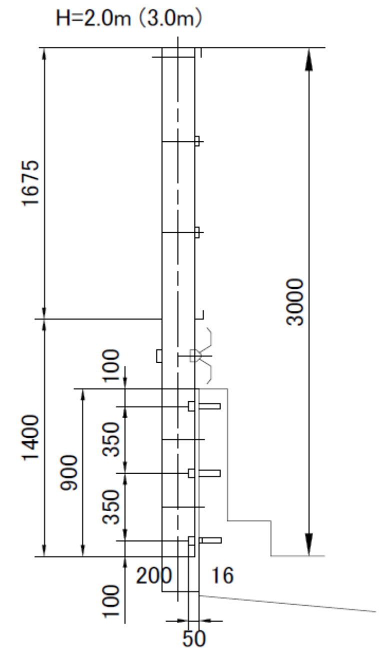
H: Height from carriageway of undercrossing road to the surface of overcrossing road (m)

α: Crossing angle between crossing roads

G: Acceleration due to gravity, 9.8 m/s^2

11.11.2 HEIGHT OF PROTECTIVE SCREEN

The height of the protective screen from carriageway of overcrossing road as shown in **Figure 11.42** should not be more than 3.0m.



All measurements in millimetres unless otherwise stated

Figure 11.42 Example of protective screen for overcrossing road

11.12 REFERENCES

1. Ghana Highway Authority Road Design Guide, 1991.
2. Standard Details, Road Signs and Markings for Urban and Trunk Road (1991), Ministry of Roads and Highways.
3. Ministry of Roads and Transport (MRT) Standard Specifications for Road and Bridge Works, 2007.
4. Draft Manual for Signs and Markings, 2007.
5. Japanese Road Structure Ordinance, April 2021.
6. Geometric Design Manual of Uganda, (2005).
7. A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.
8. AASHTO. A Guide for Development of Rest Areas on Major Arterials and Freeways. American Association of State Highway and Transportation Officials, Washington, DC, 2001.
9. ARRB Transport Research 2005, National guidelines for the provision of rest area facilities: final report, National Transport Commission, Melbourne, Vic.
10. South African Geometric Design Guidelines (2003).
11. Asian Highway Design Standard for Road Safety, 2017.
12. Geometric Design Manual, Federal Democratic Republic of Ethiopia, Ethiopian Roads Authority, 2013.
13. Highway Capacity Manual (HCM), 2016.
14. Study for the Harmonization of Vehicle Overload Control in the East African Community
15. Heavy Vehicle Interception Site – Design Manual, The State of Queensland (Department of Transport and Main Roads), July 2021.
16. Zoning guidelines, Town and Country Planning Department (2011)
17. Austroads Guide to Traffic Management Part 11, 2017
18. Southwest Ohio Regional Transit Authority (SORTA) Bus Stop Design Guidelines, 2019
19. Highway Research Board Special Report No. 125. Parking PRINCIPLES
20. Truck Parking Development Handbook, Federal Highway Administration (FHWA) Office of Operations (HOP), 2022
21. Off-Street Car Parking Guidelines, Melton City Council
22. County of San Diego Parking Design Manual, 2013

23. AS/NZS 2890.1-2004, Parking facilities: part 1: off-street car parking.
24. AS 2890.2-2002, Parking facilities: part 2: off-street commercial vehicle facilities.
25. AS 2890.5-1993, Parking facilities: part 5: on-street parking.
26. Austroads 2014, Guide to traffic management part 5: road management, 2nd edn, AGTM05-14, Austroads, Sydney, NSW.
27. Austroads Guide to Road Design Part 6B: Roadside Environment, 2021.
28. WSDOT Design Manual M 22-01.21
29. Design Manual For Roads And Bridges, TA 98/08, Volume 6, Section 3, Part 6, The Layout of Toll Plazas.
30. Schaufler, A. E. "NCHRP synthesis of highway practice 240: Toll Plaza design." Transportation Research Board of the National Academies, Washington, DC (1997).
31. Tanzania Road Geometric Design Manual 2012
32. Gotic Project In Sweden, 2002
33. Intelligent Transport Systems, Tom Rye (Napier University, Edinburgh) For The Steer Training Project Competence, 2006.
34. http://www.utmc.gov.uk/research/index.htm
35. http://www.utmc.gov.uk
36. http://www.its-assist.org.uk/links.htm
37. http://www.trafficlinq.com/its.htm
38. http://www.eu-spirit.com/
39. http://www.itsnetwork.org/
40. http://www.cordis.lu/telematics/tap_transport/home.html
41. Blokland, M., Mouris, R. (2005) Gedragseffecten Multimodale Reisinformatie (Behavioural Effects of Multimodal Travel Information). Report to Dutch Ministry of Transport, AdviesDienst Verkeer en Vervoer (Travel and Transport Advisory Service), Rotterdam
42. Carsten O.M.J. and Tate F.N. (2005) Intelligent speed adaptation: accident savings and cost benefit analysis. Accident Analysis & Prevention, Volume 37, Issue 3, May 2005, Pages 407 - 416
43. Winnett M A & Wheeler A H (2002) Vehicle-activated signs – a large-scale evaluation, TRL Report 548.
44. FHWA. Manual on Uniform Traffic Control Devices for Streets and Highways. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2009.
45. Traffic Signal Supports, Indications & Signing by Jeffrey W. Buckholz, PhD, PE, PTOE.

46. Tennessee Department of Transportation (TDOT) Traffic Design Manual, 2018.
47. Areas, Rest, and Stopping Places–Location. "Design and Facilities–Guideline." State of Queensland (Department of Transport and Main Roads) (2014).
48. Plovnick, Amy, Angela Berthaume, Carson Poe, and Tina Hodges. Sustainable Rest Area Design and Operations. No. DOT-VNTSC-FHWA-17-20; FHWA-HEP-18-006. John A. Volpe National Transportation Systems Center (US), 2017.
49. Reierson, James S. Safety Rest Area: Planning, Location, Design. The Division, 1981.
50. City of Denver 2010, Denver strategic parking plan, City of Denver, Colorado, USA.
51. Shoup, DC 2005, The high cost of free parking, University of California Transportation Center, Berkeley, CA, USA.

CHAPTER 12

ROAD FURNITURE

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CHAPTER 12 ROAD FURNITURE

12.1 INTRODUCTION

The term "road furniture" encompasses all fixtures on the road, above the road or within the roadside used for safety and control of traffic. Road furniture items are intended to provide information or safety to a road user and includes the following:

i. Delineators
ii. Road makings
iii. Road signs
iv. Traffic safety barriers
v. Fences
vi. Noise barriers
vii. Roadway lighting
viii. Anti-glare systems
ix. Sign, signal, luminaire and other supports
x. Road furniture for temporal works
xi. Truck and trailer – mounted attenuators (TMAs)

12.2 DELINEATORS

Delineators are installed on road surfaces and roadsides to provide a preview and vital information of the roadway to enable drivers navigate the road safely during daylight, night-time and in adverse weather.

Delineators generally consist of reflectors or retroreflective strips attached to ground-mounted posts or reflectors mounted directly onto safety barriers, retaining walls or any other structure along the roadway. They are particularly beneficial for roads without road lighting and where the performance of road markings is unsatisfactory. Delineators are classified into five (5) categories as shown in **Table 12.1**.

Table 12.1 Categories of delineators

Category	Function
Alignment delineator	Highlighting road alignment and curves
Access delineator	Demarcating accesses and intersections
Lane Delineator	Separating opposing traffic or slow traffic
Hazard Marker	Highlighting roadside hazard
Diverge Gore Delineator	Highlighting diverge gore on high-speed roads

Alignment delineator may be integrated with kilometre markers which display distances at 100m intervals. The appearance and colour of different categories of delineators should be distinctly different and consistently applied according to their functions.

Generally, delineators are provided as vertical posts with an overall height between 1m and 1.3m. They should consist of reflectors that are clearly visible at night-time. The reflector should be mounted more than 0.8m above the road surface to minimise masking by plants.

Delineators should be frangible or flexible for passive safety. To reduce maintenance needs, self-restoring delineation posts are encouraged at high impact sites and on high-speed roads. Concrete foundation for delineator support in soft soil should be used as directed by the Engineer.

12.2.1 ALIGNMENT DELINEATOR

Alignment delineators should be used on road sections with increased risks of an errant vehicle leaving the road such as zones of narrowing or lane reduction and merging or diverging areas. They are also recommended on straight sections where delineation by line markings is unsatisfactory and road lighting is not provided. Typical alignment delineators are illustrated in **Plate 12.1**.

Plate 12.1 Alignment delineators

They are generally mounted on circular, square or quadrangle posts. Alternatively, reflectors may be directly attached to safety barriers or retaining walls. Reflectors on safety barriers should be set back from the traffic face. For mounting onto retaining walls, protruding fixtures should not constitute a safety hazard upon impact.

Reflectors should enable a delineator to be clearly visible 300m ahead when illuminated by a high beam light in clear conditions. They should have a size equivalent to at least 75mm or 100mm diameter and are mounted at heights in the order of 800mm to 1200mm irrespective of the mounting method. The recommended colour of the post is white with a red front face and white rear face reflector.

Table 12.2 provides a guide to the spacing of alignment delineators.

Table 12.2 Spacing of alignment delineators

Curve Radius (m)	Spacing (m)	
	Outside of curves	Inside of curves*
< 100	6	12
100 – 199	10	20
200 – 299	15	30
300 – 399	20	40
400 – 599	30	60
600 – 799	40	60
1200 – 1199	60	60
1200 – 2000	90**	90**
2000 and Straight	150***	150***

* Position to match delineator on opposite side

** Adopt 60m in areas susceptible to fog

*** Adopt 75m in areas susceptible to fog and 90m where low beam light is used, adopt 90m where wire rope safety barriers are used.

Delineators for sharp curves should be substituted by larger and taller design at closer spacing to highlight the hazard. This is illustrated in **Plate 12.2**.

Plate 12.2 Alignment delineator for sharp curves

12.2.2 ACCESS DELINEATOR

Access delineators are provided at direct frontage accesses and intersections to highlight their presence and position. This is illustrated in **Plate 12.3.** They are not required for major intersections and roundabouts where the layout is clearly defined. They are not necessary on urbanised sections where lighting is provided.

Plate 12.3 Access delineators

At least one access delineator is required on each side of an access. For larger intersections, it is desirable to install two or more units to reveal the layout. A possible colour scheme is white with a red colour retroreflective strip wrapping around the post.

12.2.3 HAZARD MARKERS

Hazard markers are used to alert drivers of roadside hazards with the risk of frontal collisions. An example is illustrated in **Plate 12.4.** They are needed if the hazard cannot be readily treated.

Plate 12.4 Hazard markers

Hazard markers preferably have a rectangular sign face in stripes. They may be preceded by alignment delineators guiding approach traffic to avoid the hazard. Additionally, it could be desirable to apply reflective paints in white colour or strips of yellow and black colour (or strips of red and white colour) directly onto the hazard e.g., ending of bridge parapet or drainage structures (see **Plate 12.5**). If the hazard presents relatively low risk, painting alone may substitute the hazard marker. reflective paints in white colour or strips of yellow and black colour (or strips of red and white colour) directly onto the hazard

Plate 12.5 Strips of white and red reflective paints applied directly onto bridge hazard

12.2.4 LANE DELINEATOR

Lane delineators are circular flexible posts installed along centrelines of undivided roads. They are also appropriate for installation over narrow traffic islands separating opposing traffic lanes, slow vehicle lanes or parking areas. They should be 0.7m or more in height for conspicuity.

The main purposes of lane delineators are:

i. Deterrence of corner cutting along curves
ii. Deterrence of overtaking on straight sections or curves
iii. Prevention of wrong-way traffic
iv. Highlighting the presence of traffic islands

Where the interface of an undivided road and a divided road lies on a curve, it may be beneficial to provide a section of lane delineators to reduce the risk of wrong way driving. Similar usage may be applied ahead of traffic islands located within a curve.

Lane delineator may be used along both straight sections and curves to deter overtaking in addition to solid line markings. This is shown in **Plate 12.6**.

Plate 12.6 Centreline delineators

12.2.5 DIVERGE GORE DELINEATOR

Diverge gores are generally delineated by chevron road markings and raised pavement markers without the need for delineators. However, it could be desirable to provide delineators on the approach to serve as additional buffer zone for diverge gores which:

i. are located on curves
ii. have poor visibility
iii. have a high risk of collision

Delineators for diverge gores which are shown in **Plate 12.7** may be of a distinct colour or based on lane delineators. They should be flexible and preferably reboundable.

Plate 12.7 Delineators at diverge gore

12.3 ROAD MARKINGS

12.3.1 LINE MARKING

Line markings are traffic control devices that are installed on paved road surfaces by using lines, characters, and symbols based on a unified format. They serve a variety of functions, including:

i. Lane definition
ii. Separation of opposing flows
iii. Passing control
iv. Lane usage and designation
v. Pedestrian crosswalks
vi. Stop lines
vii. Parking areas

The two general categories of pavement markings are longitudinal and transverse markings.

Classic examples of longitudinal markings are centre, lane and edge line markings. Transverse markings include crosswalk lines, stop lines, and symbol markings.

The motoring public depends heavily upon road markings for guidance, vehicle positioning and information. Unless road markings are clearly visible, consistent and uniform in their application, drivers, pedestrians and other road users may become confused and uncertain of their purpose.

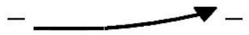
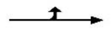
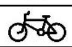
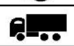
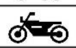
Road markings are often used to supplement the regulations or warnings of other traffic control devices, such as traffic signs or signals. Sometimes they are used alone to convey regulations or warning, which would not be obtainable by other traffic control devices.

The most common facilities for guiding the line of sight of drivers are shown in **Table 12.3** and **Table 12.4**.

Table 12.3 Standards for lane marking

Table 1	No MRT	Line Type	l =line length, b =line width, a =line spacing, m =distance between line marking. All unspecified measures are in meters		TYPE A Moterway				TYPE B Outside built-up area regardless of speed Within Urban area V>60 km/h				TYPE C Within Urban area V<60 km/h			
					l	m	b	a	l	m	b	a	l	m	b	a
Longitudinal line markings	L1		Lane marking line	Central line-lane divider line	5	10	0.15		5	10	0.1		2.5	5	0.1	
	L2		Warning line	Central line	10	5	92		10	5	0.1		5	2.5	0.1	
				Lane divider line	10	5	92		10	5	0.1		5	2.5	0.1	
			Double warning line					0.2				0.1				0.1
	L2A		Continuous line				0.2				0.1				0.1	
	L3		Double Continuous warning line					0.2				0.1				0.1
			Lane marking and continuous line					0.2				0.1				0.1
			Lane marking line and warning line					0.2				0.1				0.1
			Hatch marking					0.2				0.1				0.1
		X = 0.00-0.10 marks = 1/30; Y; X; b a 45	Chevron	Hatch Normal			1	2			0.5	1			0.3 / 0.5	0.5 / 1
			Chevron	Hatch Min.							0.3	0.5			0.3	0.5
			Chevron	Hatch Max.							0.5	0.2				
			Chevron	Continuous marking			0.3				0.1				0.1	
			Chevron	Broken marking Either					1	0.5	0.1		1	0.5	0.1	
			Chevron	Broken marking Or					2.5	1.25	0.1		2.5	1.25	0.1	
			Wide continuous edge line				0.3				0.3				0.3	
	L4		Thin continuous edge line								0.1				0.1	
	L6		Wide broken edge line						0.6	0.6	0.2		0.6	0.6	0.2	
Transverse line markings	L7		Stop line either or				0.5				0.3				0.3	
	L8		Ending of turning lane						0.6	0.6	0.3		0.6	0.6	0.3	
		m; b	Pedestrian crossing	Min.					0.35	0.35	3		0.35	0.35		
			Pedestrian crossing	Max.					0.5	0.5	4		0.5	0.5	4	
			Rumble strips - see specific guidline													
			Road hump max.						0.5	0.5	0.5		0.5	0.5	0.5	
Marking for stopping and parking	L10		No stopping line. Yellow colour								0.1				0.1	
	L9		No parking line. Yellow colour						0.5	0.5	0.1		0.5	0.5	0.1	
			parking bay								0.1				0.1	

Table 12.4 Standards for lane marking

Table 2	Line Type	l =line length b =line width a =line spacing m =distance between line marking All unspecified measures are in meters		TYPE A Moterway				TYPE B Outside built-up area regardless of speed Within Urban area V>60 km/h				TYPE C Within Urban area V<60 km/h			
				l	m	b	a	l	m	b	a	l	m	b	a
Arrow making		Straight arrow	Either	7.5		0.15		5		0.15		5		0.15	
			Or									2.6		0.15	
		Left turn arrow	Either	7.5		0.15		5		0.15		5		0.15	
			Or									2.6		0.15	
		Right turn arrow	Either	7.5		0.15		5		0.15		5		0.15	
			Or									2.6		0.15	
		Broken straight left	Either	7.5		0.15		5		0.15		5		0.15	
			Or									2.6		0.15	
		Broken straight right	Either	7.5		0.15		5		0.15		5		0.15	
			Or									2.6		0.15	
		Curved arrow left	Either					5		0.15		5		0.15	
			Or									2.6		0.15	
		Curved arrow right	Either					5		0.15		5		0.15	
			Or									2.6		0.15	
		Curved arrow left - right arrow	Either					5		0.15		5		0.15	
			Or									2.6		0.15	
		Straight - right arrow	Either	7.5		0.15		5		0.15		5		0.15	
			Or									2.6		0.15	
		Left - right arrow	Either					5		0.15		5		0.15	
			Or									2.6		0.15	
		Straight left - right arrow	Either					5		0.15		5		0.15	
			Or									2.6		0.15	
		Double lane arrow - left						5		0.15		5		0.15	
		Double lane arrow - right						5		0.15		5		0.15	
		Lane arrow with no left turn						5		0.15		5		0.15	
		Lane arrow with no right turn						5		0.15		5		0.15	
		Arrow for change of lane - left		6				6				3			
		Arrow for change of lane - right		6				6				3			
		Straight arrow for cyclist						1.7		0.1		1.7		0.1	
		Left turn arrow for cyclist						1.7		0.1		1.7		0.1	
		Right turn arrow for cyclist						1.7		0.1		1.7		0.1	
		Straight - left arrow for cyclist						1.7		0.1		1.7		0.1	
		Straight - right arrow for cyclist						1.7		0.1		1.7		0.1	
Text and symbols		Triangle symbol give way	Either	6				6				2			
			Or									4			
	STOP	Stop symbol	Either	4				4				1.6			
			Or									4			
	N1	Route numbers	Either	4				4				1.6			
			Or									4			
		Bicycle symbol	Either					1				1			
			Or					2				2			
		Car for disabled W...		See "Text / symbol in parking bay"											
	P	Parking symbol	Either					4				1.6			
			Or									4			
	BUS	Bus symbol	Either	4				4				1.6			
			Or									2.4			
	TAXI	Taxi symbol	Either	4				4				1.6			
			Or									2.4			
Text / symbol in parking bay	BUS	Bus W...								1				1	
	TAXI	Taxi W...								1				1	
		Car for disabled W...								1				1	
		Lorry / Truck W...								1				1	
		Motorbike W...								1				1	

Source GHA: Road Signs - Final Draft 2007

12.3.2 PROFILE MARKING

Profile marking (also known as audio-tactile line markings or raised rib markings) is a type of road marking that generates noise and vibration when a vehicle's tyre rolls over it. It's also called a rumble strip. It consists of a white painted line with raised markings as shown in **Figure 12.1**. The frequency of the markings affects the frequency of the sound generated when a vehicle's tyre comes in contact with it.

It can be used to mark the outer edge of the right lane to help drivers know if they are drifting right (especially where the verge is either soft or non-existent), and on the centre lane to help stop drivers drifting across into opposing traffic.

They are useful to alert sleepy drivers who might have fallen asleep at the wheel, or drivers who are distracted and drifting within the lane. When used in the centreline they can help reinforce no passing rules. In fog, when the line is less visible, they provide tactile and auditory feedback of the lane.

Because it's irritating to a driver to drive on them continuously the lines have a longer life and therefore lower maintenance costs; lines that are driven over regularly wear off quickly. In wet weather at night they provide better visibility of the line. **Table 12.5** and **Plate 12.8** show the specification and example of profile marking respectively.

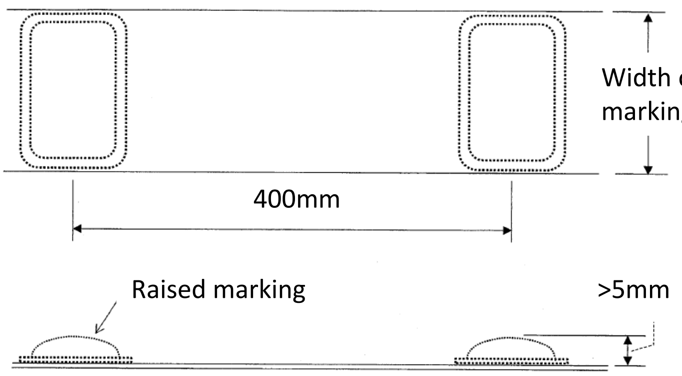


Figure 12.1 Profile marking

Table 12.5 Specification of profile marking

Specification	Dimension
Width (mm)	Same as width of markings
Length (mm)	50
Height (mm)	8mm (Roads with bicycles and pedestrians) 11mm (Roads without bicycles)
Spacing (mm)	200 - 500

Plate 12.8 Example of profile marking

12.3.3 RAISED ROAD MARKERS

Raised road markers (also known as road studs or cat eyes) are discrete markers inserted on to the pavement along edge lines and lane lines for delineation of the road layout. They are generally equipped with a reflector in glass or plastic. Their retro-reflective property reveals the road alignment under the headlights of approach traffic. Raised road markers are desirable to supplement line markings.

They are desirable to improve visibility at night, in adverse weather and where road markings are susceptible to dusts and blackening.

Raised road markers are particularly useful to emphasize curves or on road sections with increased safety risk, including:

i. undivided multi-lane
ii. transition zones with changes in the number of traffic lanes
iii. transition from divided road to undivided road
iv. interchange diverges, merges and weaving sections
v. intersections with turn lanes
vi. tunnels and approaches
vii. narrow bridges and approaches

12.3.3.1 CLASSIFICATION OF ROAD STUDS

The classification of road studs based on colour configuration is as follows:

i. White colour

The white-coloured road studs indicate the traffic lane line and centre of carriageway.

ii. **Red colour**

As the colour red indicates danger, the red-coloured road studs are to be used to indicate a line that should not be crossed and mainly to delineate the right-hand edge of the running carriageway.

iii. **Yellow colour**

The yellow colour road studs are deployed to indicate a line that should not be crossed with the aim to delineate the left-hand edge of the running carriageway in case of the multi-lane divided carriageways.

iv. **Green colour**

Green road studs are to be employed to indicate crossable edge line like the lay byes and to show the boundary of acceleration or deceleration line on right-hand side of the carriageway in case of the multi-lane divided carriageways.

Based on the number of reflective sides, abrasion resistance and flexural strength, road studs are divided as follows-

a. Type A - Two-way reflective markers, one colour.

b. Type B - One-way reflective markers, one colour.

c. Type C - Two-way reflective markers, two colours.

d. Designated H - Stud with a hard, abrasion-resistant lens surface.

e. Designated F - Stud with sufficient longitudinal strength for application to flexible, asphaltic concrete pavement.

12.3.3.2 STANDARD REQUIREMENT OF ROAD STUDS

The standard requirements of road studs are divided into 2 categories:

i. **Construction requirement**

a. The road stud shall be manufactured of materials with adequate chemical, water, and Ultraviolet (UV) resistance.

b. The height and width of the road stud shall not exceed 20.3 mm and 130 mm respectively.

c. The angle between the face of the road stud and the base shall be no greater than 45°.

d. The base of the road stud shall be flat within a tolerance of 1.3 mm.

e. The base of the road stud shall be substantially free from gloss or substances that may reduce its bond to adhesive.

ii. Performance requirement

a. The coefficient of luminous intensity of road studs shall be not less than the values mentioned in **Table 12.6**.

b. The adhesive bond used for the installation of the road stud must be of superior quality and the failure of road studs due to adhesive strength must be 1.5 times lesser than the failure of road studs without the adhesive strength.

Table 12.6 Required Coefficient of Luminous Intensity (CIL) of Road Studs

Entrance Angle, β (^{0})	Observation Angle, α (^{0})	Minimum value CIL (R_i) mcd/lx				
		White	Yellow	Red	Green	Blue
0	0.2	279	167	70	93	28
+20/-20	0.2	112	67	28	37	10

12.3.3.3 PLACEMENT OF ROAD STUDS

i. The road studs installed on the longitudinal markings shall always be placed at the centre of the gap and shall never be installed upon the line segment or by the side of the line segment.

ii. In the case of the road studs placed on the carriageway having a paved shoulder, it shall be placed outside the shoulder side edge line and shall have a set back by a distance of 50 mm from the edge line.

iii. Road studs provided on the median side shall not be on the median line marking but shall be in the hard strip or borderline width with a setback of 50mm from the median and 100 mm from the vertical face of the raised kerb.

iv. In extreme circumstances, if the width of hard strip or borderline is not adequate enough to accommodate the required set back distance of 50 mm from the edge line, the road studs can be placed adjacent to the edge line and even directly on the edge lines.

Table 12.7 and **Table 12.8** indicate the placements, colour and spacing of road studs on undivided and divided roads respectively. The placement, colour and spacing of road studs on undivided roads and divided roads are shown in **Figure 12.2** and **Figure 12.3** respectively.

Table 12.7 Placement colour and spacing of road studs on undivided roads

Description			Normal Section			Warning Section			Overtaking Section		
Road Category	Traffic Movement	Carriageway width (m)	Centre Line	Edge Line	Traffic Lane Line	Centreline	Edge Line	Traffic Lane Line	Centreline	Edge Line	Traffic Lane Line
Single/ Intermediate Lane Road	Two way	<5.5	N A	Red-White Bi Directional at 20 m interval (optional)	N A	N A	Road Wheel Bi Directional at 10 m interval (optional)	N A	N A	Red White Bidirectional at 5 m interval (Desirable)	N A
Two Lane road	Two way	5.5 to 7	Red-White Bi Directional at20m interval (optional)	Red-White Bi Directional at 20 m interval (optional)	N A	Road Wheel Bi Directional at 10 m interval (Desirable)	Road Wheel Bi Directional at 10 m interval (Desirable)	N A	Yellow Bidirectional at 5 m interval (Desirable)	Red White Bidirectional at 5 m interval (Desirable)	N A
Two Lane road with paved shoulder	Two way	> 7	Red-White Bi Directional at 20 m interval (optional)	Red-White Bi Directional at 20 m interval (optional)	N A	Road Wheel Bi Directional at 10 m interval (Desirable)	Road Wheel Bi Directional at 10 m interval (Desirable)	N A	Yellow Bi Directional at 5 m interval (Desirable)	Red White Bidirectional at 5 m interval (Desirable)	N A
Three Lane undivided road	Two way	> 11	Yellow-Yellow Bi Directional at20 m interval (Desirable)	Red-White Bi Directional at 20 m interval (optional)	Not Required	Red-Wheel Bi Directional at 10 m interval (Desirable)	Red-Wheel Bi Directional at 10 m interval (Desirable)	Not Required	Yellow Bi Directional at 5 m interval (Desirable)	Red White Bi Directional at 5 m interval (Desirable)	White- White Bi Directional at 5 m interval (Desirable)
Four Lane undivided road	Two way	> 14	Yellow-Yellow Bi Directional at 20 m interval (Desirable)	Red White Bi Directional at 20 m interval (optional)	Not Required	Red-White Bi Directional at 10 m interval (Desirable)	Red-White Bi Directional at 10 m interval (Desirable)	Not Required	Yellow Bi Directional at 5 m interval (Desirable)	Red-White Bi Directional at 5 m interval (Desirable)	White- White Bi Directional at 5 m interval (Desirable)

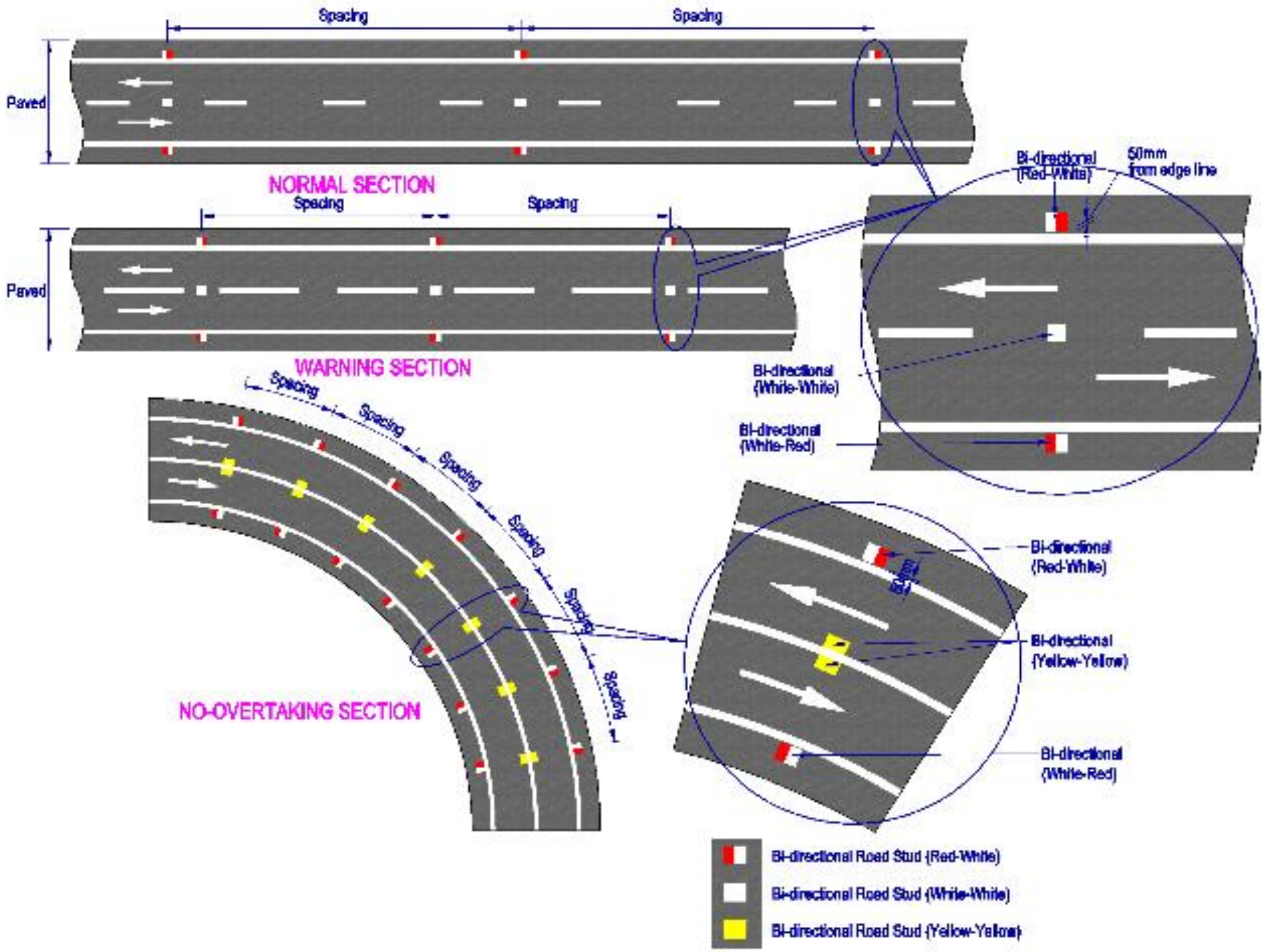


Figure 12.2 Placement, colour and spacing of road studs on undivided roads

Table 12.8 Placement, colour and spacing of roads studs on divided roads

Description	One Side Carriageway Width (m)	Normal Section			Warning Section			No overtaking Section		
Road Category		**Traffic Lane**	**Shoulder Side Edge line**	**Median Side Edge Line**	**Traffic Lane Line**	**Shoulder Side Edge line**	**Median Side Edge Line**	**Traffic Lane Line**	**Shoulder Side Edge line**	**Median Side Edge Line**
Four Lane divided Carriage ways	< 7.8	Not required	Red- Unidirectional at 20 m interval (desirable)	Yellow - Unidirectional at 20 m interval (desirable)	White- White Bi Directional at 10 m interval (optional)	Yellow - Unidirectional at 20 m interval (desirable)	Red- Unidirectional at 10 m interval (desirable)	White unidirectional at 5 m interval (Desirable)	Red unidirectional at 5 m interval (Desirable)	Yellow unidirectional at 5 m interval (Desirable)
Six Lane divided Carriage ways	> 10.8	Not required	Red- Unidirectional at 20 m interval (desirable)	Yellow - Unidirectional at 20 m interval (desirable)	White- White Bi Directional at 10 m interval (optional)	Yellow - Unidirectional at 20 m interval (desirable)	Red- Unidirectional at 10 m interval (desirable)	White unidirectional at 5 m interval (Desirable)	Red unidirectional at 5 m interval (Desirable)	Yellow unidirectional at 5 m interval (Desirable)
Two Lane road with paved shoulder	> 9.5	Not required	Red- Unidirectional at 20 m interval (desirable)	Yellow - Unidirectional at 20 m interval (desirable)	White- White Bi Directional at 10 m interval (optional)	Yellow - Unidirectional at 20 m interval (desirable)	Red- Unidirectional at 10 m interval (desirable)	White unidirectional at 5 m interval (Desirable)	Red unidirectional at 5 m interval (Desirable)	Yellow unidirectional at 5 m interval (Desirable)
Four Lane divided expressway	12	Not required	Red- Unidirectional at 20 m interval (desirable)	Yellow - Unidirectional at 20 m interval (desirable)	White- White Bi Directional at 10 m interval (optional)	Yellow - Unidirectional at 20 m interval (desirable)	White unidirectional at 5 m interval (Desirable)	Yellow Bi Directional at 5 m interval (Desirable)	Red unidirectional at 5 m interval (Desirable)	Yellow unidirectional at 5 m interval (Desirable)

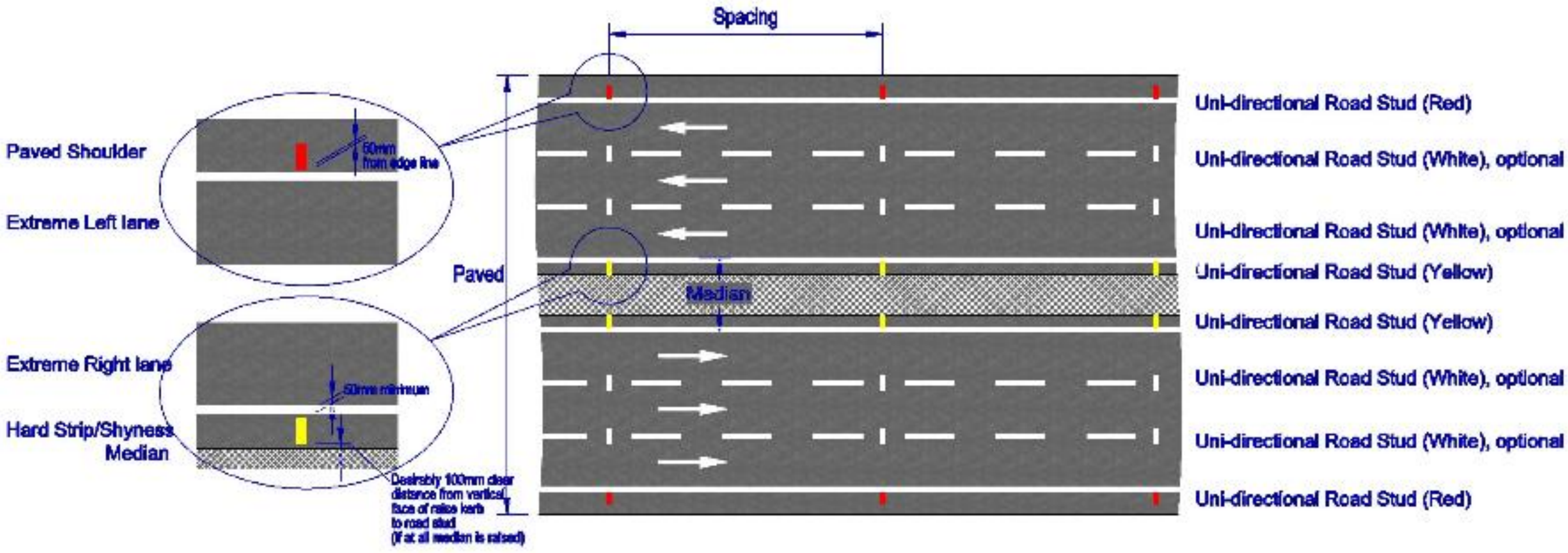


Figure 12.3 Placement, colour and spacing of road studs on divided roads

12.4 ROAD SIGNS

Road signs (Traffic signs) provide essential information to drivers for their safe and efficient manoeuvring on the road. They have the function of conveying clear and unambiguous information such as guidance, warnings, controls, and instructions to road users so that they can be understood quickly and easily. Road signs should be consistent in terms of provision criteria, size and mounting arrangement.

To be effective, signs need to be sited so that the correct information is given to road users when they need it - not too soon or too late – so that they are given sufficient time to carry out the required manoeuvre in safety.

Traffic signs are of four (4) general types:

i. Warning Signs: indicate a potential hazard, obstacle or condition requiring special attention.
ii. Regulatory sign:
    - Prohibitory Signs: are used to indicate prohibitions or restrictions which the road user must not violate.
    - Mandatory Signs: are used to give positive instructions with which the road user must comply.
iii. Informatory Signs: are used to convey information of use to the driver.
iv. Guidance Signs: are used to show route designations, destinations, directions, distances, services, points of interest, and other geographical, recreational, or cultural information.

Standards for road signs and their placement are provided in the MRH's Standard Details, Road Signs and Markings for Urban and Trunk Road (1991), MRT Standard Specification for Road and Bridge Works and GHA Road Signs – Final Draft (2007).

12.4.1 TUNNEL SIGNING

For unidirectional tunnels, lane changing should be discouraged or prohibited. For bidirectional tunnels, lane changing should be prohibited using double solid line markings. A widened centreline may be considered for additional safety.

Traffic signs and directional signs should be coordinated in positioning and information flow. They should not constitute sign clutter or overload of information.

The following traffic or informatory signs should be provided ahead of tunnels:

i. Tunnel name or symbol and length
ii. Speed limit, and minimum speed limit if appropriate

iii. Headroom
iv. Turn on Radio if radio break-in system is in place
v. Use low head beam lights
vi. Keep distance apart

A sign giving essential information may be erected ahead of the tunnel. This is illustrated in **Figure 12.4**. For very long tunnels, notably those longer than 3 km, it is advisable to provide informatory signs inside tunnels stating the remaining length of the tunnel every 1000m.

Figure 12.4 Possible sign display containing essential information

12.4.1.1 EMERGENCY SIGNING SYSTEM

An emergency signing system is required to assist road users to access emergency equipment and to evacuate a tunnel. The system should consist of self-illuminated signs fed from a separate power source. Signs should be based on graphics rather than worded messages. Essential emergency signs are given in **Table 12.9**.

Table 12.9 Emergency signing system

Sign Type	General Requirements	Typical Design
Layby sign	At the start of layby	 
Layby advance sign	200m - 300m ahead of the start of a layby	 
Telephone and fire equipment	At emergency stations, mounting height at 1.2 to 1.5m	
Emergency exit signs	At emergency exits, mounting height at 1.2 to 1.5m	
Direction to emergency exits	towards both directions, mounting height at 1.2 to 1.5m	

12.4.1.2 FIXED AND VARIABLE SIGNING

A. Directional Signing

Where a tunnel is located within the directional signing sequence of an interchange, directional signs should be provided ahead of the tunnel. The provision and positioning of directional signs should take into account any restriction of lane changing within and in the vicinity of the tunnel tube.

It may be desirable to install directional signs inside a tunnel tube due to presence of interchange inside or in the proximity of tunnel exits. This would require adequate headroom or roadside space.

Directional signs at reduced size and simplified sign faces may be acceptable if there are severe technical or economic constraints.

Where a directional sign is installed shortly downstream of a tunnel tube, there should be sufficient distance to ensure that the sign can be properly read by drivers.

B. Delineation

Where a wide paved shoulder on the approach becomes a narrow shoulder inside the tunnel, narrowing should be introduced progressively in a taper. Hatched markings may also be laid as illustrated in **Figure 12.5**. If adopted, solid lane line markings preferably commence or terminate not shorter than 20m from the tunnel portal.

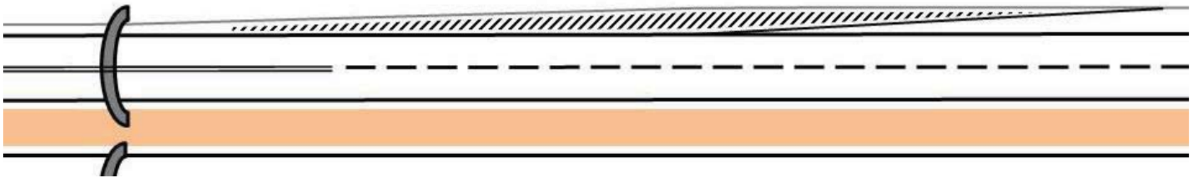


Figure 12.5 Road marking at narrowing of shoulders

Consideration should be given to adopting a wide centreline marking to separate opposing traffic in a bidirectional tunnel tube. This is illustrated in **Plate 12.9**.

Plate 12.9 Wide centreline in a bidirectional tunnel

Visual guidance inside tunnels may be enhanced with reflectors or LED light sources at regular intervals along the walkway and/or the tunnel wall. This is illustrated in **Plate 12.10**.

Plate 12.10 Delineation in a unidirectional tunnel

C. Variable Signing

Appropriate facilities are required to implement various operation modes and may include all or part of the followings:

i. Traffic signals
ii. Lane control signals
iii. Variable directional signs
iv. Variable speed limit signs
v. Variable message signs

Lane control signals will need to be double-sided for bidirectional traffic or contra-flow operation.

Messages on variable message signs may be in the form of “Event”, “Location”, “Action”. Information about the event and location is beneficial but may be omitted if these are evident. Symbolic warning signs are also desirable as long as they are readily understood.

Useful messages for severe incidents or fire inside tunnels may include “REDUCE SPEED”, “SWITCH OFF ENGINES”, “EVACUATE”.

Flashing lights or flashing messages could be desirable to alert drivers of fixed or variable signs.

D. Barrier Systems

Barrier gates should be provided to close off a tunnel tube in case of emergency or maintenance. They may be horizontally or vertically operated. In order to allow emergency vehicles to access a closed tunnel tube, a gap should be allowed on one side, in the middle or by staggering two barrier gates.

If lane closure is frequently envisaged, consideration should be given to a horizontal swing gate system on the approach.

Barrier systems should be operated in conjunction with variable message signs and lane control signals to provide sufficient warning and guidance for approaching drivers.

Barrier systems should be passively safe if collided by an errant vehicle. Their support mechanisms should be guarded by safety barriers.

12.5 TRAFFIC SAFETY BARRIERS

Traffic safety barriers are systems utilized to shield road users from potential hazards alongside the travelled way and should be able to redirect or contain:

i. An errant vehicle without imposing intolerable vehicle occupant forces.
ii. Vehicles in range of sizes, weights and designs.
iii. An errant vehicle over a range of impact speeds and impact angles.

Traffic safety barriers are themselves a source of crash potential, and vehicles striking them can cause injury or death to vehicle occupant(s). A traffic safety barrier should be installed only if it is likely to reduce the severity of potential collisions. It is therefore of the utmost importance that, in the selection of the traffic barrier, due cognisance be taken of the characteristics of the proposed barrier system. Barrier systems differ not only in purpose but also in terms of deflection and redirecting properties.

Traffic barriers are either classified as being impact attenuation devices or longitudinal barriers. The purpose of an impact attenuation device is to cause a vehicle to decelerate and come to a halt. A longitudinal traffic barrier redirects a vehicle parallel to the roadway.

Figure 12.6 shows a functional classification of traffic barriers.

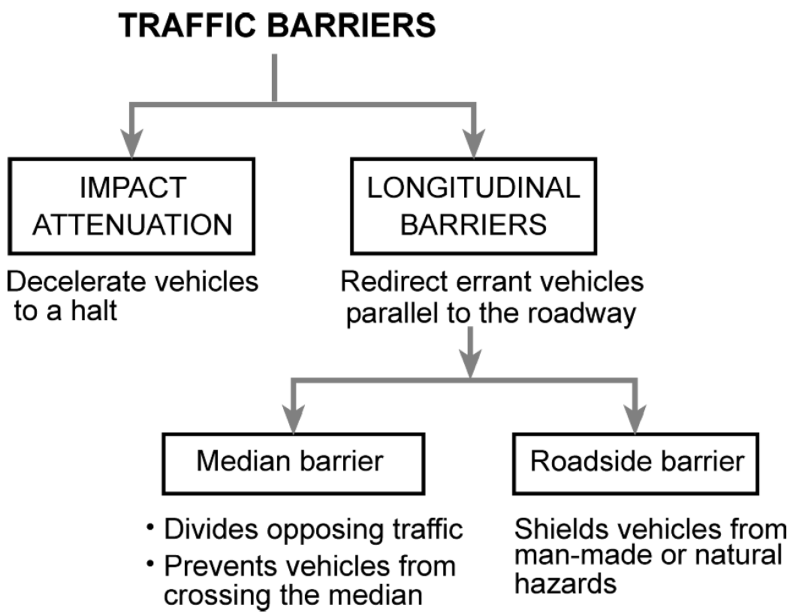


Figure 12.6 Classification of traffic barriers

12.5.1 WARRANTS FOR SAFETY BARRIERS

Barriers are installed on the basis of warrant analysis. Traditionally, these warrants have been based on a subjective analysis of certain roadside elements or conditions within the clear zone. If the consequences of a vehicle running off the road and striking a barrier are believed to be less serious than the consequences if no barrier existed, the barrier is considered warranted.

Although this approach can be used, there are often instances where the distinction between the two conditions is not immediately obvious.

In addition, this approach does not allow for consideration of the cost-effectiveness of treatment or non-treatment.

In recent years, techniques have been developed which allow warrants for barrier installation to be established on the basis of a cost-benefit analysis in which such factors as design speed, traffic volume, installation and maintenance costs, and collision costs are taken into consideration.

Typically, such an approach is used to evaluate three options:

i. The removal or alteration of the area of concern so that it no longer requires shielding.
ii. The installation of an appropriate barrier.
iii. Leaving the area of concern unshielded (usually only considered on low-volume and/or low-speed facilities).

Once a barrier is found to be necessary for an embankment, it should be provided over the entire length of the embankment and not simply terminated when the embankment height becomes less than the warranted height.

Barrier warrants for roadside obstacles are based on their location within the clear zone and are a function of the nature of the obstacle, its distance from the travelled portion of the roadway and the likelihood that it will be hit by an errant vehicle.

Conventional criteria used for embankments and roadside hazards are not usually applicable to the pedestrian/bicyclist case, and these are usually resolved through a careful individual evaluation of each potential project.

As with roadside barriers, warrants for median barriers have been established on the basis that a barrier should be installed only if the consequences of striking the barrier are less severe than the consequences that would result if no barrier existed. The primary purpose of a median barrier is to prevent an errant vehicle from crossing a median on a divided highway and encountering oncoming traffic. As such, the development of median barrier warrants has been based on an evaluation of median crossover collisions and related research studies.

In determining the need for barriers on medians, median width and average daily traffic volumes are the basic factors generally used in the analysis. However, the incidence of illegal cross-median movements may also justify the use of median barriers.

Warrants for implementing impact attenuation devices (crash cushions) are based on shielding a fixed object within the clear zone that is considered to be a hazard and cannot be removed, relocated, made breakaway, or adequately shielded by a longitudinal barrier.

In considering the use of traffic barriers, designers should note that, even when these are properly designed and constructed, they might not protect errant vehicles and their occupants completely.

After installation of these, the severity of collisions generally decreases but, as the number of installations increases, the frequency of minor collisions may also increase. For this reason, where cost-effective, the designer should make every effort to design without traffic barriers. This can be done by clearing the roadside of obstacles, flattening embankment slopes and introducing greater median separation where possible.

It should be noted, however, that, whilst a particular barrier system is chosen based on the containment level required, regular monitoring is essential to allow the system to be replaced by a more adequate one if experience indicates the need for this.

12.5.2 LONGITUDINAL ROADSIDE BARRIERS

Longitudinal roadside barriers are vehicle restraint systems provided on the roadside to contain and smoothly redirect an errant vehicle.

Safety barriers should be considered where clear zones cannot be satisfactorily provided. Their need may be evaluated initially in terms of roadside conditions and approach speeds with priority given to:

i. Traffic speeds >= 70km/h
ii. horizontal curves less than 250m radius on Design Class A and B roads
iii. horizontal curves less than 150m radius on other roads
iv. road sections on and at the bottom of long steep downhill grades
v. High embankment
vi. Road sections adjacent to railway. An adjacent railway line which is at a lower level than the road should require a roadside barrier provided the distance of separation is less than 10m as shown in **Figure 12.7 (a)**. On the other hand, adjacent railway lines at almost the same level or higher may be provided with roadside where the distance between them is less than 5m as shown in **Figure 12.7 (b)** and **Figure 12.7 (c)**.

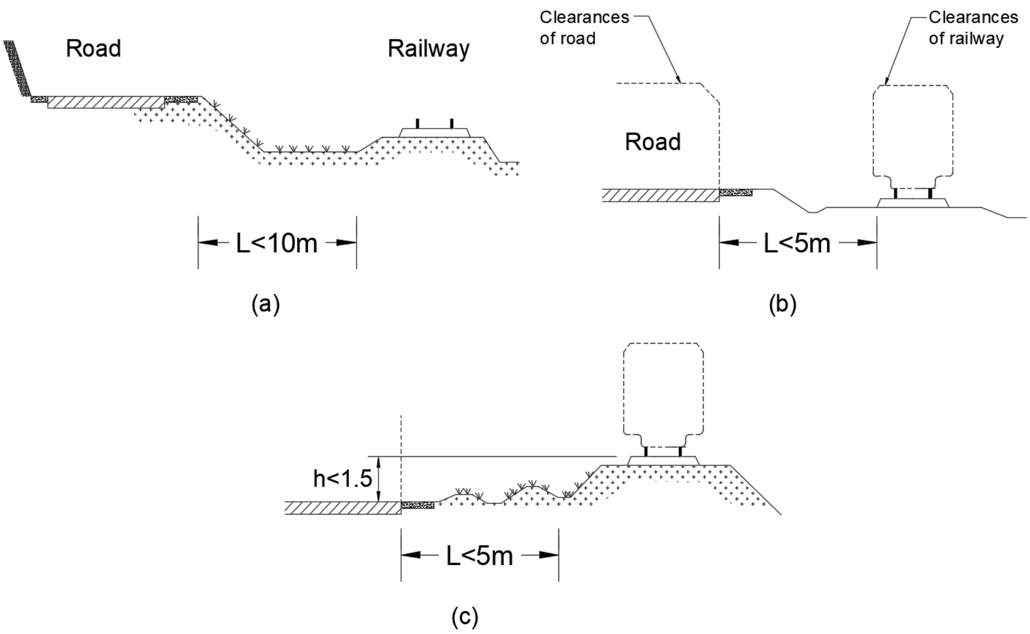


Figure 12.7 Road sections adjacent to railway

vii. Sudden change in alignment Safety fences may be set up along section with sudden reduction in lane widths and small curve radii (bellow 100m) in embankment sections.
viii. Other road structures. Structures like piers, abutments on carriageways or within 2m from the edge of carriageways are protected from the damaging effects of defiant vehicles by installing roadside barriers.
ix. Adjacent road features Roadside barriers may also be installed on roadsides adjacent to features such as seas, rivers, lakes, swamps, cut face and structures like rock outcrops and building in embankment.

12.5.2.1 CLASSIFICATION AND PERFORMANCE CHARACTERISTICS

Figure 12.8 shows the classification of longitudinal barriers based on their deflection characteristics. It should be noted that the deflection characteristics of a barrier system are not an indication of its effectiveness or safety.

Misconceptions exist regarding the advantages of the different longitudinal barrier types. Some engineers firmly believe that one system is better than another based on its deflection characteristics, however, the deflection characteristics of a particular system are not a measure of its effectiveness.

The mechanisms by which a vehicle is restrained after impacting a traffic barrier differ completely depending on the type of barrier selected. The reaction of a vehicle on impact with different types of barriers is thus also different.

In accomplishing their task of guiding and redirecting striking vehicles, a longitudinal barrier should balance the need to prevent penetration of the barrier with the need to protect the occupants of the vehicle.

Various barrier technologies achieve this in various ways and can be grouped into three distinct types as shown in **Figure 12.8**.

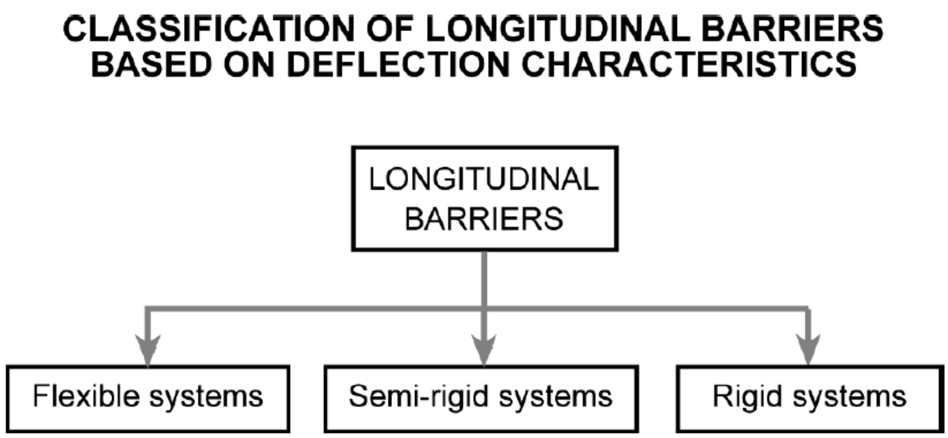


Figure 12.8 Classification of longitudinal barriers

i. **Flexible systems**
   This results in large lateral barrier deflections, but the lowest vehicle deceleration rates. Such systems have application in places where a substantial area behind the barrier is free of obstructions and/or other hazards within the zone of anticipated lateral deflection. These barriers usually consist of a weak post-and-beam system, and their design deflections are typically in the range of 3.2 to 3.7 m but can be as low as1.7m.

ii. **Semi-rigid systems**
   This provides reduced lateral barrier deflections, but higher vehicle deceleration rates.

These barrier systems have application in areas where lateral restrictions exist and where anticipated deflections have to be limited. They usually consist of a strong post-and-beam system and have design deflections ranging from 0.5 to 1.7 m.

iii. **Rigid systems**

This usually takes the form of a continuous concrete barrier. These technologies result in no lateral deflection but impose the highest vehicle deceleration rates. They are usually applied in areas where there is very little room for deflection or where the penalty for penetrating the barrier is very high. Numerous shapes are available, including a high version for use where there is a high percentage of trucks.

Designers should familiarize themselves with, and design to, the specific performance characteristics of their selected or candidate technologies.

12.5.2.2 SELECTION GUIDELINES

Once it has been decided that a roadside barrier will be installed, a specific barrier type is selected. Although the number of variables and the lack of objective criteria complicate this selection process, there are some general guidelines that may be followed. The preferred system is usually the one that offers the selected degree of shielding at the lowest cost for the specific application. Some other factors to be considered in selection of a roadside barrier include: route classification, speed, traffic volume and composition, roadway alignment, deflection space available behind the barrier, intersection sight distance, impact frequency, and construction and maintenance issues. **Table 12.10** summarizes some of the factors that should be considered before making a final selection.

Table 12.10 Selection criteria for roadside barrier

Criteria	Comments
Performance capability	Barrier should be structurally able to contain and redirect the design vehicle for the appropriate test level. The "design conditions" for a particular barrier need to be assessed carefully because areas with poor geometrics, high traffic volumes, high speeds and a large proportion of heavy vehicles might not be consistent with the "conditions" assumed when the barrier had been tested. Such sites might require barriers with a higher-than-normal performance level.
Deflection	Expected deflection of barrier should not exceed available deflection distance. Zone Of Intrusion (ZOI) should be considered. ZOI is the region measured above and behind the face of a barrier system where an impacting vehicle or any major part of the system may extend during an impact.

Criteria	Comments
Site conditions	Site conditions play a major role in the selection of appropriate barriers. The slope approaching a flexible barrier should, for example, not exceed 10 per cent and rigid barriers should not be used where the expected impact angle is large. Narrow fill sections could result in conditions where post spacing and post support might be inadequate to allow them to perform as intended. A number of site-specific aspects will have a major influence on the selection of a particular type of barrier to meet the performance requirements at that location.
Compatibility	Barrier should be compatible with planned terminal or anchorage and capable of transitioning to other barrier systems (such as bridge railing).
Life cycle costs	It is prudent to realize that any barrier system accrues costs throughout its life. High initial costs could mean low maintenance costs, whilst low initial costs could mean higher maintenance costs. In addition, expected accident costs should be considered in the calculation of life cycle costs.
Maintenance	Most systems require very little routine maintenance. When the barrier has been involved in a crash, the subsequent rehabilitation costs may be significant, to the point of being excessive in the case of a high accident location. It should be noted that only material specifically designed for that particular system should be used for maintenance, and the tendency to "mix and match" should be avoided.
Aesthetics	It is important to realize that all traffic barriers are visual obstructions. Should this become a particular concern, it is necessary to ensure that alternative systems that may be considered are able to meet the performance requirements. Aesthetics should, under no circumstances, be given preference before safety considerations.
Field experience	Site personnel's' experience of the performance, cost and maintenance requirements of installed systems as well as the traffic police services' experience of the performance of particular barrier systems under impact conditions, should not be under-estimated by the designer. Early identification of potential problems can ensure that future installations operate effectively.

12.5.2.3 WARRANTS FOR USE

Roadside hazards that warrant shielding by barriers include embankments and roadside obstacles. Warrants for the use of barriers on embankments generally use embankment height and side slope as the parameters in the analysis and essentially compare the collision severity of hitting a barrier with the severity of going down the embankment. **Figure 12.9** provides guidance for the installation of such barriers on embankments. However, such warrant procedures are regarded as less than adequate because they do not take into account the probability of a crash occurring against the barrier or the cost of installing a barrier versus leaving the slope unprotected. The development of cost analysis techniques provides the designer with an approach to analysing the need for roadside embankment protection barriers.

In Ghana, however, there is a lack of reliable data to carry out such analyses and it is necessary for the designer to make site-specific analyses, using **Figure 12.9** as a guide. The significance of this figure is that, it provides a range of values of fill slope for which, at certain heights of fill, a barrier may be more or less hazardous than the embankment it protects. For example, at a fill height of 6 metres, a fill slope steeper than 1:3 would warrant the use of a barrier while a fill slope flatter than 1:4 would not require protection. On the intervening slopes, the designer should use his or her discretion in determining the need for a barrier.

In respect of roadside obstacles that need protection from errant vehicles (or vice versa), warrants for shielding or otherwise can be developed using a quantitative cost-effective analysis, which takes the characteristics of the obstacle and its likelihood of being hit into account.

However, once again the designer must examine each site specifically to determine the necessity or otherwise for shielding. **Table 12.11** provides an overview of the types of non-traversable terrain and fixed objects that are normally considered for shielding. While roadside obstacles immediately adjacent to the travelled way are usually removed, relocated, modified, or shielded, the optimal solution becomes less evident as the distance between the obstruction and the travelled way increases. **Table 12.11** is intended as a guide to aid the designer in determining whether the obstruction constitutes an obstacle to an errant motorist that is significant enough to justify action. Most man-made objects incorporated into a highway project can be designed to minimize the concern they present to a motorist and thus make shielding unnecessary. This is particularly true of drainage features such as small culverts and ditches.

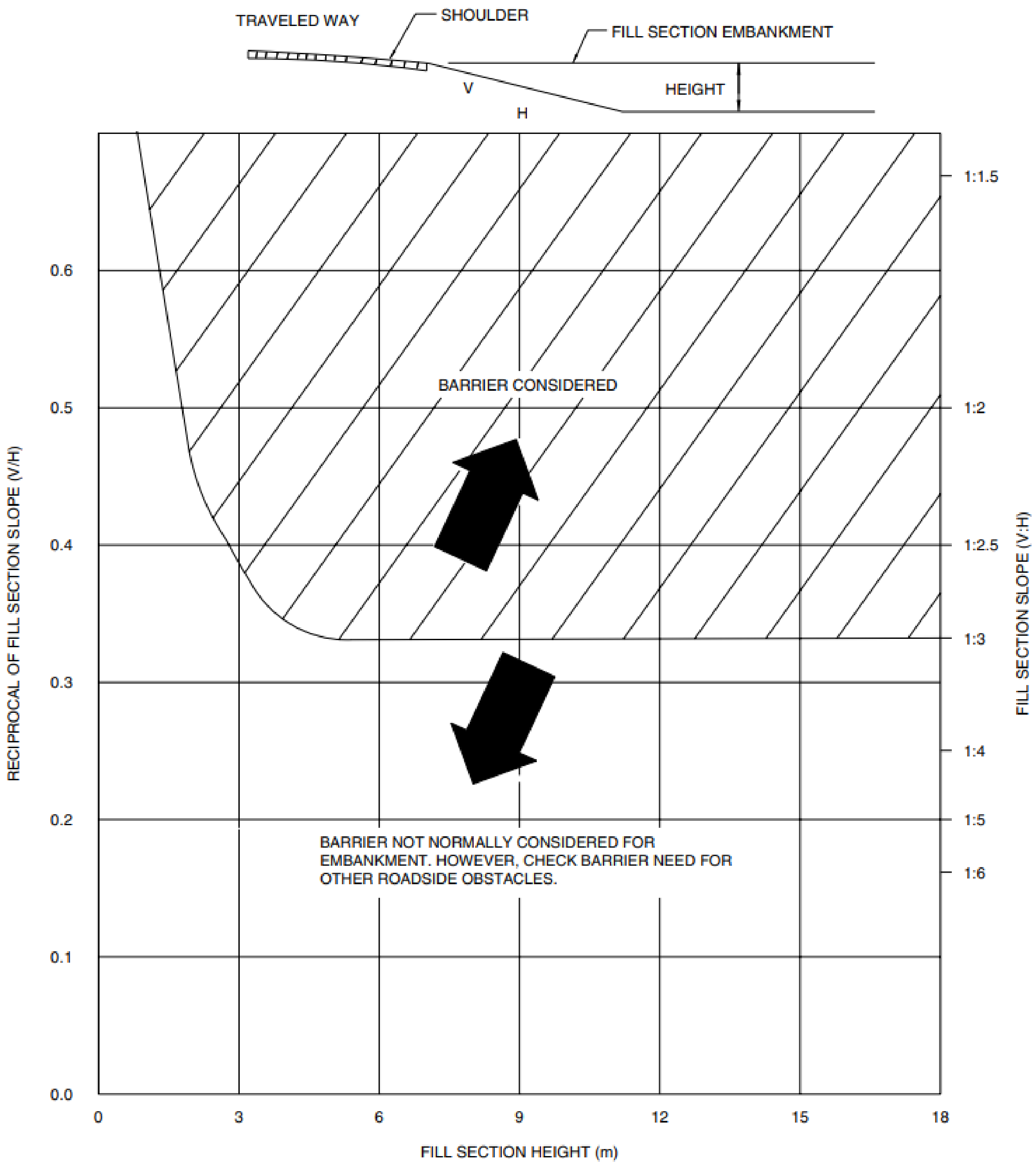


Figure 12.9 Warrants for use of roadside barriers

Table 12.11 Barrier Guidelines for Non-Traversable Terrain and Roadside Obstacles[a,b]

Terrain or obstacle	Comment
Bridge piers, abutments, railing ends	Shielding generally needed
Boulders	Judgment decision based on nature of fixed object and likelihood of impact
Culverts, pipes, headwalls	Judgment decision based on size, shape and location of obstacle.
Foreslopes and backslopes (smooth)	Shielding not generally needed.
Foreslopes and backslopes (rough)	Judgment decision based on likelihood of impact
Ditches (parallel)	Judgement based on likelihood of impact
Ditches (transverse)	Shielding generally needed if likelihood of head-on impact is high.
Embankments	Judgment decision based on fill height and slope (see **Figure 12.9)**
Retaining walls	Judgment decision based on relative smoothness of wall and anticipated maximum angle of impact.
Sign and luminaire support[c]	Shielding generally needed for non-breakaway supports.
Traffic signal supports[d]	Isolated traffic signals within clear zone on high-speed (≥ 80km/h) rural facilities may need shielding
Trees	Judgment decision based on site-specific circumstances.
Utility poles	Shielding may be needed on a case-by-case basis.
Permanent water bodies	Judgment decision based on location and depth of water and likelihood of encroachment

Notes:

a) Shielding non-traversable terrain or a roadside obstacle is usually necessary when it is within the clear zone and cannot practically or economically be removed, relocated, or made breakaway, and it is determined that the barrier provides a safety improvement over the unshielded condition.

b) Marginal situations, with respect to placement or omission of a barrier, will usually be decided by crash experience, either at the site or at comparable site(s).

c) Where appropriate, most sign and luminaire supports should be of a breakaway design regardless of their distance from the roadway if there is reasonable likelihood of their being hit by an errant motorist. The placement and locations for breakaway supports also should consider the safety of pedestrians from potential debris resulting from impacted systems.

d) In practice, relatively few traffic signal supports, including flashing light signals and gates used at railroad crossings, are shielded. If shielding is deemed necessary, however, crash cushions are sometimes used in lieu of a longitudinal barrier installation.

In some situations, a measure of physical protection may be required for pedestrians or bicyclists using, or in close proximity to, a major road or highway.

Examples of such cases could include:

i. A barrier adjacent to a school boundary or property to minimize potential vehicle contact.
ii. Shielding businesses or residences near the right of way in locations where there is a history of run-off-the road crashes.
iii. Separating pedestrians and/or cyclists from vehicle flows in circumstances where high-speed vehicle intrusions onto roads or walkway areas might occur.

In all these cases, conventional criteria will not serve to provide warrants for barriers, and the designer should be aware of the needs and circumstances of the individual situation when deciding on appropriate action.

12.5.2.4 SAFETY BARRIER TYPES

Safety barrier types should satisfy the requirements given in "Manual for Assessing Safety Hardware" (MASH, US), EN1317-2 (EU) or equivalent standard. **Figure 12.10** shows the shape of common safety barrier types. **Table 12.12** and **Table 12.13** provide a summary of their typical performance. Actual performance depends on design details and is subject to crash testing. **Table 12.14** and **Table 12.15** are additional considerations for the use of typical safety barrier types. **Table 12.16** shows special consideration for usage of typical safety barrier types.

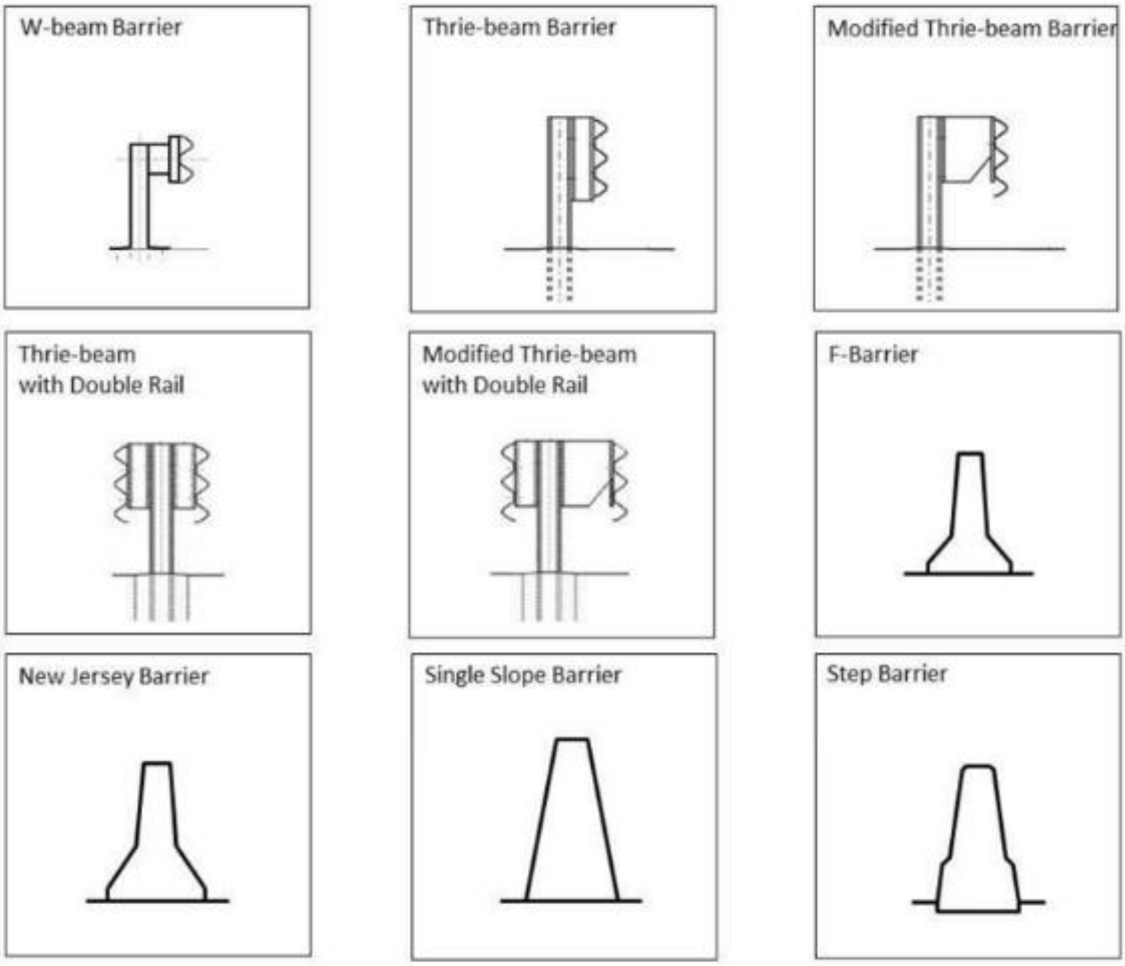


Figure 12.10 Common safety barrier types

Table 12.12 Typical safety barrier performance

Safety Barrier Type	Category	MASH	EN1317-2			Note
			Containment Level	Working width*	ASI**	
Flexible and Semi flexible systems						
Wire Rope Barrier	Flexible	TL-3	N2/H1	W4-W8	A	Need for substantial space for dynamic deflection
Box-beam barrier	Semi-flexible	TL-2/TL-3	N2	W1-W5	A	Susceptible to penetration and rollover for larger vehicles
W-beam barrier	Flexible/semi flexible	TL-2/TL-3/TL4	N2/H1/H2	W2-W6	A	Susceptible to penetration and rollover for larger vehicles
Thrie-beam barrier	Semi-flexible	TL-3/TL-4	No data	W4-W5	No Data	
Modified Thrie-beam barrier	Semi-flexible	TL-3/TL-4	No data	W4-W5	No Data	Suitable for larger vehicles
Thrie-beam or modified Thrie-beam barrier with rails on both sides	Semi-flexible	TL-5	H2	W4-W6	No Data	Suitable for heavy vehicles, Modified Thrie-beam barrier on the traffic face is preferred for larger vehicle
Rigid systems: Concrete Safety Barriers						
Low profile barrier	Rigid	TL-2	No data	W1-W2	No data	
New Jersey barrier	Semi-rigid/Rigid	TL-3/TL-4/TL-5	H2/H3	W1-W2	B/C	More potential for rollover of light vehicles
F-barrier	Semi-rigid/Rigid	TL-4/TL-5	H2/H3	W1-W2	B/C	Less potential for rollover of light vehicles

Safety Barrier Type	Category	MASH	EN1317-2			Note
			Containment Level	Working width*	ASI**	
Vertical barrier	Semi-rigid/Rigid	TL-4/TL-5	H2/H3	W1-W2	B/C	More potential for redirection onto oncoming vehicles
Single slope barrier	Semi-rigid/Rigid	TL-4/TL-5	H2/H3	W1-W2	B/C	
Step barriers	Semi-rigid/Rigid	TL-4/TL-5	H2/H3	W1-W2	B/C	Less potential for rollover of light vehicles
Very high containment barriers	Rigid	TL-4/TL-6	H4a/H4b	W1-W2	B/C	

* Typical range for reference only, working widths vary with system details

**Accident severity index (A, B, C in order of high potential for severe injury)

Table 12.13 General considerations for usage of typical safety type*

Safety Barrier Type	Roadside Single Face Width (mm)	Median Double face Width** (mm)	Height (mm)	Post Spacing (m)
Flexible and Semi Flexible System				
Wire Rope barrier	94	113	780 - 900	3.2
W-beam barrier	150 - 600	660	730 - 780	1 - 4
Thrie-beam barrier	500	700	875	1 - 4
Modified Thrie-beam barrier	600	1110	900	1 - 2
Thrie-beam or Modified Thrie-beam barrier with rails on both sides	310-510	620 -1020	875	2
Rigid System: Concrete Safety Barrier				
Low profile barrier	No Data	No Data	400 - 510	-
New Jersey Barrier	470	610 - 820	810 - 1070	-
F-barrier	600	610 - 820	810 - 1070	-
Vertical barrier	600	600	700 - 1070	-
Single slope barrier	No Data	610	810 - 1070	-
Step barrier	No Data	540	900	-

* Dimensions are only for indication and depend on design

** Double face safety barriers may be used at medians or on the roadside

Table 12.14 General considerations for usage of typical safety barrier types

Type	Category	Advantages	Disadvantages	Containment Level	Working Width (m)
Cable	Flexible	-Reuseable and readily reparable -Provide good visibility with minimum visual impacts - High flexibility of installation	-Limitation on small radius curves -Some concerns for injuries to motorcyclists and cyclists -Proprietary products	Possible to contain heavier vehicles if specified	1.0 – 2.8 (High tension) 2.9 - 3.5 (Low tension)
Box-beam	Semi-rigid	-Narrow and pleasing appearance - Minimum blockage of view	Limitation on small radius curves	Mainly for light vehicles	0.6 – 1.7
W-beam	Flexible/ Semi-rigid	-Lower installation costs -Relatively flexible placement criteria -Less injury potential	-Damage upon impact -Deflection distance is required -Vehicle damage	Mainly for light vehicles	0.6 - 1.2 (Strong post) 1.5 - 2.1 (weak post)
Thrie-beam	Semi-rigid	Similar to W-beam	Similar to W-beam	Suitable for larger vehicles with modified design	1.0 - 1.7
Concrete or Steel	Semi-rigid*/ Rigid	-Minimmal damage due to impact -Little or no deflection -Less vehicle damage at low angle impact	-Higher installation costs -More injury potential -Stricter placement criteria -Blocking of view -Storm drainage may be needed	Suitable for all vehicles including heavy vehicles	0.6 if rigid

* If free standing

Table 12.15 Special considerations for usage of typical safety barrier types

Safety Barrier Type	Small radius curves	Visibility and comfort	Narrow roadside	Sections with large differential settlement	Long straight sections	High risk sections, sheer drops and bridges
Cable	+	*	**	*	**	+
Box-beam			+			+
W-beam			+			+
Thrie-beam			+			*
Concrete or steel	+	+		+	*	*

*Very suitable
** Suitable
+ Potential problems or inadequacies

12.5.2.5 SELECTION OF SAFETY BARRIER TYPES

Selection of safety barrier types should be based on the likelihood of roadside crashes and their consequences. Other considerations are economics, roadside conditions and the number of transitions between different barrier types. **Table 12.16** and **Table 12.17** provides some general guidance.

MASH presents specific test level (TL) impact conditions for conducting vehicle crash tests. The specified test conditions include vehicle mass [weight], impact speed, approach angle, and point of impact on the safety feature. Standard test vehicle types are defined for small passenger cars (1100C), pickup trucks (2270P), single-unit van trucks (10,000S), tractor/van-type trailer units (36,000V), and tractor/tanker trailer units (36,000T). The design impact test conditions for each type of roadside hardware have been established to reflect the vast majority of real-world crash conditions. **Table 12.18** shows the test matrix for traffic barrier systems.

Table 12.16 General guidance on selection of roadside safety barriers

<table>
<tr><th colspan="2">Containment Level</th><th rowspan="2">Typical Application</th></tr>
<tr><th>MASH</th><th>EN1317-2</th></tr>
<tr><td>TL-2</td><td>N1</td><td>Speed limit ≤ 60km/h and traffic volume ≤ 12,000 veh/day
Speed limit ≥ 70km/h and traffic volume ≤1,500 veh/day</td></tr>
<tr><td>TL-3</td><td>N2</td><td>- Design Class A and B roads with roadside of moderate risk
-Along loops at grade-separated intersections with roadside moderate risk
-Speed limit ≤ 80km/h with low to moderate volume of heavy vehicles</td></tr>
<tr><td>TL-4</td><td>H1/L1/H2/L2</td><td>- Design Class A and B roads with roadside of moderate to high risk
- Sections of roads with speed limit ≤70km/h:
- over bridges, retaining walls or sheer drops higher than 4m
- over high risk water bodies
- where collisions with roadside features would have severe consequences</td></tr>
<tr><td>TL-5</td><td>H2/L2/H3/L3
H4a/H4b/L4a/L4b</td><td>- Sectons on Design Class A roads and other roads with speed limit ≥80km/h and high volume of heavy vehicles or buses, where the risk of running off the road is greater than usual with potentially very severe consequences.
- Where collapses or serious damage to roadside structures could lead to severe secondary consequences
- On bridge and over sheer of significant height
- On bridges that cross railways and along roads where railways lie in the proximity</td></tr>
<tr><td>TL-6</td><td>H4a/H4b/L4a/L4b</td><td>- Over or adjacent to major or high speed railways and areas with potential for catastrophic secondary events
- On high bridges and over sheer along Design Class A roads with high volume of heavy vehicles or buses</td></tr>
</table>

The use of flexible or semi-rigid safety barriers with containment level in the region of TL-3 and TL-4 to (MASH) or N2 to H2 (EN1317-2) is sufficient for the majority of roadside conditions with moderate risks. Thrie-beam safety barriers have higher containment level and therefore are preferred with frequent buses or high volume of heavy vehicles.

Higher containment levels should be adopted if there is an increased risk e.g. due to high proportion of buses, heavy vehicles, sharp curves, long steep grades, fogy conditions etc. For medians and embankments of moderate heights, containment level of TL-4 (MASH) or H2 (EN1317-2) are generally recommended.

Containment levels of TL-5 (MASH), H3 (EN1317-2) or higher are generally required to prevent a fully loaded bus from breaking through the safety barrier in very high risk situations including bridges and sheer drops. Where there is a high risk of catastrophic consequences such as a container truck falling onto a railway track, containment levels of TL-6 (MASH) or H4a/H4b (EN1317-2) should be adopted.

An errant vehicle may collide with the roadside perpendicularly or at high impact angles for certain road layouts such as very sharp curves, roads ending at a T-junction or roundabout central islands. Yet safety barriers may be needed at these locations due to presence of bridges, sheer drops or other roadside risks. The type and layout of safety barriers will need to be carefully balanced between injury potential for vehicle occupants and the consequences of the vehicle breaking through the barrier. It is advisable that comprehensive treatments are adopted to lower the overall risk.

Table 12.17 General guidance on selection of median safety barriers

Test Criteria		Typical Applications*
MASH	EN1317-2	
TL-2/TL -3	N1/N2	- Wide medians - Medians of full-access-controlled Design Class B roads in built-up areas and their periphery (possible need of additional pedestrian fences)
TL-3	N2/H1	- Wide medians - Narrow medians ≤ 3m of Design Class A roads with low proportion of heavy vehicles - Design Class B roads with speed limit ≤ 70km/h with high proportion of heavy vehicles
TL-4	H2/L2	- Narrow median ≤ 3m on Design Class A and B roads with speed limit ≥ 80km/h and high proportion of heavy vehicles
TL-5	H4a/H4b/L4a/L4b	- Medians on the outside of sharp curves and along long steep grade sections on Design Class A roads, access-controlled Class B roads and other roads with speed limit ≥ 80km/h and a high portion of heavy traffic

*Refer to **Table 12.16** on roadside safety barriers for aggressive features on medians and roads with separate formations for the two traffic directions.

Along wide medians on rural sections with relatively low traffic volume, flexible or semi-flexible safety barriers could be appropriate. These barriers may be laid out asymmetrically to provide variable widths of clear zones according to safety risks for each travel direction. They may also be discontinuous with an overlapping layout.

In order to limit the risk of errant vehicles breaking into the opposite carriageway on Design Class A roads and access-controlled Class B roads, median barriers of higher containment level should be adopted:

i. on urban sections with high traffic volume
ii. where there is a high volume of heavy vehicles
iii. on the outside of sharp curves
iv. on road sections with long steep grades

Higher containment level may be obtained with the use of Thrie-beam barriers or rigid barriers on the median. Where there is a high risk of heavy vehicles overcoming the median, containment level TL-5 (NCHRP 350/MASH) should be attained by using a height of 1,070mm for concrete rigid safety barriers.

Table 12.18 MASH Test Matrix for Traffic Barrier Systems

Test Level (TL)	Test Vehicle Designation and Type	RDG Design Vehicle	Test Conditions		
			Vehicle Weight (kg)	Speed km/h	Angle (°)
1	1100C (Passenger Car)	S-5	1,100	50	25
	2270P (Pickup Truck)	M-6	2,270	50	25
2	1100C (Passenger Car)	S-5	1,100	70	25
	2270P (Pickup Truck)	M-6	2,270	70	25
3	1100C (Passenger Car)	S-5	1,100	100	25
	2270P (Pickup Truck)	M-6	2,270	100	25
4	1100C (Passenger Car)	S-5	1,100	100	25
	2270P (Pickup Truck)	M-6	2,270	100	25
	10000S (Single Unit Truck)	L-9	10,000	90	15
5	1100C (Passenger Car)	S-5	1,100	100	25
	2270P (Pickup Truck)	M-6	2,270	100	25
	36000V (Tractor/Van Trailer)	T-17 & T-21	36,000	80	15
6	1100C (Passenger Car)	S-5	1,100	100	25
	2270P (Pickup Truck)	M-6	2,270	100	25
	36000V (Tractor/Van Trailer)	T-17 & T-21	36,000	80	15

12.5.2.6 LONGITUDINAL BARRIER PLACEMENT

A typical longitudinal roadside barrier installation, with its associated elements is illustrated in **Figure 12.11**. The length of need is illustrated in more detail in **Figure 12.13.**

The factors to be considered in barrier installation are the following:

i. Offset of the barrier from the travelled way (shy-line offset)
ii. Rail deflection distance
iii. Terrain effects
iv. Flare rate
v. Length of need

Roadside barriers should be placed as far from the travelled way as practical, while maintaining the proper operation and performance of the system. This ensures that:

i. There is more recovery area to regain control of the vehicle without crashing into the barrier.
ii. There is better sight distance, particularly at intersections.
iii. Less barrier is required to shield the hazard.
iv. Adverse driver reaction to the barrier is reduced.

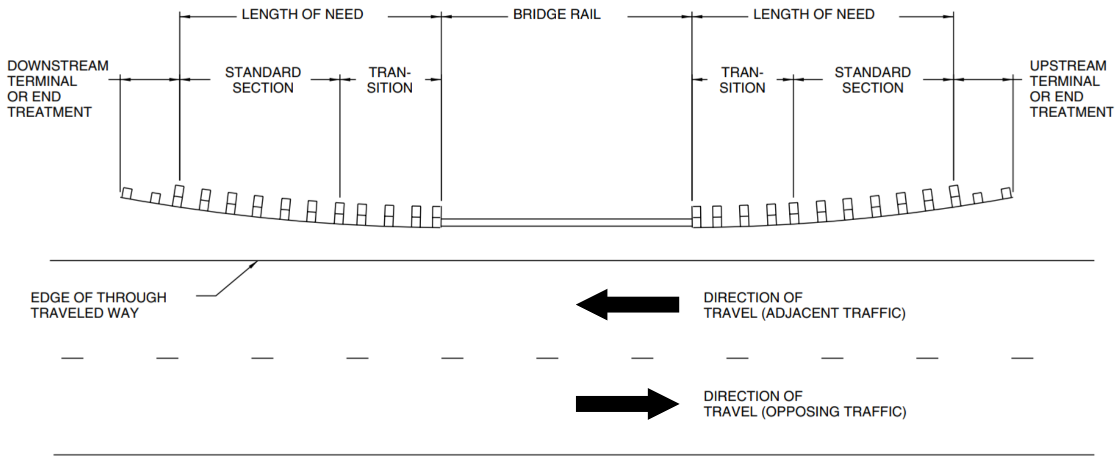


Figure 12.11 Definition of roadside barriers

However, placing the barrier away from the roadway and closer to the hazard may have disadvantages. These are:

i. The possible impact angle increases, leading to higher risk of the vehicle penetrating the rail as well as increased collision severity.
ii. The roadside area in front of the barrier must be traversable and as flat as possible.

It is generally desirable that there be uniform clearance between traffic and roadside features such as bridge railings, parapets, retaining walls, and roadside barriers, particularly in urban areas where there is a preponderance of these elements. Uniform alignment enhances highway safety by providing the driver with a certain level of expectation, thus reducing driver concern for and reaction to those objects. The distance from the edge of the travelled way beyond which a roadside object will not be perceived as an obstacle and result in a motorist's reducing speed or changing vehicle position on the roadway is called the shy-line offset. This distance varies for different design speeds as indicated in **Table 12.19**.

If practical, a roadside barrier should be placed beyond the shy-line offset, particularly for relatively short, isolated installations. For long, continuous runs of barrier, this offset distance is not as critical, especially if the barrier is first introduced beyond the shy line and gradually transitioned to toward the roadway. Shy-line offset distance is seldom a controlling criterion for barrier placement. As long as the barrier is located beyond the perceived shoulder of a roadway, it will have minimum impact on driver speed or lane position. Problems arise when the roadway appears narrower or is narrowed, such as at a bridge that is narrower than the approach roadway. On facilities with no shoulders, barriers or other fixed objects 1.8 m or more from the edge of the travelled way may not create driver reactions.

It also is worth noting that median barriers can be set closer to the edge of the driving lane without affecting vehicle placement. When the barrier is to the left, the driver can clearly see how close the barrier is; however, for a right shoulder installation, depth perception becomes more of a problem for many drivers, and they tend to position their vehicles farther from the barrier than is necessary.

Barriers are typically placed at a distance of 0.3 metres beyond the edge of the usable shoulder so that the greater of the distance in **Table 12.19** or the width of the shoulder plus 0.3m should be used.

Table 12.19 Recommended minimum shy-line (L_s) offset values

Design Speed (km/h)	Shy-line offset distance measured from the edge of the travelled way, L_S (m)
20 - 50	1.1
60	1.4
80	2.0
100	2.4
120	3.2

12.5.2.7 WORKING WIDTH

Adequate lateral space is required relative to roadside aggressive features in order to prevent an errant vehicle colliding onto the safety barrier to reach the feature. The expected deflection of a barrier should not exceed the available space between rail and the object being shielded. The various terms according to EN1317-2 is illustrated in **Figure 12.12**.

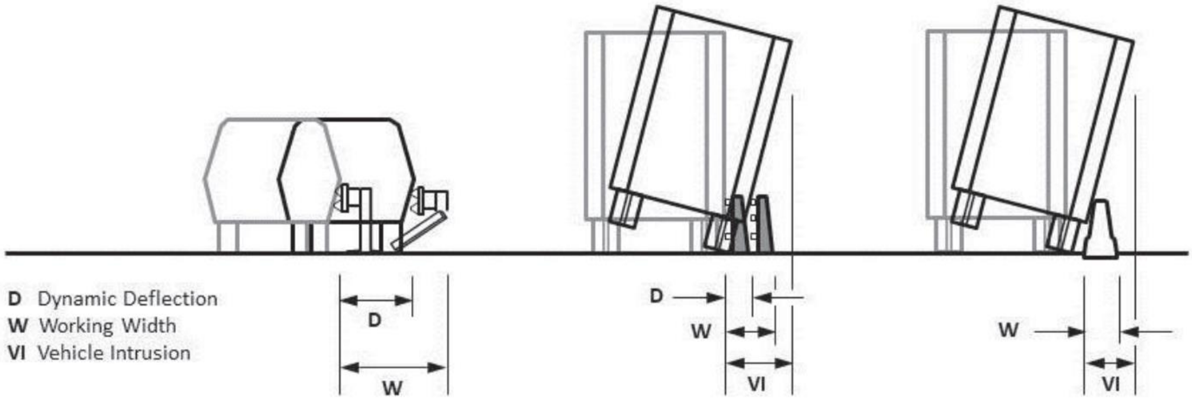


Figure 12.12 Definition of terms in EN1317-2

The working width (W) is the maximum lateral distance between any part of the barrier on the undeformed traffic side and the maximum dynamic position of any part of the barrier. The vehicle intrusion (VI) of the Heavy Goods Vehicle (HGV) is its maximum dynamic lateral position from the undeformed traffic side of the barrier.

Working width is specific to safety barrier types ranging from less than 0.6m for a rigid system to 3.5m for a flexible system. Working width class to EN1317-2 is given in **Table 12.20**.

Table 12.20 Working width definition To En1317-2

Working Width Class							
W1	**W2**	**W3**	**W4**	**W5**	**W6**	**W7**	**W8**
≤0.6m	≤0.8m	≤1.0m	≤1.3m	≤1.7m	≤2.1m	≤2.5m	≤3.5m

Safety barriers should be set back from the carriageway with a horizontal clearance. Unless there are no other feasible solutions, installation of safety barriers should not normally result in a reduction of shoulder width at any location.

If the available space between the rail and the obstacle is not adequate for non-rigid barrier systems then the barrier can be stiffened in advance of, and alongside, the fixed object. This can be achieved through reducing the post spacing, increasing post sizes or increasing the rail stiffness by nesting rail elements. However, care should be exercised when considering this step since the total system characteristics might be altered.

Other areas of concern include the possibility of rolling over when vehicles with a high centre of gravity impact a barrier or of vehicles dropping over the edge when a barrier, positioned too close to the edge, deflects on impact.

A minimum width of 600mm is needed between the back of a semi-rigid barrier (e.g., guardrails and the top edge of a steep embankment in order that adequate passive resistance can be developed by the posts. In case of difficulties to achieve the above width, taller mounting posts up to 2.5m in lengths should be adopted.

12.5.2.8 TERRAIN EFFECTS

Roadside features such as kerbs and drainage inlets affect the bumper height and suspension and may cause errant vehicles to snag or vault the barrier.

Kerbs should preferably be sited behind the guardrail face. Barrier offsets less than 230mm behind the kerb would still be acceptable. The height of the rail should be carefully considered to limit the possibility of the bumper or a wheel under-riding the rail. This may be achieved by setting the rail height relative to the road surface in front of the kerb.

12.5.2.9 SLOPES

Roadside barriers perform best when installed on slopes of 1:10 or flatter. Slope changes may cause vehicles to impact higher on the barrier than normal, increasing the possibility of vaulting.

Should barriers be installed beyond a slope change, they should be set back at least 3.5m from the slope break line to allow the vehicle trajectory to stabilize. Installation of guardrails on slopes steeper than 1:6 is not recommended because inadequate lateral support for the guardrail posts would result. If this location is unavoidable, consideration should be given to deeper postholes.

12.5.2.10 FLARE RATE

A roadside barrier is considered flared when it is not parallel to the edge of the travelled way. Flare is normally used to locate the barrier terminal farther from the roadway to:

i. minimize a driver's reaction to an obstacle near the road by gradually introducing a parallel barrier installation.
ii. transition a roadside barrier to an obstacle nearer the roadway such as a bridge parapet or railing.
iii. reduce the total length of guardrail needed.

The use of a flared barrier also reduces the number of barrier and terminal impacts as well as provides additional roadside space for an errant motorist to recover.

A barrier flare may be used to increase the barrier offset from the edge of the roadway. This is normally used to position the barrier terminal further from the roadway, to adjust the existing roadside features, to reduce the total length of rail and to reduce driver reaction to the close proximity of the barrier rail next to the road.

Flared barriers can, however, also lead to increased impact angles causing higher impact severity, as well as to larger rebound angles causing greater conflicts with other vehicles.

Table 12.21 shows the recommended maximum flare rates for semi-rigid and rigid barriers. Note that the recommended flare rate for barriers within the shy line is approximately twice that for barriers located outside the shy-line distance. Flatter rates may be used particularly where extensive grading would be required to provide a 1:10 approach slope to the barrier.

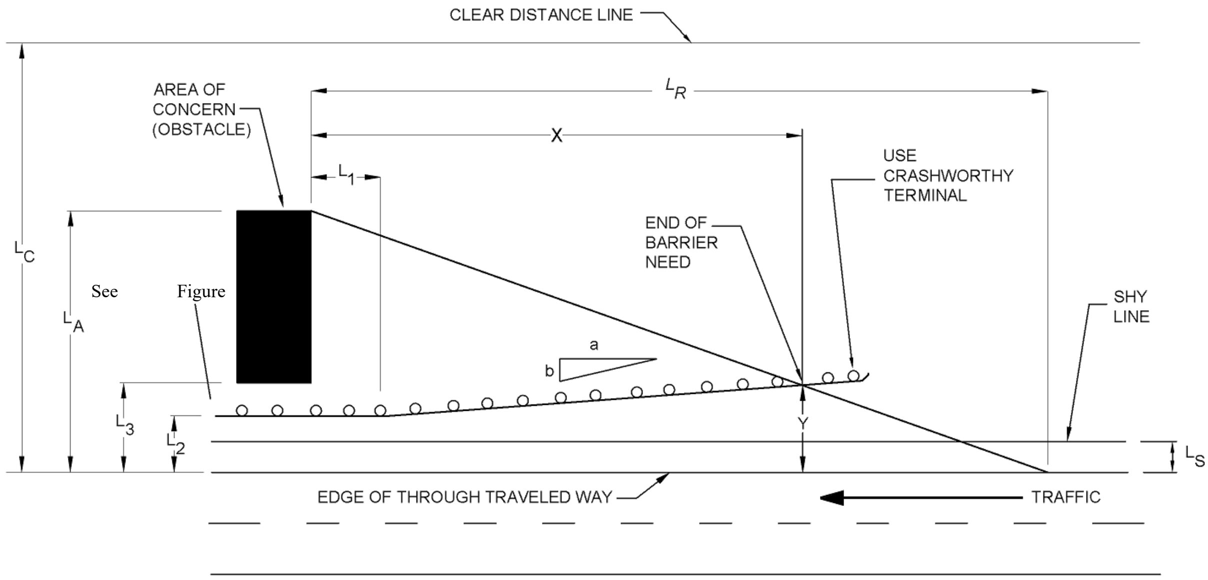
Table 12.21 Recommended maximum flare rates for barrier design

Design speed (km/h)	Barrier inside the min. shy-line offset	Barrier beyond the min. shy-line offset	
		Rigid barriers	Non-rigid barriers
20 - 50	1:13	1:8	1:7
60	1:16	1:10	1:8
80	1:21	1:14	1:11
100	1:26	1:18	1:14
110	1:30	1:20	1:15
120	1:40	1:25	1:18

12.5.2.11 LENGTH OF NEED

The "Length of Need" (LoN) is the required minimum length of safety barrier with full height and strength to shield an aggressive roadside feature. LoN should be determined in one direction for divided roads and both directions for undivided roads.

Figure 12.13 illustrates the variables that should be considered in designing a roadside barrier to shield an obstruction effectively. The primary variables are the Lateral Extent of the Area of Concern (L_A) and the Runout Length (L_R). Both of these factors should be clearly understood by the designer to be used properly in the design process.



X-Length of need (m)

Y- Lateral offset (m)

L_1 – Tangent length of barrier upstream from area of concern (m)

L_2 – Lateral distance from the edge of the travelled way (m)

Ls - Shy-line offset (m)

L_C - Clear zone (m)

L_R - Runout Length (m)

L_A - Lateral Extent of the Area of Concern (m)

a:b - Flare rate

Figure 12.13 Approach barrier layout variables

The Lateral Extent of the Area of Concern (L_A) is the distance from the edge of the travelled way to the far side of the fixed object or to the outside edge of the clear zone (L_C) of an embankment or a fixed object that extends beyond the clear zone. Selection of an appropriate L_A distance is a critical part of the design process and is illustrated in the examples at the end of this section.

Run-out length is the theoretical distance required for a vehicle leaving the roadway to come to a stop prior to impacting a hazard. The design of a traffic barrier requires provision to be made for sufficient length to restrict such a vehicle from reaching the hazard.

Once L_R and L_A have been selected, the length of barrier required at a specific location depends on the tangent length of barrier upstream from the Area of Concern (L_1), its lateral distance from the edge of the travelled way (L_2), and the flare rate (a:b) specified for the installation.

The recommended run-out lengths are shown in **Table 12.22**. The run-out length is measured along the edge of the road. A control line is established between the end of the run-out length and the far side of the hazard to be shielded. The length of need for a standard barrier would then be the length between the near side of the hazard and the position where the barrier intersects the control line.

If the barrier is designed for a continuous hazard such as a river or a critical fill embankment, then the control line would be between the end of the run-out length and the end of the desirable clear zone. The same principle is adopted to determine the length of need for opposing traffic.

The standard guardrail ends at the end of the length of need. An acceptable end-treatment should be added to this length to determine the total length of installation.

Table 12.22 Recommended runout length for barrier design

Design speed (km/h)	**Run-out length, L_R (m)**			
	ADT<800	**800>ADT<2000**	**2000<ADT<6000**	**ADT>6000**
20 - 50	40	45	50	50
60	50	55	60	70
80	75	80	90	100
100	100	105	120	130
120	125	135	150	160

Once the appropriate variables have been selected, the required length-of-need (X) in advance of the area of concern for straight or nearly straight sections of roadway can be calculated with using **Equation 12.1**.

$$X = \frac{L_A + \left({}^{b}/_{a}\right)(L_1) - L_2}{\left({}^{b}/_{a}\right) + \left({}^{L_A}/_{L_R}\right)} \tag{12.1}$$

Note that for a parallel installation (i.e., no flare rate), the **Equation (12.1)** reduces to **Equation (12.2)**.

$$X = \frac{L_A - L_2}{{}^{L_A}/_{L_R}} \tag{12.2}$$

The lateral offset (Y) from the edge of the travelled way to the beginning of the length-of-need can be calculated using the **Equation (12.3)**.

$$Y = L_A - \left({}^{L_A}/_{L_R}\right) X \tag{12.3}$$

These formulas are intended to provide the designer with an approximation for the approach barrier length-of-need. The calculated length-of-need should be adjusted to account for the following factors:

Standard Manufactured Length of Guardrail Sections—The calculated length-of-need should be adjusted upward to account for the industry's manufactured lengths of barrier sections. For example, the typical manufactured lengths of W-beam and thrie-beam guardrail are in two nominal dimensions of 3.81m or 7.62m and 1.91m for the thrie-beam–to–W-beam transition component.

Beginning of Length-of-Need—Most W-beam guardrail terminals are designed to contain and redirect vehicles striking at or beyond the third post from the end of the terminal unit, but vehicles striking within the first 3.81m of the terminal unit may not be redirected and could penetrate the rail system and be exposed to the shielded feature. The designer should extend the barrier so the length-of-need is at least at the point on the selected terminal where redirection can be expected. In some cases, the rounding of the calculated length-of-need to the nearest industry dimension of the manufactured beam rail systems will accomplish this.

Buried in Backslope—If the barrier ends near a cut section, it may be possible for the designer to consider anchoring the barrier in the backslope. This eliminates the possibility of an end-on hit into the terminal unit and possible penetration behind the rail system. Refer to **Section 12.5.5.1** for additional information on the design of buried-in-backslope terminals.

Parabolic Flare—The use of parabolic layout for a flared section is acceptable provided the maximum slope of the curve does not exceed the suggested flare rates in **Table 12.21** or per the manufacturer's recommendations. However, these rates may be exceeded in the terminal section if the greater flare rates are essential for proper impact performance of the terminal.

Design Chart—As an alternative to computing a length-of-need (X), design charts have been developed to enable a length of barrier to be selected directly based on standard conditions. **Figure 12.14** and **Figure 12.15** show examples of such charts for flared and parallel installations, respectively. If charts, such as these examples, are used to address the length-of-need, then the designer will need to review the site plans to determine if the area of concern has been adequately protected.

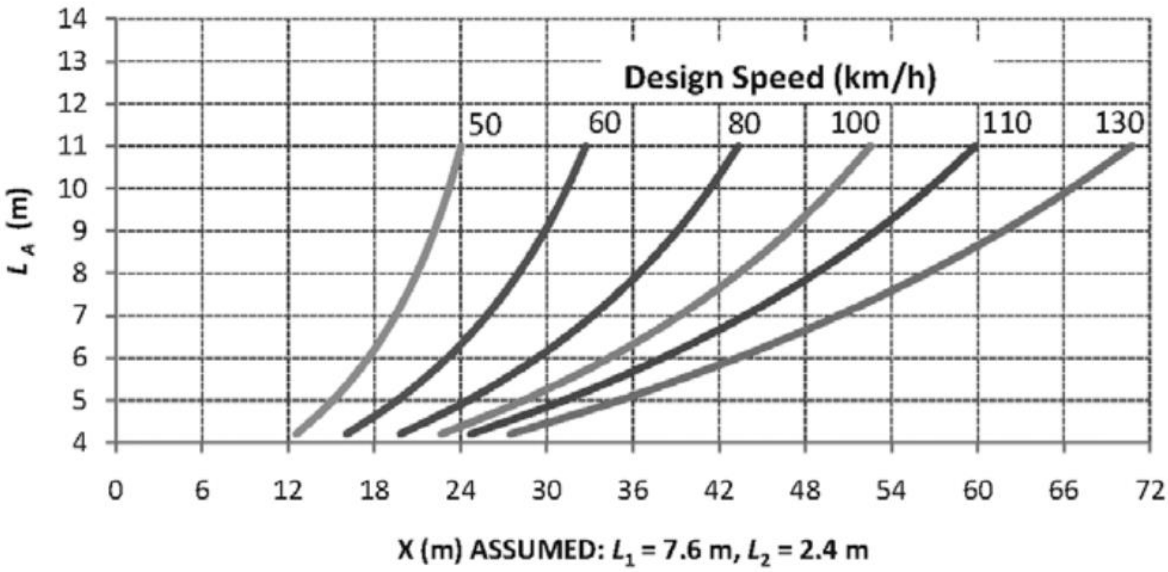


Figure 12.14 Design Chart for a Flared Roadside Barrier Installation (flare rate =20:1)

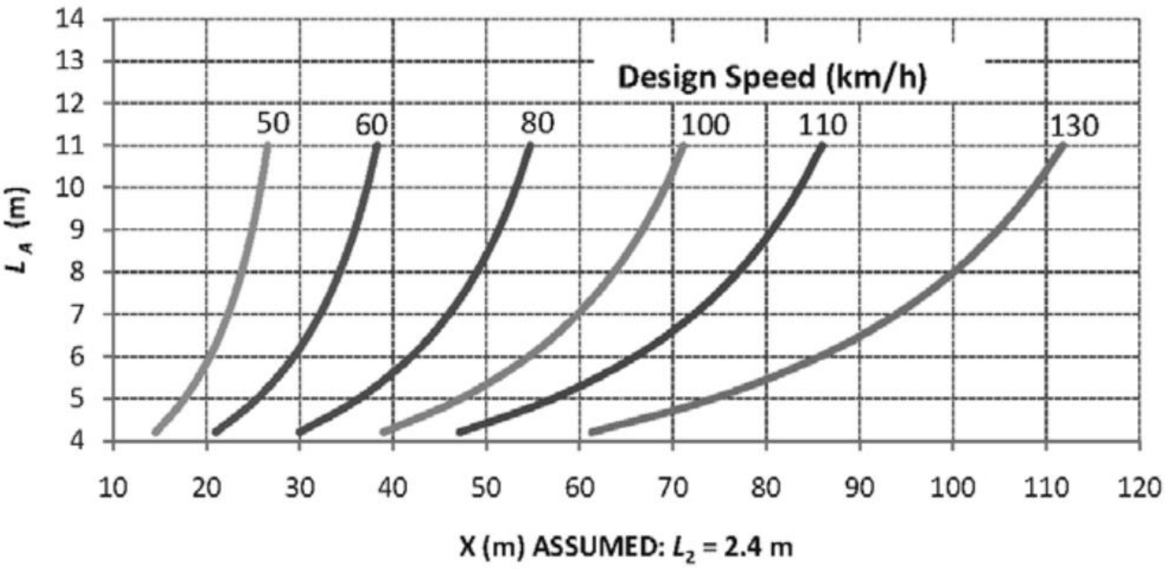


Figure 12.15 Design Chart for a Parallel Roadside Barrier Installation

Figure 12.16 illustrates the layout variables of an approach barrier for opposing traffic. The length-of-need (X) is determined in the same manner as previously described, but all lateral dimensions are measured from the left edge of the travelled way of the opposing traffic (i.e., from the centreline for a two-lane roadway). If there is a two-way divided roadway, the edge of the travelled way for the opposing traffic would be the edge of the driving lane on the median side.

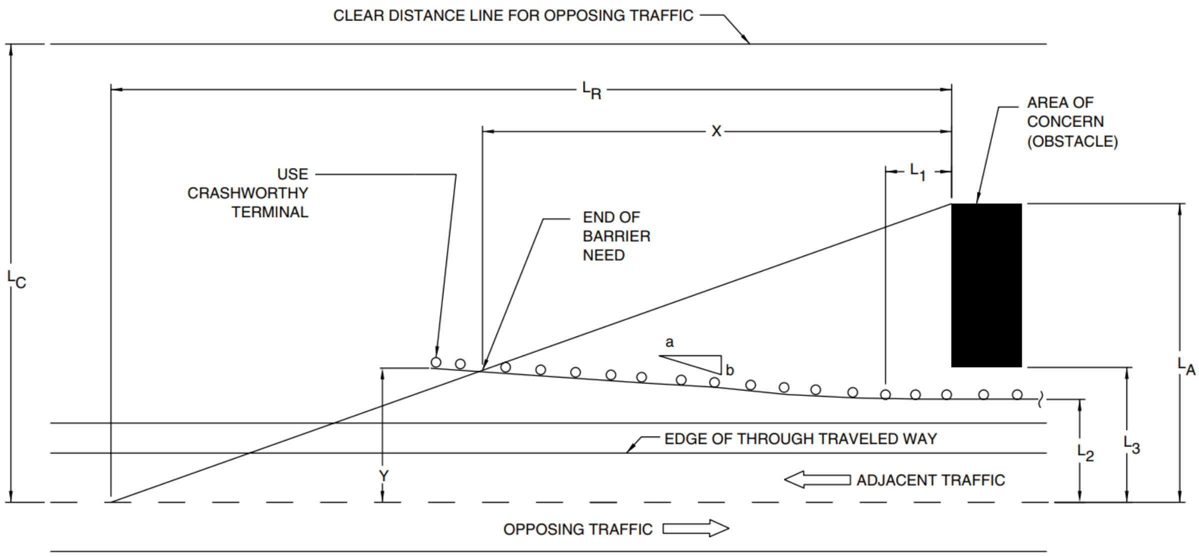


Figure 12.16 Approach Barrier Layout for Opposing Traffic

Three ranges of clear zone width (L_C) deserve special attention for an approach barrier for opposing traffic (see **Figure 12.16**):

i. If the barrier is beyond the appropriate clear zone, no additional barrier and no crashworthy terminal is needed. For this case, use a crashworthy terminal on 2-lane undivided roadways and not on divided roadways.
ii. If the barrier is within the appropriate clear zone but the area of concern is beyond it, no additional barrier is needed, but a crashworthy terminal should be used.
iii. If the area of concern extends well beyond the appropriate clear zone (e.g., a river), the designer may choose to shield only that portion that lies within the clear zone by setting L_A equal to L_C.

On divided highways and roadways with one-way traffic, the length of guardrail to protect the downstream corner of the area of concern is determined by plotting a line at a defined exit angle. The guardrail should have the end anchor assembly downstream of this exit angle line. It is recommended that the guardrail be extended at least 3.81m beyond the exit length-of-need line, as shown in **Figure 12.17**. Exit angles typically vary from 25 degrees to perpendicular. The designer can use this exit angle to determine the amount of guardrail on the trailing end that can be removed. Preferably a 90-degree exit angle could be used so that the guardrail is extended to a location adjacent to the downstream corner of the area of concern. This also results in additional guardrail to develop and transmit rail tension forces into the anchorage. By using a perpendicular line, the design, construction, and maintenance of the guardrail is simplified. Refer to **Figure 12.17** for trailing end guardrail termination details.

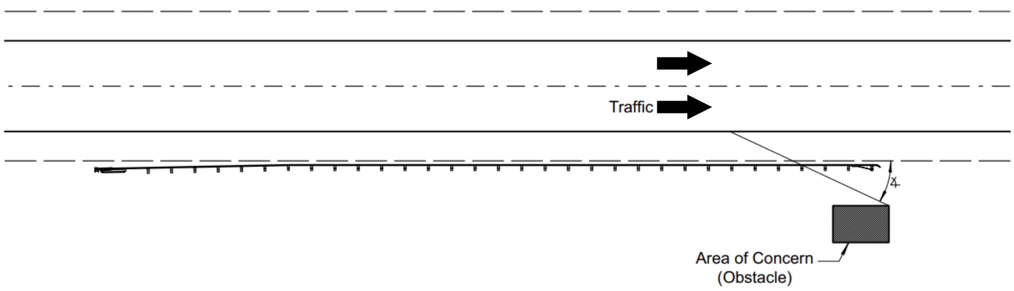


Figure 12.17 Determination of Trailing End Guardrail Layout

12.5.3 MEDIAN BARRIERS

Median barriers are longitudinal barriers most commonly used:

i. to separate opposing traffic on a divided highway.

ii. to deter vehicles from overtaking or U-turning across the median.

iii. along heavily travelled roadways to separate through traffic from local traffic or to separate high occupancy vehicle (HOV) lanes from general-purpose lanes.

Most of the principles and designs with respect to longitudinal roadside barriers as described in **Section 12.5.2** also apply to median barriers. However, median barriers, discussed in this Section, are those designed to redirect vehicles striking either side of the barrier.

Regarding warrants for their use, median barriers should only be installed if the consequences that would result if they did not exist are more severe than the consequences of striking them.

However, excessive incidence of illegal cross median movements might justify the use of median barriers. **Figure 12.18** provides general guidance on the need for median barriers.

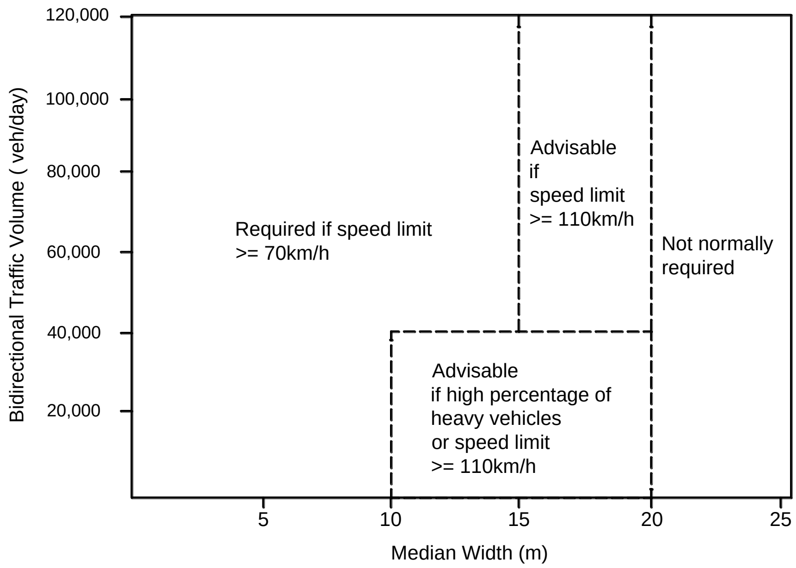


Figure 12.18 Recommended requirements for median safety barriers

For non-access-controlled Design Class B roads with speed limit of 70km/h or below, the following options may be considered:

i. Adoption of a wider median in the order of 5m to 10m.
ii. Provision of median safety barrier if the road has a high volume of heavy vehicles.
iii. Adoption of low profile median safety barrier.
iv. Adoption of raised median on kerbs if speed limit ≤ 50km/h.

Median safety barriers may be provided as a double-sided unit with each side having the function of a safety barrier. They may also be provided as independent single sided safety barriers for each travel direction.

Along wide medians on rural sections with relatively low traffic volume, flexible or semi-flexible safety barriers could be appropriate. These barriers may be laid out asymmetrically to provide variable widths of clear zones according to safety risks for each travel direction. They may also be discontinuous with an overlapping layout.

In order to limit the risk of errant vehicles breaking into the opposite carriageway on Design Class A roads and access-controlled Class B roads, median barriers of higher containment level should be adopted:

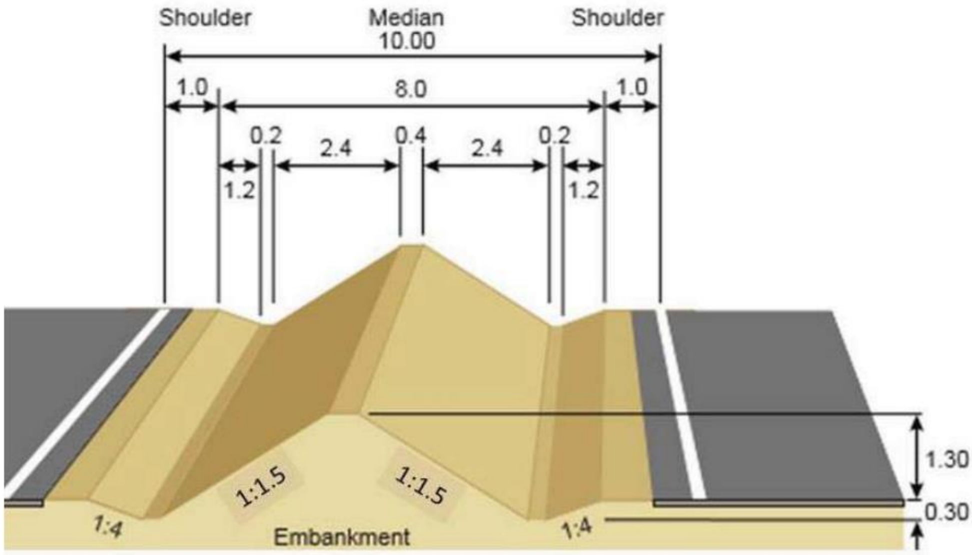
i. on urban sections with high traffic volume
ii. where there is a high volume of heavy vehicles:
iii. on the outside of sharp curves
iv. on road sections with long steep grades

Higher containment level may be obtained with the use of Thrie-beam barriers or rigid barriers on the median. Where there is a high risk of heavy vehicles overcoming the median, containment level TL-5 (NCHRP 350/MASH) should be attained by using a height of 1,070mm for concrete rigid safety barriers.

The use of median safety barriers should be planned in conjunction with the overall road cross-section. If lighting columns, traffic signs, gantry supports or bridge columns are envisaged on medians less than 4m in widths, particular care is needed to ensure that horizontal clearance and working width of safety barriers can be accommodated.

Median safety barriers have implications on visibility at bends and intersections and the problem could be aggravated by horizontal curves and crest profiles. It is important to ensure that visibility criteria are met with respect to safety barrier types, heights and layouts. It may be necessary to widen the median shoulder at bends, terminate the safety barrier earlier at intersections or adopt lower barrier types.

For medians wider than 10m, an earth embankment may be provided to serve as a combined safety barrier and anti-glare screen as shown in **Figure 12.19**. The embankment may be vegetated with grass or shrubs. Special treatments are required in terms of visibility and passive safety at median openings and terminations.



units: m

Figure 12.19 Median Earth Embankment

Once the need for a median barrier is established, the designer should consider several factors in developing the barrier layout. These include:

i. Terrain effects.
ii. Flare rate of the barrier.
iii. Treatment of rigid objects in the median.
iv. Openings in the median as a result of underpasses.

12.5.3.1 TERRAIN EFFECTS

For a median barrier to be effective, it is essential that, at the time of impact, the vehicle has all its wheels on the ground and that its suspension system is neither compressed nor extended. Kerbs and sloped medians are of particular concern, since a vehicle, which traverses one of these features prior to impact, may go over or under the barrier or snag on its support posts.

Kerbs offer no safety benefits on high-speed roads and are not recommended where median barriers are present.

Medians should be relatively flat (slopes of 1:10 or less) and free of rigid objects. Where this is not the case, carefully considered placement of the median barrier is needed. There are three conditions where specific guidelines for median barrier placement should be followed:

i. In depressed medians or medians with a ditch section, the slopes and ditch section should first be checked to determine whether a roadside barrier is warranted. If both slopes require shielding, a roadside barrier should be placed near the shoulder on each side of the median. If only one slope requires shielding, a median barrier should be placed near the shoulder of the adjacent travelled way.

ii. If neither slope requires shielding but both are steeper than 1:10, a median barrier should be placed on the side with the steeper slope, when warranted.

iii. If both slopes are relatively flat, then a median barrier may be placed at or near the centre of the median if vehicle override is not likely.

For stepped medians that separate travelled ways with significant differences in elevation, a median barrier should be placed near the shoulder adjacent to each travelled way if the embankment slope is steeper than 1:10. If the cross-slope is flatter than 1:10, a barrier could be placed at or near the centre of the median.

Placement criteria are not clearly defined for raised medians or median berms. Research suggests that the cross section of a median berm itself, if high and wide enough, can redirect vehicles impacting at relatively shallow angles.

As a general rule, if the cross section is inadequate for redirecting errant vehicles, a semi-rigid barrier should be placed at the apex of the cross-section. If the slopes are not traversable, roadside barriers should be used near the shoulder adjacent to each of the travelled ways.

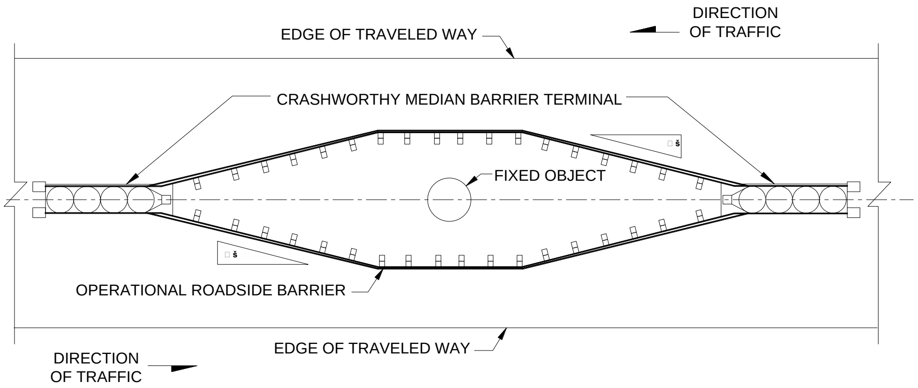
12.5.3.2 FLARE RATES

If a median barrier has to be flared at a rigid object in the median, the flare rates for roadside barriers should be used for the median barrier flare as well.

12.5.3.3 RIGID OBJECTS

A special case may result in circumstances where a median barrier is not warranted but where a rigid object warrant shielding. Typical examples are bridge piers, overhead sign support structures, and high mast lighting installations. If shielding is necessary for one direction of travel only, or if the object is in a depressed median and shielding from either or both directions of travel is necessary, the criteria for roadside barriers should be used.

If shielding for both directions of travel is necessary and if the median side slopes are steeper than 1:10 the designer may investigate the possibility of a crash cushion (or an earth berm) to shield the object. A second possibility involves the use of semi-rigid barriers with crash cushions or earth berms to shield the barrier ends as illustrated in **Figure 12.20**. If semi-rigid systems are used, the distance from the barrier to the obstruction should be greater than the dynamic deflection of the barrier. If a concrete barrier is used, the barrier can be placed adjacent to the obstruction unless there is a concern that a high-centre-of-gravity vehicle will strike the obstruction because its contact with the barrier causes the top of the vehicle to lean over the railing.



*Flare rate should not exceed suggested limits (Refer to **Table 12.21**)

Figure 12.20 Suggested layout for shielding a rigid object in a median

12.5.3.4 MEDIAN OPENINGS AS A RESULT OF UNDERPASSES

In certain instances, the cost implications of providing underpasses have the result that an opening in the median occurs. In such instances the use of transverse barriers (or concrete barriers) shielded by impact attenuation devices should be considered.

12.5.4 SAFETY ROLLER BARRIER

'Safety Roller Barrier' is a safety fixture that prevents drivers and passengers from fatal accidents by not only absorbing shock energy but also converting shock energy into rotational energy.

The Safety Roller Barrier is a steel rail safety barrier consisting of vertical steel posts that support a series of yellow ethylene vinyl acetate rollers.

It is installed at sites where vehicles are exposed to frequent accidents. Safety Roller Barrier will safely lead a vehicle back to the road or stop the vehicle by absorbing shock energy. It effectively functions for drivers to properly control vehicles with its noticeable colour and self-luminescence.

Advantages of rolling barriers are:

i. It increases the safety of humans and vehicles.
ii. It has shock absorbent system, which reduces sudden shocks on vehicles.
iii. It converts shock energy to rotary rotational energy.
iv. It is easy to install, and maintenance required is also less than normal barriers.
v. It gives good visibility at night also, with help of reflective tape.

vi. It has more serviceable life than normal barriers.

vii. It prevents sudden stoppage and overthrowing of vehicles after collision.

viii. It can be made by recyclable materials, thus it’s eco-friendly.

ix. It may have high initial cost, but the final cost is less as maintenance required is less and it has more life.

Safety Roller Barrier consists of both flexible property and semi rigid property stiffness. It can be placed at mountainous places, U-turns, barriers, curved alignment, corner curves etc as shown in **Plate 12.11**, **Plate 12.12** and **Plate 12.13**.

Plate 12.11 Wall mounted roller barriers

Plate 12.12 Roller barrier installed in a sharp curve

Plate 12.13 Roller barrier installed at the approach to a tunnel

12.5.4.1 CONFIGURATION

The Safety Roller Barrier system consists of vertical steel posts which support a series of horizontal rollers. Two top rails and two bottom rails run horizontal along the length of the barrier system. The steel posts are driven 1235mm below ground level at 1334mm centres. Intermediate posts, placed centrally between line posts are supported by the top and bottom rail members only. The finished nominal rail height of the system is 890mm, with all steel line posts finishing 80mm above the top of the rail.

12.5.4.2 DESIGN CONSIDERATIONS

i. Design to be in accordance with the KSI Global Australia Safety Roller Barrier System Manual Version 2.1, Dated 29.08.17.
ii. The length of need should be determined in accordance with the methodology detailed in **Section 12.5.2.11**.
iii. Kerbs should not be placed in front of the barrier as depicted in **Plate 12.14**.
iv. Objects should not be attached to the barrier or placed behind the barrier within the deflection zone.
v. The approach to the barrier should be a trafficable running surface at a slope of 1 in 10 or flatter, clear of objects and grade change to allow for an errant vehicle to hit the barrier at an appropriate height.
vi. The ends of the barrier must be fitted with end terminals.
vii. The offset from the back of the barrier edge to the batter hinge point should be a minimum of 1.85m.

viii. The minimum length of Safety Roller Barrier is 60m (terminal lengths not included)

Plate 12.14 Mountable kerbs in from of roller barrier

12.5.4.3 TERMINAL PERMITTED

i. Approved W-Beam terminal in conjunction with crash tested transition should be used with the Safety Roller Barrier.

ii. Alternatively, the approach end of Safety Roller Barrier may be shielded with an approved crash cushion. This treatment is not accepted on the departure end of Safety Roller Barrier.

12.5.4.4 LIMITATION

i. The Safety Roller Barrier is a proprietary system that has been successfully crash tested in soils equivalent to an AASHTO standard soil (i.e. CBR ≈ 60) for the full depth of the posts. If the Safety Roller Barrier is to be installed in soil conditions weaker than an AASHTO standard soil, then advice from the Supplier should be sought.3

ii. When installed in embankment conditions in soils equivalent to AASHTO standard soil or stronger (i.e. CBR ≥ 60) the hinge point shall be offset a minimum of 2.0m from the Safety Roller Barrier post.

iii. Should not be installed behind kerbs if possible. If kerbing is required, then mountable is acceptable. Semi-mountable and non-mountable kerbing shall not be used in front of barrier.

12.5.5 END TREATMENTS

Traffic barriers (both roadside and median types) themselves represent fixed objects. Impact with their untreated terminal sections can have severe consequences, primarily because of the very high deceleration rates experienced by vehicle occupant under such circumstances, but also often because penetration of the passenger compartment by the barrier itself is a distinct possibility. A wide variety of devices are considered to be end treatments:

i. **Anchorages:** These devices anchor a flexible or semi-rigid barrier to the ground to develop its tensile strength during an impact. Anchorages are not considered crashworthy; thus, they typically are used on the trailing end of a roadside barrier on one-way roadways or on the approach or trailing end of a flexible or semi-rigid barrier that is located outside the clear zone or that is shielded by another barrier system.

ii. **Terminals:** Essentially crashworthy anchorages, these devices are used to anchor a flexible or semi- rigid barrier to the ground, normally at the end of a barrier either located within the clear zone or likely to be impacted by errant vehicles. Most terminals are designed for vehicular impacts from only one side of the barrier; however, a few terminal designs have been developed for median applications and may be installed where there is potential for impact from either side.

iii. **Crash cushions:** Also known as impact attenuators, these devices typically are attached to or placed in front of rigid concrete barriers (i.e., median barriers, roadside barriers, or bridge railings) or other rigid fixed objects, such as bridge piers. Generally, crash cushions may be used in either a median or roadside application. More detailed descriptions are provided in **Section 12.5.7**.

A proper end terminal has two functions:

i. In any non-rigid barrier system, the end terminal should act as an anchor to allow the full tensile strength of the system to be developed during downstream angled impacts on the barrier.

ii. Regardless of the type of barrier, the end terminal should be crashworthy, i.e. it must keep the vehicle stable and it must keep the vehicle occupants away from rigid points creating high deceleration resulting in serious injuries or death during impact.

Upstream terminals of safety barriers should not constitute a hazard by stopping an errant vehicle abruptly, penetrating into a vehicle or launching the vehicle air-borne. This is particularly important for roads with speed limit of 70km/h or above and also applies to locations susceptible to impacts by errant vehicles on roads with lower speed limit.

Experience has shown that metal beam systems often result in penetration of the passenger compartment, and that high-speed impacts with concrete barriers result in intolerable deceleration forces. In designing crashworthy end treatments, designers must create treatments that provide vehicle deceleration rates that are within recommended limits for survivability. **Plate 12.15** to **Plate 12.20** and **Plate 12.21** to **Plate 12.24** illustrate the various types of end treatment for longitudinal roadside and median barriers respectively.

A number of principles relevant to barrier end treatments are offered:

i. Crashworthy end treatments are essential if a barrier terminates within the clear zone. Such a terminal must not spear, vault, or roll a vehicle in either head-on or angled hits.

ii. Barrier end treatments should gradually stop or redirect an impacting vehicle when a barrier is hit end on. The end treatment should also be capable of redirecting a vehicle impacting the side of the terminal.

iii. The end treatment should have the same redirectional characteristics as the barrier to which it is attached for impacts at or near the end of the terminal and within the length of need. The end should be properly anchored and capable of developing the full tensile strength of the barrier elements.

iv. Where space is available, a barrier can sometimes be introduced far enough from approaching traffic so that the end can be considered non-hazardous and no additional end treatment is required. Flare rates, in this case, should be in accordance with those mentioned above. Positive end anchorage is required in semi-flexible systems in order to preclude penetration of the barrier within the length of need. Care should be taken, however, to ensure that this flaring back does not create a hazard for traffic in the opposing direction.

v. End treatments involving turned down terminals parallel to the direction of travel may cause impacting vehicles to vault and roll over or ride up the terminal and hit the object the barrier is intended to protect. Consequently, turned down terminals should not be used on the approach ends of roadside or median barriers on high-speed, high-volume roads unless they are also flared.

vi. Termination of a barrier in a back slope eliminates the danger of an untreated barrier end and reduces the opportunity for errant vehicles to penetrate the end of the barrier.

vii. A number of end treatments have been developed for metal beam barriers that utilize a combination of a breakaway mechanism and a cable with a flared configuration to address the spearing and roll-over potential and to develop the full tensile strength of the rail for downstream impacts.

viii. Where an end treatment is designed as a "gating" device, i.e., to allow for controlled penetration of a vehicle when impacted, through a breakaway mechanism, care should be taken to provide an adequate run-out area behind the end treatment.

ix. The concrete safety shape barrier can be terminated by tapering the end. However, this treatment should only be used where speeds are low (60 km/h or less) and space is limited. Flaring the barrier beyond the clear zone should be considered on higher speed facilities where space is available.

x. Proprietary mechanical end treatments are often suitable only for limited types of barrier applications. When adopting such technologies, designers should ensure not only the efficacy of the technology of their choice but also its compatibility with the barrier technology being used. In addition to information generally available from the manufacturers and suppliers of these treatments, road agencies and others compile and provide appropriate guidance in respect of crash testing results and system compatibility recommendations.

xi. All systems should be installed with a level surface leading to the treatment. The use of kerb and gutter is discouraged, but if they are needed, only the mountable type should be specified.

The principles noted above provide a rule of thumb approach. Road designers should still investigate physical site restrictions such as longitudinal space, hazard width, slopes and surface types. At locations with a high likelihood of collisions, the costs of accidents and repair should be factored into the decision matrix in addition to the initial installation costs.

Designers should note that new technologies are continually being developed and tested. Nothing in this Guide relieves the designer of the responsibility of keeping abreast of these new technologies and their potential application to the roadside barrier end treatment problem.

Plate 12.15 Training end w-beam guardrail anchorage

Plate 12.16 Terminals for high tension cable barrier systems

Plate 12.17 CASS^{TM} cable terminal (CCT)

Plate 12.18 W Beam guardrail anchored in back slope

Plate 12.19 Modified eccentric loader terminal (MELT)

Plate 12.20 Bursting energy absorbing terminal

Plate 12.21 Brakemaster® 350

Plate 12.22 Crash Cushion Attenuating Terminal (CAT-350™)

Plate 12.23 FLEAT Median Terminal (FLEAT-MT™)

Plate 12.24 X-Tension™ Median Attenuator System (X-MAS)

12.5.5.1 TREATMENT STRATEGY

The following end terminals should be avoided on roads with speed limit of 70km/h or above:

i. Bullnose terminals
ii. Fishtail terminals
iii. Ramped down terminals
iv. Blunt terminals

Safety barrier terminals should not be positioned around bends where there is an increased risk of loss of control. Attention should be given to combination of roadside features which could aggravate the consequences of a crash. Examples include a ramped down terminal located shortly ahead of highly aggressive features such as a bridge pier or a sheer drop. Roadside ditches may also guide an errant vehicle to collide with an end terminal.

The following strategy should be considered:

i. Closing short gaps e.g. gaps < 50m of sections of safety barriers
ii. Extending the safety barrier upstream so that the terminal is located in an area of lower traffic speeds e.g. a slip road
iii. Extending the safety barrier upstream of curves and other locations susceptible to roadside crashes
iv. Extending and flaring the safety barrier to anchor onto an upstream slope that is steeper than 1:2 as illustrated in **Plate 12.18**.

For any remaining safety barrier end terminals with speed limit of 70km/h or above, crashworthy terminals or crash cushions are appropriate end treatments if economics, availability and maintenance are not a constraint.

For any remaining safety barrier end terminals with speed limit of 60km/h or below, the following end treatments may be tolerated:

i. Blunt end terminals with flaring
ii. Ramped down terminals preferably with flaring
iii. Blunt end terminals without flaring (speed limit ≤ 50km/h)

These end terminals should be located on straight sections well ahead of bends and other locations susceptible to loss of control. Where ramped-down terminals are adopted, the adjacent roadside should not be highly aggressive in case a vehicle passes or rolls over the terminal.

End terminals should be highly visible to alert drivers in keeping their lateral position if they are in close proximity to the edge of carriageway. Possible treatments include the use of delineators, hazard markers and reflective paints or retro-reflective sheeting on the end terminal.

12.5.5.1.1 DOWNSTREAM END TERMINALS

Downstream end terminals for unidirectional traffic may be anchored with a ramped down terminal or left unanchored. They should be provided with the following arrangements:

i. Where a clear zone commences
ii. Overlapping another roadside safety barrier or acceptable uphill side slope
iii. Terminating near intersections or road sections where speed is low with no outstanding roadside aggressive features

Downstream end terminals for bidirectional traffic should be treated as upstream end terminals if they lie within the clear zone for opposing traffic including overtaking vehicles.

12.5.5.1.2 MEDIAN OPENINGS

Where median barriers are terminated at median openings, priority intersections, signalised intersections or roundabouts, the layout will need to minimise the risk of collision and to tie up with visibility requirements. It would be desirable to flare the median barrier away from approach traffic and to defer the commencement of end terminals.

12.5.5.1.3 CRASHWORTHY TERMINALS

Crashworthy terminals are generally non-redirective products associated with safety barrier systems, typically W-beams. They should be tested to conform to MASH (US), EN1317-4 (EU) or equivalent standard. There are four classes under EN 1317-4 as shown in **Table 12.23**. A typical installation of P4 crashworthy terminal is illustrated in **Plate 12.25**.

As crashworthy terminals are non-redirective, an adequate clear zone is required in case an errant vehicle passes in front of or breaks through the system.

Table 12.23 Crashworthy terminals classification TO EN1317-4

Performance Class	Test Speed (km/h)
P1	80
P2	
P3	100
P4	110

Plate 12.25 P4 Crashworthy terminal to EN1317-4

12.5.6 TRANSITION

Transition sections are necessary to provide continuity of protection when two roadside or median barriers of differing lateral stiffness are joined together. Transition sections with gradually increasing lateral stiffness are necessary when a barrier joins another barrier system with dissimilar deflection characteristics, such as a W-beam guardrail connected to a bridge rail.

The transition design should produce a gradual stiffening of the overall approach protection system so that vehicular pocketing, snagging, or penetration can be reduced or avoided at any position along the transition. Typical transitions are illustrated in **Plate 12.26** and **Plate 12.27**.

Plate 12.26 Transition between concrete safety barrier and metal parapet

Plate 12.27 Transition between W-Beam safety barrier and concrete parapet

A transition section is needed where a semi-rigid approach barrier joins a rigid bridge railing. Transitions may not be necessary when bridge railings with some flexibilities are used.

Details of special importance for transitions are as follows:

i. The approach-rail/bridge-rail splice or connection must be as strong as the approach rail itself so it will not fail when struck by pulling out and allowing a vehicle to strike the end of the bridge railing. The use of a cast-in-place anchor or through-bolt connection is recommended. The transition also must be designed to minimize the likelihood of snagging by an errant vehicle, as well as one from the opposing lane on a two-way facility.

ii. Strong-post systems (usually blocked out) or post-and-strong-beam systems can be used on transitions to rigid bridge railings or other rigid objects. These systems usually should be blocked out from their posts unless the railing member is of sufficient width to prevent or reduce snagging to an acceptable level. However, blockouts or railing offsets alone may not be sufficient to prevent potential snagging at the immediate upstream end of the rigid bridge railing. A rubrail may be desirable in some designs that use flexible W-beam or box-beam transition members. Tapering of the rigid bridge railing end behind the transition members at their connection point also may be desirable, especially when the approach transition is recessed into the concrete end of the bridge railing or other rigid object.

iii. The transition section should be long enough so that significant changes in deflection do not occur within a short distance. Generally, the transition length should be 10 to 12 times the difference in the lateral deflection of the two systems in question.

iv. The stiffness of the transition should increase smoothly and continuously from the less rigid system to the more rigid one. This usually is accomplished by decreasing the post spacing, increasing the post size, or both, as well as by strengthening the rail element. W-beam or thrie-beam rail elements typically are strengthened by nesting two rails together.

v. When drainage features (e.g., kerbs, raised inlets, kerb inlets, ditches, or drainage swales) are constructed in front of barriers, especially in the transition area, they may initiate vehicle instability that can, in some instances, adversely affect the crashworthiness of the transition. However, some transition designs incorporate a kerb to reduce the probability of a vehicle snagging on the end of a rigid bridge railing. The slope between the edge of the travelled lane and the barrier should be no steeper than 1V:10H.

12.5.6.1 CONNECTION BETWEEN SAFETY BARRIER SECTIONS

Discontinuity of sections of safety barriers could lead to an errant vehicle colliding frontally with the downstream safety barrier section. In the case of W-beam or Thrie-beam safety barriers, individual sections have to be securely fastened with overlapping in the direction of traffic. In the case of precast segments of rigid safety barriers, they have to be designed in such a manner that connection between successive units could be maintained during an impact.

12.5.6.2 OPENINGS ALONG SAFETY BARRIERS

Openings may be required along safety barriers to provide a refuge or to facilitate maintenance and management. Where openings are provided, the arrangement should prevent an errant vehicle leaving the road at 30 degrees to collide with the end terminal of the downstream safety barrier. This is illustrated in **Figure 12.21**. This arrangement does not apply for bidirectional roads where an errant vehicle may crash from either direction.

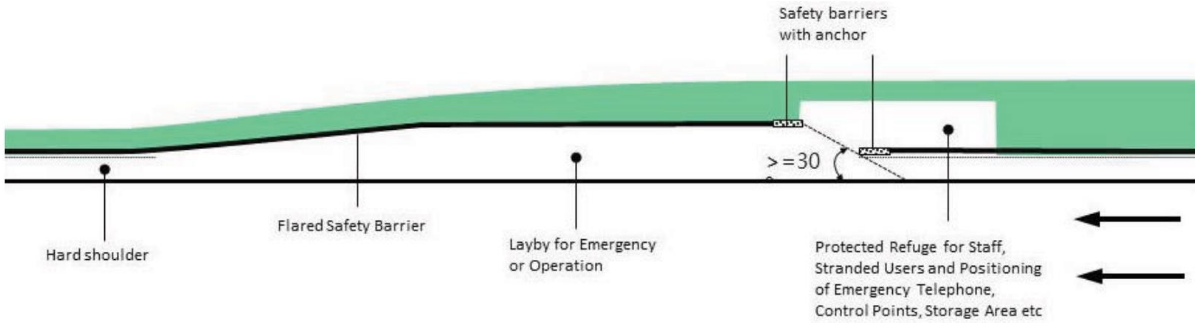


Figure 12.21 Openings along safety barriers

12.5.7 CRASH CUSHION

Crash cushions, also known as impact attenuators, are protective devices that significantly reduce the severity of impacts with fixed objects. This function is accomplished by gradually decelerating a vehicle to a safe stop for head-on impacts and by redirecting a vehicle away from the fixed object for side impacts. Crash cushions are ideally suited for use at locations where fixed objects cannot be removed, relocated, or made to break away, and where they cannot be adequately shielded by a longitudinal barrier.

Typical objects and areas that can benefit from the use of impact attenuators include:

i. An expressway exit ramp gore area in an elevated or depressed structure where a bridge rail end or a pier requires shielding
ii. The ends of roadside or median barriers
iii. Rigid objects like cantilever sign gantries within the clear zone
iv. Construction work zones
v. Toll booths

12.5.7.1 DESIGN PRINCIPLES

A crash cushion's major contribution to highway safety is its ability to absorb the vehicle's kinetic energy at a controlled rate, decelerating an impacting vehicle to a stopped or nearly stopped condition in a relatively short distance and in such a way that the potential for serious injury to its occupants is reduced. Commonly used crash cushions generally employ one or both of two concepts to accomplish this task:

i. the work-energy principle.
ii. the conservation of momentum principle.

The various types of crash cushions based on these principles are summarized in **Table 12.25**. **Plate 12.28** to **Plate 12.38**, show some of these crash cushions.

12.5.7.1.1 WORK-ENERGY PRINCIPLE

The work-energy principle of crash cushion design involves the reduction of an impacting vehicle's kinetic energy to zero, which is the condition of a stopped vehicle.

The vehicle's initial kinetic energy is determined by the following formula, $KE = \frac{1}{2}MV^2$ where KE is the kinetic energy, M is the vehicle's mass, and V is the vehicle's initial velocity.

As the vehicle is slowed by the crash cushion, the vehicle's kinetic energy is reduced through conversion of kinetic energy into other forms of energy, including:

i. Mechanical energy, dissipated through the deformation of the vehicle and crash cushion components,
ii. Potential energy, to the extent that crash cushion components and vehicle components will deform during the impact event and later will rebound toward a pre-crash shape,
iii. Heat energy, dissipated through friction generated by sliding components, and
iv. Sound energy, a minor factor, which is evidenced by the noise produced by the impact.

Work (W) done on an object, the object being the vehicle in the case of crash cushion design, is defined as the net change in kinetic energy of the object. Based on an assumption that the vehicle will be in a stopped condition at the conclusion of the impact, the "work" done on the vehicle is equal to the vehicle's initial kinetic energy; thus the "work-energy" principle.

In practice, each crash cushion system uses a unique combination of methods to convert the kinetic energy of an impacting vehicle into other forms of energy. Some devices will utilize "crushable" or "plastically deformable" materials. Other devices will utilize "elastically-deformable" materials that will rebound to, or near to, their pre-crash shapes. All impacts with crash cushions will result in some damage to the impacting vehicle, which is a viable strategy for slowing the impacting vehicle as long as the potential for serious injury to the vehicle occupants is low. The conservation of momentum principle, as discussed below, is involved in all crash cushion impacts, since some portion of the vehicle's kinetic energy is transferred to components of the crash cushion by accelerating and moving them during the impact event.

There are many ways to manage the energy in a crash, and because these many ways can be applied in unique combinations, there are a wide variety of crash cushions systems available. These types of systems may be referred to as compression crash cushions. Crash cushions of this type need a rigid support structure or anchorage to resist the vehicle impact force that deforms the energy-absorbing material. There are currently no widely accepted methods to determine the performance of this type of device without full-scale crash tests, although computer simulation is frequently used to analyse new or modified designs prior to crash testing.

12.5.7.1.2 CONSERVATION OF MOMENTUM PRINCIPLE

The conservation of momentum principle of crash cushion design involves the transfer of the momentum from the impacting vehicle to an expendable mass of material located in the vehicle's path. The expendable mass usually consists of containers filled with sand.

Devices of this type need no rigid back-up or support to resist the vehicle's impact force. Instead, the vehicle's velocity and kinetic energy are incrementally reduced through momentum transfer by accelerating the sand particles found within the containers or barrels. This type of crash cushion is generally referred to as an "inertial" crash cushion and is the only type whose design can be readily determined analytically.

A. Sacrificial Crash Cushions

Sacrificial crash cushions are crashworthy roadside safety devices designed for a single impact. Most of the systems absorb impact energy by crushing the steel rail elements. Other devices have expendable plastic cartridges containing foam, sand, or water, which also absorb energy by crushing. These systems' major components are destroyed in impacts, but many of the other parts can be reused. These devices generally offer low initial costs and can be cost-effective if placed in locations where the designer expects infrequent crashes to occur.

B. Reusable Crash Cushions

Reusable crash cushions have some major components that may be able to survive most impacts intact and can be salvaged when the unit is being repaired. Some of the components, however, need to be replaced after a crash to make the entire unit crashworthy again. The initial purchase and installation of reusable products generally are more expensive than sacrificial products, but in locations where designers expect to have frequent crashes, these devices may very well be cost-beneficial and appropriate.

C. Low-Maintenance and/or Self-Restoring Crash Cushions

They are typically considered for use at locations where a high frequency of impacts may be expected. The category of "Low Maintenance and/or Self Restoring" crash cushions include those devices that either suffer very little, if any, damage upon impact and are easily pulled back into their full operating condition, or they partially rebound after an impact and may only need an inspection to ensure that no parts have been damaged or misaligned. Although some attenuators can still function and save lives after being struck once, no device is completely maintenance free. It is important to note that devices in this category may be low-maintenance, self-restoring, or both. Inclusion of a device in this combined category does not imply that the device has both attributes. Often these products are installed in high-speed, high-traffic volume ramps or medians to reduce the exposure of maintenance workers to the traffic.

Table 12.24 Types of Crash Cushions

Design Principle	Classification	Crash Cushion	Test Level (TL)
Work-Energy	Sacrificial Crash Cushions	Thrie-Beam Bullnose Guardrail System	3
		ABSORB 350®	3, 2
		Advanced Dynamic Impact Extension Module (ADIEM™)	3
		Bursting Energy Absorbing Terminal–Single Sided Crash Cushion (BEAT-SSCC™) System	3
		Bursting Energy Absorbing Terminal–Bridge Pier (BEAT-BP™) System	3
		QuadTrend® 350	3
		Narrow Connecticut Impact Attenuation System (NCIAS)	3
	Reusable Crash Cushions	QuadGuard® Family 3-bay unit 6-bay unit	 2 3
		Universal TAU-II® Family	2, 3
		Trinity Attenuating Crash Cushion (TRACC™) Family	2 & 3
		QUEST® Crash Cushion	3
	Low-Maintenance and/or Self-Restoring Crash Cushions	Compressor™ Attenuator	3
		EASI-CELL® Cluster	1
		Hybrid Energy Absorbing Reusable Terminal (HEART™)	3
		QuadGuard Elite 7-bay unit 8-bay unit 9-bay unit	 2 3 3
		QuadGuard Low-Maintenance Cartridge (LMC)	3
		Reusable Energy-Absorbing Crash Terminal (REACT 350®) 4-cylinder array 9-cylinder array	 2 3
		Smart Cushion Innovations (SCI) SCI-70GM SCI-100GM	 2 3
Conservation of Momentum	Sand-filled plastic barrels	Fitch Universal Barrel	3
		ENERGITE III	3
		Big Sandy	3
		CrashGard	3

Plate 12.28 Bullnose Guardrail System

Plate 12.29 Advanced Dynamic Impact Extension Module (ADIEM™)

Plate 12.30 Hybrid Energy Absorbing Reusable Terminal (HEART™)

Plate 12.31 QuadGuard Elite

Plate 12.32 QuadGuard® Crash Cushion

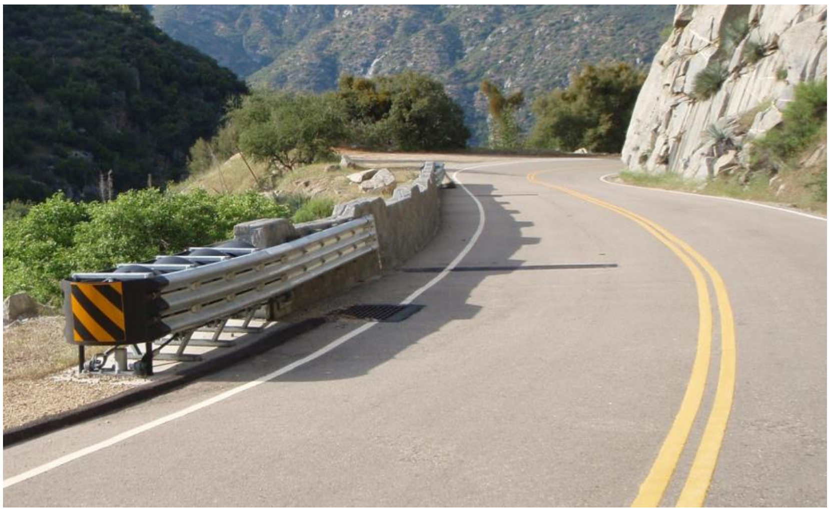


Plate 12.33 TAU-II Crash Cushion

Plate 12.34 QUEST® Crash Cushion

Plate 12.35 QuadGuard Low-Maintenance Cartridge (LMC)

Plate 12.36 Reusable Energy-Absorbing Crash Terminal (REACT 350®)

Plate 12.37 Smart Cushion Innovations (SCI-100GM) Crash Cushion

Plate 12.38 The Fitch Universal Barrel

12.5.7.1.3 SAND FILLED PLASTIC BARREL IMPACT ATTENUATORS

Sand-filled plastic barrel impact attenuators work on the principle of conservation of momentum and therefore does not necessarily need a backdrop in front of the hazard being shielded. Recommendation pertaining to these devices are as follows:

i. Single rows of barrels should not be allowed for permanent installation
ii. Barrels should be spaced some 150 mm apart and stop 300 mm to 600 mm short of the hazard being shielded
iii. Barrels should be positioned in such a way that rigid corners of the hazard are overlapped by barrels by some 760 mm (300 mm minimum) to reduce the severity of angled impacts near to the rear of the attenuator. Where such attenuators are subject to bi-directional traffic flow, the array of barrels should be flush with the edge of the hazard

so as to ensure that reverse direction traffic does not inadvertently impact the rear end of the barrel arrangement.

If speeds higher than 95 km/h are anticipated, barriers can be lengthened. Since most serious accidents occur at excessive speeds, an "over-design" is acceptable where space permits.

Plate 12.39 shows a typical example of plastic drum attenuators. A number of typical arrangements of sand-filled barrel attenuators are shown in **Figure 12.22.** The legend illustrates the mass of sand contained in each barrel.

Plate 12.39 Sand filled plastic barrel attenuators located at a gore

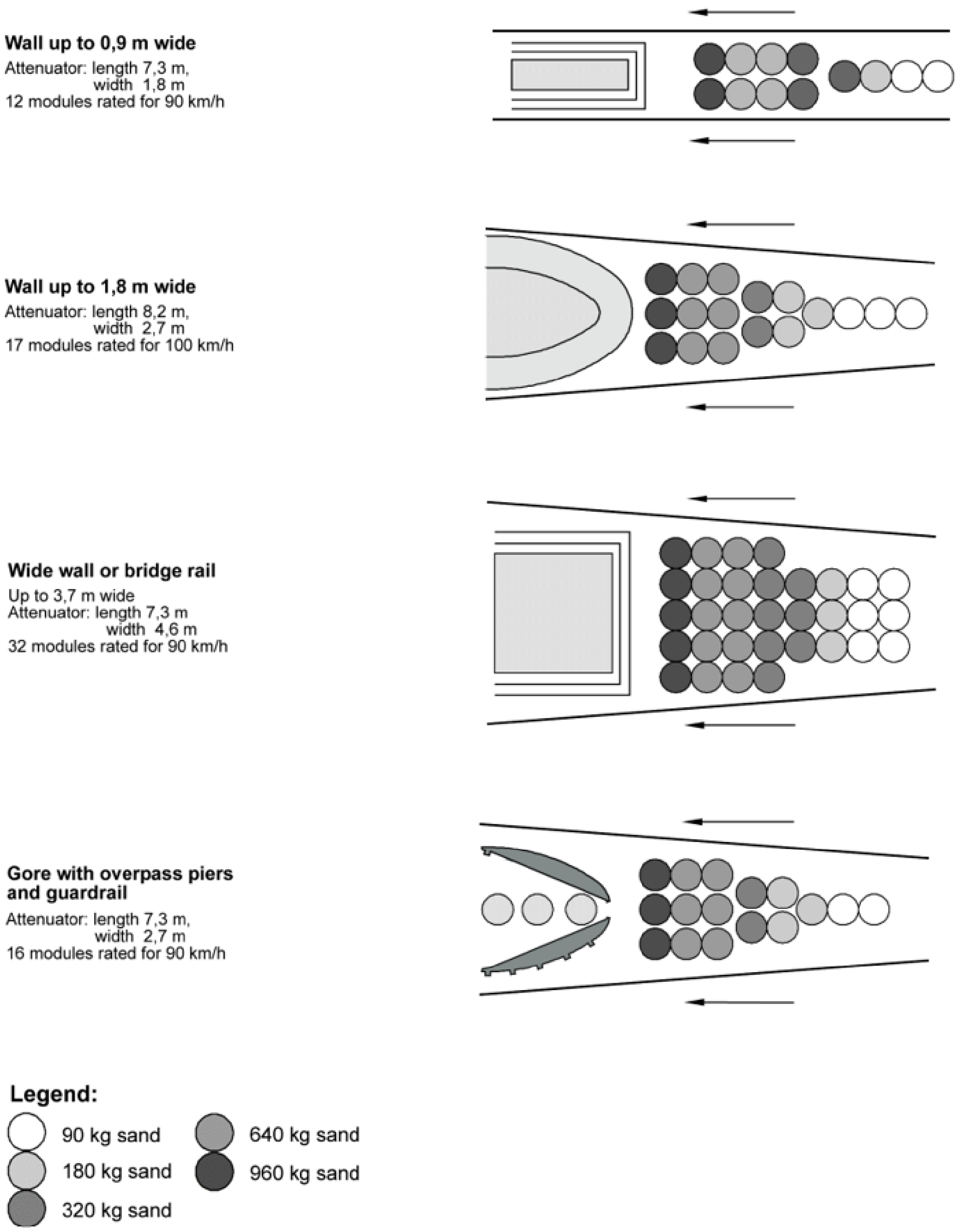


Figure 12.22 Typical arrangement of sand-filled barrel attenuators

12.5.7.2 TYPES OF CRASH CUSHIONS

Crash cushions are categorised into redirective (non-gating) or non-redirective (gating) products. Redirective products redirect an errant vehicle hitting the side of the crash cushion back onto its travel path. They should be specified in most circumstances to limit secondary consequences. Non-redirective products are more appropriate where adequate space or clear zone is available on the roadside.

Crash cushions are also categorised into unidirectional or bi-directional products. Uni-directional products are generally appropriate on the roadside and at diverge gores of divided roads as well as toll booths. In the following situations, bi-directional products are required to allow for vehicles colliding from the opposite direction:

i. Roadside and diverge gores on a two-way road
ii. Isolated bridge piers in the middle of a road separating opposing traffic
iii. Hazards on traffic islands separating opposing traffic
iv. Installations on the median

12.5.7.3 FUNCTIONAL CONSIDERATIONS

Attenuators as well as barrier end-treatments can be installed as bi-directional or uni-directional as well as with re-directive or non-redirective capabilities. As a general rule, the chosen system should be able to redirect an errant vehicle if the hazard being shielded is less than 3m from the edge of the travelled way. Typical functional considerations for attenuators and barrier end terminals are given in **Table 12.25**.

Table 12.25 Impact attenuator and end terminal application

Appropriate Device Type	Distance (m)		Probable Impact Angle $(^{0})$
	Hazard to traffic	Opposing to traffic hazard	
Re-directive			
Bi-directional	< 3	< 9	> 5
Uni-directional	< 3	> 9	> 5
Non-redirective	< 3	> 9	> 5

12.5.7.4 CRASH CUSHION SELECTION GUIDELINES

The number and complexity of factors that enter the selection process for crash cushions preclude the development of a simple selection procedure. Each operational system has its own unique physical and functional characteristics. In some cases, one crash cushion will stand out as the most appropriate, but in most instances, two or more types of crash cushions will provide satisfactory protection to an errant motorist and the designer should choose between them. Once a decision has been made that a roadside feature should be shielded and that a crash cushion is the appropriate way to shield it, the designer should consider the following factors before making a final selection:

i. Site characteristics
ii. Structural and safety characteristics of candidate systems
iii. Cost
iv. Maintenance characteristics
v. Selection criteria

vi. Inclusion criteria.

Each of these factors is discussed in the following subsections.

12.5.7.4.1 SITE CHARACTERISTICS

During the preliminary design stages for new construction and for rehabilitation or reconstruction of existing highways, making provisions for adequate space for crash cushions to shield non-removable fixed objects should be considered. This will promote compatibility between the final design and the crash cushion that is to be installed. **Figure 12.23** and **Table 12.26** suggests the area that should be made available for crash cushion installation. Although it depicts a gore location, the same recommendations will generally apply to other types of fixed objects that are to be shielded. The unrestricted conditions represent the minimum dimensions for all locations except for those sites where it can be demonstrated that the increased costs for obtaining these dimensions (as opposed to those for restricted conditions) will be unreasonable. Note that the information provided in this table is generic and may not be adequate for some systems. Therefore, it is recommended that the designer look at the various available systems that will adequately shield the obstacle and determine the space needed from the manufacturer's specifications.

The designer should be aware of site conditions that might dictate the type of crash cushion needed. For example, fixed objects, such as rigid barrier ends that are less than 900mm wide, should be shielded by a narrow crash cushion. Similarly, wide obstacles, such as those greater than 5m, can be effectively shielded by sand barrel arrays.

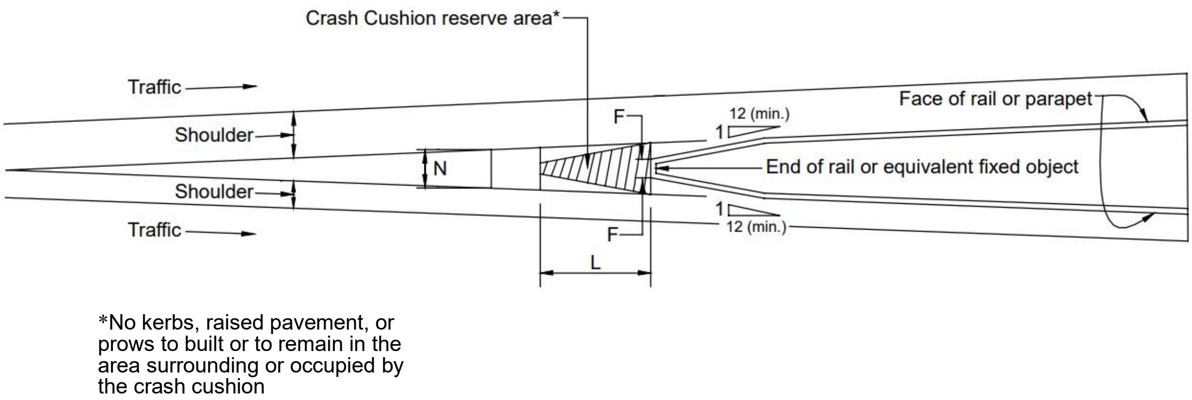


Figure 12.23 Area Available for Crash Cushion Installation

Table 12.26 Area Available for Crash Cushion Installation

Design Speed (km/h)	Dimensions for Crash Cushion, Reserve Area (m)								
	Minimum						Preferred		
	Restricted Conditions			Unrestricted Conditions					
	N	L	F	N	L	F	N	L	F
50	2	2.5	0.5	2.5	3.5	1	3.5	5	1.5
80	2	5	0.5	2.5	7.5	1	3.5	10	1.5
100	2	8.5	0.5	2.5	13.5	1	3.5	17	1.5
120	2	11	0.5	2.5	17	1	3.5	21	1.5

Note:
N = for preliminary design purposes, an assumed width of space necessary for the placement of a crash cushion.
L = for preliminary design purposes, an assumed length of space necessary for the placement of a crash cushion.
F = for preliminary design purposes, an assumed maximum width of a fixed object that will need to be shielded with a crash cushion.

12.5.7.4.2 STRUCTURAL AND SAFETY CHARACTERISTICS

When more than one crash cushion system is under consideration, the designer should carefully evaluate the structural and safety characteristics of each candidate system, including such factors as impact decelerations, redirection capabilities, anchorage and backup structure needs, and debris produced by impact.

All of the systems described in this chapter as meeting MASH or NCHRP Report 350 TL-3 evaluation criteria have the capability to stop both compact cars and pickup trucks impacting head-on at 100 km/h within tolerable deceleration levels and redirect or contain those vehicles impacting on the sides of the units within the system’s length-of-need.

Most of the crash cushion systems previously described may be configured to accommodate lesser impact speeds when site and operational conditions permit. Note that additional lower mass sand barrel modules sometimes could be added to an array to reduce the expected deceleration forces to lower levels. This is especially true when the shielded object is well off the roadway and the additional modules do not significantly reduce a motorist’s ability to avoid a crash.

12.5.7.4.3 COSTS

Cost considerations should include initial material costs, site preparation costs, installation costs, maintenance costs, and repair or replacement costs. Site preparation costs can be significant when accommodating certain systems. At locations where frequent hits are expected, life-cycle costs for repairing or replacing a crash cushion system also may become a significant factor in the selection process.

12.5.7.4.4 MAINTENANCE CHARACTERISTICS

Frequently, the most appropriate crash cushion still will not be evident after analysing the site conditions, operational characteristics, and the initial costs of candidate systems. The maintenance characteristics of each crash cushion will, in many cases, play an important role in the selection process. **Table 12.27** summarizes pertinent maintenance characteristics of each crash cushion. This information is based primarily on subjective evaluations. When available, individual agency maintenance records should be used to establish costs associated with the types of crash cushions in actual use. Although the information in **Table 12.27** will permit a designer to compare the relative maintenance characteristics of candidate systems, there is no substitute for knowing the actual maintenance needs and costs for in-service installations. Each agency should consider documenting this information, so it is available to the designer.

Maintenance characteristics can be conveniently categorized as regular (or routine) maintenance, crash maintenance, and material inventory needs. Each category is described in **Table 12.27**.

Many systems described in this chapter need relatively little regular or routine maintenance. However, it is important that periodic maintenance checks are performed, so that each installed unit remains fully functional. If a crash cushion is located in an area that is accessible to pedestrians, vandalism may be a problem. Some cracking problems have occurred in the past with the plastic containers used in the inertial systems. These problems have been attributed in part to vibration (when the sand barrels were located on structures), to calcium chloride), and to design problems with the seams of some first-generation modules. It appears that these problems have been solved through improved designs. Plastic sand barrels eventually will degrade from exposure to ultraviolet light; barrels older than 10 years should be inspected more frequently and replaced when necessary.

Crash maintenance characteristics demand special consideration because they may dictate the most effort and expenditure during the life of an installation. If a particular site has a relatively high frequency of crashes, using a crash cushion that has some degree of reusability or self-restoration is recommended. Similarly, if nuisance hits are relatively common, a crash cushion with redirection capability should reduce or eliminate the maintenance effort for minor repairs or partial replacement of a system. The availability of the replacement parts needed to restore a damaged crash cushion to its original capacity is closely associated with repair time and cost.

Thus, the type and number of spare parts that should be kept on hand or quickly obtainable to repair each type of crash cushion in use by an agency may play an important role in the final selection process. The fewer different types of crash cushions used by an agency, the easier it becomes to establish and maintain an adequate inventory of replacement parts. Ideally, permanent repairs should be made very quickly. If not, appropriate temporary measures should be taken to afford a reasonable level of protection or delineation until the original crash cushion can be restored.

Table 12.27 Comparative Maintenance Characteristics

Crash Cushion	Routine Maintenance	Crash Maintenance	Material Inventory
Sacrificial Crash Cushions			
Thrie-Beam Bullnose Guardrail System	Can be inspected on a drive-by. Cable tension should be checked periodically.	Rail elements and posts should be replaced. Cables and foundation tubes are normally reusable.	Slotted thrie-beam rail elements and wood posts.
ABSORB 350®	Normally can be inspected on a drive-by. Periodic on-site inspections should be performed to be certain that all parts are properly connected. Need to check water level.	Nosepiece and damaged energy-absorbing elements should be replaced.	Replacement nosepiece, energy-absorbing elements, and fluid supply. Other parts per manufacturer's recommendation.
ADIEM™	Modules should be closely inspected for damage.	Damaged concrete modules should be replaced. Damaged covers also should be replaced. Most other parts normally are reusable.	Replacement concrete modules, covers, and other parts per the manufacturer's recommendation.
BEAT-SSCC™	Normally can be inspected on a drive-by. Periodic on-site inspections should be performed to be certain that all parts are properly connected, and anchor cable is not slack.	Damaged tubes and posts should be replaced. Impact head is normally reusable.	End tube, second tube, breakaway posts.
BEAT-BP™	Normally can be inspected on a drive-by. Periodic on-site inspections should be performed to be certain that all parts are properly connected, and anchor cable is not slack.	Damaged tubes and posts should be replaced. Impact head is normally reusable.	End tube, second tube, standard line tubes, breakaway posts, and standard line posts.
QUADTREND 350®	Can be inspected on a drive-by for external damage. If lids are not riveted on, sand content should be checked periodically.	Most major components should be reusable after a crash.	Spare parts per manufacturer's recommendation.
NCIAS	Can be inspected on a drive-by.	Crushed units should be removed from site; minor damage can be repaired on-site by jacking.	Spare cylinders to replace badly damaged units.
Sand-Filled Barrels	Can be inspected on a drive-by for external damage. If lids are not riveted on, sand content should be checked periodically.	Individual sand barrels should be re-placed after a crash; units damaged by nuisance hits also should be replaced. Debris should be removed from the site.	Spare barrels, sand support inserts, and lids; supply of sand.
Reusable Crash Cushions			
QuadGuard®	Normally can be inspected on a drive- by; missing or displaced cartridges can be readily noted. Should be periodically inspected on-site to be certain that all parts are properly connected.	Nose, expended cartridges, and damaged fender panels should be replaced. Unit should be repositioned.	Spare cartridges, nose units, fender panels, and other parts per manufacturer's recommendation.
Universal TAU-II™ Family	Normally can be inspected on a drive- by. Periodic on-site inspections should be performed	After a frontal impact, the system can be pulled out to restore the	Cartridges, sliding panels, pipe panel mounts, and nose

Crash Cushion	Routine Maintenance	Crash Maintenance	Material Inventory
	to be certain that all parts are properly connected.	proper length. Replace damaged cartridges. During some side impacts, the sliding panels may be damaged.	pieces per manufacturer's recommendations.
TRACC™	Normally can be inspected on a drive-by. Periodic on-site inspections should be performed to be certain that all parts are properly connected.	The rip plates need replacement. Newer versions of the TRACC eliminate need for extensive disassembly. The nose and fender panels also may need replacement.	Replacement rip plates, nose sections, fender panels, and other replacement parts per manufacturer's recommendation.
QUEST®	Normally can be drive-by inspected. Periodic on-site inspections should be performed to be certain that all parts are properly connected.	The nose, fender panels, and energy- absorbing rails or tubes need replacement after impacts. Open design allows for easy repair.	The nose, fender panels, and energy-absorbing rails or tubes and other parts per manufacturer's recommendations.
Low Maintenance and/or Self-Restoring Crash Cushions			
Compressor	Normally can be inspected on a drive-by.	This unit is designed to take repeated impacts without any additional recovery procedures and with minimal or no repairs.	Spare parts per manufacturer's recommendation.
EASI-CELL®	Normally can be inspected on a drive-by. Plastic cylinders may deteriorate after several years of exposure to the elements.	This unit is designed to withstand multiple impacts without cylinder replacement. All cylinders need to be replaced when the minor axis of the cylinders in the rear most row measures 230mm or less.	Spare parts per manufacturer's recommendation.
HEART™	Normally can be inspected on a drive-by.	Repair will depend on the severity of the impact. Minor side impacts may require no repair. End-on impacts may require only pulling the system back into place and replacing the nose bolt.	Spare parts per manufacturer's recommendation.
QuadGuard LMC and Elite	Normally can be inspected on a drive-by. Periodic on-site inspections should be performed to be certain that all parts are properly connected.	Much of unit is reusable after a crash. Unit tends to self-restore to some extent but should be evaluated after each impact. Unit may need to be repositioned. When diameter of last cartridge becomes less than 660mm, all cartridges should be replaced.	Fender panels and other replacement parts per manufacturer's recommendation.

Crash Cushion	Routine Maintenance	Crash Maintenance	Material Inventory
REACT 350®	Can be inspected on a drive-by.	The system is considered fully reus-able. Repositioning is normally all that is needed after an impact. After side impacts, inspect stabilizer rods. If the cylinders cannot be restored to 90percent of the original diameter, they should be replaced.	Spare parts per manufacturer's recommendation.
Smart Cushion SCI	Can be inspected on a drive-by for external damage. If the frontal collapse has been initiated, the unit should be inspected and reset.	The system will need two shear bolts and possibly a new delineator plate under design criteria impacts.	Shear bolts and delineator panel.

12.5.7.4.5 SELECTION CRITERIA

Criteria for selection of crash cushion types should be objective in order to promote consistency in selection of devices within a given agency. Crash history is obviously an important guide in selecting appropriate crash cushions. For new installations and those where crash history is not available, Average Daily Traffic (ADT) has been shown to be an accurate barometer of impact frequency. Repair times are also a very important factor as some roads have a narrow time window when repairs can be performed. Proximity to the roadway will affect impact frequency and is also important due to lane closure needs. If night repairs are necessary, repair crew exposure should be minimized. Lastly, gore areas are subject to driver indecision which causes sudden lane changes, potentially resulting in increased frequency of impacts.

Road Agencies will need to select classifications of devices, and among the devices that may fit within a classification, based on the conditions, the crash cushion performance that is desired, and their desired balance between initial and repair costs. Guidelines which may be considered for adoption by Road Agencies include:

i. **Sacrificial Crash Cushions**
    - ADT < 25,000veh/day
    - low history or expectation of impacts occurring during lifetime of the crash cushion
    - locations >3m from travelled way and/or outside of the clear zone.

ii. **Reusable Crash Cushions**
    - ADT < 25,000 veh/day
    - history or expectation of one or fewer impacts each year
    - unlimited repair time locations
    - locations >3m from the travelled way.

iii. **Low Maintenance and/or Self-Restoring Crash Cushions**
    - ADT ≥ 25,000veh/day

- history or expectation of multiple impacts each year
- sites with repair time limitations
- locations within 3m of the travelled way
- sites requiring night repairs, and gore locations.

12.5.7.4.6 INCLUSION CRITERIA

Inclusion criteria guidelines that should be considered include:

- **Sacrificial Crash Cushions**—Full replacement or substantial field repairs may be needed following an impact. Damaged components are identified in the field, as needed repair parts may not be readily predictable. Repair time and costs will vary greatly depending on the impact conditions.

- **Reusable Crash Cushions**—Typically, these devices will be field-repairable. Many major components will be reusable. Needed replacement components are more predictable and devices are designed with the intent to make these repair components replaceable/restorable within moderate parts cost and time constraints.

- **Low Maintenance and/or Self-Restoring Crash Cushions**—To be included in this category, a threshold on repair parts could be considered. Similarly, a threshold on repair time could be established, such as a repair time for a four-person crew of one hour or less. Such criteria may be defined based on frontal and side impacts approximating the severity of crash test impacts. It may be reasonable to expect reporting of repair cost and time results from a yet-to-be established minimum number of real-world impacts before a device would be formally included in this category.

12.5.7.5 PLACEMENT RECOMMENDATIONS

Most crash cushions and other types of end treatments were designed and tested on flat, level terrain. Consequently, a system installed on or behind certain terrain conditions may perform unpredictably at best and ineffectively at worst. It is highly desirable that crash cushions be placed on a relatively flat surface and that the path between the roadway and the crash cushion is clear of any obstructions or irregularities. For optimal performance of any system, an impacting vehicle should strike the unit at normal height, with the vehicle's suspension system neither compressed nor extended.

Two prominent features with which the designer often contends are roadside kerbs and slopes. Both of these features can cause an impacting vehicle to become airborne and reach undesirable roll and pitch angles. For new construction, kerbs should not be built where crash cushions are to be installed. Existing crash cushion locations should be reviewed to determine if the presence of a kerb or a slope is likely to affect the performance of the unit, and if so, appropriate modifications should be made when major roadway rehabilitation occurs. In general, a kerb no higher than 102mm may be considered acceptable on existing construction and left in place unless it has contributed to poor crash cushion performance in the past.

The surface on which a crash cushion is installed should be smooth, flat, and compacted. All of the crash cushions should be placed on a hard, smooth pad or surface (usually concrete) to enable the unit to compress uniformly during an impact. In the case of inertial crash cushions, a paved surface, although not needed, provides uniform support for the sand barrels and, perhaps more importantly, provides a surface on which the pattern of the array and the design masses of the modules can be marked. This information should be readily available to maintenance personnel if a damaged or destroyed array is to be restored to its original capacity.

If a crash cushion is installed on a structure, the location of expansion joints may dictate the type of attenuator to be used; if not, some modifications to the standard design may be needed. Non-anchored units such as sand barrels may be susceptible to vibration induced movement. Climatic conditions in a particular area also should be considered because some crash cushions are affected by above or below average temperatures.

12.5.7.6 DESIGN OF IMPACT ATTENUATORS

The detail design of impact attenuators should be done in conjunction with the manufacturer of a specific attenuator and will be dependent on the actual attenuator chosen for installation.

The following factors should be considered for the preliminary design of the impact attenuator:

i. Hazard characteristics - type width and height
ii. Site geometry - including space available for installation
iii. Traffic pattern - bi-directional or unidirectional traffic
iv. Slopes - preferably on flat surface, but with a slope of no more than 1:50 over the length of the attenuator
v. Design speed
vi. Kerb and roadway elevation; preferably no kerb within 16m of attenuator
vii. Probable angle of impact
viii. Base type and base feature
ix. Site features - are there any unique site features
x. Orientation - an attenuator should be oriented to maximize likelihood of head-on impact, though a maximum angle of up to 10 degrees between roadway centre line and attenuation device is acceptable
xi. Placement area.

12.5.7.7 DELINEATION OF END TREATMENTS

End treatments, including terminals and crash cushions, are not intended to reduce the frequency of crashes but to lessen their severity. Nevertheless, if a particular installation is struck frequently, it is important to determine why the crashes are occurring. Frequently, improved signing, pavement markings, or delineation may result in fewer crashes. In this regard, conspicuous, well delineated crash cushions and terminals are significantly less likely to be hit than those that blend into the background, especially at night or during inclement weather. If a system is not reflective, standard object markers make it more conspicuous at night and under conditions of reduced visibility.

12.6 FENCES

Fencing is used to contribute to safe traffic movement. Fences along and within road reserves are primarily used to control entry to road reservations or crossing of roadways by vehicles, pedestrians, or animals. They are also used to provide security for private property, to restrict access to specific areas in order to protect people from hazards within the area, or to secure infrastructure or plant within an area. Specific purposes of fences include:

i. control of access
ii. safety and control of pedestrians
iii. safety and control of cyclists
iv. protection of wildlife and prevention of associated crashes
v. restriction to hazardous areas
vi. security of infrastructure and plant.

Where protection of residences from road noise or screening for aesthetic reasons is required, noise walls or visual screens may be provided and they also function as a fence at or near the reservation boundary.

Fences should be designed to ensure that they restrict the movement of the particular class of vehicle, pedestrian or animal.

In both urban and rural situations, fences should be designed so that they do not constitute a hazard for road traffic, particularly when located in the clear zone of the road. In particular, horizontal rails and other features that could spear or snag vehicles, and large posts, should not be used within the clear zone.

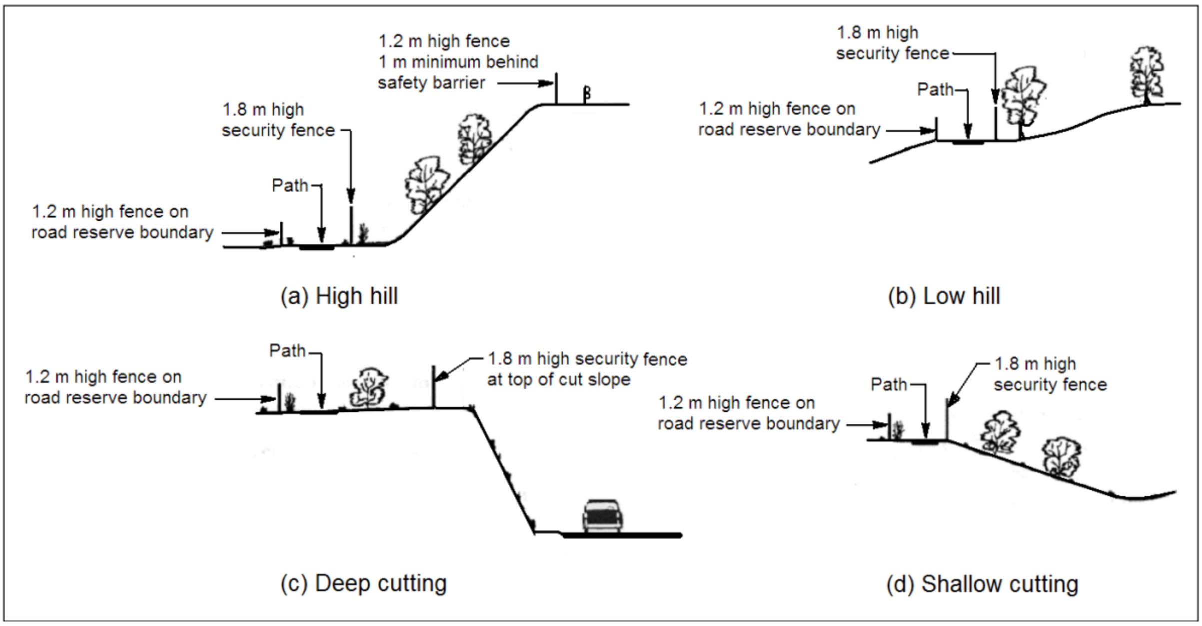
All fences should be designed to ensure that they achieve their intended purpose. Fences should therefore:

i. have sufficient structural strength to resist wind loading and penetration by persons, animals or vehicles that are to be excluded
ii. be high enough to deter people or animals from climbing over
iii. have a mesh size corresponding to the animals that are to be excluded or protected
iv. incorporate barbed wire as an added deterrent, where appropriate
v. be aesthetically pleasing, where necessary.

12.6.1 EXPRESSWAY/MOTORWAY FENCES

Fences on rural expressways/motorways are normally required to define the property boundaries and where appropriate, to prevent the straying of stock onto the road. Urban expressways/motorways will usually require security fencing, but it may or may not be required on major arterial roads. In some cases, noise walls that are provided also function as a fence.

Where practicable, paths for pedestrians and cyclists should be provided within the road reservation and located outside the security fence in order to prevent encroachment onto the road. Options for the location of the fences are shown in **Figure 12.24**.



Note: Fences should be erected on the roadside of the road reserve boundary.

Figure 12.24 Location of security fences for Design Class A/B roads

12.6.2 PEDESTRIAN FENCES

Pedestrian barriers and fences are used along roads or at pedestrian facilities where there is a serious hazard adjacent to the road or facility. Typical hazards may include a high-speed road or railway, a steep drop-off or excavation, large open drains or deep water. Hazardous areas also include locations where there is an unacceptable number (or risk) of crashes involving pedestrians crossing the road and the approaches to signalised intersections and pedestrian crossings.

Where it is proposed to use fences or similar structures in association with a path provided for pedestrians and/or cyclists they should:

i. be high enough to discourage pedestrians from climbing over the top
ii. be located and of a height and construction that will not obstruct sight lines between motor vehicle drivers and path users or between motor vehicle drivers, remembering

that children often use paths

iii. not have horizontal rails that could spear a vehicle, other road users or pedestrians (e.g., post and rail fencing constructed of galvanised pipe or timber)
iv. not have horizontal fence rails that can act as a ladder for children
v. not have gaps wide enough (in a 'full barrier' fence) to enable children to climb through
vi. be finished with no sharp edges or protrusions which could be a danger to those who use the railing for support or guidance; in some situations, finishing the fencing or barrier in a colour in contrast to its surroundings is appropriate to assist its identification by people with impaired vision
vii. not have protrusions that could snag cyclists or the pedals of cycles as they pass
viii. not be constructed of material (particularly fence railings) likely to have (or develop) burrs, splinters, sharp or rough edges or surfaces; in general, steel fences are preferred.

Where fencing with horizontal rails cannot be avoided, it should be constructed so that:

i. the fence is not located within the desirable clear zone appropriate to the road if this is practicable
ii. if the fence must be located within the desirable clear zone, the rail height above the ground should not exceed 300 mm
iii. the fence does not obstruct the necessary sight distances at entrances and intersections.

Barriers, fences, and other treatments placed for the purpose of protecting, controlling or guiding pedestrians should also be designed so that they are safe and appropriate for cyclists.

Pedestrian fences are generally inappropriate where traffic speeds exceed 70km/h and should not be used to substitute safety barriers (refer to **Section 12.5**). When collided by a vehicle, there should be minimum risk of detachment of horizontal rails which may penetrate into a vehicle or dislodgement of components as projectiles. In particular, top rails with rigidity substantially higher than other members of the fence panel should not be permitted unless there is remote likelihood of vehicles colliding with the fence.

There are several types of pedestrian barriers, including:

i. numerous varieties of architectural metal railings
ii. concrete, brick, timber or other fabricated (low) walls as used in many off-road situations
iii. chain-link, weld mesh or other commercially available fencing
iv. bollards and other forms of posts supporting a chain catenary.

Situations where the provision of pedestrian barriers or fencing is common are:

i. Pedestrian bridges, where the provision of a barrier or railing is essential to the safety of users and needs to take account of the special requirements of people with disabilities and, where relevant, of cyclists.

ii. Vehicle (road) bridges, where there is a significant pedestrian demand across the bridge that carries a high volume of traffic at speeds of 80 km/h or greater. Where a risk assessment shows that physical separation of the road traffic and the pedestrians is desirable, then a fence should be placed between the roadway and the pedestrian footpath.
iii. Median fences, which are often proprietary fences erected to prevent pedestrians from crossing dual carriageway roads at mid-block locations. This type of barrier may be added solely for separation of pedestrians, or may be incorporated with vehicle barriers (e.g., guardrail or concrete barriers).
iv. Footpath fences, railings or bollards, located between the footpath and the roadway to prevent pedestrians from crossing at hazardous locations, and to channel pedestrians to crossing locations. Footpath fences are typically constructed of chain-link, weld mesh, post or bollard and chain, concrete or other planters and hedges.

The bollard and chain is sometimes used to provide a delineation between the road traffic and pedestrian activity, particularly around strip shopping facilities. The chain (instead of a rail) treatment addresses the traffic safety concerns of using horizontal rails of timber and metal and is sometimes preferred for aesthetic reasons. However, the bollard and chain barrier has disadvantages in that:

i. it can result in multiple projectiles from a single hit
ii. people with impaired vision and who use a cane and/or a sound reflection device for guidance can be confused by the post and chain arrangement
iii. it is not particularly effective in providing a barrier to restrict pedestrian movement.

Solid barriers which reflect sound and provide a continuous solid edge at pavement level are preferred for guidance of people relying on these aids and are more effective in restricting unsafe pedestrian movements.

However, solid barriers are more likely to obstruct sight lines and are often considered unattractive in the street environment. The provision of tactile tiles and use of high colour contrasts are suitable ways of alleviating some of the concerns that the post and chain barrier poses for people with vision impairments.

Where planters or hedges are used, they should be well maintained, kept at a height which maintains driver sight lines and be trimmed to avoid foliage encroaching on the width of the footpath.

Various types of vehicle barriers are also used at some locations to benefit pedestrians by re-routing or limiting vehicular traffic movements. For example, street closure barriers are sometimes used on a temporary or permanent basis to restrict motor vehicles in neighbourhoods of high pedestrian activity (particularly young children). Traffic diversion barriers are sometimes installed within selected intersections to prevent certain through or turning movements. Examples of pedestrian fences are shown in **Plate 12.40** and **Plate 12.41**.

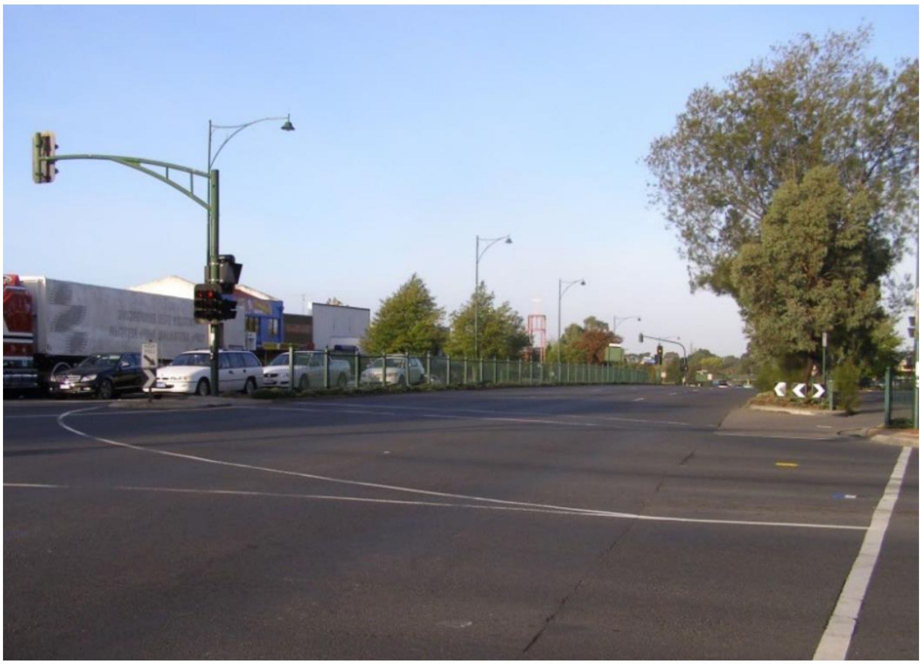


Plate 12.40 Median fence directing pedestrians to controlled crossings

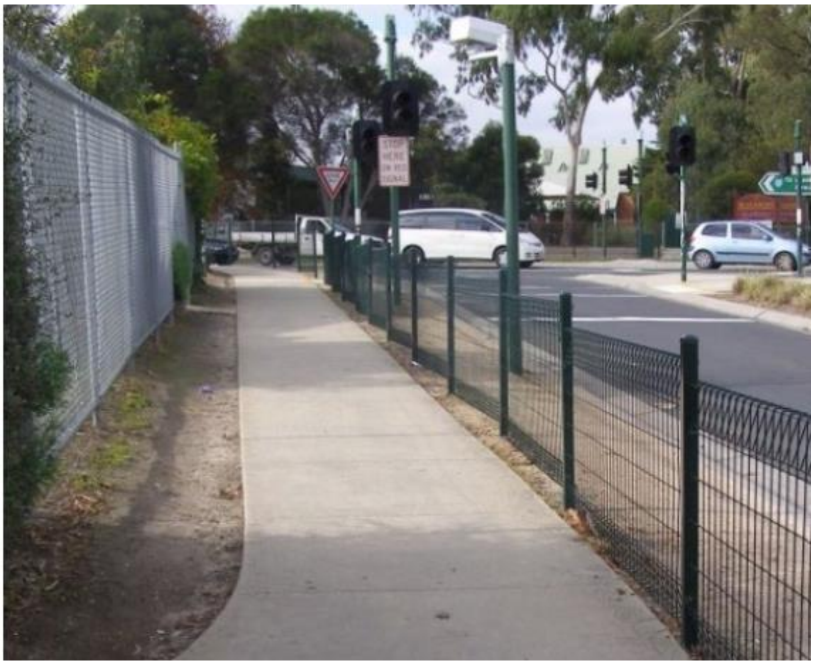


Plate 12.41 Weld mesh fence controlling pedestrians in the vicinity of a right-turn island

12.6.3 FENCES FOR CYCLISTS

The installation of a fence at the side of a path used by cyclists is desirable where:

i. there is a steep batter or large fall located in close proximity to the path

ii. the path is adjacent to an arterial road and it is necessary to separate cyclists from motor vehicles
iii. a bridge or culvert exists on a path
iv. a hazard exists adjacent to a particular bicycle facility
v. cyclists' desire lines of travel may mean that they choose a different path at an intersection, between paths or around a path terminal.

The width of paths and lanes should also account for the presence of fences and reference should be made to Chapter 6 for details of required clearances. The following types of fence should not be used in close proximity to bicycle lanes or paths. They should be located at least 1 m from the edge of bicycle facilities and preferably should be much further away:

i. Treated pine log – these are often constructed with exposed ends and are invariably too low to be used adjacent to bicycle routes.
ii. Chain mesh – these may catch pedals, have exposed elements (e.g. bolts and nuts, loose wire) and in some instances have been responsible for spearing injuries.
iii. Post and wire – these have exposed elements.

Irrespective of the type of fence used, the main requirement is that adequate clearance is provided between the edge of the path and the fence. Examples of cyclist/pedestrian fences on shared paths are shown in **Plate 12.42** and **Plate 12.43**.

It is also very important to provide fences that cater for people who have impaired mobility, particularly on ramps and bridges. **Plate 12.44** shows a pedestrian/cyclist bridge with additional rails to provide for people who use wheelchairs.

Plate 12.42 A high fence on a shared path

Plate 12.43 Full barrier fence on top of a high, steep slope

Plate 12.44 Rails and kerbs to cater for impaired people on a ramp to an overpass

12.7 NOISE BARRIERS

The need to incorporate road traffic noise barriers in a road project is identified in a noise study, usually as part of a wider environmental impact assessment. A typical noise study report will indicate noise level criteria and any noise barriers required to achieve them, in terms of their location, extent and height. The design process and community consultation usually involve consideration of options and review to ensure a suitable final design.

A noise barrier may take the form of a bund (earth mound), a wall or solid fencing, or a combination of these. It may also be combined with a road safety barrier. Examples of attractive noise walls are shown in **Plate 12.45** and **Plate 12.46**.

Plate 12.45 Textured and coloured concrete noise walls on an urban expressway/motorway

Plate 12.46 Example of a noise wall with transparent panels

Careful consideration should be exercised so that the construction and placement of these noise barriers will not increase the severity of crashes that may occur. Every effort should be made to locate noise barriers to allow for sign placement and to provide lateral offsets to obstructions outside the edge of the carriageway. It is recognized, however, that such a setback may sometimes be impractical. In such situations, the largest practical width commensurate with cost-effectiveness considerations should be provided. Stopping sight distance is another important design consideration. Therefore, horizontal clearances should be checked for adequate sight distances.

Construction of a noise barrier should be avoided at a given location if it would limit stopping sight distance below the minimum values shown in **Table 7.10**. This situation could be particularly critical where the location of a noise barrier is along the inside of a curve. Some designs use a concrete safety shape either as an integral part of the noise barrier or as a separate roadside barrier between the edge of the roadway and the noise barrier. On non-tangent alignments, a separate concrete barrier may obstruct sight distance even though the noise barrier does not. In such instances, it may be appropriate to install metal rather than concrete roadside barriers in order to retain adequate sight distance.

Care should be exercised in the location of noise barriers near gore areas. Barriers at these locations should begin or terminate, as the case may be, at least 60 m from the theoretical nose. Other considerations in determining barrier locations include development and assessment of alternative designs, ease and cost of maintenance, and aesthetics.

Potential noise problems should be identified early in the design process. Line, grade, earthwork balance, and right-of-way should all be worked out with noise in mind. Noise attenuation may be inexpensive and practical if built into the design and expensive if not considered until the end of the design process. An effective method of reducing traffic noise from adjacent areas is to design the highway so that some form of solid material blocks the line of sight between the noise source and the receptors. Advantage should be taken of the terrain in forming a natural barrier so that the appearance remains aesthetically pleasing.

A depressed highway section is the most desirable for noise abatement. Depressing the roadway below ground level has the same general effect as erecting barriers (i.e., a shadow zone is created where noise levels are reduced [see **Figure 12.25**]). Where a highway is constructed on an embankment, the embankment beyond the shoulders will sometimes block the line of sight to receptors near the highway, thus reducing the potential noise effects (see **Figure 12.26**).

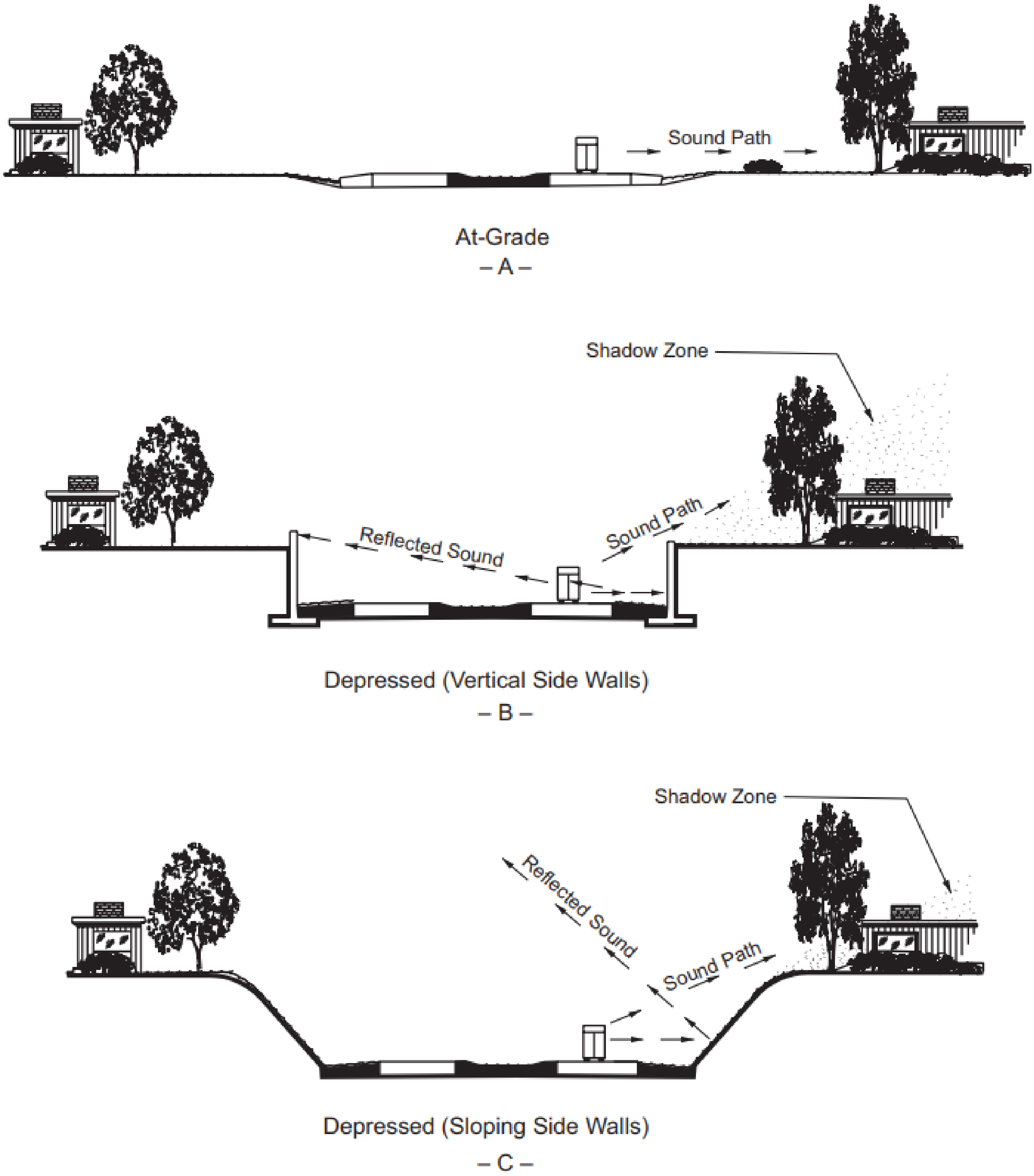


Figure 12.25 Effects of Depressing the Roadway

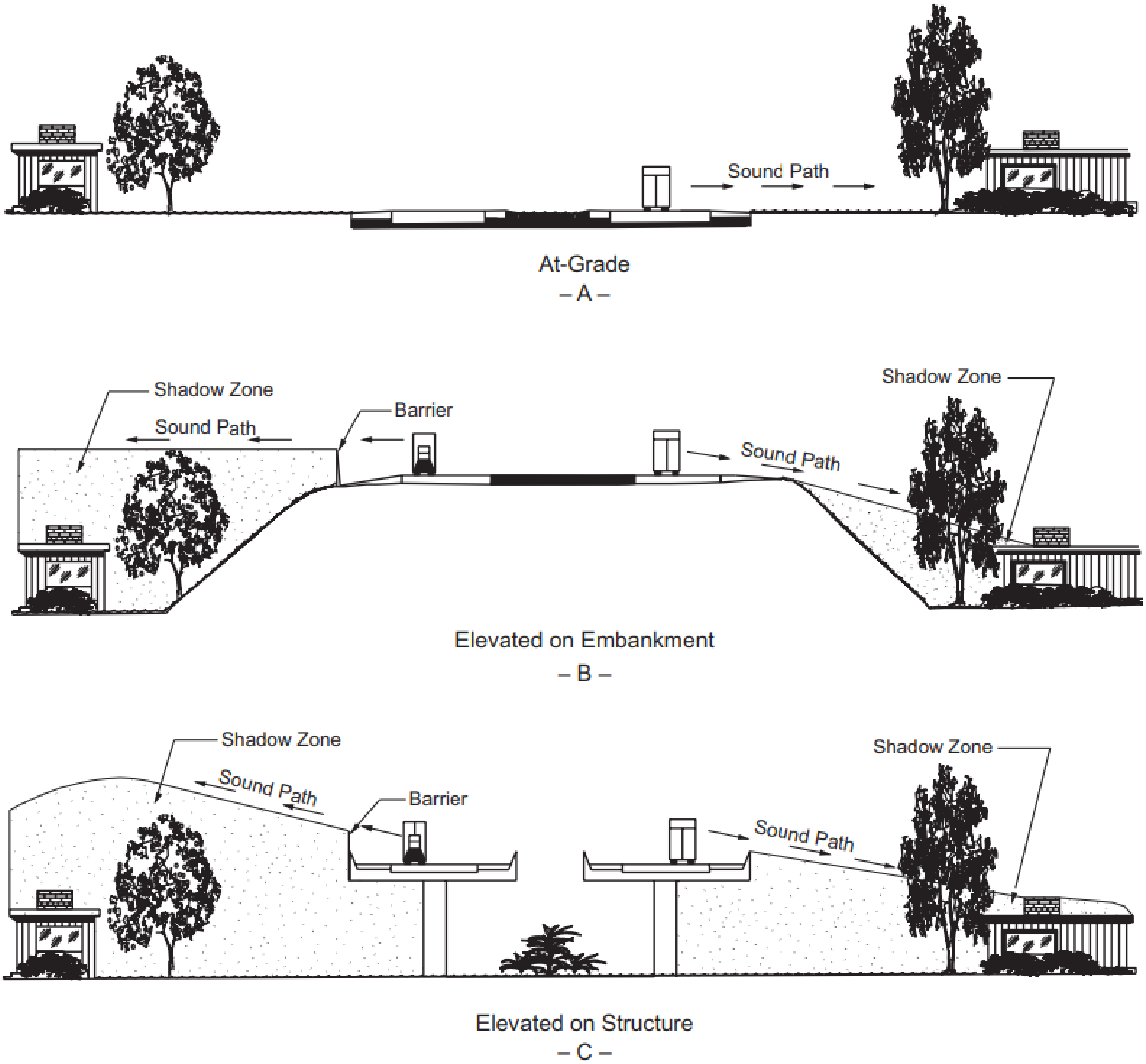


Figure 12.26 Effects of Elevating the Roadway

12.7.1 ACOUSTIC REQUIREMENTS

The minimum acoustic requirements for noise barriers are that they should:

i. have a minimum superficial mass of 15 kg/m^2
ii. be free of cracks and gaps for their design lives
iii. be located and of sufficient height and length to achieve the noise-level criteria determined in the noise study and required by local policy
iv. be solid with no air gaps.

12.7.2 AESTHETIC REQUIREMENTS

It is desirable that noise walls are designed so that they:

i. are acceptable to the local community and local government authority, in terms of appearance, location and maximum height, and security matters for abutting property owners/occupiers

ii. are aesthetically pleasing and suit the streetscape and local environment
iii. integrate planting or vegetated screening where space permits. Preference should be given to screening the full height of the wall, where suitable, to soften the appearance and to reduce graffiti and vandalism opportunities
iv. have materials and treatments that are contextually appropriate to the landscape setting
v. utilise high-quality themed treatments to create visual interest only where warranted (e.g., at highly visual locations and key focus points such as interchanges and entrances to a specific area), otherwise treatments should be minimal for ease of maintenance
vi. retain views through the design and location of barriers where there are high-quality scenic views from the roadway.

12.7.3 STRUCTURAL AND MATERIAL REQUIREMENTS

Noise walls should be designed and constructed so that they:

i. are structurally sound.
ii. are durable so that they perform satisfactorily with respect to degradation, corrosion and appearance throughout their design life. A wall that is expected to last for 25 years or more without reconstruction will need highly durable materials, especially in areas with aggressive climate conditions such as along the coast.
iii. utilise acceptable materials such as brick, concrete, stone, steel, plywood, and can include fibre-cement, glass, acrylic and polycarbonate sheets where considered appropriate for a specific site. Timber may not be suitable because of durability issues (e.g., warping/cracking may particularly affect acoustic performance), vandalism and fire risk but may be used if suitably treated
iv. have appropriately designed and located footings to facilitate maintenance between the wall and the property boundary or be coincident with it. The footings of any noise wall located on the boundary should desirably be located completely outside the abutting property
v. have a logical termination point satisfying noise objectives as well as fitting with the road design
vi. have consistent colour.

12.8 ROADWAY LIGHTING

The purpose of road lighting is to enable the safe and smooth flow of traffic at night time. In cases where it is necessary to secure the safe and smooth flow of traffic at night time, road lighting facilities should be installed. Compared to having no lighting, road lighting imparts outstanding effects in terms of expanding the field of vision, improving visibility and enhancing visual guidance. It is necessary to secure the following visual environment:

i. Road structural elements such as road alignment and road width.
ii. Left and right turn lanes at intersections and branches and positions of installation.
iii. Confirmation of presence of obstructions or pedestrians, etc. on the road.
iv. Position and behaviour of drivers on the road.
v. Position and behaviour of other drivers.
vi. Conditions abutting the roadway.

Drivers need good visibility under day or night conditions to travel along roads at the following section:

i. within built-up areas and their peripheries
ii. road sections with sub-standard alignment
iii. road sections with heavy traffic at night time
iv. road sections with regular pedestrians or slow traffic at night time
v. grade-separated pedestrian or slow vehicle facilities
vi. at major interchanges or intersections with heavy traffic at night time
vii. through tunnels, long bridges and their immediate approaches
viii. toll plazas and immediate approaches to ports or border control points.

Properly designed and maintained street lighting will produce comfortable and accurate visibility at night, which will facilitate and encourage both vehicular and pedestrian traffic. Thus, where adequate illumination is provided, existing roads can be efficiently used at night. Determinations of need for lighting should be coordinated with crime prevention programs and other community needs.

Warrants for the justification of lighting and its associated illumination levels involve the following:

i. functional classification of the roadway
ii. pedestrian and vehicular volume
iii. night-to-day crash ratios
iv. roadway geometry
v. merging lanes
vi. curves, and
vii. intersections.

12.8.1 PURPOSE OF ROADWAY LIGHTING

The basic goal of roadway lighting is to provide patterns and level of horizontal pavement luminance and of horizontal and vertical illuminance of objects. A driver's eye discerns an object on or near the roadway due to contrast between the brightness of the object and the brightness of the background or pavement, or by means of surface detail, glint, or shadows.

12.8.1.1 TRAFFIC ENGINEERING OBJECTIVES

The following are traffic engineering objectives of roadway lighting:

i. Promotion of safety at night by providing quick, accurate, and comfortable visibility for drivers and pedestrians.
ii. Improvement of traffic flow at night by providing light, beyond that provided by vehicle lights, which aids drivers in orienting themselves, delineating roadway geometries and obstructions, and judging opportunities for overtaking.
iii. Illumination in long underpasses and tunnels during the day to permit drivers entering such structures from the daylight to have adequate visibility for safe vehicle operation.

12.8.1.2 OTHER OBJECTIVES

The following are other objectives of roadway lighting:

i. Reduction of street crimes at night.
ii. Enhancement of socio-economic activities at night time.
iii. Beautification of street scape
iv. Not all of these objectives are necessarily achieved by good lighting alone.

12.8.2 VISIBILITY OF OBJECTS

Visibility is the state of being perceived by the eye. The purpose of roadway lighting is to attain a level of visibility which enables the motorist and pedestrian to see quickly, distinctly, and with certainty all significant roadway details, such as the alignment of the road (its direction and its surroundings) and any obstacles that influences visibility such as:

i. Brightness of an object on or near the roadway
ii. General brightness of roadway background – ambient light
iii. Size of object and identifying detail
iv. Contrast between an object and its surroundings
v. Contrast between pavement and its surroundings as seen by the observer
vi. Time available for seeing the object
vii. Glare
    a. Discomfort glare: Ocular discomfort that doesn't affect visual performance.
    b. Disability glare: Reducing ability to see or spot an object.

c. Blinding glare: Glare so intense that for an appreciable length of time no object can be seen.

viii. Driver vision

ix. Condition of windshield

12.8.3 CLASSIFICATION OF ROAD LIGHTING

Road lighting can be classified into three types:

i. continuous lighting (roadway with or without median barrier lighting)

ii. localized lighting

  a. partial interchange lighting

  b. complete interchange lighting

  c. bridge lighting

  d. intersection lighting

  e. roundabout lighting

iii. tunnel lighting

Continuous lighting entails installing lights at set intervals over a road section to illuminate the entire section. This should be adopted in urban areas that have a lot of traffic.

Localized lighting entails illuminating the necessary localized spots and should target intersections, bridges, bends, pedestrian crossings, and places where road elements change.

Tunnel lighting targets inside tunnels and underpasses.

12.8.3.1 CONTINUOUS LIGHTING

Continuous lighting places continuous lighting that encompasses the roadway and area immediately adjacent to the roadway over a substantial distance along the road.

12.8.3.1.1 CONDITIONS FOR CONTINUOUS LIGHTING

To secure a good visual environment in road continuous lighting, it is necessary to consider the following conditions.

i. The average luminance of the road surface should be appropriate. Values in **Table 12.28** should be adopted as the standard for average road surface luminance.

ii. The luminance distribution of the road surface should be even.

iii. The glare imparted to drivers should be amply controlled.

iv. The lighting should have inductivity.

Table 12.28 Standard for average road surface luminance

Design Class	External conditions (cd/m^2)		
	a	b	c
A	1.0	1.0	0.7
B & C	1.0	0.7	0.5
D & E	0.7	0.5	0.5

External conditions: This refers to the extent to which light from building lighting, advertisement lights, neon signs, etc. exerts an impact on road traffic.

a. Condition where there are continuous lights on the roadside that exert an impact on road traffic.
b. Condition where there are intermittent lights on the roadside that exert an impact on road traffic.
c. Condition where there are hardly any lights on the roadside that exert an impact on road traffic.

Concerning inductivity, for example, on road curves, as is shown in **Plate12.47 (b)**, if the installation interval of lighting is too wide and the arrangement is irregular, it becomes difficult to predict the road alignment. Such cases should be avoided. When the installation interval of lighting is appropriate as shown in **Plate12.47 (a)**, it becomes possible to correctly predict the road alignment.

(a) Good example

(b) Bad example

Plate12.47 Inductivity of Road Lighting

12.8.3.1.2 ROADWAYS WITH MEDIAN BARRIER LIGHTING

The median barrier twin mast arm lighting units have certain advantages such as providing the same number of luminaires with fewer poles, utilizing back light from luminaires, and are less likely to be knocked down.

The disadvantages of median lighting are that traffic control is required when working on median lights and the potential danger to employees working on the median lights. In high volume urban areas, it is very difficult to maintain barrier lighting and, if possible, luminaires should be placed on the outside edge of the roadway (side-mounted).

12.8.3.2 LOCALIZED LIGHTING

Localized lighting is installed in places where the traffic flow becomes locally complicated or places where the road's horizontal alignment or vertical alignment becomes complicated, with the objective of clarifying traffic conditions, road conditions, etc. Accordingly, the arrangement of lighting fixtures and lighting units should be appropriately selected upon fully considering the purpose of installation in each case. Lamps should be LED lamps that use light-emitting diodes as standard, however, in unavoidable cases where safe night-time road traffic cannot be secured, use of conventional High-intensity discharge (HID) lamps (mercury lamps, metal halide lamps, natrium lamps) should be considered.

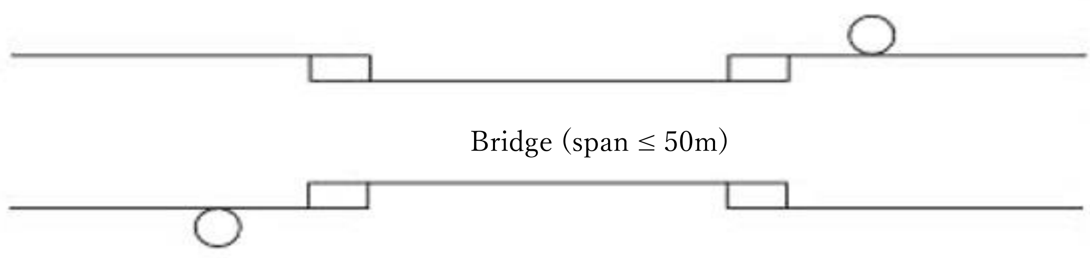
Localized lighting should be installed as standard in the following places and according to the following objectives.

i. **Intersections**: In addition to the general effects of continuous lighting, localized lighting are intended to indicate the intersection to approaching drivers and give them awareness of conditions inside and around the intersection.

ii. **Pedestrian crossing**: Localized lighting are intended to indicate the pedestrian crossing to approaching drivers and inform them of pedestrians, etc. who are crossing or waiting to cross.

iii. **Walkway/cycle lane**: Localized lighting are intended to secure a good visual environment to ensure the safe and smooth movement of pedestrians/cyclist.

iv. **Other localized lighting**: It is desirable to install lighting facilities according to necessity in places where the road width or alignment suddenly changes, bridges, railway crossings, interchanges, toll booths, rest facilities and so on. Especially in cases where there is no continuous lighting on general sections in bridges of 50m or more, as standard, one light should be installed on both ends according to necessity to indicate changes in the width composition as shown in **Figure 12.27**.



(○: Position of lighting unit)

Figure 12.27 Arrangement of Localised Lighting on a Bridge General Section

12.8.3.2.1 PARTIAL INTERCHANGE LIGHTING

Partial expressway/motorway lighting is the illumination of only the parts of the interchange that are most critical to the night driver, which are merge-diverge areas of the ramp connections, interchanges, and other critical roadway features.

12.8.3.2.2 COMPLETE INTERCHANGE LIGHTING

Complete interchange lighting is applying lighting to the interchange to achieve illumination of all roadways in the interchange.

12.8.3.2.3 BRIDGE LIGHTING

The roadway on a bridge is normally treated the same as other parts of the roadway. If there is no lighting on the adjacent roadway, there is normally no need for lighting on the bridge. An exception is a very long bridge, which may be lit even though the roadway is not lit at other locations.

Where lights are to be installed on a bridge, the desirable locations for the lighting units are at abutments and at pier locations, or at a distance from an abutment or pier not to exceed 25 percent of the length of the span. This placement of the lighting units reduces the effects of vibration. The light poles should utilize davit type mast arms and shorter mast arm lengths so that there are no joints to be weakened by vibration.

12.8.3.2.4 INTERSECTION LIGHTING

Lighting at intersections is usually justified and will alert the driver to an approaching intersection. Notes regarding intersection lighting are as follows:

i. Luminaires should be placed on or near prominent conflict points.
ii. Lighting should be provided at all signalized and flashing beacon intersections.
iii. A signal pole shaft extension with a luminaire mast arm should be utilized whenever possible to avoid adding more poles at the intersection.
iv. Streetlights on traffic signal poles should be fed from the traffic signal service point.
v. The level of illumination of a signalized intersection is dictated by the area classification (commercial, residential) of the roadway.
vi. Additional light poles may be necessary when the intersection has channelization or complex turning lanes.

12.8.3.2.5 ROUNDABOUT LIGHTING

The need for illumination varies somewhat based on the location in which the roundabout is located (urban, suburban, or rural conditions). Generally, roundabouts should always be lit. The following features are recommended:

i. Good illumination should be provided on the approach nose of the splitter islands, at all conflict areas where traffic is entering the circulation stream, and at all places where the traffic streams separate to exit the roundabout.

ii. It is preferable to light the roundabout from the outside in towards the centre. This improves the visibility of the central island and the visibility of circulating vehicles to vehicles approaching the roundabout.

12.8.3.3 TUNNEL /UNDERPASS LIGHTING

Where underpass lighting is desirable, the lights are mounted on the abutment of the bridge or on a pier for each direction of travel on the roadway. If such mounting would lower light levels to a non-acceptable level, then the luminaire is typically mounted on the bottom of the diaphragm.

Generally, for continuously lit expressway/motorway, underpass lighting should be installed for structures greater than 15m in length. For underpasses that are longer than 60m, underpasses should be lit all day.

12.8.3.3.1 VISIBILITY OPTIMIZATION OF THE TUNNEL AND APPROACH FEATURES

It is important in the physical design of a tunnel structure that due consideration be given to lighting. The physical features of a tunnel can have a significant effect on reducing the day lighting needs.

The following items contribute to improved tunnel visibility and should be explored in the development of daytime tunnel lighting designs.

i. Reduction of Ambient Daytime Brightness

Tunnel portals, adjacent walls, approach pavement, and other external features in the motorist's field of view should be darkened to the extent possible. Admixtures, overlays, vegetation, or other methods that result in low reflectance, non-specular surfaces are recommended. Dark features increase the degree of advance eye adaptation of the entering motorist and improves contrast with the lower luminance levels in the tunnel interior. Tunnels with a predominant sky background above the entrance should be reviewed for the use of plantings, screens, or panels that increase the size of the darkened area above the portals.

ii. Portal Design Factors

Upsweep ceilings may increase daylight penetration, but can result in increased tunnel structure costs. Sunscreens have not been effective. Dirt accumulation, permanent depreciation of reflective and light-transmitting properties have posed serious problems. The high initial costs of sunscreens coupled with high maintenance costs have practically eliminated their use.

iii. Visibility Optimization of Tunnel Interiors

It is recommended that ceiling and wall surfaces be of an easily maintained finish with a non-specular reflective efficiency of at least 70 percent. High wall brightness is of great value in meeting visibility needs in tunnels that have curved roadways or approach roadways. Relatively narrow tunnels where the width-to-height ratios are approximately 3 or less will develop inter-reflectivity that can enhance tunnel visibility as a result of the reflected light from the walls.

Natural sunlight penetration in entrance portal areas can be improved by the use of wall, ceiling, and roadway surface texture control. The use of vertical wall corrugations, coarse finished pavements, or other treatments which produce surface relief, increase the retro-reflection of light entering the portal over that of smooth surfaces.

iv. Types of Pavement Surfaces

The use of bituminous concrete on the approach road surface to the tunnel portal and portland cement concrete on the road surface inside the portal for a distance at least equal to the safe sight stopping distance will reduce the luminance contrast between the outside and the inside of the tunnel. This will in turn reduce entrance zone luminance and illuminance requirements. Future resurfacing should account for the designed roadway surface.

12.8.3.3.2 DAYTIME LIGHTING OF TUNNEL INTERIORS

A. Short Tunnels-Silhouette Visibility

Short vehicular tunnels that have relatively straight and level approach alignments with corresponding straight and level tunnel roadways may offer adequate visibility to the entering motorist by silhouette viewing of other vehicles and objects on the roadway against the far side exit portal. These tunnels are treated as underpasses in the guide. Silhouette visibility should be carefully evaluated with respect to the tunnel geometry to provide visibility of objects within the tunnel. The roadway surface details will normally be indiscernible to the motorist with silhouette visibility.

In multi-lane one-way tunnels, or unseparated two-way tunnels, lighting should be provided to the extent the motorist can distinguish lane markings or other delineation important to safe travel through the tunnel.

B. Entrance Portal Lighting

The most critical portion of a tunnel that affects visibility is at the portal. This is commonly called the “black hole” effect. Visibility of this first entrance zone, while still outside the tunnel, is essential to the motorist in identifying and safely reacting to the presence of vehicles and objects that may be present on the tunnel roadways. This is accomplished by lighting the entrance zone in proper proportion to the outside ambient luminance to which the motorists' eyes are adapted.

The luminance of the approach pavement, adjacent landscape, sky, and the portal area itself, are all integrated over time by the motorist's eyes in adapting to overall ambient conditions. It is suggested that an evaluation of brightness conditions be made for the actual roadway and tunnel prior to establishing a lighting design.

A model simulation may be necessary for new facilities in order to duplicate the anticipated tunnel approach conditions. The motorist's field of view of adapted luminance should be evaluated at a location along the approach roadway equal to the minimum stopping sight distance in advance of the portal.

Two and three lane one-way tunnels having favourable alignments of the approaches and tunnel structure, and which are of relatively short length, have been adequately lighted with relatively low artificial lighting levels. The optimization of portal entrance conditions, in some cases, has produced adequate entrance visibility at artificial luminance levels in the range of about 100 to 200 candelas per square meter reflected from an in-service roadway surface.

Entrance zone lighting levels should be designed to accommodate the greatest ambient luminance expected at the location. The stopping sight distance defined previously determines the length of entrance zone lighting. Most tunnel approach roadways, except for extreme cases of vertical and horizontal curvature, have entrance characteristics such that a point relatively close to the tunnel portal will confine the motorist's view to the predominance of the darkened tunnel structure. It is an acceptable practice to include this "fixation" distance in the minimum stopping sight distance to reduce the length of the entrance interior lighting. Pre-adaption should not normally be used to reduce portal lighting levels.

C. Lighting Beyond the Entrance Zone

If the tunnel is classified as a short tunnel, the entrance zone lighting level applies throughout its entire length. However, in long tunnels, lighting beyond the minimum stopping sight distance should be reduced progressively until an established minimum level is reached. It is recommended that beginning at the end of the entrance zone lighting the levels be reduced in steps to a level not less than 5 horizontal footcandIes (54 lux) or 5 candelas per square meter on the roadways. Each stepped zone should have a length at least equal to the minimum stopping sight distance.

Typical lighting profiles are illustrated in **Figure 12.28** and **Figure 12.29**.

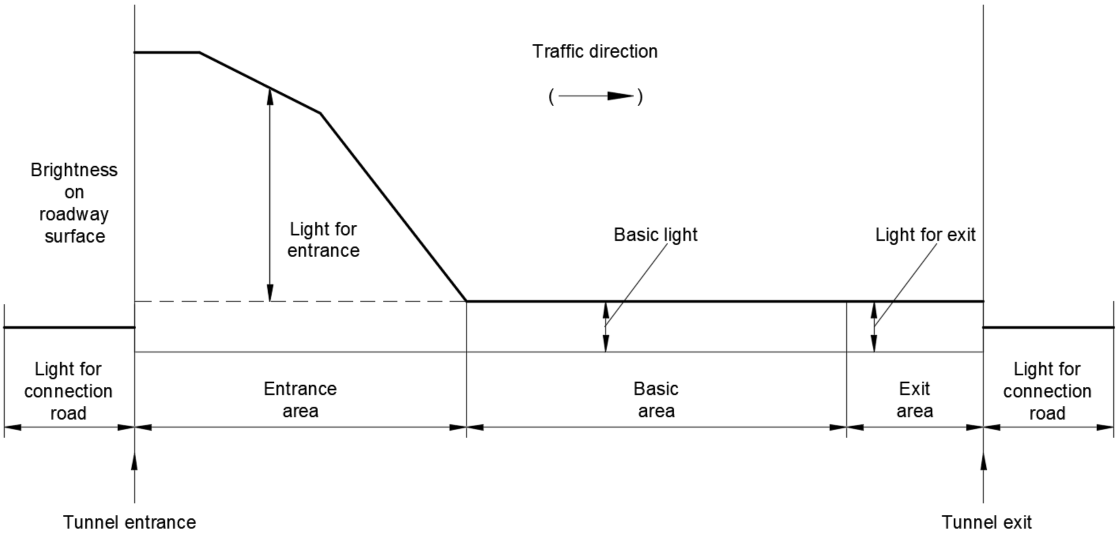


Figure 12.28 Lighting profile for unidirectional tunnel tube

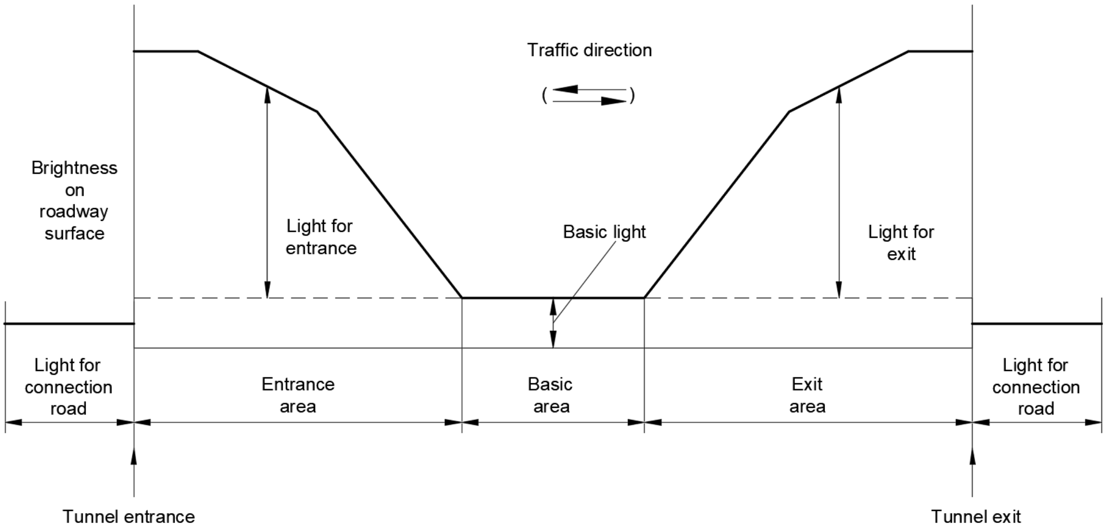


Figure 12.29 Lighting profile for bidirectional tunnel tube

D. Nighttime Tunnel Lighting

Nighttime lighting should, if practical, make use of a portion of the daytime lighting system, rather than be a separate system. Nighttime levels in a tunnel should be somewhat higher, but not exceeding three times that of the lighting requirements for the roadways adjacent to the tunnel. Uniformity of lighting should closely match that of the requirements for the adjacent roadways.

Tunnels located on non-continuously lighted roadways should be lighted to the minimum standards required for the highway type and character as contained in this guide.

E. Selection and Placement of Tunnel Luminaires

The choice of particular types of tunnel luminaires and light sources should be made by considering such items as luminous efficacy, source glare, light distribution characteristics, physical placement limitations, frequency of maintenance, and resistance to damage. An important consideration in the choice of a particular system for both daytime and nighttime lighting in long tunnels is the stroboscopic effect of alternate bright-dark areas where luminaires do not provide a continuous line of luminance. Frequencies in the range of 5 to 10 cycles per second have been observed to result in eye annoyance and should be avoided at the particular design speed of the tunnel.

F. Tunnel Lighting Control Systems

Lighting levels for the entrance zone may be adjusted to match the ambient conditions created due to varying light levels from season to season and during cloudy or inclement weather. If such system variances are determined to be economical and feasible, lighting levels in subsequent tunnel zones should vary in the same proportion. Lighting systems for tunnels should be designed as fail safe as practical to reduce the possibility of a total tunnel outage in the event of a circuit failure or other malfunction.

G. Maintenance Factor for Tunnel Lighting Design

The reduction of initial lighting levels becomes an important factor in tunnel lighting design. Initial design levels should consider the frequency and degree of maintenance that is to be performed. Factors in the range of 50 percent are commonly applied to tunnel lighting designs.

12.8.3.4 HIGH-MAST LIGHTING

High-mast lighting is comprised of groups of luminaires mounted on free-standing poles at mounting heights that can vary from approximately 18 - 55 m. High mast poles are usually provided with luminaire lowering devices that lower the luminaires for maintenance.

High-mast lighting is used principally at interchanges, toll plazas, rest areas, parking areas, and for continuous lighting on highways that have wide cross sections.

Recommended lighting levels are on **Table 12.29** to **Table 12.33**. Higher lighting levels than indicated in **Table 12.29** to **Table 12.33** may be required after consideration of such factors as the complexity of the interchange, the existence of high brightness from competing light sources near the roadway, and the prevailing level of lighting on connecting roadways.

The benefits of high-mast lighting include excellent uniformity, lower glare, and fewer pole locations when compared to conventional lighting systems. Traffic control requirements are reduced for maintenance because the poles can be located out of the clear zone or recovery area and away from the roadway.

Surrounding off road areas receive incidental illumination that provides the motorist with a panoramic view, compared to the "tunnel of light" effect provided by conventional systems. Another benefit is the visibility of vertical surfaces of the roadway system such as guardrail, bridge columns, abutments and drainage headwalls. High-mast lighting systems perform well under adverse weather conditions such as rain and fog.

The most common type of luminaire used in high-mast lighting is the area type which is usually offered with symmetric or asymmetric distribution. Both types of distribution are frequently used. Cut-off style luminaires are recommended.

Scheduled inspections of the pole and lowering system may detect corrosion, fatigue, or other problems.

Structural supports for high mast lighting systems should be placed outside the clear zone or they should be protected with proper guide rail or other deflecting barrier. High mast lighting supports are considered fixed-base support systems that do not yield or break away on impact. The large mass of these support systems and the potential safety consequences of the systems when they fall to the ground necessitate a fixed-based design.

12.8.4 ROAD LIGHTING DESIGN CRITERIA

The design of road lighting should be based on the reflection characteristics of the road surface in order to obtain the optimum quality and quantity of illumination. Road lighting should be designed in conjunction with bridges, noise barriers, tree plantings, overhead signs and signals and any other overhead utility to minimise shadow patches. Safe working clearances should be maintained between lighting and overhead power lines.

Lighting design is concerned with the selection and location of lighting equipment so as to provide improved visibility and increased safety while making the most efficient use of energy within minimum expenditure.

There are two basic concepts of lighting design, i.e., the illumination concept and the luminance concept.

The illumination concept is based on the premise that, by providing a given level of illumination and uniformity of distribution, satisfactory visibility which is related to the luminance of the pavement and the objects on the pavement can be achieved.

The appropriate standard of lighting for each Road Class is determined by the visibility requirements of the road users. The visibility requirements of vehicle drivers are far more demanding than those of other road users. Consequently, the standard of road lighting provided is directed towards satisfying the requirements of vehicle drivers. When these are met the requirements of other road users such as pedestrians will be satisfied. The vehicle driver must be able to see and recognize the following in a limited space of time as his vehicle moves forward:

i. The carriageway, kerbs, directional signs, and the general direction his route is taking him.
ii. The road surrounds, buildings, route signs etc., which help him/her to recognize the location through which he/she is passing.
iii. Possible hazards of a fixed or static nature such as intersections, pedestrian crossings, roadworks, potholes, etc. These must be seen and recognized in time for him to take the necessary action.
iv. Moving hazards such as pedestrians, and other vehicles moving unexpectedly near the proposed path of the driver.

The speed and accuracy with which the vehicle driver sees and recognizes the above is directly related to:

i. The luminance (brightness) of the scene in front of him/her
ii. The contrast in brightness in the scene
iii. The presence of glare sources such as car headlights, roadside floodlights, badly aligned or designed streetlight luminaries.
iv. The time available to see and recognize details of the scene in front of him/her.

12.8.5 STREET LIGHTING STANDARDS

The provision of Street lighting in Ghana shall be based on the principles and techniques which are applied in normal international practice. The standards of lighting to be applied for the various road types shall conform generally with standards in other countries such as British Standard European Norm (BSEN), Illuminating Engineering Society (IES) of North America or the Commission Internationale de L'Eclairage (CIE), but shall take account of special circumstances which obtain or may arise in Ghana.

All street lighting and electrical systems must comply with the following general legislation and more specific street lighting industry standards:

i. Ghana Highway Authority Act, 1997
ii. Ministry of Transportation Standard Specification for Roads & Bridge Works, July 2007, Section 27
iii. ECG Street lighting In Ghana Standards and Policy
iv. ECG Technical Requirements for Street lighting Installations
v. Traffic Signs Regulations and General Directions

vi. BS 7671: Regulations for Electrical Installations 1992

vii. BS 5489: Parts 1 – 10," Code of Practice for Road Lighting"

viii. BS EN 60529: Specification for Clarification of Degrees of Protection provided by Enclosures

ix. BS EN 605589-2-3: 1994 Luminaires for Road and Street Lighting

x. BS 5649: Lighting Columns

xi. BS EN 40: Lighting columns1992

xii. Standard BD26/94 - Design of Lighting Columns

12.8.6 IMPLEMENTATION OF DESIGN STANDARDS

Table 12.29 to **Table 12.33** indicates the main design features of straight road installations which would satisfy the quality criteria for each of the adopted road classifications in Ghana. The main assumptions made are:

i. Road Reflectance characteristics are for C11 described in CIE Publication No. 66. This is the type of smooth surface texture which asphalt achieves after approx. 12 months constant use.

ii. The surface is reasonably free from pronounced humps or dips.

iii. The performance of luminaires complies with the light distribution and glare control characteristics required by BS. 4533:103:1:1981.

The special treatment required for special situations such as bends, junctions, roundabouts, sloping roadways, pedestrian crossings, near harbours and airports will usually result in closer spacings, (hence more luminaires) than the straight road installations.

Table 12.29 Road type: Design Class A

Carriageway Details			Design Guidelines for Straight Roads			
Dual or single	**Median Width (m)**	**Carriageway Width (m)**	**Arrangement**	**Mounting height (m)**	**Maximum spacing (m)**	**Lamp type (lumen output)**
Dual	≥ 6	≥ 12	Two at opposite sides of the median	12	48-55	400W (46,000 to 50,000 lumens)
		< 12		10	40-45	
Two or three lanes with hard shoulder	< 6	≥ 12	Center of the median	12	48-55	
		< 12		10	40-45	

Table 12.30 Road type: Design Class B & C

Carriageway details			Design guidelines for straight roads			
Dual or single	**Median width (m)**	**Carriageway width (m)**	**Arrangement**	**Mounting height (m)**	**Maximum spacing (m)**	**Lamp type (lumen output)**
Dual	≥ 2	≥ 12	In central median	10	38-42	250W (26,000 to 28,000 Lumens)
	< 2	< 12			40-45	
Single	No median	≥ 12	At both sides staggered		38-42	
		10-12			40-45	
		< 10	Single side			

Table 12.31 Road type: Design Class D

Carriageway details			Design guidelines for straight roads			
Dual or single	**Median width (m)**	**Carriageway width (m)**	**Arrangement**	**Mounting height (m)**	**Maximum spacing (m)**	**Lamp type (Lumen output)**
Single	No median	> 12	Both sides staggered	10	38-42	150W (15,000 to 16,000 lumens)
		10 - 12				
		< 10	Single side	8	33-36	

Table 12.32 Road type: Design Class E – Local roads

Carriageway details			Design guidelines for straight roads			
Dual or single	**Median width (m)**	**Carriageway width (m)**	**Arrangement**	**Mounting height (m)**	**Maximum spacing (m)**	**Lamp type (Lumen output)**
Single	No median	> 10	Both sides staggered	8 or 10	32-36	70W (5,500 to 6,000 lumens)
		8 - 10	Single side	8	32-36	
		< 8		6	38-36	

Table 12.33 Road type: Unpaved Roads (Minor towns and villages)

Layout	Position light at corners and junction to provide guidance fill in between where spacing is more than 65m
Mounting height (m)	4-5 - fixed to network columns under conductors 5 - on special wood poles
Spacing (m)	maximum – 65 minimum – 33
Light source	50W (3000 to 3,500 lumens) or 50W (1,900 to 1,200 lumens)
Supply	Connect direct to overheading cables or services fit photos witch on each luminaire

12.8.7 LIGHTING QUALITY CRITERIA

The objective of the road lighting installations shall be to provide adequate visibility conditions for the safe, quick and comfortable movement of all road users. To meet this objective, installations shall be designed to satisfy the quality criteria set out in **Table 12.34**.

Table 12.34 Quality criteria for road lighting in Ghana

Road class	**Average road luminance level (L) cd/m^2**	**Minimum uniformity ratios**		**Glare restrictions**
		Overall uniformity Ratio U_0	**Longitudinal Uniformity Ratio U_1**	**Threshold increment (%)**
A	2	0.4	0.7	10
B	1.5	0.4	0.7	20
C	1	0.4	0.5	20
D	0.5	0.4	0.4	20
E	The use of this criteria is not applicable			

L: Average luminance over a defined area of the road surface viewed from a specified observer position.

U_0: Ratio of the minimum to average luminance of a defined area of the roadway.

U_1: Ratio of the minimum to the maximum luminance along a longitudinal line drawn through the observer position.

12.9 ANTI-GLARE SYSTEMS

Anti-glare systems are facilities to limit the glare of headlights from opposing vehicles or other external light sources. They can be provided in the following forms:

i. Manufactured net using expanded metal
ii. Manufactured plate typically in synthetic resin
iii. Hedges

iv. Earth embankments
v. Raised concrete safety barriers.

Anti-glare systems should be considered on Class A & B roads without road lighting in the following situations:

i. Median less than 9m in width
ii. Heavy night time traffic
iii. Low standard horizontal curves and sag curves
iv. Opposing traffic at a different level smaller than 2m
v. Existence of a parallel road with opposing traffic
vi. Near tunnel portals where tunnel tubes are at close proximity
vii. Glare from other light sources

At intersections, median openings and pedestrian crossings, anti-glare systems have to be terminated well in advance to satisfy visibility requirements.

12.9.1 GENERAL REQUIREMENTS

Anti-glare systems are designed to block visibility between drivers and the front headlights of opposing vehicles. They are based on the eye-height of drivers and the height of front lights in **Table 12.35**.

Table 12.35 Design parameters for anti-glare facilities

Design Vehicle	Observer eye-height (m)	Height of headlight (m)
L-9, L-12, T-17 & T-21	2.0*	1.05
S-5 & M-6	1.05	0.6

*Up to 2.45m depending on vehicle type.

It is generally sufficient to provide screening from opposing headlights between zero and 10 degrees. This is illustrated in **Figure 12.30**.

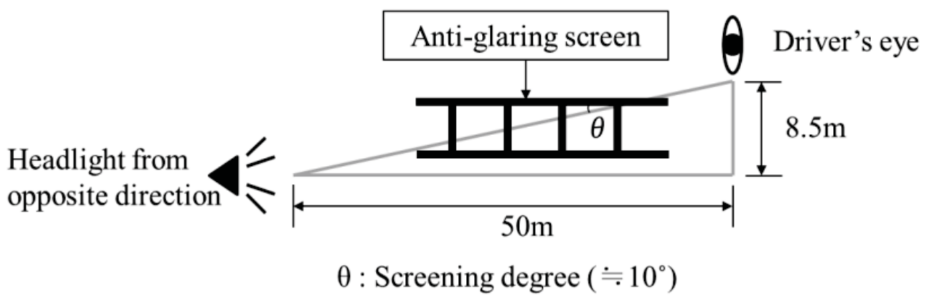


Figure 12.30 Screening angle for Anti-Glare System

The height of anti-glare systems is generally in the order of 1.4m above the pavement. The width of anti-glare screen is in the range of 80 to 250 mm at a spacing of 0.5 to 1.0 m. For hedges, the spacing of trees should be based on the effective width of tree crowns. It is desirable to alternate between screens or nets and hedges.

Anti-glare screens should be able to shield opposing traffic headlamps at an angle not less than 8 degrees. On horizontal and vertical curves, the minimum angle should be increased to the range of 8 to 15 degrees.

Anti-glare facilities on curves should not result in unacceptable forward visibility required for the prevailing operating speeds of traffic. Otherwise, the median has to be adequately widened.

Anti-glare screens or nets should not be made of reflective materials. They may be attached onto safety barriers without compromising their normal safety function. Typical designs are illustrated in **Plate 12.48** and **Plate 12.49**.

Plate 12.48 Safety barrier mounted Anti-Glare Screens

Plate 12.49 Safety Barrier mounted Anti-Glare Nets

The use of hedges should take into account the adequacy of space for planting and maintenance. Preference should be given to species which are strong, slow growing, have few falling leaves and require little maintenance. An example is shown in **Plate 12.50**.

Plate 12.50 Hedges as Anti-Glare System

Raised concrete median safety barriers may be considered if screens or nets are susceptible to damage and additional containment level is also desirable. This is illustrated in **Plate 12.51**.

Plate 12.51 127mm high concrete Safety barrier as Anti-Glare System

12.10 SIGN, SIGNAL, LUMINAIRE AND OTHER SUPPORTS

When a driver loses control of the vehicle to the point that it departs from the travel lane or the roadway, much of the ability to regain full control of the vehicle is lost. Any object in or near the path of the vehicle becomes a potential contributing factor to crash severity. The concept of a forgiving roadside should not be independently applied to each design element but rather as a comprehensive approach to roadway design.

Although a traversable and unobstructed roadside is highly desirable from a safety standpoint, some appurtenances simply should be placed near the travelled way. Man-made fixed objects that frequently occupy highway rights-of-way include highway signs, roadway lighting, traffic signals, railroad warning devices, intelligent transportation systems (ITS) and utility poles.

The highway designer is charged with providing the safest facility practicable within given constraints. There are six options for mitigation of objects within the design clear zone:

i. Remove the obstacle.
ii. Redesign the obstacle so it can be traversed safely.
iii. Relocate the obstacle to a point where it is less likely to be struck.
iv. Reduce impact severity by using an appropriate breakaway device.
v. Shield the obstacle with a longitudinal traffic barrier designed for redirection or use a crash cushion or both if it cannot be eliminated, relocated, or redesigned.
vi. Delineate the obstacle if the above alternatives are not appropriate.

While the first two options are the preferred choices, these solutions are not always practical, especially for highway signing and lighting, which should remain near the roadway to serve their intended functions. This section deals primarily with the fourth option: the use of breakaway hardware, which has become a cornerstone of the forgiving roadside concept since its inception in the mid-1960s.

12.10.1 BREAKAWAY SUPPORTS

The term breakaway support refers to all types of signs, luminaire, and traffic signal supports that are designed to yield, fracture, or separate when impacted by a vehicle. The release mechanism may be a slip plane, plastic hinge, fracture element, or a combination of them.

A breakaway support is designed for loading in shear and normally for impact at bumper height (typically 500 mm above ground level). It is critical that the support be properly installed as to ensure that loading takes place at the correct height. Loading above the design height may cause the breakaway device to fail to activate because the bending moment in the breakaway support may be sufficient to keep the support in place. Incorrect loading can also take place when the support is installed close to ditches or steep slopes, causing a vehicle to become airborne and hit the support at the wrong position.

The soil type of the breakaway system is important as it may also affect the activation of the mechanism. In the case of fracture-type supports such as high carbon U-channel posts, telescoping tubes and wood supports, the supports may slip through saturated or loose soil during impact, absorbing energy and changing the breakaway mechanism.

Plate 12.52 to **Plate 12.55** show the various types of breakaway support.

Plate 12.52 Multidirectional coupler

Plate 12.53 Typical unidirectional slip base

Plate 12.54 Slotted fuse plate design

Plate 12.55 Perforated fuse plate design

If a sign support is installed at a depth less than 1m, it will pull out of the soil during impact.

Installations with anchor plates or those installed deeper than 1m are particularly sensitive to the foundation conditions. For small sign supports using base-bending or yielding mechanisms, the performance of the supports in strong soils is more critical.

The maintenance requirements are critical in the selection of a particular breakaway device. The following maintenance requirements should be considered:

i. The availability of breakaway devices will influence the costs associated with installation and maintenance or replacement after impact. An installation that can be reused can be more cost-effective than mechanisms that have to be replaced.
ii. The durability of a support is important as it will determine the life span of a support that is not struck as compared to that of a non-breakaway support.

A breakaway device yields when hit if it is properly installed and maintained. The mechanism should then be replaced or repaired. Consequently, the availability of material, maintenance personnel and availability of personnel after an impact for each breakaway design should influence the selection thereof.

12.10.1.1 ACCEPTANCE CRITERIA FOR BREAKAWAY SUPPORTS

The broad criteria which breakaway supports should meet include:

i. Dynamic performance criteria, i.e., implicit velocity breakaway thresholds.
ii. Maximum remaining stub height of 100mm.
iii. The need for the vehicle to remain upright during and after the collision.
iv. No significant deformation of the vehicle or intrusion into the passenger compartment during or after impact.

12.10.2 NON-BREAKAWAY SIGN SUPPORTS

The first requirement for sign supports is the need to structurally support the devices that are mounted upon them. Signs and other devices should be carefully placed in order to minimize the hazard that they can represent to motorists.

The following practices should be borne in mind by designers when developing signing plans for their projects:

i. Sign supports should not be placed in drainage ditches, where erosion might affect the proper operation of breakaway supports.
ii. Wherever possible, signs should be placed behind existing roadside barriers (beyond the deflection distance), on existing structures, or in non-accessible areas. If this cannot be achieved, then breakaway supports should be used.
iii. Only when the use of breakaway supports is not practicable should a traffic barrier or crash cushion be used to shield sign supports.

12.10.3 DESIGN AND LOCATION CRITERIA FOR SIGN SUPPORTS

Roadway signs fall into three primary classes:

i. overhead signs
ii. large roadside signs and
iii. small roadside signs.

12.10.3.1 OVERHEAD SIGNS

Since overhead signs, including cantilevered signs, require massive support systems that cannot be made breakaway, they should be installed on or relocated to nearby overpasses or other structures, where possible.

All overhead sign supports located within the clear zone should be shielded with a crashworthy barrier. In such instances, the sign gantry should be located beyond the design deflection distance of the barrier.

12.10.3.2 LARGE ROADSIDE SIGNS

Large roadside signs are generally greater than $5.0m^2$ in area. Typically, they have two or more supports that are breakaway. The hinge for breakaway supports on large roadside signs should be at least 2100 mm above ground, so that the likelihood of the sign or upper section of the support penetrating the windshield of an impacting vehicle is minimized.

The required impact performance is shown in **Figure 12.31**. No supplementary signs should be attached below the hinges if their placement is likely to interfere with the breakaway action of the support post or if the supplementary sign is likely to strike the windscreen of an impacting vehicle.

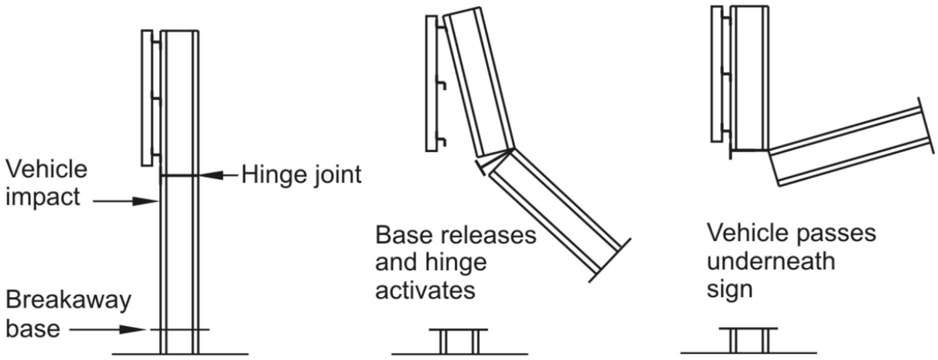


Figure 12.31 Impact Performance of a Multiple-Post Sign Support

12.10.3.3 SMALL ROADSIDE SIGNS

Small roadside signs are supported on one or more posts and have a sign panel area of less than 5.0m^2. Although not perceived as significant obstacles, small signs can cause serious damage to impacting vehicles, and wooden posts should be used as far as possible.

The bottom of the sign panel should be a minimum 2100mm above ground and the top of the panel should be a minimum 2700mm above ground to minimize the possibility of the sign panel and post rotating on impact and striking the windshield of a vehicle.

Consideration to various factors should be given when selecting, designing and locating breakaway and other supports. These include:

i. Road environment: urban or rural.
ii. Terrain where device is installed.
iii. Proximity to drainage ditches or structures.
iv. Soil type used as a base for the breakaway support.
v. Maintenance requirements of the support (i.e., the simplicity of maintenance, availability of material and the durability of the support)
vi. Expected impact frequency.

Unidirectional, multi-directional and Oregon 3-bolt ship base for small signs are shown in **Plate 12.56, Plate 12.57** and **Plate 12.58** respectively.

Plate 12.56 Unidirectional Slip Base for Small Signs

Plate 12.57 Multidirectional Slip Base for Small Signs

Plate 12.58 Oregon 3 -Bolt slip base

12.10.4 LUMINAIRE SUPPORTS

12.10.4.1 BREAKAWAY LUMINAIRE SUPPORTS

Lighting supports should be of the frangible base, slip base or frangible coupling type. They are designed to release in shear when hit at a typical bumper height of about 500 mm. As long as the side slopes between the roadway and the luminaire support are 6:1 or flatter, vehicles should strike the support appropriately, and breakaway action can be assured.

Superelevation, side slope, rounding and vehicle departure angle and speed will influence the striking height of a typical bumper. Designers should consider this fact when developing illumination plans for their projects.

As a general rule, a lighting support will fall near the line of the path of an impacting vehicle. Designers should be aware that these falling poles represent a threat to bystanders such as pedestrians, bicyclists and uninvolved motorists. Poles with breakaway features should not exceed 18.5m in height - the current maximum height of accepted hardware. The mass of a breakaway lighting support should not exceed 450kg.

Foundations for lighting supports should be designed with consideration being given to the surrounding soil conditions that could influence the effectiveness of the breakaway mechanism.

When a lighting support is located near a traffic barrier and if it is within the design deflection distance of the barrier, it should be either a breakaway design, or the barrier should be strengthened locally to minimize its deflection.

Examples of each type in common use are shown in **Plate 12.59** to **Plate 12.61**.

Plate 12.59. Example of a Cast Aluminium Frangible Luminaire

Plate 12.60. Example of a Luminaire Slip Base Design

Plate 12.61 Example of a Frangible Coupling Design

A final consideration on roadway lighting is a reduction in the total number of luminaires used along a section of highway. Higher mounting heights may significantly reduce the total number of supports needed. The ultimate design in this respect is the use of tower or high-mast lighting that requires far fewer supports located much farther from the roadway. From a roadside safety perspective, this is a preferred method for lighting major interchanges.

12.10.4.2 HIGH-LEVEL LIGHTING SUPPORTS

High-level lighting supports also are fixed-base support systems that do not yield or break away on impact. The length and large mass of these support systems and the potential safety consequences of the systems falling necessitate a fixed-base design that cannot be made breakaway. Where possible, these supports should be located outside of the clear zone. Otherwise, they should be shielded with a crashworthy barrier. The location of the support should provide clearance between the back of the rail and the face of the support to ensure that the rail will function as intended when struck by a vehicle. This clearance will vary depending on the type of guardrail used.

12.10.5 TRAFFIC SIGNAL SUPPORTS

Traffic signal supports include structures for post mounted traffic signals, structures with cantilevered arms, overhead mounted traffic signals, and span wire mounted traffic signals.

Traffic signal supports present a special situation where a breakaway support may not be practical or desirable. As with luminaire supports, a fallen signal post support may become an obstruction. However, the potential risks associated with the temporary loss of full signalization at the intersection should be considered.

When traffic signals are installed on high-speed facilities (generally defined as those having speed limits of 80 km/h or greater), the signal supports and, if not mounted on one of the signal support poles, the signal support box, should be placed as far away from the roadway as practicable. Shielding these supports can be considered if they are within the clear zone for that particular roadway.

Traffic signal supports with mast arms, or those that have a support on both sides of the roadway and a wire (span wire) or other components (overhead) that spans the facility, normally are not provided with a breakaway device. Post-mounted signals are commonly installed in close proximity to traffic lanes or in wide medians; therefore, consideration should be given to using breakaway devices for these supports.

12.10.6 UTILITY POLES

Globally, motor vehicle crashes with utility poles account for a significant proportion of all fixed-object fatal crashes annually. This degree of involvement is related to the number of poles in use, their proximity to the travelled way, and their unyielding nature.

As with sign and luminaire supports, the most desirable solution is to locate utility poles where they are least likely to be struck. One alternative unique to power and telephone lines is to bury them, thereby eliminating the obstacles. For poles that cannot be eliminated or relocated, breakaway designs have been developed and successfully crash tested.

For new construction or major reconstruction, every effort should be made to install or relocate utility poles as far from the travelled way as practical.

12.10.7 TREES

Unlike the roadside hardware previously addressed in this section, trees are not generally a design element over which highway designers have direct control. With the exception of landscaping projects in which the types and locations of trees and other vegetation can be carefully chosen, the problem most often faced by designers is the treatment of existing trees that are likely to be impacted by an errant vehicle.

Trees are potential obstructions by virtue of their size and their location in relation to vehicular traffic. Generally, an existing tree with an expected mature size greater than 100 mm at stub height is considered a fixed object. When trees or shrubs with multiple trunks or groups of small trees are close together, they may be considered as having the effect of a single tree with their combined cross-sectional area.

Routine maintenance can minimize future problems by clearing the clear zones to prevent seedlings from becoming established. The location factor is more difficult to address than tree size. Typically, large trees should be removed from within the selected clear zone for new construction and for reconstruction. The extent of the clear zone depends on several variables, including highway speeds, traffic volumes, and roadside slopes.

Essentially, there are two methods for addressing the issue of roadside trees. These are:

i. On-roadway treatment
ii. Off-roadway treatment

A. On-roadway treatments include:

- Pavement marking
- Rumble strips
- Signs
- Delineators
- Roadway improvements (pothole patching, resealing etc.)

Pavement markings are one of the most effective and least costly improvements that can be made to a roadway. Centreline and edge line markings are particularly effective for roads with heavy night-time traffic, frequent fog, and narrow lanes. Shoulder rumble strips also can be used to warn motorists that their vehicles have crossed the edge line and may run off the road.

The installation of advance warning signs and roadway delineators also can be used to notify motorists of sections of roadway where extra caution is advised. Typically, these will be used in advance of curves that are noticeably sharper than those immediately preceding it.

Roadway improvements such as curve reconstruction to provide increased superelevation, shoulder widening, and paving are relatively expensive countermeasures that may not be cost-effective in all cases.

B. Off-roadway treatment is to mitigate the danger inherent in leaving a roadway with trees along it. Off-roadway treatments consist primarily of two options:

- Tree removal
- Shielding

The removal of individual trees should be considered when those trees are determined to be both obstructions and in a location where they are likely to be hit. Such trees often can be identified by past crash histories at similar sites, by scars indicating previous crashes, or by field reviews. Removal of individual trees will not reduce the probability that a vehicle will leave the roadway at that point, but it should reduce the severity of any resulting crash. For example, 1V:3H and flatter slopes may be traversable, but a vehicle on a 1V:3H slope usually will reach the bottom. If numerous trees are at the toe of the slope, removal of isolated trees on the slope will not significantly reduce the risk of a crash. Similarly, if the recommended clear zone for a particular roadway is 7 m, including the shoulder, removal of trees 6 to 7 m from the road will not materially change the risk to motorists if an unbroken tree line remains at 8 m and beyond.

However, isolated trees noticeably closer to the roadway may be candidates for removal. If a tree or group of trees is in a vulnerable location but cannot be removed, a properly designed and installed traffic barrier can be used to shield them. Roadside barriers should be used only when the severity of striking the tree is greater than striking the barrier. Specific information on the selection, location, and design of roadside barriers is in **Section 12.5.2.5**.

12.10.7.1 PLACEMENT OF LANDSCAPING, TREES, AND SHRUBS

Along most urban roads, some type of landscaping exists. Trees, shrubs, lawns, decorative rock, and other materials are used to provide a pleasing setting for drivers, pedestrians, bicyclists, and abutting landowners. The presence of roadside landscaping is known to have a positive influence on the health of drivers as well as other users of the facility. Roadside landscaping also can aid in providing drivers visual cues about the road environment. Maintenance of urban forestry similarly can aid in improving the environmental quality in the region. The design process, therefore, should balance the benefits of landscaping with the requirements for roadside safety when possible.

The designer always should be consulted in the decisions regarding landscaping, particularly because they relate to sight distance and possible future lane needs. Considerations in the design of landscaping include the following:

i. The mature size of trees and shrubs, and how it will affect safety, visibility, and maintenance cost.
ii. Adequacy of border area to accommodate the type of landscaping planned (i.e., if parking is allowed along the kerb, the landscaping should allow kerbside access to parked vehicles).
iii. Potential future changes in roadway cross-sections. For example, adding a second left-turn lane at major intersections by taking approximately 3m of additional space from the median island is becoming a common practice. Landscaping in the affected area should be minimal or should not be included in the plan.

Visibility restrictions resulting from landscaping are of principal concern to the designer. Points that must be considered include the following:

i. Border area landscaping should allow full visibility for drivers and pedestrians at driveways and intersections.
ii. A clear vision space from 1 to 3 m above grade is desirable along all roads and at all intersections. This space allows drivers in cars, trucks, and buses to have good sight distance.
iii. Landscaping of very small islands should be avoided to reduce maintenance needs.
iv. Large trees or rocks should not be used at decision points (e.g., gore areas, island noses) to protect poles and other appurtenances.
v. Longitudinal placement of trees and landscaping should separate these items from underground utility lines, power poles, street lights, existing trees, light standards, fire hydrants, water meters, or utility vaults to assure root systems do not conflict with utilities.
vi. Canopy trees should not be positioned under service wires and, where present, should be of sufficient height to provide clearance for taller vehicles, including buses and trucks.

With respect to pedestrians, it is desirable to have a grass strip separating the sidewalk from the kerb, thus further separating the pedestrian from vehicular traffic.

Another planting strategy that can improve roadside safety is layering plants so that rigid plants are shielded by smaller, more frangible ones. This approach creates an attractive roadside landscaping while also naturally creating energy dissipation in an accident through the creative use of plants.

12.10.8 ADVERTISEMENT SIGNS

These are signs which do not have a function related to traffic control and road safety. They are often provided for commercial purposes but sometimes non-commercial publicity. They could range from billboards on massive overhead structures, bridge-mounted signs to more modest roadside signs.

The main concerns for advertisement signs are:

i. Distraction to drivers
ii. Impairment of visibility of intersection
iii. Impairment of visibility of traffic signs and directional signs
iv. Passive safety

Prominent advertisement signs, whether within or outside the right-of-way, should therefore be avoided and minimised along major routes, especially where there is a high demand of driving tasks at high speeds. Less dominating advertisement signs may be acceptable where they do not constitute a significant problem.

12.11 ROAD FURNITURE FOR TEMPORAL WORKS

The purpose of temporal road furniture is to inform, warn and guide road users safely past the work zones and in that way protecting the road users and the workforce. All road works, no matter how small, must be properly signed. To keep the respect of the road users for the signing of road works, and with it the road safety, it is also important to continuously maintain and adjust the signing to the current work situation. If some or all of the signing are no longer needed, it must be removed.

When carrying out works on an existing road, in addition to installing the traffic safety facilities described above, traffic guides (flaggers or flag men) and security personnel should be assigned and traffic regulations such as shown in the MRT Guidelines for the Signing at Road Works - Final Draft September 2007 should be adhered to.

12.12 TRUCK- AND TRAILER-MOUNTED ATTENUATORS (TMAs)

In many short-term, mobile, and moving work zones, trucks can be used as blocking vehicles to protect workers. Large trucks are effective in preventing vehicle encroachment into the work site; however, serious injury to occupants of the impacting vehicle and the truck could result when an errant vehicle strikes the back of the truck.

Crash cushions called TMAs can be attached to the rear of these shadowing vehicles to reduce the severity of rear-end crashes. The TMAs may be directly mounted onto the rear of the truck or towed by the vehicle as a trailer. They may be used for moving operations such as pavement marking, roadway sweeping, and maintenance activities in high-volume, high-speed areas or at long-term, stationary construction sites. **Table 12.36** shows suggested priorities for consideration of their use.

TMAs are used on the following three classes of protective vehicles in work zones:

i. Shadow Vehicle

   A moving truck spaced a short distance from a moving operation, giving physical protection to workers from traffic approaching from the rear.

ii. Barrier Vehicle

   A truck parked upstream from a stationary operation and usually unoccupied.

iii. Advance Warning Truck

   A truck parked a considerable distance upstream of a moving or stationary operation and displaying an arrow panel and other signs as appropriate.

Shadow trucks and barrier vehicles may be equipped with a TMA. Advance sign trucks may use TMAs if they encroach on the travelled way. Protective vehicles usually are equipped with arrow boards, changeable message signs, or flashing amber lights. To increase the protection for the truck drivers, the trucks should have lap/shoulder restraints and headrests. Existing TMAs generally are not suitable for specialized vehicles such as motor graders, mowers, and tow trucks; however, there are crash-tested interfaces for use between TMAs and some types of street sweepers.

Table 12.36 Suggested priorities for application of protective vehicles and Truck - mountain attenuators

Closure/Exposure Conditions	Examples of Typical Construction Maintenance Activities	Design Class*			
		A & B	C, D & E		
			Design Speed (km/h)		
			80	70	60
Mobile Activities					
A. No Formal Lane Closure					
Shadow vehicle for operation involving exposed personnel	Crack pouring, patching, utility work, striping, coning	A-1	A-2	A-3	A-4
Shadow vehicle for operation not involving exposed personnel	Sweeping, chemical spraying	E-1	E-2	E-3	E-4
B. No Formal Shoulder closure					
Shadow vehicle for operation involving exposed personnel	Pavement repair, pavement marking, delineator repair	B-2	B-3	C-3	C-3
Barrier vehicle for operation not involving exposed personnel	Open excavation, temporarily exposed bridge pier	E-2	E-3	E-4	E-5
Stationary Activities					
A. Formal Lane Closure					
Barrier vehicle for operation involving exposed personnel	Pavement repair, pavement marking	B-2	B-3	C-4	C-5
Barrier vehicle for condition involving significant obstruction	Open excavation	E-2	E-3	E-4	E-5
B. Formal Shoulder Closure					
Barrier vehicle for operation involving exposed personnel	Pavement repair, pavement marking, guardrail repair	C-3	C-4	D-5	D-5
Barrier vehicle for condition involving significant obstruction	Open excavation	E-3	E-4	E-5	E-5

*The alphabetic ranking includes the priority assigned to the use of protective vehicles as follows:

A— is very highly recommended
B— is highly recommended
C— is recommended
D— is desirable
E—may be justified on the basis of special conditions encountered on an individual project when an evaluation of the circumstances indicates that an impact with a protective vehicle is likely to result in less serious damage than would impact with a working vehicle on the obstruction.

*The numerical ranking indicates the level of priority assigned to the use of a TMA on an assigned protective vehicle.
The use of a TMA under the defined numerical conditions are as follows:

1—is very highly recommended
2—is highly recommended
3—is recommended
4—is desirable
5—may be justified on the basis of special conditions encounter on an individual project.

12.12.1 BUFFER DISTANCE

The buffer distance is the space between the protective vehicle and the work activity. It provides for a roll-ahead, post-collision movement of the protective vehicle. This distance is typically a compromise between anticipated roll-ahead movement and excessive space that would permit traffic to move into the buffer zone. Buffer distances ranges from 15 to 60 m. Buffer distances should be based on horizontal and vertical geometrics, available sight distance, average speed of traffic, and type of operation. **Table 12.37** shows an example of guidelines for spacing shadow vehicles.

When tested with a 2,000kg passenger car at 70km/h, a truck with a TMA moved forward less than 10 m. Therefore, a minimum distance of 9m between the truck and work zone is recommended. Based on the manufacturer's recommendation, if approach speeds are higher than 70 km/h, a longer distance should be used. The truck's parking brake should be set, the transmission placed in gear, and, when possible, the front wheels turned away from the work area. These recommendations are for trucks weighing 4,500kg or more.

Table 12.37 Examples of Guidelines for spacing of Shadow Vehicles

For Shadow Vehicles Weighing 10,000 kg or more		
Operating Speed/Speed Limit (km/h) [a]	Recommended Spacing [a]	
	Stationary Operation (m)	Moving Operation [c] (m)
>90	45	52.5
70-90	30	45
<70	22.5	30
For Shadow vehicle Weighing Less than 10,000kg but Greater than 4,500 kg [d]		
Operating Speed/Speed Limit (km/h) [a]	**Recommended Spacing [b]**	
	Stationary Operation (m)	**Moving Operation [c] (m)**
>90	52.5	67.5
70-90	37.5	52.5
<70	30	30

Footnotes

a) Should use operating speed of higher than posted speed limit.
b) Recommended spacing is distance between front of shadow vehicle and beginning of work area, that is, the first worker/operation/vehicle to be protected.
c) Distances are appropriate for shadow vehicle speed up to 25 km/h.
d) Shadow vehicle shall weigh 8,000 to 9,000 kg on all road construction projects.

Notes:

1. The heaviest shadow vehicle should be used to optimize protection of maintenance or construction workers. Because roll-ahead is minimized with heavier shadow vehicles, they can be placed closer to the workspace to minimize the risk of vehicles cutting in ahead of the shadow vehicles.
2. The spacing distance is good with or without a TMA. A vehicle equipped with a TMA may move less than a truck not equipped with a TMA. However, the recommended spacing is conservative enough to allow the same spacing for a TMA versus a vehicle without a TMA.
3. Distances are intended as guidelines. However, engineering judgment should be used to alter distance to take into account traffic conditions, vehicle mix, sight distance, and other site conditions.

12.12.2 MASS OF A SHADOW VEHICLE

The mass of the shadow vehicle should be similar to the mass of the vehicle with which the TMA was crash tested, generally 9,000kg ± 450 kg. If a significantly lighter or heavier shadow vehicle is used, the manufacturer’s recommendations should be followed.

12.12.3 DELINEATION

Delineation should be used on TMAs to make them conspicuous at night.

12.12.4 CRASHWORTHY TMAs

Several types of TMAs have met the requirements of NCHRP Report 350 or MASH. They can be categorized as follows:

i. Energy-absorbing cartridge mounted in a frame (see **Plate 12.62**)
ii. Telescoping steel frame with a cutter assembly (see **Plate 12.63**)
iii. Trailer-mounted steel frame with burster or kinker assembly (see **Plate 12.64**)
iv. Steel or polyethylene cylinder assembly (see **Plate 12.65**)
v. Mobile Barrier Trailer (see **Plate 12.66**)

Plate 12.62 TMA with Energy-Absorbing Cartridge

Plate 12.63 TMA with Telescoping Steel Frame and Cutter Assembly

Plate 12.64 TMA with Steel Frame and Burster or Kinker Assembly

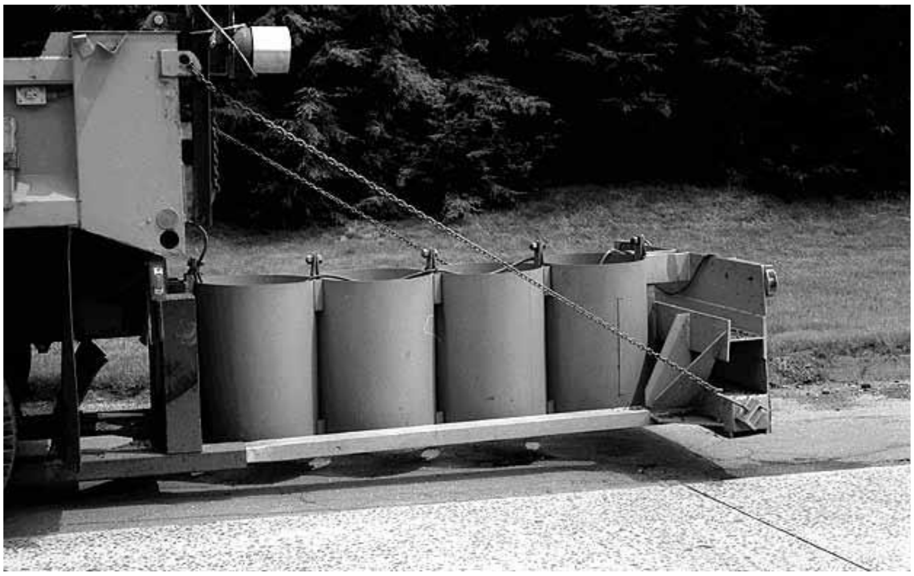


Plate 12.65 TMA with Steel or Polyethylene Cylinder Assembly

Plate 12.66 Mobile Barrier Trailer

12.13 REFERENCES

1. Ghana Highway Authority Road Design Guide, 1991.
2. Standard Details, Road Signs and Markings for Urban and Trunk Road (1991), Ministry

of Roads and Highways.

3. Ministry of Roads and Transport (MRT) Standard Specifications for Road and Bridge Works, 2007.
4. Draft Manual for Signs and Markings, 2007.
5. Japanese Road Structure Ordinance, April 2021.
6. Geometric Design Manual of Uganda, (2005).
7. A Policy on Geometric Design of Highway and Street, American Association of State Highway and Transportation Officials (AASHTO), 2018.
8. South African Geometric Design Guidelines (2003).
9. Geometric Design Manual, Federal Democratic Republic of Ethiopia, Ethiopian Roads Authority, 2013.
10. Roadside Design Guide. United States: American Association of State Highway and Transportation Officials, 2011.
11. Asian Highway Design Standard for Road Safety, 2017.
12. AASHTO. Roadway Lighting Design Guide, Sixth Edition with 2010 Errata, GL-6. American Association of State Highway and Transportation Officials, Washington, DC, 2005. Seventh edition pending 2018.
13. Ministry of Transportation Standard Specification for Roads & Bridge Works, July 2007, Section 27
14. ECG Street lighting In Ghana Standards and Policy
15. ECG Technical Requirements for Street lighting Installations
16. Traffic Signs Regulations and General Directions
17. BS 7671: Regulations for Electrical Installations 1992
18. BS 5489: Parts 1 – 10,” Code of Practice for Road Lighting”
19. BS EN 60529: Specification for Clarification of Degrees of Protection provided by Enclosures
20. BS EN 605589-2-3: 1994 Luminaires for Road and Street Lighting
21. BS 5649: Lighting Columns
22. BS EN 40: Lighting columns1992
23. Standard BD26/94 - Design of Lighting Columns
24. Plaxico, Chuck A., Malcolm H. Ray, and Kamarajugadda Hiranmayee. "Impact performance of the G4 (1W) and G4 (2W) guardrail systems: comparison under NCHRP Report 350 test 3-11 conditions." Transportation Research Record 1720, no. 1 (2000): 7-18.

Volume V

Chapter 13 Road Drainage

GHANA ROAD DESIGN GUIDE 2023

CHAPTER 13
ROAD DRAINAGE

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CHAPTER 13 ROAD DRAINAGE

13.1 INTRODUCTION

Highway hydraulic structures perform the vital function of conveying, diverting, or removing surface water from the highway right-of-way. They should be designed to be commensurate with risk, construction cost, importance of the road, economy of maintenance, and legal requirements. One type of drainage facility will rarely provide the most satisfactory drainage for all sections of a highway. Therefore, the designer should know and understand how different drainage facilities can be integrated to provide complete drainage control.

Drainage design covers many disciplines, of which two are **hydrology** and **hydraulics**. The determination of the quantity and frequency of runoff, surface and groundwater, is a hydrologic problem. The design of structures with the proper capacity to divert water from the roadway, remove water from the roadway, and pass collected water under the roadway is a hydraulic problem.

13.1.1 TYPES OF DRAINAGE FACILITIES

Highway drainage facilities can be broadly classified into two major categories based on construction: (1) open-channel or (2) closed-conduit facilities.

Open-channel facilities include roadway channels, median swales, kerb and gutter flow, and others. Closed-conduit facilities include culverts and storm drain systems. Note that from a hydraulic classification of flow condition, open-channel or free-surface flow can occur in closed-conduit facilities.

Figure 13.1 shows a typical divided highway where a variety of open-channel and closed conduit facilities are needed to drain the highway. Starting at the outer edge of the right-of-way are the intercepting channels on the natural ground outside the cut-and-fill or on benches breaking the cut slope. In an arid region, intercepting channels (or dikes) may also be used for great distances along the roadway to capture overland flow runoff from large upstream watersheds. Next are the roadway channels between the cut slope and shoulder of the road and the toe-of-slope channels which take the discharges from the roadway channels and convey it along or near the edge of the roadway embankment to a point of disposal. A shallow depression or swale drains the median to an inlet that conveys water to the culvert. The culvert itself provides for cross drainage of a relatively large stream channel.

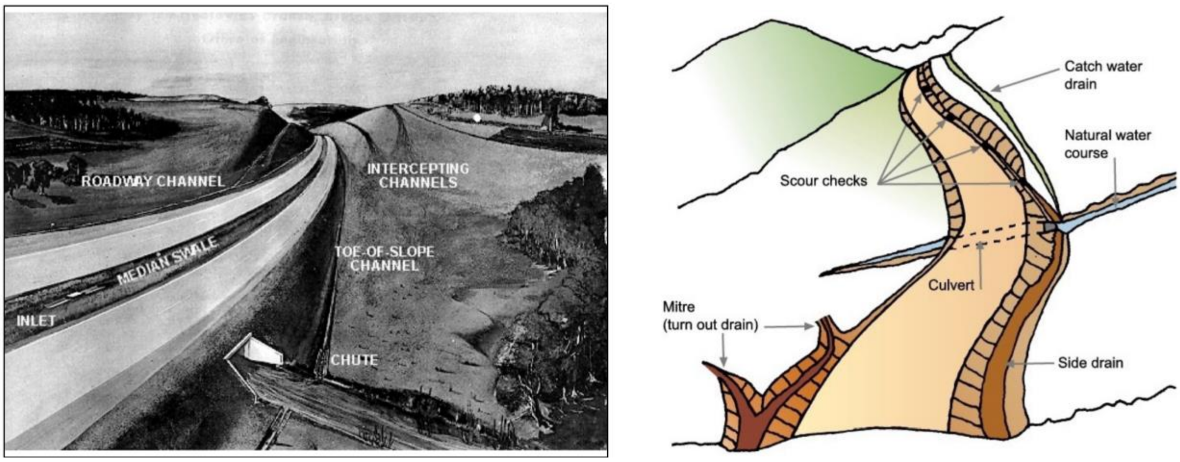


Figure 13.1 Types of road drainage facilities

13.1.2 DRAINAGE CONSIDERATIONS IN ROAD PLANNING AND LOCATION

The planning and locating of road facilities are the first steps in a challenging process of providing a safe and efficient transportation system. Hydrologic and hydraulic requirements are among the facets that must be considered during the early phases of the design process.

Water and its related resources are important considerations in the planning and locating of roads and their appurtenant facilities.

The possible effects that road construction may have on existing drainage patterns, river characteristics, potential flood hazards, and the environment in general, and the effects the river

and other water features may have on the road, should be considered at this time.

Hydrologic and hydraulic specialists must be actively involved during the initial project phases to ensure that proper consideration is being given to drainage aspects. This involvement should include participation during the route location selection phase. Early input from these specialists will result in a better design, both hydraulically and economically.

Sound planning and design of road drainage system is supposed to achieve the following:

i. avoid unnecessary alteration of natural drainage pattern
ii. prevent loss of life and minimize property damage
iii. minimize traffic hazards and interruptions from surface pounding and flooding
iv. minimize stream bank erosion

In order to develop the most appropriate drainage solution, the project team for each project must select applicable drainage considerations from the following categories:

i. Geometric
ii. Geographic
iii. Environmental
iv. Crossing Type
v. Economic
vi. Maintenance
vii. Safety

13.1.3 GEOMETRIC CONSIDERATIONS

There are two aspects of geometry that must be considered in the drainage design of a road project. The first aspect deals with the geometry of the watercourse and the second aspect deals with the geometry of the road-watercourse crossing.

13.1.3.1 WATERCOURSE GEOMETRY

It is important to determine the geometry of the watercourse or flow path, in particular: watercourse longitudinal alignment; watercourse gradient; and channel shape.

Watercourse alignment refers to the natural meanders of the watercourse channel. While most watercourses have only one alignment for all flows, it is possible to have the situation where the alignment for a low flow differs from the alignment for a high flow in the same watercourse. This situation must be identified and considered when designing the road-watercourse crossing.

It is possible to alter the alignment of existing watercourses to improve the hydraulic performance of the road-watercourse crossing, however it is preferable to maintain or preserve the existing watercourse alignment as changes will affect the existing flow parameters (velocity, depth of flow and energy).

Watercourse gradient refers to the vertical alignment of the watercourse and changes to gradient will also affect flow parameters. Gradient has a significant influence on flow velocity and velocity in turn has a significant effect on sediment transport and scour potential.

Channel shape needs to be considered as it will tend to dictate the size and configuration of drainage structures. Altering the channel shape to accommodate a drainage structure will affect flow parameters and could increase the risk of erosion. It is preferable to maintain or preserve the existing channel shape as closely as possible and culvert structures should be designed to ‘fit’ the shape of the watercourse. Some channels may not contain all of the design storm run-off and overtopping of the banks will occur. Multiple culvert installations for the one catchment will be required and in this instance, specialist advice / design will be required.

Lastly, road drainage designers must have an understanding of stream morphology when considering stream geometrics. Streams are dynamic and can change over time. It is important for this aspect to be considered.

13.1.3.2 ROAD GEOMETRY

Drainage is an integral component of road infrastructure and therefore drainage design cannot be undertaken in isolation from the geometric design of the road. In the design of the road-watercourse crossing, it is important to consider the skew angle between the road alignment and drainage structure. Keeping the skew angle as small as possible (or eliminating it altogether) reduces costs and construction difficulty and is therefore the most desirable option.

Given that it is highly recommended to preserve watercourse alignment, this consideration, however, does not imply any priority of drainage over road alignment and high skew angles may be unavoidable at times.

Two aspects of the proposed alignment must be considered. First, the hydraulics engineer must consider how the streams or storm drain systems may affect the roadway and, second, how the roadway may affect the flow characteristics of such streams or systems. Slight changes in alignment can sometimes alter the flooding characteristics significantly.

Whether or not changes to the horizontal alignment can be made often depends on whether the project is an improvement to an existing highway or the construction of a highway in a new location.

There is often little opportunity to change horizontal alignments when the project is an improvement to an existing highway. The alignment should still be reviewed, though, to identify locations where:

i. slopes need to be protected against scour,
ii. abutments moved or skewed differently,
iii. drainage structures protected against headcutting, and
iv. meanders are endangering the roadway.

Changes to the horizontal alignment of the highway at stream crossings can also result in hydraulic consequences. Many older structures were constructed to cross the stream at a right angle to the flow. This sometimes resulted in sharp curves in the roadway approaches to the bridges. Replacement structures are often planned to correct this poor alignment by crossing the stream at a skew. Proper abutment and pier alignment of the replacement structure must be ensured. If the existing substructures are to be used as part of the replacement, their alignment with the channel must be considered. If the substructures will not be used in the new alignment, it is generally preferable to remove them.

The construction of a roadway on a new alignment affords the greatest opportunity for the hydraulics engineer to influence the alignment during the location phase. During this phase, changes can be recommended to locate the roadway away from a stream or situate a bridge at a more stable channel location. These recommendations should be made early in the development of a project to avoid delays during the design or right-of-way acquisition phase when the horizontal alignment is difficult to change.

During relocation, there may also be constraints that control the alignment. Topographic and cultural features may have to be avoided, resulting in the use of the river environment for the highway. In these cases, the constraints noted in the previous section will often exist. Besides these constraints, there may be other alternatives that should be studied because of other considerations, such as cost-effective designs or land development plans.

The effect of the vertical alignment, commonly called the profile, on highway drainage facilities is significant and must be assessed in comparing alternative locations. Although the profile usually is of greater interest to the hydraulics engineer than the horizontal alignment, it is normally easier to alter and is not firmly set as early in the project development.

The profile is that feature, along with the hydraulic opening, that determines when, and where, the highway will be overtopped. By raising or lowering the profile, the frequency of overtopping can be either decreased or increased. Not only does the profile affect the frequency of overtopping, but it also determines the level of upstream flooding.

Depressed roadways act as drainage interceptors and may require that upstream surface runoff be accommodated in storm drains or diversion channels. Fills on wide, flat areas may intercept surface flows and require special drainage treatments. These problems will be of special concern with large urban expressways and deserve careful evaluation at the location phase. On streams where navigation exists, clearances required for waterway vessels may become the factor controlling vertical alignment.

The profile not only affects the flow from streams either over the roadway or through the structure opening, but it also affects the flow of the roadway runoff water. Sag vertical curves are critical profile areas, because they can serve to trap roadway drainage unless adequately sized and spaced outlets or catch basins are provided. Steepness of the highway grade also determines the spacing of inlets in areas where the roadway has kerbs.

Vertical alignment together with cross-sectional cross fall of the road also affects longitudinal drainage channels (such as table drains) and therefore must be designed considering minimum grade requirements for flows and minimising steeper grades where higher erosive velocities could result. Another important aspect related to the geometric design of roads is storm water run-off from the road surface. This aspect is critical as water flow (and depth) on the road surface can result in aquaplaning.

Where the possibility of storm water crossing over the road exists (whether intentional or unintentional), adequate stopping sight distance must be provided, and this factor could affect the vertical alignment design.

13.1.4 GEOGRAPHIC CONSIDERATIONS

Geographic conditions play a significant role in the determination of what type of drainage structure and/or controls may be adopted at a given location. Structures and controls that are appropriate in one part of the country may not be suitable in other areas.

The design of drainage systems in should ensure that the road level and associated drainage infrastructure is adequate to provide the specified level of flood immunity. Furthermore, the drainage structures should be sized to ensure that flow velocities and afflux are acceptable.

Specific issues to be addressed in rural areas include:

i. Awareness of local drainage and management plans
ii. Ensuring property and crops will not be affected by an increase in water levels or duration of inundation
iii. Changes to flow patterns, and consideration of seasonal variations in hydraulic roughness linked to changes in vegetation cover
iv. Concentration of flow on floodplains should be minimized because of the risk of scour; maintaining free drainage, and not creating ponding at low flows.

Urban regions have similar issues to rural areas but may also present other constraints. Constraints may be present in the form of adjacent infrastructure (including businesses and housing) or a limit in available space (rights-of-way). Because of the more intense level of development, afflux is usually of more concern in urban areas than in rural locations.

Considerations in urban regions include:

i. Provision for higher peak flows arising from uncontrolled upstream development
ii. Assessment of the requirements of any catchment management plan or storm water management plans prepared for the watercourse
iii. The need for pollution control measures
iv. Interaction of road drainage provisions with existing services
v. Minimization of ground disturbance during construction, as urban environments often have limited space for large control measures such as sediment basins
vi. Consideration and control of afflux effects. There is often a requirement that negligible afflux increases be generated upstream/downstream of the proposed drainage structure.

With respect to possible change in water levels, it is important that each case is assessed fully in keeping with a risk management approach. Design of road drainage in flat terrain is often difficult for several reasons, including:

i. Flows velocities in flat areas are usually low so larger structures are needed to convey the flow
ii. Flow may be widespread and/or shallow and minor obstructions may divert the flow; these minor obstructions include levees and other floodplain works
iii. Even the road itself may cause major diversions.

It is often difficult to determine the catchment areas accurately because of minimal relief in terrain and the presence of minor obstructions as discussed above. Poorly defined flow paths also mean that it is sometimes difficult to place culverts in the most suitable locations.

In flat terrain, the impacts of the road on flood levels may extend for significant distances upstream of the road. Where afflux is a concern, this impact may often be critical. There is usually an increased risk of erosion at culvert outlets because flow will be concentrated by drainage structures, particularly where there are poorly defined flow paths and/or most flow occurs across the floodplain.

In mountainous or steep terrain, the most common factor influencing design is the gradient of the natural ground. Issues for consideration where topography is steep include:

i. Control of velocities in roadside drains and culvert outlets
ii. Collection and discharge of water from the upward side of the road to the downward side
iii. Prevention of erosion at outlets onto steep areas
iv. The need for small scale drop structures, weirs or drop manholes.

Locations subject to inundation by water, such as floodplains by backwater, require careful consideration of how drainage infrastructure will operate under a range of water levels. The presence of high and low water levels requires significantly different approaches:

i. When downstream water levels are high, the hydraulic capacity of a structure may be limited; and
ii. When downstream water levels are low, high velocities can result, thereby maximising the potential for erosion to occur.

It is therefore very important that both cases are considered during the design of drainage infrastructure. Regular inundation (i.e., change in water levels) can also accelerate the erosion process, through the saturation of banks, which may then fail as water levels drop.

13.1.5 ENVIRONMENTAL CONSIDERATIONS

A road embankment may form an obstruction to a natural waterway or grass/farmland corridor if there are insufficient bridge and/or culvert openings in the road. Considerations of the following may therefore be required:

i. An unacceptably high increase in upstream flood level (afflux) may cause water to reach property or crops which would otherwise be flood free.
ii. An increase in stream velocities through the structures may initiate or cause continuing erosion downstream of the road and limit fish movement.
iii. Restricted flow may cause increased time of inundation of the land upstream of the road. However, it would be extremely rare for this to cause a problem unless the road is designed as a dam wall or part of a large retention basin. In this situation there is also a tendency to reduce peak discharges in floods.
iv. Roads may restrict the movement of fauna from one side of the road to the other. At some locations, local government or the Environmental Protection Agency may have minimum size requirements for openings in the road for fauna.

13.1.6 CROSSING TYPE

Determining the type of structure for any crossing is an important consideration and there are a number of factors that need to be addressed in this process. It may be necessary to assess several options of different crossing types and sizes in order to appropriately meet the design requirements and objectives. There are three main types of cross-drainage structures used on roads, and each has particular advantages and disadvantages. The three types are bridges, culverts and low-level river crossings (ford/drift and vented ford/drift/causeway) as shown in **Plate 13.1**.

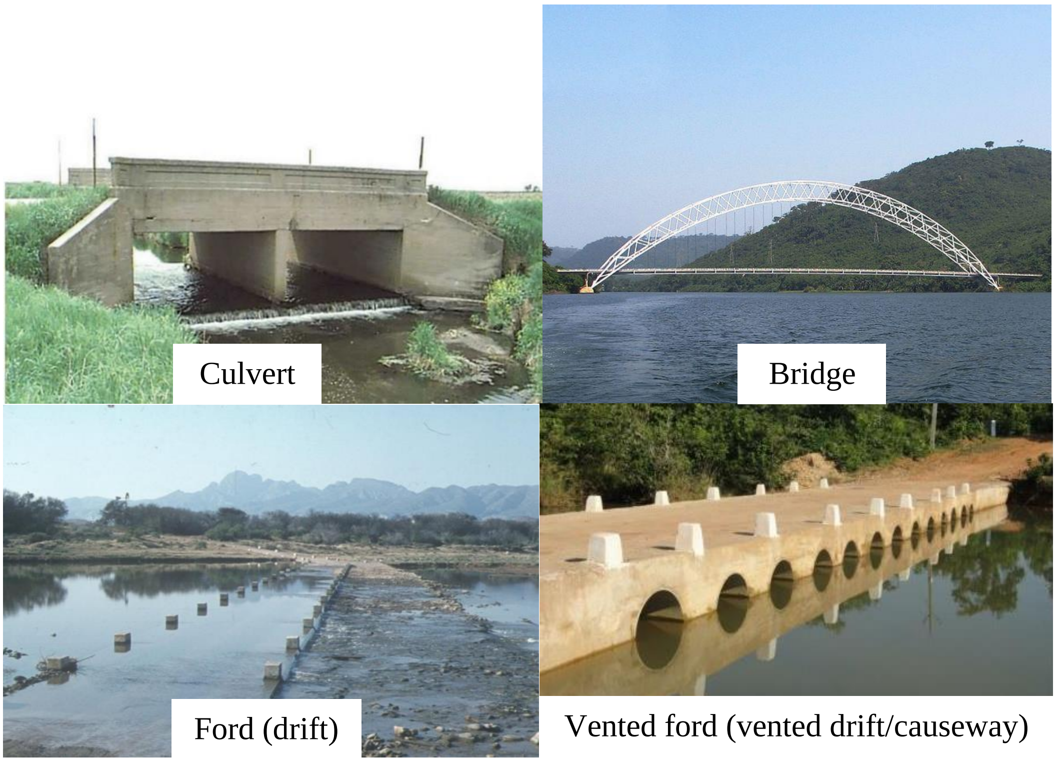


Plate 13.1 Primary Drainage Infrastructure Types

The relevant factors that need to be considered in selecting drainage infrastructure are grouped into 'hydraulic' and 'other' factors.

The hydraulic factors include:

i. Flood discharge: Defined waterways with a large discharge are more suited to a bridge because of the larger waterway area. The large discharge will also generally occur in rivers or large creeks, where a bridge is more appropriate and cost effective. Depending on location and importance of road, in flat terrain where the waterways are less defined and road embankment is typically low, a low-level river crossing may be a better option.
ii. Stream channel conditions and topography: Similarly, with the consideration of discharge, the shape and size of the channel and the catchment will also indicate a bridge or culvert. Large and well-defined channels will be better suited to a bridge, while less well-defined, smaller channels will be more suited to a culvert, especially where multiple openings are required (such as on floodplains). Low-level river crossings could also be considered, particularly in flat terrain/low embankment situations.
iii. Afflux constraints: The most suitable structure may be indicated by the amount of flow that can pass through/over the structure with acceptable afflux. The location and extent of afflux needs to be considered in detail and the alternatives assessed to minimise afflux.
iv. Debris properties: Culverts will normally have a smaller waterway area and present a greater obstruction to the flow. They are more prone to collection of debris. If there is a large amount of debris conveyed by a stream, a bridge or larger culvert may be more suitable.
v. Scour risk: Scour can be affected by the size and type of waterway structure. If a structure concentrate flow significantly, risk of scour may be increased, so structures that spread the flow may be favoured in these locations. This is especially important for drainage in floodplains where the flow paths may not be well-defined.
vi. Fish passage requirements: where the structure is on a mapped waterway for waterway

barrier works the selection of the structure is impacted by legislative requirements. This can impact economics, hydraulic and structural design.

Other relevant factors that need to be considered include:

i. Road alignment: Sometimes the alignment of the road is well-defined and this may not be the best arrangement for drainage. This may sometimes occur where land tenure needs to be considered and the alignment follows streams rather than crossing at a zero skew. In these cases, the sizing and locating of drainage structures must be carefully considered.
ii. Level of serviceability: This includes the required flood immunity or ‘trafficability’, and the type of structure that will be best for meeting this requirement.
iii. Navigation: Structures crossing rivers where boat traffic needs to be considered must allow for specified clearances for this traffic.
iv. Soil conditions: Particular soil conditions, such as marine mud or acid sulphate soils, for example, may be a problem and this can affect the selection of drainage structures.
v. Fauna movement: This is an important consideration in many locations.

13.1.6.1 BRIDGE, CULVERT OR LOW-LEVEL RIVER CROSSINGS

There are a number of factors and issues that need to be considered in the selection of the most suitable / appropriate structure for a particular crossing. These are listed in **Table 13.1**.

Table 13.1 General Selection Factors - Structure Advantages & Disadvantages

Structure	Advantages	Disadvantages
Bridges	• Waterway area generally increases with increased deck height. • Provides greatest flood immunity. • Large flow capacity. • Fewer problems with debris. • Deck widening does not affect capacity. • Minimal impact on aquatic environment and wetlands. • Capacity increases with stage. • Flowline is flexible.	• Higher design, construction and maintenance costs. • More structural maintenance required. • Spill slopes can be affected by erosion (potential for costly batter protection requirements particularly for higher/exposed approach embankments). • Bridge railing and parapets hazardous as compared to recovery areas. • Deck drainage may require frequent maintenance cleanout. • Pier and abutment can be affected by scour. • Increased buoyancy, drag and impact risks. • Susceptible to stream/channel meander migration.

Structure	Advantages	Disadvantages
Culverts	• Provides an uninterrupted view of the road. • Roadside recovery area can be provided. • Usually easier and quicker to design and build than bridges. • Generally, most cost-effective option. • Scour is localized, more predictable, and easier to control. • Can be used to arrest headcutting. • Storage can be utilized to reduce peak discharge. • Capacity increases with stage. • Capacity can sometimes be increased by installing improved inlets. • Can accommodate future changes to road geometry. • Less structural maintenance. • Can spread flows.	• Generally, require higher levels of general maintenance. • No increase in waterway as stage rises above soffit. • Most susceptible to failure. • Higher siltation/debris risk (blockage) • Increased environmental impacts (fauna/fish passage). • Susceptible to erosion of fill slopes and scour at outlets. • Susceptible to abrasion and corrosion damage. • Future extension may reduce capacity. • Potential for separation at joints. • Potential for failure by piping (leading to failure of embankment).
Low-level river crossings	• Generally simple to design. • May offer environmental advantages over culverts and bridges, since they will tend to spread flows more widely. • Typically have low embankments. • Risk of scour to waterway and surrounding land is reduced.	• Allow water flow over road – immunity and safety issues. • Increased disruption to traffic due to overtopping. • Can have higher construction costs than culvert. • Batter slopes can be affected by erosion / scour (particularly for higher embankments). • Generally, have costly batter protection requirements. • Susceptible to stream / channel migration. • Can have environmental impacts (fauna / fish passage). • Potential for failure of embankment (depending on provided protection).

13.1.7 ECONOMIC CONSIDERATIONS

The provision of road drainage infrastructure is invariably linked to the trade-off between the desired level of immunity and the available budget. Hence, there will be occasions when the preferred level of immunity cannot be provided. This may be countered through stagged construction (i.e., initial construction to a lower standard) or acceptance of the lower standard, particularly if alternative routes are available.

Similarly, it will not always be possible to provide the desired level of environmental protection. In such cases, a risk analysis may be required, detailing the costs of providing infrastructure versus those of not providing drainage infrastructure to the nominated standard.

13.1.8 MAINTENANCE CONSIDERATIONS

The provision for maintenance is an integral component of the planning and design phases of road drainage. Adequate maintenance is necessary for the proper operation of the drainage system. The lack of maintenance is one of the most common causes of failure of drainage systems (and erosion and sediment controls). This may be attributed to reasons such as a significant reduction in hydraulic or storage capacity (such as blockage by debris or sediment). Inspection, mowing, channel clearing and repair, cleaning out culverts and repairs to protective treatments are just some of the maintenance operations that need to be easily and safely undertaken over the life of the structure.

13.1.9 SAFETY CONSIDERATIONS

An integral aspect of the detailed design of all road drainage systems is the underlying consideration of safety. Some of the safety issues that require consideration as part of the road drainage design process, excluding workplace health and safety issues are described below.

i. Maintenance Access: - Safe access needs to be provided to all drainage structures that require either ongoing (e.g., moving of drains) or occasional (e.g., removal of debris) maintenance. This access is required for vehicles and maintenance crews depending on the type of maintenance that will be undertaken. Safe access to erosion and sediment control devices during the construction phase should also be allowed.
ii. Human Safety: - Where long culverts potentially provide a hazard (particularly in urban areas) to human safety, preventative measures should be considered. Safety measures include fencing, swing gates and grates at culvert inlets. Any safety device needs to ensure that it prevents both accesses to the culvert and trapping of people against the grate. The effect of any proposed human safety measure on culvert capacity and efficiency needs to be checked.
iii. Traffic Safety: - Projecting culvert ends have the potential to act as obstructions to ‘out of control’ vehicles. Where there are no safety barriers; culvert ends should be designed so as not to present an obstruction. If obstructions from projecting culverts or head walls are unavoidable, then safety barriers should be considered.
iv. Ford Safety: - The main issue associated with safety at fords is adequate sight distance for drivers to ensure vehicles can stop before entering the ford. Preferably, the ford longitudinal profile should be horizontal so that the same depth of water exists over the entire ford length. The ford length should be limited and be on a straight stretch of road where possible. Adequate permanent and temporary signing must be erected. As flood water recedes, silt and debris can be left on the road surface of a ford and this can be a hazard to road users.
v. Energy Dissipaters: - Energy dissipation is necessary due to high flow velocities. Dissipation devices usually consist of large obstructions to the flow and result in a high degree of turbulence. For these reasons, energy dissipation structures should be avoided in urban areas where possible. Otherwise, access should be limited by appropriate fencing. Energy dissipaters are also very costly to build and maintain and changes to the design, such as flattening of channel to reduce high velocities, is preferred.

13.2 HYDROLOGY

Hydrology is the study of the properties, distribution, and effects of water on the earth's surface, and in the soils, underlying rocks, and atmosphere. For the purpose of this guide, hydrology will deal with estimating flood magnitudes as the result of precipitation. In the design of highway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic meters per second (m^3/s) and hydrographs as discharge per time. For structures that are designed to control volume of runoff, like detention storage facilities, or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest.

The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of highway drainage structures. Errors in the estimates will result in a structure that is either undersized and causes more drainage problems or oversized and costs more than necessary. On the other hand, it is important to realise that any hydrologic analysis is only estimation. Although some hydraulics analysis is necessary for all highway drainage structure design, the extent of such studies should be commensurate with the hazards associated with the hydraulics structures and with other concerns, including economic, engineering, social, and environmental factors. Because hydrology is not an exact science, different hydrologic flow estimation methods developed for determining flood runoff may produce different results for a particular situation. Therefore, the engineer should exercise sound engineering judgment to select the proper flow estimation method or methods in estimation design flows. While performing the hydrological and hydraulics analysis for the design of highway drainage systems, the hydraulic engineer should recognize and evaluate potential environmental problems that would impact the specific design of a drainage structure early in the design process.

13.2.1 FACTORS AFFECTING FLOOD RUNOFF

i. Runoff: Two main factors influence runoff from a watershed: precipitation and abstractions. Precipitation in Ghana is represented as rainfall. Rainfall rate distributions within a watershed vary both temporally and spatially. For most determinations of peak flows for use in road drainage design and analysis efforts, it is commonly assumed that rainfall rates not to vary within the watershed during the rainfall event. However, this assumption only holds true for small and medium size catchments. Generally, the entire volume of rainfall occurring on a watershed does not appear as runoff. Losses, known as abstractions, tend to reduce the volume of water appearing as runoff. Abstractions of precipitation in its evolution into runoff are numerous. However, for the typical highway drainage design problem, only six abstractions are commonly considered. These are discussed below in the order of their significance to the rainfall runoff.
ii. Infiltration: Infiltration is the amount of precipitation that percolates into the ground within the watershed. This abstraction is a function of soil type and characteristics, terrain slopes, and ground cover. In order to assess infiltration, detailed topographical, soil data and land cover/use map is required.
iii. Depression storage: Depression storage is the precipitation stored permanently in inescapable depressions within the watershed. It is a function of land use, ground cover, and general topography
iv. Detention storage: Detention storage is the precipitation stored temporarily in streams, channels, and reservoirs within the watershed. It is a function of the general drainage network of streams, channels, ponds, etc. within the watershed.
v. Interception: Interception is the precipitation that serves to first “wet” the physical features of the watershed (e.g., leaves, rooftops, pavements). It is a function of most watershed characteristics.
vi. Evaporation: Evaporation is the precipitation that returns to the atmosphere as water vapor by the process of evaporation from water concentrations. It is mostly a function of climate factors, but it is associated with exposed areas of water surface.
vii. Transpiration: Transpiration is the natural process of vegetation foliage which generates this. It is a function of ground cover and density of vegetation.

viii. Watershed Area Information: Most runoff estimation techniques use the size of the contributing watershed as a principal factor. Generally, runoff rates and volumes increase with increasing drainage area. The size of a watershed will not usually change over the service life of the road drainage facility. However, agricultural activity and land development may cause the watershed area to change over time. Flow diversions and catchment area changes due to urbanization and other development inevitably will also occur at some point in the future. The drainage designer should try to identify or otherwise anticipate such changes. Urbanization, deforestation, infrastructure, development including roads, railways, and water resources projects (dams and reservoirs) will be anticipated to occur in the future. This should be taken into consideration and consultation with the district/municipal and regional government offices is required in order to establish and identify areas allocated for future development. On the other hand, the watershed shape will also affect rainfall runoff rates. For example, a long, narrow watershed is likely to experience lower runoff rates than a short, wide watershed of the same size and other characteristics. Some hydrologic methods accommodate watershed shape explicitly or implicitly; others may not. If a drainage area is unusually irregular extremely narrow, the designer should consider using a hydrologic method that explicitly accommodates this watershed shape.

ix. Geographic Location: The geographic location of the watershed within Ghana is a significant factor for the drainage designer. Rainfall intensities and distributions, empirical hydrologic relations, and hydrologic method applications vary because of geographic location.

x. Land Use: Land use significantly affects the parameters of a runoff event. Land use and human activity within most watersheds vary with respect to time. For example, a rural watershed can be developed into a commercial area in a short period. Factors subject to change with general variations in land use include the following:
   a. Permeable and impermeable areas.
   b. Vegetation
   c. Minor topographic features; and
   d. Drainage systems.

   All of these factors usually affect the rate and volume of runoff that may be expected from a watershed. Therefore, it is important to consider current land use and future potential land use change in the development of the parameters of any runoff hydrograph.

xi. Soil Type: Soil type can have considerable effect on the discharge rates of the runoff hydrograph; the soil type directly affects the permeability of the soil and thus the rate of rainfall infiltration. The Natural Resources Conservation Service (NRCS) is a good repository for information about soils but the soil parameters specified in NRCS should be calibrated and validated with site-specific local data within Ghana before it can be used in hydrological analysis.

xii. Topography: Topography mostly affects the rate at which runoff occurs. The rate of runoff increases with increasing slope. Furthermore, rates of runoff decrease with increasing depression storage and detention storage volumes. Many methods incorporate a watershed slope factor, but fewer methods allow the designer to consider the effects of storage on runoff. The drainage designer should take this limitation of the chosen method into consideration.

xiii. Vegetation: In general, runoff decreases with increasing density of vegetation; vegetation helps to reduce antecedent soil moisture conditions and increases interception such as to increase initial rainfall abstractions. Vegetative characteristics can vary significantly with the land use; therefore, consider this in the assessment of potential future land use changes of the watershed.

xiv. Detention Storage Systems: Detention storage systems are mostly aimed at controlling increased runoff from developed areas. The drainage designer should identify any detention storage systems that might exist within the subject watershed. A detention storage facility can attenuate the runoff hydrograph, thus reducing the peak discharge.

xv. Flow Diversions: Flow diversions within a watershed can change the runoff travel times and subsequent peak discharge rates. They can decrease discharge at some locations and increase discharge elsewhere. Flow diversions may redirect flow away from a location during light rainfall but overflow during heavy rainfall. Assess the likely effect of diversions that exist within the watershed. Also, ensure that the potential impact of necessary diversions resulting from the highway project is minimized.

xvi. Channelization: Channelization in an urban area includes the following:

  e. Improved open channels
  f. Kerb and gutter street sections
  g. Inverted crown street sections; and
  h. Storm drain systems.

Any of these channelization types serve to make drainage more efficient. This means that flows in areas with urban channelization can be greater, and peak discharges occur much more quickly than where no significant channelization exists.

xvii. Future Conditions: Changes in watershed characteristics and climate directly affect runoff rates. A reasonable service life of a designed system is expected. Therefore, base the estimate of design flood upon runoff influences within the time of the anticipated service life of the facility.

xviii. Prediction Information: In general, consider estimates for future land use and watershed character within some future range. It is difficult to predict the future, but the designer should make an effort at such a prediction, especially with regard to watershed characteristics. District and municipal officials and planners can often provide information on potential future characteristics of the watershed. In estimating future characteristics of the watershed, consider changes in vegetative cover, surface permeability, and controlled drainage systems. Climatic changes usually occur over extremely long periods of time however, it is reasonable to consider potential climatic changes during the anticipated life span of the facility.

13.2.2 DESIGN FREQUENCY

As with other natural phenomena, occurrence of flooding is governed by chance. The chance of flooding is described by a statistical analysis of flooding history in the subject watershed or in similar watersheds. Because it is not economically feasible to design a structure for the maximum possible runoff from a watershed, the designer must choose a design frequency appropriate for the structure.

All proposed structures are sized using the specified design frequency as provided in **Table 13.2**.

Table 13.2 Design and Check Storm Frequency (years) by Design Class

Structure type	Design Class									
	A1/A2		B1/B2		C1/C2		D1/D2		E	
	Storm Frequency (years)									
	Design	Check	Design	Check	Design	Check	Design	Check	Design	Check
Gutters and Inlets	10	25	10	25	10	25	5	10	5	10
Side Ditches	10	25	10	25	10	25	5	10	5	10
Bridge Deck (small and medium bridges)	10	25	10	25	10	25	5	10	5	10
Bridge Deck (large bridge)	25	50	25	50	25	50	10	25	10	25
Vented Fords							5	10	5	10
Minor Culverts Span ≤ 2 m	25	50	25	50	25	50	10	25	10	25
Major Culverts 2m < Span ≤ 6m	50	100	50	100	25	50	25	50	25	50
Short Span Bridges 6m < span ≤ 15	50	100	50	100	25	50	25	50	25	50
Medium Span Bridges 15m < span ≤ 50m	100	200	100	200	100	200	50	100	50	100
Long Span Bridges span >50m	100	200	100	200	100	200	50	100	50	100

Note: Span in the above table is the total clear-opening length of a structure. For example, the span for a double 1.2m diameter pipe is 2.4m, and the design storm frequency is therefore "major culvert, 2m<span≤6m." Similarly, a double box culvert having two 4.5m barrels should use the applicable design storm frequency for a short span bridge and a bridge having two 10m spans is a medium span bridge.

A design frequency shall be selected to match the facility's cost, amount of traffic, potential flood hazard to property, expected level of service, political considerations, and budgetary constraints, considering the magnitude and risk associated with damages from larger flood events. With long highway routes having no practical detour, where many sites are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. In selecting a design frequency, potential upstream land use changes that could reasonably occur over the anticipated life of the drainage facility shall be considered.

The frequency with which a given flood can be expected to occur is the reciprocal of the probability or chance that the flood will be equalled or exceeded in a given year. If a flood has a 20 percent chance of being equalled or exceeded each year, over a long period of time, the flood will be equalled or exceeded on an average of once every five years. This is called the return period or average recurrence interval (ARI). Thus, the exceedance probability equals

100/ARI.

Average recurrence interval is defined as the average interval in years between exceedances of a specified event (rainfall or discharge). The word “average” is the important part of the definition of recurrence interval. As hydrological events are generally random in their occurrences, it cannot be inferred that a flow of particular average recurrence interval is equalled or exceeded at regular interval. This important point is also to be explained to decision makers and to the public at large who are affected by them. For example, a 5-year flood is not one that will necessarily be equalled or exceeded every five years. There is a 20 percent chance that the flood will be equalled or exceeded in any year; therefore, the 5-year flood could conceivably occur in several consecutive years. The same reasoning applies to floods with other return periods.

Exceedance probability is the probability that an event of a given average recurrence interval will be equalled or exceeded during a given period of time. Consideration of the probability of exceedance during a given span of time, particularly the design life of a structure can be used as an aid in the selection of the level of serviceability to be provided.

Mathematically, the probability of non-occurrence and occurrence is given by **Equations 13.1** and **13.2** respectively.

$$P_n = \left(1 - \frac{1}{f}\right)^n \qquad (13.1)$$

$$P = 1 - P_n \qquad (13.2)$$

Where,

f = year flood

n = return period

For example, the chance of a 50-year flood, f, or greater event occurring in any 10-year period, n, is: $P = 1 - (1-1/50)^{10} = 0.18$

This is an 18% chance of occurrence and an 82% chance of non-occurrence. The equation can be applied to floods with other return periods.

The annual exceedance probability (AEP) is also used in relation to flood flows. The AEP is the probability of exceedance of a given discharge within a period of one year. It is commonly considered as the reciprocal of the average recurrence interval (ARI) in years expressed as a percentage (e.g., 50 years ARI = 2% AEP).

13.2.3 CHECK FREQUENCY

From the standpoint of drainage structure utilization, design a structure that will operate in the following manner:

i. Efficiently for lesser floods
ii. Adequately for the design flood; and
iii. Acceptably for greater floods.

After sizing a drainage facility using the peak flow for the design frequency, it is necessary to evaluate the proposed facility with a review flood or check flood. This is done to ensure that no unexpected flood hazards are inherent in the proposed facility. The review (check) flood shall be at least as provided in **Table 13.2**. In some cases, a flood event larger than the specified review flood might be used for analysis to ensure the safety of the drainage structure and downstream communities.

Where roadway or property inundation and associated damage is judged to be severe, a higher design frequency should be considered. A superflood [Any flood or tidal flow with a flow rate greater than that of the 100-year flood or the 500-year flood or a designated ratio (e.g., 1.7) times the 100-year flood] may be used to evaluate sites that have large potential risks or substantial initial costs.

If a catastrophic failure of a bridge or culvert can release a flood wave that would result in loss of life, disruption of essential services, or excessive economic damage, the bridge or culvert design should be evaluated in terms of a probable maximum flood or PMF. For example, a culvert under normal flood operation will act like a dam. PMF considers the conditions under which the culvert/dam may fail. The PMF is not related to an event frequency but is a specialized analysis.

13.3 HYDROLOGIC ANALYSIS METHOD

Stream flow measurements for determining a flood frequency relationship at or near a site are usually unavailable. In such cases, it is an accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. In general, results from using several methods should be compared, not averaged. The discharge that best reflects local project conditions, with the reasons documented, should be used. In general, follow the following guidelines to select Flow Estimation Methods (FEMs):

i. Compare results from several methods
ii. Use the discharge that appears to best reflect local project conditions. Averaging of results of several methods is not recommended; and
iii. Document reasons supporting the selection of the results.

The peak discharge is adequate for design of conveyance systems such as storm drains, open channels, culverts, and bridges. However, if the design necessitates flood routing through areas such as storage basins and complex conveyance networks, a flood hydrograph is required.

Many hydrologic methods are available for estimating peak discharges and runoff hydrographs. The omission of other flow estimation methods from this guide does not necessarily preclude their use. The methods to be used and the circumstances for their use are listed in **Table 13.3**. Each method has a range of application and limitations, which the engineer should clearly understand prior to using them. Basin size, hydrologic and geographic region, dominant precipitation type, elevation, and level of development are all important factors. The engineer must ensure that the selected hydrologic method is appropriate for the basin conditions and that sufficient data is available to perform the required calculations. If possible, the method should be calibrated to local conditions and flood history. Several methods will be appropriate for predicting peak flood rates and volumes at most sites. Comparison of hydrologic prediction methods on recurrence interval curves should be performed in selection of peak flow rates for a drainage design.

The following are some of the most widely used flow estimation methods:

i. Rational Method
ii. Modified Rational Method
iii. NRCS Runoff Curve Number Method
iv. Statistical analysis of stream data
v. Hydrograph Method.

Table 13.3 Application and limitation of flood estimation methods

Method	Input data	Recommended maximum area (km^2)	Return period of flood that could be determined (years)
Rational Method	Catchment area, watercourse length, average slope, catchment characteristics, rainfall intensity	≤1.0	2 – 200
Modified Rational Method		No limitation, large areas	2 – 200
NRCS Method	Catchment area, watercourse length, length to catchment centroid (centre), mean annual rainfall, veg. type, soil cover and synthetic regional unit hydrograph	0.5 to 65	2 – 200
Statistical Analysis of stream data	Historical flood peak records	No limitation, large areas	2 – 200 (depending on the record length)
Hydrograph Method	Catchment area, watercourse length, length to catchment centroid (centre), mean annual rainfall, veg. type and synthetic regional unit hydrograph	0.5 to 5000	2 -200

13.3.1 RATIONAL AND MODIFIED RATIONAL METHODS

The Rational Method provides estimates of peak runoff rates for small urban and rural watersheds of less than 1 square km and in which natural or man-made storage is small. It is a fairly simple and accurate method especially when the basin is primarily impervious. It is best suited to the design of urban storm drain systems, small side ditches and median ditches, and driveway pipes. It shall be used with caution if the time of concentration exceeds 30 minutes. Rainfall is a necessary input for this method of flow estimation.

The Rational Method formula is expressed by **Equation 13.3** as follows:

$$Q = \frac{CIA}{3.6} \qquad (13.3)$$

Where,

Q: Maximum run-off from the catchment area (m^3/s)

C: Runoff coefficient (a coefficient which represents ratio of runoff rainfall)

I: Average rainfall intensity (mm/h)

A: Catchment area (km^2)

This formula is based on a series of theoretical simplifications needed to justify this implied direct linear relationship between rainfall and runoff.

These include assumptions that:

i. The peak flow at the inlet of the cross drainage structure is reached at the moment that the entire watershed is contributing to that flow. This is known as the Time of Concentration (t_c)
ii. The Rainfall Intensity associated with a storm varies with the applicable t_c, but is otherwise uniform over time.
iii. The Runoff Coefficient as estimated for a specific catchment is independent of other variables such as the moisture content of the soil, and the rainfall intensity and duration.

None of these assumptions holds strictly true, and the application of the method to large catchments can result in significant over-estimation of peak flows. Nevertheless, they are sufficiently valid to allow the Rational Method to be used in the Ghana context for catchments of up to 1km^2. Beyond this, the inclusion of an Area Reduction Factor (ARF) results in the **Modified Rational Method**, which can be used for larger catchments. The modified rational method is given by **Equation 13.4**. The ARF is obtained from **Equation 13.5**.

$$Q = ARF\frac{CIA}{3.6} \tag{13.4}$$

$$ARF = 1 - \left(0.04t^{-\frac{1}{3}} A^{\frac{1}{2}}\right) \tag{13.5}$$

Where,

Q: Maximum run-off from the catchment area (m^3/s)

C: Runoff coefficient (a coefficient which represents ratio of runoff rainfall)

I: Average rainfall intensity (mm/h)

A: Catchment area (km^2)

ARF: Area reduction factor

t: storm duration in hours (equal to t_c).

13.3.1.1 RUN-OFF COEFFICIENT

The runoff coefficient "C" represents the percentage of rainfall that becomes runoff. The Rational method implies that this ratio is fixed for a given drainage basin. In reality, the coefficient may vary with respect to prior wetting and seasonal conditions. The use of an average coefficient for various surface types is quite common and it is assumed to stay constant through the duration of the rainstorm. This Guide recommends **Table 13.4** and **Table 13.5** for drainage facilities with low rainfall probability years such as road surface drainage facilities, and **Table 13.6** for facilities with relatively high rainfall probability years such as culverts and bridges.

If the land use is not the same within the catchment, use a weighted average value based on the percentage of the land area that it comprises. The weighted (composite) C value is given by **Equation 13.6**.

$$C_w = \frac{C_1A_1 + C_2A_2 + \cdots + C_iA_i}{A_1 + A_2 + \cdots + A_i} \tag{13.6}$$

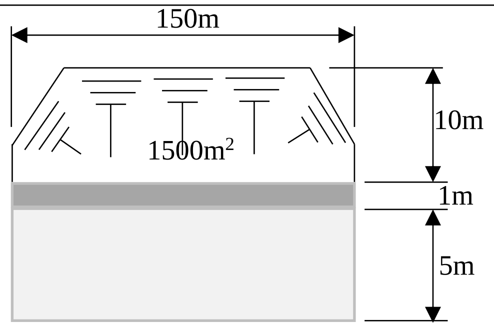
Where,

A_1, A_2 and A_i = subareas within the watershed, km^2.

C_1, C_2, and C_i = corresponding C values for the subareas $A_1 + A_2 + \ldots + A_i$

C_w = Weighted (composite) C

Figure 13.2 is an example of weighted coefficient of runoff calculation.



<Condition>

Shoulder surface: Gravel (C=0.70)

Slope: Grass (C=0.25)

Road surface: Pavement (C=0.95)

$$C = \frac{(5 \times 150 \times 0.95) + (1 \times 150 \times 0.70) + (10 \times 150 \times 0.25)}{(1 + 5) \times 150 + 1500} = 0.495 \cong 0.50$$

Figure 13.2 Example of weighted coefficient of run-off calculation

Table 13.4 Run-off coefficient by type of surface

Type of surface		**Run-off coefficient (c)**
Road surface including shoulder	Paved	0.70-0.95
	Gravel	0.30-0.70
Shoulder, slope	Fine grained soil (Clay, Silt)	0.40-0.65
	Coarse grained soil (Sand, Gravel)	0.10-0.30
	Hard rock	0.70-0.85
	Soft rock	0.50-0.75
Grass on sandy soil	Gradient 0-2%	0.05-0.10
	Gradient 2-7%	0.10-0.15
	Gradient more than 7%	0.15-0.20
Grass on clayey soil	Gradient 0-2%	0.13-0.17
	Gradient 2-7%	0.18-0.22
	Gradient more than 7%	0.25-0.35
Roof Garden, vegetation Grass, Park with lots of green Loose slope mountains Steep slope mountains		0.75-0.95 0.20-0.40 0.10-0.25 0.20-0.40 0.40-0.60
Rice field, pond Farm		0.70-0.80 0.10-0.30

Table 13.5 Run-off coefficient by type of land use

Land use		Run-off coefficient (c)
Commercial District	Downtown	0.70-0.95
	Adjacent area of downtown	0.50-0.70
Industrial District	Areas that are not too densely populated	0.50-0.80
	Not densely populated areas	0.60-0.90
Residential District	Residential area with few landscaped gardens	0.65-0.80
	Residential area	0.50-0.70
	Residential area with many landscaped gardens	0.30-0.50
Green space	Park, cemetery	0.10-0.25
	Stadium	0.20-0.35
	Traffic Terminal	0.20-0.40
	Rice field, farm, woods	0.10-0.30

Table 13.6 Run-off coefficient by terrain

Terrain	Run-off coefficient (c)
Road and slope surface	0.70-1.00
Steep mountainous area	0.75-0.90
Loose mountainous area	0.70-0.80
Rugged land and forests	0.50-0.75
Flat arable area	0.45-0.60
Rice field with water ponded	0.70-0.80
Urban area	0.60-0.90
Forest area	0.20-0.40
Watershed on mountainous area	0.75-0.85
Watershed on flat and small rivers	0.45-0.75
Large rivers with more than half of the land flat	0.50-0.75

13.3.1.2 TIME OF CONCENTRATION

The time of concentration, (t_c), is the period of time required for runoff to flow from hydraulically most distant point, within the tributary area, to the point under consideration. Although usually the most hydraulically remote point is located at the drainage basin divide, it is not necessarily the point having the longest distance to the outlet. After rainfall begins, the entire drainage basin starts to contribute flows at the outlet at Time of Concentration. The time of concentration is used to estimate the design rainfall intensity from the rainfall intensity-duration-frequency curve for the area of study.

A storm equal to this duration will permit direct runoff to arrive from all points in the watershed concentrating at the outlet. This time measure is taken to be the critical time by many flood-estimating approaches, in that it is assumed that the use of any other time would result in a lower flood estimate. A shorter time, although resulting in higher rainfall intensity, will not permit the entire basin to contribute flow simultaneously. A longer duration allows the entire basin to contribute, but with a lower intensity.

The time of concentration usually has two components. The first component is the initial time, t_i, which is the time runoff is sheet flowing (overland flow) as shown in **Figure 13.3**. The second component, the travel time, t_t, is the time runoff is in a channel. The summation of these two components as given in **Equation 13.7** gives the t_c.

$$t_c = t_i + t_t \tag{13.7}$$

Where,

t_c = time of concentration, min.

t_i = overland flow time, min.

t_t = travel time through ditch, pipe, channel etc, min.

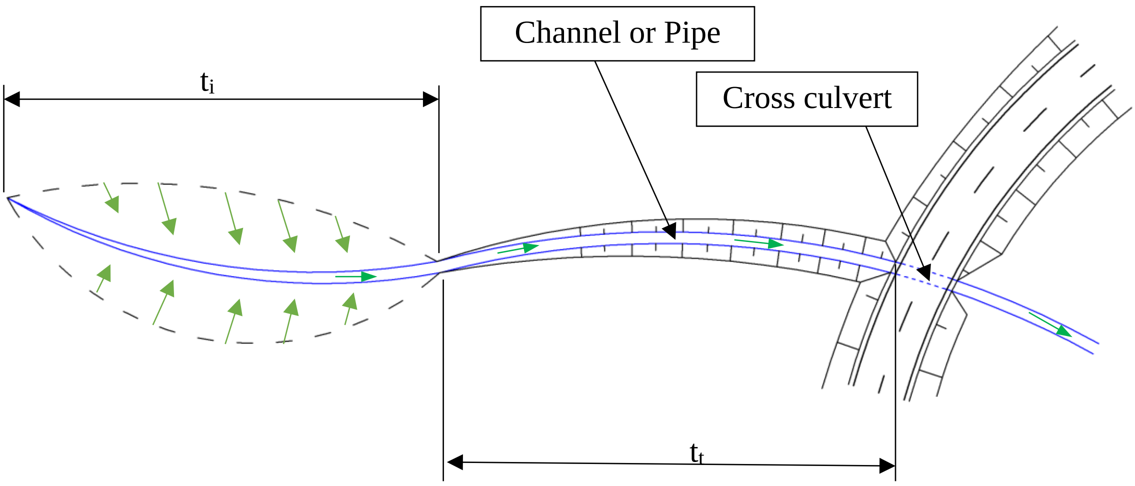


Figure 13.3 Components of time of concentration (T_c)

It is recommended in this guide that for the design of most drainage structures, the minimum time of concentration is taken as 12 minutes. The design of gutters and inlets may be based on shorter rainfall durations (runoff is assumed to have travelled from the centreline of the road, therefore, in such designs 5 minutes is recommended as the minimum), but this isn't serious conservatism. The overall drainage system - drainage conduit - will usually be designed for storm duration of nearly 12 minutes or more, thus the most expensive part of the drainage system will not be unnecessarily over-designed.

13.3.1.2.1 SELECTION OF METHOD

There are many empirical and semi-empirical methods that are in frequent application to estimate the time of concentration. The user of the guide is to choose the appropriate one/s for the case/s under consideration taking into account the physical characteristics, hydro meteorological conditions, and the dominant parameters of each formula as it relates to the drainage area being evaluated and the type of facility being designed. Time of concentration calculation using multiple methods should always be carried out to cross-check each other's results. The results from different methods should not be averaged.

13.3.1.2.2 NRCS TR-55

This method is a velocity-based time of concentration method which divides a flow path into sections of sheet flow, shallow concentrated flow, and pipe and channel Flow, and each is calculated individually. The time of concentration is the sum of the travel times in all three flow paths.

i. **Sheet flow**

Sheet flow occurs in the headwaters, before any visible channels are present. TR-55 suggests a maximum of 130m for any sheet flow segment. Sheet flow is given by **Equation 13.8**.

$$T_{sh} = \frac{5.5}{P_2^{0.5}}\left(\frac{nL}{\sqrt{S}}\right)^{0.8} \tag{13.8}$$

Where,

T_{sh} = Sheet flow time (min)
n = Manning's roughness coefficient for overland flow (see **Table 13.7**)
L = flow length (m)
P_2 = 2-year, 24-hour rainfall depth (mm)
S = slope of the surface (m/m)

Table 13.7 Manning's Roughness Coefficient (n) for Overland and Sheet Flow

n	Surface Description
0.011	Smooth asphalt
0.012	Smooth concrete
0.013	Concrete lining
0.014	Good wood
0.014	Brick with cement mortar
0.015	Vitrified clay
0.015	Cast iron
0.024	Corrugated metal pipe
0.024	Cement rubble surface
0.050	Fallow (no residue)
	Cultivated soils
0.060	Residue cover ≤ 20%
0.170	Residue cover > 20%
0.130	Range (natural)
	Grass
0.150	Short grass prairie
0.240	Dense grasses
0.410	Bermuda grass
	Woods *
0.400	Light underbrush
0.800	Dense underbrush

*When selecting n for woody underbrush, consider cover to a height of about 30 mm. This is the only part of the plant cover that will obstruct sheet flow.

ii. Shallow Concentrated Flow

After short distances, sheet flow tends to concentrate in rills and then gullies of increasing proportions. Such flow is usually referred to as shallow concentrated flow. The velocity of such flow can be estimated using an empirical relationship between the velocity and the slope as provided in **Equation 13.9**.

$$V = 10kS^{0.5} \tag{13.9}$$

Where,

V = velocity, m/s
S = slope, m/m
k = dimensionless function of land cover (see **Table 13.8**)

Table 13.8 Intercept Coefficients for Velocity vs. Slope Relationship (McCuen, 1989)

k	Land Cover/Flow Regime
0.076	Forest with heavy ground litter; hay meadow (overland flow)
0.152	Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)
0.213	Short grass pasture (overland flow)
0.274	Cultivated straight row (overland flow)
0.305	Nearly bare and untilled (overland flow); alluvial fans in western mountain regions
0.457	Grassed waterway (shallow concentrated flow)
0.491	Unpaved (shallow concentrated flow)
0.619	Paved area (shallow concentrated flow); small upland gullies

From the flow velocity and length of shallow concentrated flow, you can find the travel time using standard physics as shown in **Equation 13.10**.

$$T = \frac{L}{60V} \tag{13.10}$$

Where,

T = Travel time (min)
L = flow length (m)
V = average velocity (m/s)

iii. Pipe and Channel Flow

Flow in gullies empties into channels or pipes. In many cases, the transition between shallow concentrated flow and open channels may be assumed to occur where either the blue-line stream is depicted on Topographical sheets (scale equals 1:50000) or when the channel is visible on aerial photographs. Channel lengths may be measured directly from the map or scale photograph. However, depending on the scale of the map and the sinuosity of the channel, a map-derived channel length may be an underestimate. Pipe lengths should be taken from as-built drawings for existing systems and design plans for future systems. Cross-section information (i.e., depth-area and roughness) can be obtained for any channel reach in the watershed. Manning's equation as given by **Equation 13.11** can be used to estimate average flow velocities in pipes and open channels.

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{13.11}$$

Where,

V = average velocity (m/s)
R = hydraulic radius (m) = Wetted Area (m^2) / Wetted Perimeter (m)
S = channel slope (m/m)
n = Manning’s roughness coefficient (see **Table 13.9**)

Equation 13.10 can be used to determine the travel time.

Table 13.9 Manning’s Roughness coefficient, n

Type	Condition	Range	Average
Culvert	In-situ concrete		0.015
	Concrete pipe		0.013
	Steel pipe		0.015
	Vinyl chloride pipe		0.010
	Precast concrete pipe		0.013
Lined open channel	Metal without painting but smooth	0.011-0.014	0.012
	Mortar	0.011-0.015	0.013
	Wood	0.012-0.018	0.015
	Concrete with bottom side gravel	0.015-0.020	0.017
	Stone masonry (mortal joint)	0.017-0.030	0.025
	Dry masonry	0.023-0.035	0.032
Unlined open channel	Soil in straight uniformed alignment	0.016-0.025	0.022
	Soil in straight alignment with weeds	0.022-0.033	0.027
	Gravel in straight alignment	0.022-0.030	0.025
	Rock in straight alignment	0.025-0.040	0.035
Natural channel	Uniformed	0.025-0.033	0.030
	Very irregular shape with a lot of trees and weeds	0.075-0.150	0.100
Asphalt Pavement	Smooth texture		0.013
	Rough texture		0.017
Concrete pavement	Float finish		0.014
	Broom finish		0.017
Concrete gutter	Trowelled finish		0.012
Concrete gutter-asphalt pavement	Smooth		0.013
	Rough		0.015

13.3.1.2.3 AIRPORT (FAA) METHOD

Back in 1965, the U.S. Federal Aviation Agency (FAA) developed a simple estimation of T_c to be used with the Rational Method. The FAA method, as it is commonly referred to, calculates the Time of Concentration over a uniform slope and surface cover (airfields, car park). The FAA method is probably most valid for small watersheds in an urban basin where sheet flow or overland flow controls. FAA method tends to overestimate inlet time if the inlet time flow path has a significant portion of shallow concentrated flow. For the reasons above, FAA method is not recommended for time of concentration calculation unless overland flow overwhelmingly dominates the entire flow path. It is given by **Equation 13.12**.

$$T_c = \frac{3.26(1.1 - C)L^{0.5}}{S^{0.333}} \qquad (13.12)$$

Where,

T_c = time of concentration (min)
C = Rational run-off coefficient
L = flow length (m)
S = catchment slope (%)

13.3.1.2.4 KINEMATIC WAVE EQUATION

When calculating the time of concentration for a catchment, the alternative to using an empirical formula is to calculate the surface runoff flow directly from overland flow formulas. The first part of runoff flow is the overland sheet flow which can be modelled using the kinematic wave equation as given in **Equation 13.13**. The wave formula is generally applicable for the first 100m or so of overland flow on shallow slopes. For steeper slopes or longer flow paths the surface runoff flow should be modelled as shallow concentrated flow or channel flow rather than overland sheet flow.

$$T_c = \frac{6.92}{i^{0.4}}\left(\frac{nL}{\sqrt{S}}\right)^{0.6} \qquad (13.13)$$

Where,

Tc = sheet flow travel time (min)
n = Manning's roughness coefficient
L = flow length (m)
i = rainfall intensity (mm/hr)
S = surface slope (m/m)

As the wave formula requires the rainfall intensity, the determination of the time of concentration must be done via an iterative approach. This is because the design rainfall intensity is usually dependent on the time of concentration. Therefore, an estimate must be made of the rainfall intensity to calculate a time of concentration, then this time of concentration must be used to recalculate the rainfall intensity. This process must usually be repeated several times before an accurate time of concentration value is reached.

The kinematic wave theory has been shown to be applicable only for turbulent flow and where the product of rainfall intensity (mm/hr) and flow length (m) is greater than 750. It should also only be used on planes that are fairly homogenous in terms of slope and roughness. For these reasons the kinematic wave model is most suitable for large paved areas such as car parks or airfields, not for heterogeneous rural catchments.

13.3.1.2.5 KERBY METHOD

The Kerby method was developed from a very small watershed, thus care should be used in applying it to larger watersheds. It is generally suggested that the upper limit should be a flow length (L) of about 365 m. Also, the Kerby method is meant for overland flow only, not flow through a channel. Therefore, if a significant amount of the time of concentration of your drainage basin consists of channel flow, it can be combined with the Kirpich equation. The Kerby method is given by **Equation 13.14**.

$$T_c = 1.445\left(\frac{n_d L}{\sqrt{S}}\right)^{0.467} \qquad (13.14)$$

Where,

T_c = time of concentration (minutes)
n_d = dimensionless retardance coefficient (see **Table 13.10**)
L = the overland flow length (m)
S = average slope (m/m)

Table 13.10 Coefficient of Retardation, n_d

Type of Surface	Coefficient of Retardation, n_d
Asphalt, concrete	0.013
Smooth and hard surface	0.02
Smooth and gravel surface	0.10
Arable land	0.20
General grass land	0.40
General forest land	0.60
Dense weed and dense forest zone	0.80

13.3.1.2.6 THE KIRPICH METHOD

The Kirpich equation is normally used for natural basins with well-defined channels. The Kirpich equation was developed from data for six agricultural watersheds in Tennessee, USA (ranging in size from 0.4 ha to 45 ha), with well-defined channels and slopes ranging from 3% to 10%.

If there are many undefined channels that are grassed or vegetated throughout, the Kirpich formula will likely underestimate the time of concentration, and a factor of 1.3-1.5 should be added or the Kerby formula could be combined with it to calculate the total time of concentration. Rossmiller (1980) has reviewed field application of Kirpich equation and suggested that for overland flow on asphalt surfaces, t_c should be multiplied by 0.4; on concrete surfaces by 0.2 and for general overland flow and flow in natural grass channels, by 2. The Kirpich method is given by **Equation 13.15**.

$$T_c = 0.0195\frac{L^{0.77}}{S^{0.385}} \qquad (13.15)$$

Where,

T_c = time of concentration (min)
L = Length of channel flow (m)
S = average slope (m/m)

13.3.1.2.7 IZZARD FORMULA

It was derived from experiments on overland flow on roadway and turf surfaces. The solution requires iteration similar to the kinematic wave equation. The product of intensity (mm/h) and the length of overland flow (m) should not be greater than 3800 (mm/h·m). It is given by **Equation 13.16**.

$$T_c = 525.2 \frac{L^{0.33}(2.76 \times 10^{-5} i + k)}{S^{0.33} \cdot i^{0.667}} \tag{13.16}$$

Where,

T_c = time of concentration (min)
L = Length of overland flow (m)
S = average slope (m/m)
k = Retardation coefficient (see **Table 13.11**)
i =Rainfall Intensity (mm/h)

Table 13.11 Izzard's retardation coefficient *k*

Type of Surface	Retardation coefficient k
Smooth asphalt	0.0070
Sandy tar pavement	0.0075
Slate	0.0082
Concrete	0.012
Gravelly tar pavement	0.017
Grass	0.060

13.3.1.2.8 FRIEND FORMULA

In urban areas, the length of overland flow will typically be less than 50 metres after which the flow will become concentrated against fence, paths or structures or intercepted by open drains. the Friend formula given by **Equation 13.17** can be used to estimate sheet (overland) flow times. It is recommended for surfaces with known Manning's roughness coefficient and good topographic map to estimate the slope. It is recommended to use before flow starts to concentrate into gully flows. The recommended maximum length of sheet flow to be adopted in this formula is 200 metres.

$$T_c = \frac{107 n L^{0.333}}{S^{0.2}} \tag{13.17}$$

Where,

T_c = time of concentration (min)
L = overland sheet flow path (m)
n =Manning's roughness value for the surface
S = average surface slope (%)

13.3.1.2.9 MODIFIED FRIEND FORMULA

For rural catchment areas less than 25 km^2, the time of concentration can be estimated using the modified Friend formula, assuming an estimated peak level of the design flood. This level is usually the maximum reported flood level or approximate bank level. If later hydraulic calculations show this to be in error by more than 0.3–0.6m, the value is recalculated. The modified Friend formula is given by **Equation 13.18**.

$$T_c = \frac{800L}{ChA^{0.1}S^{0.4}} \qquad (13.18)$$

Where,

T_c	= time of concentration (min)
L	= Length (km) of flow path from catchment divide to outlet
Ch	= Chezy's coefficient at the site = $(1/n)R^{1/6}$
R	= hydraulic radius = $0.75R_S$ where stream slope is fairly uniform
	= $0.65R_S$ where stream slope varies appreciably along the stream
R_S	= hydraulic radius at the initially assumed flood level at the site
n	= average Manning roughness coefficient for the entire stream length
A	= catchment area (ha)
S	= average stream flow path (%)

Because an initial overland flow component is incorporated into the modified Friend's equation, the addition of an overland flow travel time or standard inlet time is not required.

13.3.1.2.10 BRANSBY-WILLIAMS' EQUATION

For natural/landscaped rural catchments with mixed flow paths, the time of concentration can be found by the use of the Bransby-Williams' Equation. In these cases, the times for overland flow and channel or stream flow are included in the time calculated. Time of concentration based on the Bransby-Williams' Equation is given by **Equation 13.19**.

$$T_C = \frac{58.5L}{A^{0.1}S^{0.2}} \qquad (13.19)$$

Where,

T_c	= time of concentration (min)
L	= length (km) of flow path from catchment divide to outlet
A	= catchment area (km^2)
S	= slope of flow path (m/km)

Note the following:

i. **Equation 13.19** can only be used where the catchment is fully saturated, and factors such as channel storage causing flow attenuation are low. Resulting Times of Concentration 'T_c' are especially low for short duration events in catchments dominated by loess soils.

ii. In some catchments, due to shape, surface water network, and the varying permeabilities within the catchment, part of the catchment under consideration may produce a higher peak flow than the whole of the catchment. Although the area for the part catchment is smaller, this may be more than offset by the higher intensity storm associated with a shorter time of concentration and storm duration. This situation will arise where the upper reaches are rural, and the lower reaches of a catchment are densely developed.

iii. If the actual catchment slope varies significantly from the value ΔH/L (e.g. with a sudden steepening in the upper reaches), the time of concentration shall be determined by evaluating the component travel times for the hillside, and the flat.

13.3.1.3 RAINFALL INTENSITY

Rainfall along with catchment characteristics determines the flood flows upon which storm drainage design is based. Although in practice rainfall intensity varies during precipitation events, many of the procedures used to derive peak flow are based on an assumed constant rainfall intensity. Rainfall intensity, I, is defined as the rate of rainfall and is typically given in units of millimetres per hour (mm/hr). Intensity Duration Frequency (IDF) curves are normally provided by the Ghana Meteorological Agency (GMet) for use in the determination of rainfall intensity in a specific area within the country. Please refer to the MRH’s Draft Report on the Consultancy Services for Coordination of the Review of the Intensity Duration Frequency (IDF) of the Rainfall Intensity Graph, 2016, for the latest IDF curves for Ghana (Appendix C).

13.3.1.4 CATCHMENT AREA

Catchment Area, A, is any portion of the earth's surface within a physical boundary defined by topographic slopes that divert all runoff to the same drainage (basin) outlet. The physical boundary of the catchment (watershed) is called the drainage divide. The catchment area includes all the points that lie above the elevation of the outlet and within the drainage divide that separates adjacent catchments (watersheds).

Determining the size of the catchment area that contributes to flow at the drainage structure site is a basic step in a hydrologic analysis. The catchment area, usually expressed in hectares or km^2, is determined from field surveys, topographic maps, aerial photographs, satellite imagery, existing digital ground models or a combination of these items.

When manually delineating watersheds from a topographic map, drainage divides are located by analysing the contour lines. Arrows representing the flow directions are drawn perpendicular to each contour, in the direction of the steepest descent. The location of a divide is taken to be where flow directions diverge, or where the arrows point in opposite directions. The catchment area is be determined using a digital planimeter.

Manually locating the divide is a difficult process, and slight errors are unavoidable. The extent of these errors is dependent upon the engineer, and different engineers will likely present different results. On the contrary, watersheds and catchments determined through the use of raster data in a GIS software, will produce consistent results, regardless of the user. The accuracy of these results will also only be a function of the accuracy and resolution of the raster data. Even with this accuracy, manual editing of the raster-defined drainage boundaries is still necessary in order to obtain the best possible agreement with topographic data.

Figure 13.4 shows a catchment demarcated using GIS software.

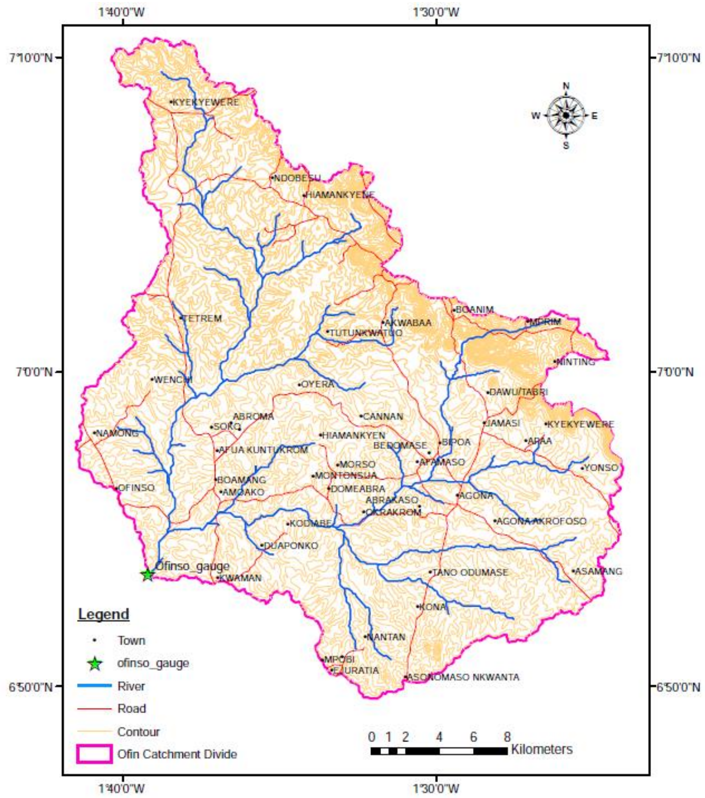


Figure 13.4 Catchment demarcation using GIS

13.3.2 NRCS RUNOFF CURVE NUMBER METHODS

The Natural Resources Conservation Service, NRCS (formerly Soil Conservation Service, SCS) developed the runoff curve number method as a means of estimating the amount of rainfall appearing as runoff. For many peak discharge estimation methods, the input includes variables to reflect the size of the contributing area, the amount of rainfall, the potential watershed storage, and the time-area distribution of the watershed. These are often translated into input variables such as the drainage area, the depth of rainfall, an index reflecting land use and soil type, and the time of concentration. The NRCS method is typical of many peak discharge methods that are based on input such as that described.

13.3.2.1 RUNOFF DEPTH ESTIMATION

The volume of storm runoff can depend on a number of factors. Certainly, the volume of rainfall will be an important factor. For very large watersheds, the volume of runoff from one storm event may depend on rainfall that occurred during previous storm events. However, when using the design storm approach, the assumption of storm independence is quite common. In addition to rainfall, other factors affect the volume of runoff. A common assumption in hydrologic modelling is that the rainfall available for runoff is separated into three parts: direct (or storm) runoff, initial abstraction, and losses. Factors that affect the split between losses and direct runoff include the volume of rainfall, land cover and use, soil type, and antecedent moisture conditions. Land cover and land use will determine the amount of depression and interception storage.

In developing the SCS rainfall-runoff relationship, the total rainfall was separated into three components: direct runoff (Q), actual retention (F), and the initial abstraction (I_a). The retention F was assumed to be a function of the depths of rainfall and runoff and the initial abstraction.

The initial abstraction includes the portion of rainfall that is not available for either infiltration or runoff and includes the portion of rainfall that is used to wet surfaces prior to reaching the ground (interception). The retention, F, is the portion of rainfall reaching the ground that is retained by the catchment and consists primarily of infiltrated volume. The SCS model is based on the basic assumption that for any rainfall event, the precipitation, P, the runoff, Q, the retention, F, and the initial abstraction, I_a, are related by **Equation (13.20)**.

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}, P > I_a \qquad (13.20)$$

Where,

P = depth of precipitation, mm

I_a = initial abstraction, mm

S = maximum potential retention, mm

Q = depth of direct runoff, mm

Given **Equation (13.20)**, two unknowns need to be estimated, S and I_a. The retention S should be a function of the following five factors: land use, interception, infiltration, depression storage, and antecedent moisture. Empirical evidence resulted in the **Equation (13.21)** for estimating the initial abstraction.

$$I_a = 0.2S \qquad (13.21)$$

If the five factors above affect S, they also affect I_a. Substituting **Equation (13.21)** into **Equation (13.20)** yields the following **Equation (13.22)**, which contains the single unknown S.

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}, P > 0.2S \qquad (13.22)$$

Equation (13.22) represents the basic equation for computing the runoff depth, Q for a given rainfall depth, P. Both P and Q are expressed in mm depth. They both reflect volumes because it is assumed that the same depths occurred over the entire catchment.

Additional empirical analyses were made to estimate the value of S. The studies found that S was related to soil type, land cover, and the hydrologic condition of the watershed. These are represented by the runoff curve number (CN), which is used to estimate S by **Equation (13.23)**.

$$S = 25.4\left(\frac{1000}{CN} - 10\right) \qquad (13.23)$$

Where,

S= maximum potential retention, mm

CN = index that represents the combination of a hydrologic soil group and a land use and treatment class.

The CN is a function of three factors: soil group, the cover complex, and antecedent moisture conditions.

Rearranging **Equation (13.23)** to make CN the subject results in **Equation (13.24)**.

$$CN = \frac{1000}{10 + 0.0394S} \qquad (13.24)$$

From **Equation (13.24)** it is clear that in the absence of available storage (S=0, i.e., impervious surface), the curve number is equal to 100; and for an infinite amount of storage, the curve number is equal to zero. The curve numbers to be used in Equation (13.24) varies between 0 and 100.

13.3.2.2 ANTECEDENT RUNOFF CONDITION (ARC)

Antecedent Runoff condition (ARC) or Antecedent Moisture content is a measure of the actual available storage relative to the average available storage at the beginning of the rainfall event. The antecedent runoff condition is grouped into three categories: ARC I, ARC II and ARC III. ARC I refers to land conditions drier than normal conditions while ARC III refers to land with wetter than normal conditions. The curve numbers cited for the above table are done for land areas corresponding to ARC II conditions, which indicate normal and average moisture condition of the land. The curve numbers can be adjusted for drier than normal ARC I or wetter than normal conditions ARC III by **Equation (13.25)** and **Equation (13.26)** respectively.

$$CN(I) = \frac{4.2CN(II)}{10 - 0.058CN(II)} \tag{13.25}$$

$$CN(III) = \frac{23CN(II)}{10 + 0.13CN(II)} \tag{13.26}$$

The range of antecedent moisture conditions for each class is provided in **Table 13.12**.

Table 13.12 Range of antecedent moisture conditions for each class

AMC group	Total 5-dayantecedentrainfall(cm)	
	Dormant season	Growing season
I	Less than 1.3	Less than 3.6
II	1.3 to 2.8	3.6 to 5.4
III	Over 2.8	Over 5.4

13.3.2.3 SOIL GROUP CLASSIFICATION

SCS developed a soil classification system that consists of four groups, which are identified by the letters A, B, C, and D. Soil characteristics that are associated with each group are provided in **Table 13.13**. Information on soils groups for Ghana can be obtained from the Soil Research Institute.

Table 13.13 Hydrologic Soil Groups of the SCS-CN method

GROUP	CSIR/FAO Soil Class	Description	Further description	Minimum Infiltration Rate (mm/hr)
A	Acrisols, Nitosols, Arenosols and Histosols	Deep sand; deep loess; aggregated silts	Sand, loamy sand and sandy loam types of soils. These soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission.	7.6 - 11
B	Alisols	Shallow loess; sandy loam	Silt, silt loam and loam soils. These soils have a moderate infiltration rate when thoroughly wetted and consists chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.	3.8 – 7.6
C	Leptosols, Gleysols, Cambisols, Fluvisols, Luvisols, Nitosols, Solonetz, Planosols	Clay loams; shallow sandy loam; soils low in organic content; soils usually high in clay	Sandy clay loam soils. They have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine structure.	1.3 – 3.8
D	Lixisols, Ferrosols, Vertisols	Soils that swell significantly when wet; heavy plastic clays; certain saline soils.	Clay loam, silty clay loam, sandy clay, silty clay, and clay soils. This group has the highest runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high-water table, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious material.	0 – 1.3

13.3.2.4 CURVE NUMBER TABLES

Table 13.14, **Table 13.15** and **Table 13.16** shows the SCS CN values for the different land uses, treatments, and hydrologic conditions; separate values are given for each soil group. For example, the CN for a wooded area with good cover and soil group B is 55; for soil group C, the CN would increase to 70. If the cover (on soil group B) is poor, the CN will be 66.

Table 13.14 Runoff Curve Numbers (average watershed condition, $I_a = 0.2S$)

Cover Type		Curve Numbers for Hydrologic Soil Group			
		A	**B**	**C**	**D**
Fully developed urban areas[a] (vegetation established)					
Lawns, open spaces, parks, golf courses, cemeteries, etc.					
Good condition; grass cover on 75% or more of the area		39	61	74	80
Fair condition; grass cover on 50% to 75% of the area		49	69	79	84
Poor condition; grass cover on 50% or less of the area		68	79	86	89
Paved parking lots, roofs, driveways, etc. (excl. right-of- way)		98	98	98	98
Streets and roads					
Paved with kerbs and storm sewers (excl. right-of-way)		98	98	98	98
Gravel (incl. right-of-way)		76	85	89	91
Dirt (incl. right-of-way)		72	82	87	89
Paved with open ditches (incl. right-of-way)		83	89	92	93
	Average % impervious[b]				
Commercial and business areas	85	89	92	94	95
Industrial districts	72	81	88	91	93
Row houses, town houses, and residential with lots sizes 0.05 ha or less (0.12 acres or less)	65	77	85	90	92
Residential: average lot size					
0.1 ha (0.25 acres)	38	61	75	83	87
0.135 ha (0.33 acres)	30	57	72	81	86
0.2 ha (0.5 acres)	25	54	70	80	85
0.4 ha (1.0 acres)	20	51	68	79	84
0.8 ha (2.0 acres)	12	46	65	77	82
Developing urban areas[c] (no vegetation established) Newly graded area		77	86	91	94

Table 13.15 Runoff Curve Numbers (Cont'd)

Cover Type		Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group			
			A	B	C	D
Cultivated Agricultural Land: Fallow						
Straight row or bare soil			77	86	91	94
Conservation tillage		Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row	Poor	72	81	88	91
		Good	67	78	85	89
	Conservation tillage	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured	Poor	70	79	84	88
		Good	65	75	82	86
	Contoured and tillage	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraces	Poor	66	74	80	82
		Good	62	71	78	81
	Contoured and terraces and conservation tillage	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Conservation tillage	Poor	64	75	83	86
		Good	60	72	80	84
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured and tillage	Poor	62	73	81	84
		Good	60	72	80	83
	Contoured and terraces	Poor	61	72	79	82
		Good	59	70	78	81
	Contoured and terraces and conservation tillage	Poor	60	71	78	81
		Good	58	69	77	80

Cover Type		Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group			
			A	B	C	D
Close-seeded or broadcast legumes or rotation meadows[e]	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
	Contoured and terraces	Poor	63	73	80	83
		Good	57	67	76	80
Non-cultivated agricultural land						
Pasture or range	No Mechanical treatment[i]	Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
		Fair	25	59	75	83
		Good	6	35	70	79
Meadow - continuous grass, protected from grazing and generally mowed for hay			30	58	71	78

Table 13.16 Runoff Curve Numbers (Cont'd)

Cover Type	Hydrologic Condition[d]	Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Forestland - grass or orchards - evergreen or Deciduous	Poor	55	73	82	86
	Fair	44	65	76	82
	Good	32	58	72	79
Brush - brush-weed-grass mixture with brush the major element[g]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[f]	48	65	73
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[f]	55	70	77
Woods - grass combination (orchard or tree farm) [h]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Farmsteads		59	74	82	86
Forest-range					
Herbaceous - mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen - mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon - juniper - pinyon, juniper, or both grass understory)	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sage-grass	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub - major plants include saltbush, greasewood, creosote bush, blackbrush, bursage, palo verde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

a For land uses with impervious areas, curve numbers are computed assuming that 100 percent of runoff from impervious areas is directly connected to the drainage system. Pervious areas (lawn) are considered to be equivalent to lawns in good condition and the impervious areas have a CN of 98.

b Includes paved streets.

c Use for the design of temporary measures during grading and construction. Impervious area percent for urban areas under development vary considerably. The user will determine the percent impervious. Then using the newly graded area CN, the composite CN can be computed for any degree of development.

d For conservation tillage poor hydrologic condition, 5 to 20 percent of the surface is covered with residue (less than 850 kg/ha row crops or 350 kg/ha small grain).
For conservation tillage good hydrologic condition, more than 20 percent of the surface is covered with residue (greater than 850 kg/ha row crops or 350 kg/ha small grain).

e Close-drilled or broadcast.
For non-cultivated agricultural land:
Poor hydrologic condition has less than 25 percent ground cover density.
Fair hydrologic condition has between 25 and 50 percent ground cover density.
Good hydrologic condition has more than 50 percent ground cover density.
For forest-range.
Poor hydrologic condition has less than 30 percent ground cover density.
Fair hydrologic condition has between 30 and 70 percent ground cover density.
Good hydrologic condition has more than 70 percent ground cover density.

f Actual curve number is less than 30: use CN = 30 for runoff computations.

g CNs shown were computed for areas with 50 percent woods and 50 percent grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

h Poor: < 50 percent ground cover.
Fair: 50 to 75 percent ground cover.
Good: > 75 percent ground cover.

i Poor: < 50 percent ground cover or heavily grazed with no mulch.
Fair: 50 to 75 percent ground cover and not heavily grazed.
Good: > 75 percent ground cover and lightly or only occasionally grazed.

In the delineation of the soil types, if more than one soil group is involved, the limits of each group should be outlined on the drainage area map to aid in computing the CN for each land

use or ground cover. Whenever detailed plans are available for either existing or proposed developed areas, the drainage area should be reduced to roof areas, various types of paved areas, and areas of various vegetative types. If detailed plans are not available, the watershed can be reduced to various types of development, i.e., residential, industrial or commercial.
Where heterogeneous drainage areas are encountered, a weighted value of CN will be used.

The formula for the weighted curve number (CN_w) is provided in **Equation (13.27)**.

$$CN_w = \frac{\sum_{i=1}^{n} CN_i A_i}{A} \tag{13.27}$$

Where, CN_i and A_i are the curve numbers and area for cover type i to the nth type respectively and A is the total drainage area.

13.3.2.5 ESTIMATION OF CN VALUES FOR URBAN LAND USES

The CN table (**Table 13.14**) includes CN values for a number of urban land uses. For each of these, the CN is based on a specific percentage of imperviousness. For example, the CN values for commercial land use are based on an imperviousness of 85 percent. Curve numbers for other percentages of imperviousness can be computed using a weighted CN approach, with a CN of 98 used for the impervious areas and the CN for open space (good condition) used for the pervious portion of the area. Thus, CN values of 39, 61, 74, and 80 are used for hydrologic soil groups A, B, C, and D, respectively. These are the same CN values for pasture in good condition. Thus the **Equation (13.28)** can be used to compute a weighted CN.

$$CN_W = CN_p(1 - f) + 98(f) \tag{13.28}$$

Where,

CN_W= Weighted CN

CN_P=CN value for open space (good condition)

f= the fraction (not percentage) of imperviousness

To show the use of **Equation (13.28)**, the CN values for commercial land use with 85 percent imperviousness are:

A soil: 39(0.15) + 98(0.85) = 89

B soil: 61(0.15) + 98(0.85) = 92

C soil: 74(0.15) + 98(0.85) = 94

D soil: 80(0.15) + 98(0.85) = 95

13.3.2.6 I_A/P PARAMETER

I_a/P is a parameter that is necessary to estimate peak discharge rates. I_a denotes the initial abstraction, and P is the 24-hour rainfall depth for a selected return period. For a given 24-hour rainfall distribution, I_a/P represents the fraction of rainfall that must occur before runoff begins.

13.3.2.7 PEAK DISCHARGE ESTIMATION

Equation (13.29) can be used to compute a peak discharge with the SCS method.

$$q_p = q_u A Q \tag{13.29}$$

Where,

q_p = peak discharge, m^3/s
q_u = unit peak discharge, $m^3/s/km^2/mm$
A = drainage area, km^2
Q = depth of runoff, mm

The unit peak discharge is obtained from **Equation (13.30)**, which requires the time of concentration (t_c) in hours and the initial abstraction/rainfall (I_a/P) ratio as input.

$$q_u = 0.000431 \times 10^{C_0 + C_1 log_{10} t_c + C_2 (log_{10} t_c)^2} \quad (13.30)$$

where,

C_0, C_1, and C_2 = regression coefficients given in **Table 13.17** for various I_a/P ratios.

The runoff depth (Q) is obtained from **Equation (13.22)** and is a function of the depth of rainfall P and the runoff CN. The I_a/P ratio is obtained directly from **Equation (13.21)**.

Table 13.17 Coefficients for SCS Peak Discharge Method

Rainfall Type	Ia/P	C_0	C_1	C_2
I	0.10	2.30550	-0.51429	-0.11750
	0.20	2.23537	-0.50387	-0.08929
	0.25	2.18219	-0.48488	-0.06589
	0.30	2.10624	-0.45695	-0.02835
	0.35	2.00303	-0.40769	0.01983
	0.40	1.87733	-0.32274	0.05754
	0.45	1.76312	-0.15644	0.00453
	0.50	1.67889	-0.06930	0.0
IA	0.10	2.03250	-0.31583	-0.13748
	0.20	1.91978	-0.28215	-0.07020
	0.25	1.83842	-0.25543	-0.02597
	0.30	1.72657	-0.19826	0.02633
	0.50	1.63417	-0.09100	0.0
II*	0.10	2.55323	-0.61512	-0.16403
	0.30	2.46532	-0.62257	-0.11657
	0.35	2.41896	-0.61594	-0.08820
	0.40	2.36409	-0.59857	-0.05621
	0.45	2.29238	-0.57005	-0.02281
	0.50	2.20282	-0.51599	-0.01259
III	0.10	2.47317	-0.51848	-0.17083
	0.30	2.39628	-0.51202	-0.13245
	0.35	2.35477	-0.49735	-0.11985
	0.40	2.30726	-0.46541	-0.11094
	0.45	2.24876	-0.41314	-0.11508
	0.50	2.17772	-0.36803	-0.09525

*Type II is typical of the rainfall type in Ghana where convection activity is the main cause of flood rainfall over small catchments.

The peak discharge obtained from **Equation (13.30)** assumes that the topography is such that surface flow into ditches, drains, and streams is relatively unimpeded. Where ponding or wetland areas occur in the watershed, a considerable amount of the surface runoff may be retained in temporary storage. The peak discharge rate should be reduced to reflect this condition of increased storage. Values of the pond and swamp adjustment factor (F_p) are provided in **Table 13.18**. The adjustment factor values in **Table 13.18** are a function of the percent of the total watershed area in ponds and wetlands. If the watershed includes significant portions of pond and wetland storage, the peak discharge of **Equation (13.29)** can be adjusted using **Equation (13.31)**.

$$q_a = q_p F_P \tag{13.31}$$

Where,

q_a = adjusted peak discharge, m^3/s

F_p= pond and swamp adjustment factor

Table 13.18 Adjustment Factor (F_p) for Pond and Wetland Areas

Area of Pond and Wetland (%)	F_p
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

The SCS has a number of limitations which can have an impact on the accuracy of estimated peak flows:

i. Basin should have fairly homogeneous CN values. Where parts of the watershed have CNs that differ by 5, the watershed should be subdivided and analysed using a hydrograph method, such as TR-20.
ii. CN should be 40 or greater.
iii. t_c should be between 0.1 and 10 hr.
iv. I_a/P should be between 0.1 and 0.5.
v. Basin should have one main channel or branches with nearly equal times of concentration.
vi. Neither channel nor reservoir routing can be incorporated.
vii. F_p factor is applied only for ponds and swamps that are not in the t_c Flow path.

Example 13.1
Given: The following physical and hydrologic conditions.

- 3.3km^2 of fair condition open space and 2.8km^2 of large lot residential
- Negligible pond and swamp land
- Hydrologic soil type C
- Average antecedent moisture conditions
- Time of concentration is 0.8 hr
- 24-hour, type II rainfall distribution, 10-year rainfall of 150 mm

Find: The 10-year peak flow using the SCS peak flow method.

Solution:

Step 1: Calculate the composite curve number using **Table 13.14** and **Equation (13.27)**.

$$CN_w = \frac{\sum_{i=1}^{n} CN_i A_i}{A}$$

$$CN_w = \frac{(3.3 \times 79) + (2.8 \times 77)}{(3.3 + 2.8)} = 78$$

Step 2: Calculate the retention, S, using **Equation (13.23)**.

$$S = 25.4\left(\frac{1000}{CN} - 10\right)$$

$$S = 25.4\left(\frac{1000}{78} - 10\right) = 72mm$$

Step 3: Calculate the depth of direct runoff using **Equation (13.22)**.

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

$$Q = \frac{(150 - 0.2(72))^2}{150 + 0.8(72)} = 89mm$$

Step 4: Determine I_a/P.

$I_a/P = 0.2S/P = (0.2x72)/150 = 0.10$

Step 5: Determine coefficients from **Table 13.17**.

$C_0 = 2.55323$ $C_1 = -0.61512$ $C_2 = -0.16403$

Step 6: Calculate unit peak flow using **Equation (13.30)**.

$$q_u = 0.000431 \times 10^{C_0 + C_1 log_{10} t_c + C_2 (log_{10} t_c)^2}$$

$$q_u = 0.000431 \times 10^{2.55323 + (-0.61512\ ) log_{10}(0.8) + (-0.16403)(log_{10}(0.8))^2}$$

$$q_u = 0.176 \text{ m}^3/\text{s}/\text{km}^2/\text{mm}$$

Step 7: Calculate peak flow using **Equation (13.29)**.

$$q_p = q_u A Q$$

$$q_p = (0.176)(3.3 + 2.8)(89) = 96 \text{ m}^3/\text{s}$$

13.3.3 STATISTICAL ANALYSIS OF STREAM DATA

Many stream gauging stations exist throughout the country where runoff data can be obtained and used for hydrologic studies. The gauging stations are maintained by the Hydrological Services Department of the Ministry of Works and Housing.

If the gauging record covers a sufficient period of time, it is possible to develop a flow-frequency relation by statistical analysis of the series of recorded annual maximum flows. The designer can then use the flow-frequency relation in several different ways:

i. If the road drainage site is near the gauging station on the same stream and watershed, the discharge can be used directly for a specific frequency (T-year discharge) from the peak stream flow frequency relationship.

ii. If the drainage structure site is within the same basin but not proximate to the gauging station, transposition of gauge analysis results is possible.

iii. If the structure site is not within a gauged basin, it is possible to develop the peak flow flood-frequency from data from a group of several gauging stations based on either a hydrologic region (e.g., regional regression equations), or similar hydrologic characteristics.

If adequate data are not available, the design peak discharge should be based on analyses of data from several stream flow-gauging stations. In some cases, a site requiring a design peak discharge is on the same stream and near an active or discontinued stream flow gauging station with an adequate length of record (see the **Table 13.19**).

Table 13.19 Recommended Minimum Stream Gauge Record Lengths

Design Frequency (Years)	Minimum Record Length (Years)
10	8
25	10
50	20
100	25

Having determined that a suitable stream gauge record exists, it is necessary to determine if any structures or urbanization may be affecting the peak discharges at the design site. Consider the following guidelines:

i. **Period of record similar to design site** - The period of record for the gauging station's annual peak discharges should represent the same or similar basin conditions as that of the design site. Therefore, exclude from the analysis any gauged peak discharges not representing the basin conditions for the design site.

ii. **Factors affecting peak discharge** - The most typical factors affecting peak discharges are regulation by urbanization and reservoirs. Densities of impervious cover less than 10 percent of the watershed area generally do not affect peak discharges. The existence in the watershed of a major reservoir or many smaller reservoirs or flood control structures can greatly affect the runoff characteristics.

iii. **Length of record** - The length of record should be adjusted to include only those records that have been collected subsequent to the impoundment of water by reservoirs and subsequent to any major urbanization. If the resulting records then become too short, do not use the procedures in this section.

13.3.3.1 PLOTTING POSITION FORMULAS

When making a flood frequency analysis, it is common to plot both the assumed population and the peak discharges of the sample. To plot the sample values on frequency paper, it is necessary to assign an exceedence probability to each magnitude. A plotting position formula is used for this purpose. A number of different formulas have been proposed for computing plotting position probabilities, with no unanimity on the preferred method. A general formula for computing plotting positions is given by **Equation (13.32)**.

$$P = \frac{m - a}{(n - a - b + 1)} \tag{13.32}$$

Where,

m = rank order of the ordered flood magnitudes, with the largest flood having a rank of 1

n = record length

a, b = constants for a particular plotting position formula.

The inverse of **Equation (13.32)** gives the recurrence interval (Return Period, T_r) which is given by **Equation (13.33)**.

$$T_r = \frac{(n - a - b + 1)}{m - a} \tag{13.33}$$

The Weibull [**Equations (13.34)** & **(13.35)**], P_w/T_w (a = b = 0), Hazen [**Equations (13.36)** & **(13.37)**], P_h/T_h (a = b = 0.5), and Cunnane [**Equations (13.38)** & **(13.39)**], P_c/P_c (a = b = 0.4) are three possible plotting position formulas.

$$P_w = \frac{m}{n + 1} \tag{13.34}$$

$$T_w = \frac{n + 1}{m} \tag{13.35}$$

$$P_h = \frac{m - 0.5}{n} \tag{13.36}$$

$$T_h = \frac{n}{m - 0.5} \tag{13.37}$$

$$P_c = \frac{m - 0.4}{n + 0.2} \tag{13.38}$$

$$T_c = \frac{n + 0.2}{m - 0.4} \tag{13.39}$$

The data are plotted by placing a point for each value of the flood series at the intersection of the flood magnitude and the exceedance probability computed with the plotting position formula. The plotted data should approximate the population line if the assumed population model is a reasonable assumption.

13.3.3.2 LOG-PEARSON TYPE III DISTRIBUTION

Flood frequency analysis uses sample information to fit a population, which is a probability distribution. These distributions have parameters that must be estimated in order to make probability statements about the likelihood of future flood magnitudes. A number of methods for estimating the parameters are available. The method of moments, which is just one of the parameter-estimation methods is applied in this guide.

Several cumulative frequency distributions are commonly used in the analysis of hydrologic data, and as a result they have been studied extensively and are now standardized. The frequency distributions that have been found most useful in hydrologic data analysis are the normal distribution, the log-normal distribution, the Gumbel extreme value distribution, and the log-Pearson Type III distribution.

Log-Pearson Type III Distribution has found wide application in hydrologic analysis. It is a three-parameter gamma distribution with a logarithmic transform of the variable. It is widely used for flood analyses because the data quite frequently fit the assumed population.

The log-Pearson Type III distribution differs from most of other distributions in that three parameters (mean, standard deviation, and coefficient of skew) are necessary to describe the distribution. By judicious selection of these three parameters, it is possible to fit just about any shape of distribution.

However, the designer is not limited to using this method, especially if the resulting flow-frequency relation does not seem to fit the data.

The Log-Pearson III Distribution is given by **Equation (13.40)**.

$$logQ_{T_r} = avg(logQ) + [k(T_r, C_s)] \times \sigma_{logQ} \tag{13.40}$$

Where,

Q_{Tr} = flood magnitude with the return period of Tr, m^3/s

avg(logQ) = mean of the log of all floods in the series, m^3/s

σ_{logQ} = standard deviation of the logarithms of the floods

k = frequency factor for particular return period and coefficient of skew. Values of k are tabulated in **Table 13.20 - Table 13.25**.

C_s = skew coefficient

Table 13.20 Frequency Factors (K) for the Log-Pearson Type III Distribution

	Skew						
Prob.	**-2.0**	**-1.9**	**-1.8**	**-1.7**	**-1.6**	**-1.5**	**-1.4**
0.9999	-8.21034	-7.98888	-7.76632	-7.54272	-7.31818	-7.09277	-6.86661
0.9995	-6.60090	-6.44251	-6.28285	-6.12196	-5.95990	-5.79673	-5.63252
0.9990	-5.90776	-5.77549	-5.64190	-5.50701	-5.37087	-5.23353	-5.09505
0.9980	-5.21461	-5.10768	-4.99937	-4.88971	-4.77875	-4.66651	-4.55304
0.9950	-4.29832	-4.22336	-4.14700	-4.06926	-3.99016	-3.90973	-3.82798
0.9900	-3.60517	-3.55295	-3.49935	-3.44438	-3.38804	-3.33035	-3.27134
0.9800	-2.91202	-2.88091	-2.84848	-2.81472	-2.77964	-2.74325	-2.70556
0.9750	-2.68888	-2.66413	-2.63810	-2.61076	-2.58214	-2.55222	-2.52102
0.9600	-2.21888	-2.20670	-2.19332	-2.17873	-2.16293	-2.14591	-2.12768
0.9500	-1.99573	-1.98906	-1.98124	-1.97227	-1.96213	-1.95083	-1.93836
0.9000	-1.30259	-1.31054	-1.31760	-1.32376	-1.32900	-1.33330	-1.33665
0.8000	-0.60944	-0.62662	-0.64335	-0.65959	-0.67532	-0.69050	-0.70512
0.7000	-0.20397	-0.22250	-0.24094	-0.25925	-0.27740	-0.29535	-0.31307
0.6000	0.08371	0.06718	0.05040	0.03344	0.01631	-0.00092	-0.01824
0.5704	0.15516	0.13964	0.12381	0.10769	0.09132	0.07476	0.05803
0.5000	0.30685	0.29443	0.28150	0.26808	0.25422	0.23996	0.22535
0.4296	0.43854	0.43008	0.42095	0.41116	0.40075	0.38977	0.37824
0.4000	0.48917	0.48265	0.47538	0.46739	0.45873	0.44942	0.43949
0.3000	0.64333	0.64453	0.64488	0.64436	0.64300	0.64080	0.63779
0.2000	0.77686	0.78816	0.79868	0.80837	0.81720	0.82516	0.83223
0.1000	0.89464	0.91988	0.94496	0.96977	0.99418	1.01810	1.04144
0.0500	0.94871	0.98381	1.01973	1.05631	1.09338	1.13075	1.16827
0.0400	0.95918	0.99672	1.03543	1.07513	1.11566	1.15682	1.19842
0.0250	0.97468	1.01640	1.06001	1.10537	1.15229	1.20059	1.25004
0.0200	0.97980	1.02311	1.06864	1.11628	1.16584	1.21716	1.26999
0.0100	0.98995	1.03695	1.08711	1.14042	1.19680	1.25611	1.31815
0.0050	0.99499	1.04427	1.09749	1.15477	1.21618	1.28167	1.35114
0.0020	0.99800	1.04898	1.10465	1.16534	1.23132	1.30279	1.37981
0.0010	0.99900	1.05068	1.10743	1.16974	1.23805	1.31275	1.39408
0.0005	0.99950	1.05159	1.10901	1.17240	1.24235	1.31944	1.40413
0.0001	0.99990	1.05239	1.11054	1.17520	1.24728	1.32774	1.41753

Table 13.21 Frequency Factors (K) for the Log-Pearson Type III Distribution (Cont'd)

	Skew						
Prob.	**-1.3**	**-1.2**	**-1.1**	**-1.0**	**-0.9**	**-0.8**	**-0.7**
0.9999	-6.63980	-6.41249	-6.18480	-5.95691	-5.72899	-5.50124	-5.27389
0.9995	-5.46735	-5.30130	-5.13449	-4.96701	-4.79899	-4.63057	-4.46189
0.9990	-4.95549	-4.81492	-4.67344	-4.53112	-4.38807	-4.24439	-4.10022
0.9980	-4.43839	-4.32263	-4.20582	-4.08802	-3.96932	-3.84981	-3.72957
0.9950	-3.74497	-3.66073	-3.57530	-3.48874	-3.40109	-3.31243	-3.22281
0.9900	-3.21103	-3.14944	-3.08660	-3.02256	-2.95735	-2.89101	-2.82359
0.9800	-2.66657	-2.62631	-2.58480	-2.54206	-2.49811	-2.45298	-2.40670
0.9750	-2.48855	-2.45482	-2.41984	-2.38364	-2.34623	-2.30764	-2.26790
0.9600	-2.10823	-2.08758	-2.06573	-2.04269	-2.01848	-1.99311	-1.96660
0.9500	-1.92472	-1.90992	-1.89395	-1.87683	-1.85856	-1.83916	-1.81864
0.9000	-1.33904	-1.34047	-1.34092	-1.34039	-1.33889	-1.33640	-1.33294
0.8000	-0.71915	-0.73257	-0.74537	-0.75752	-0.76902	-0.77986	-0.79002
0.7000	-0.33054	-0.34772	-0.36458	-0.38111	-0.39729	-0.41309	-0.42851
0.6000	-0.03560	-0.05297	-0.07032	-0.08763	-0.10486	-0.12199	-0.13901
0.5704	0.04116	0.02421	0.00719	-0.00987	-0.02693	-0.04397	-0.06097
0.5000	0.21040	0.19517	0.17968	0.16397	0.14807	0.13199	0.11578
0.4296	0.36620	0.35370	0.34075	0.32740	0.31368	0.29961	0.28516
0.4000	0.42899	0.41794	0.40638	0.39434	0.38186	0.36889	0.35565
0.3000	0.63400	0.62944	0.62415	0.61815	0.61146	0.60412	0.59615
0.2000	0.83841	0.84369	0.84809	0.85161	0.85426	0.85607	0.85703
0.1000	1.06413	1.08608	1.10726	1.12762	1.14712	1.16574	1.18347
0.0500	1.20578	1.24313	1.28019	1.31684	1.35299	1.38855	1.42345
0.0400	1.24028	1.28225	1.32414	1.36584	1.40720	1.44813	1.48852
0.0250	1.30042	1.35153	1.40314	1.45507	1.50712	1.55914	1.61099
0.0200	1.32412	1.37929	1.43529	1.49188	1.54886	1.60604	1.66325
0.0100	1.38267	1.44942	1.51808	1.58838	1.66001	1.73271	1.80621
0.0050	1.42439	1.50114	1.58110	1.66390	1.74919	1.83660	1.92580
0.0020	1.46232	1.55016	1.64305	1.74062	1.84244	1.94806	2.05701
0.0010	1.48216	1.57695	1.67825	1.78572	1.89894	2.01739	2.14053
0.0005	1.49673	1.59738	1.70603	1.82241	1.94611	2.07661	2.21328
0.0001	1.51752	1.62838	1.75053	1.88410	2.02891	2.18448	2.35015

Table 13.22 Frequency Factors (K) for the Log-Pearson Type III Distribution (Cont'd)

	Skew						
Prob.	**-0.6**	**-0.5**	**-0.4**	**-0.3**	**-0.2**	**-0.1**	**0.0**
0.9999	-5.04718	-4.82141	-4.59687	-4.37394	-4.15301	-3.93453	-3.71902
0.9995	-4.29311	-4.12443	-3.95605	-3.78820	-3.62113	-3.45513	-3.29053
0.9990	-3.95567	-3.81090	-3.66608	-3.52139	-3.37703	-3.23322	-3.09023
0.9980	-3.60872	-3.48737	-3.36566	-3.24371	-3.12169	-2.99978	-2.87816
0.9950	-3.13232	-3.04102	-2.94900	-2.85636	-2.76321	-2.66965	-2.57583
0.9900	-2.75514	-2.68572	-2.61539	-2.54421	-2.47226	-2.39961	-2.32635
0.9800	-2.35931	-2.31084	-2.26133	-2.21081	-2.15935	-2.10697	-2.05375
0.9750	-2.22702	-2.18505	-2.14202	-2.09795	-2.05290	-2.00688	-1.95996
0.9600	-1.93896	-1.91022	-1.88039	-1.84949	-1.81756	-1.78462	-1.75069
0.9500	-1.79701	-1.77428	-1.75048	-1.72562	-1.69971	-1.67279	-1.64485
0.9000	-1.32850	-1.32309	-1.31671	-1.30936	-1.30105	-1.29178	-1.28155
0.8000	-0.79950	-0.80829	-0.81638	-0.82377	-0.83044	-0.83639	-0.84162
0.7000	-0.44352	-0.45812	-0.47228	-0.48600	-0.49927	-0.51207	-0.52440
0.6000	-0.15589	-0.17261	-0.18916	-0.20552	-0.22168	-0.23763	-0.25335
0.5704	-0.07791	-0.09178	-0.11154	-0.12820	-0.14472	-0.16111	-0.17733
0.5000	0.09945	0.08302	0.06651	0.04993	0.03325	0.01662	0.00000
0.4296	0.27047	0.25558	0.24037	0.22492	0.20925	0.19339	0.17733
0.4000	0.34198	0.32796	0.31362	0.29897	0.28403	0.26882	0.25335
0.3000	0.58757	0.57840	0.56867	0.55839	0.54757	0.53624	0.52440
0.2000	0.85718	0.85653	0.85508	0.85285	0.84986	0.84611	0.84162
0.1000	1.20028	1.21618	1.23114	1.24516	1.25824	1.27037	1.28155
0.0500	1.45762	1.49101	1.52357	1.55527	1.58607	1.61594	1.64485
0.0400	1.52830	1.56740	1.60574	1.64329	1.67999	1.71580	1.75069
0.0250	1.66253	1.71366	1.76427	1.81427	1.86360	1.91219	1.95996
0.0200	1.72033	1.77716	1.83361	1.88959	1.94499	1.99973	2.05375
0.0100	1.88029	1.95472	2.02933	2.10394	2.17840	2.25258	2.32635
0.0050	2.01644	2.10825	2.20092	2.29423	2.38795	2.48187	2.57583
0.0020	2.16884	2.28311	2.39942	2.51741	2.63672	2.75706	2.87816
0.0010	2.26780	2.39867	2.53261	2.66915	2.80786	2.94834	3.09023
0.0005	2.35549	2.50257	2.65390	2.80889	2.96698	3.12767	3.29053
0.0001	2.52507	2.70836	2.89907	3.09631	3.29921	3.50703	3.71902

Table 13.23 Frequency Factors (K) for the Log-Pearson Type III Distribution (Cont'd)

	Skew						
Prob.	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**
0.9999	-3.50703	-3.29921	-3.09631	-2.89907	-2.70836	-2.52507	-2.35015
0.9995	-3.12767	-2.96698	-2.80889	-2.65390	-2.50257	-2.35549	-2.21328
0.9990	-2.94834	-2.80786	-2.66915	-2.53261	-2.39867	-2.26780	-2.14053
0.9980	-2.75706	-2.63672	-2.51741	-2.39942	-2.28311	-2.16884	-2.05701
0.9950	-2.48187	-2.38795	-2.29423	-2.20092	-2.10825	-2.01644	-1.92580
0.9900	-2.25258	-2.17840	-2.10394	-2.02933	-1.95472	-1.88029	-1.80621
0.9800	-1.99973	-1.94499	-1.88959	-1.83361	-1.77716	-1.72033	-1.66325
0.9750	-1.91219	-1.86360	-1.81427	-1.76427	-1.71366	-1.66253	-1.61099
0.9600	-1.71580	-1.67999	-1.64329	-1.60574	-1.56740	-1.52830	-1.48852
0.9500	-1.61594	-1.58607	-1.55527	-1.52357	-1.49101	-1.45762	-1.42345
0.9000	-1.27037	-1.25824	-1.24516	-1.23114	-1.21618	-1.20028	-1.18347
0.8000	-0.84611	-0.84986	-0.85285	-0.85508	-0.85653	-0.85718	-0.85703
0.7000	-0.53624	-0.54757	-0.55839	-0.56867	-0.57840	-0.58757	-0.59615
0.6000	-0.26882	-0.28403	-0.29897	-0.31362	-0.32796	-0.34198	-0.35565
0.5704	-0.19339	-0.20925	-0.22492	-0.24037	-0.25558	-0.27047	-0.28516
0.5000	-0.01662	-0.03325	-0.04993	-0.06651	-0.08302	-0.09945	-0.11578
0.4296	0.16111	0.14472	0.12820	0.11154	0.09478	0.07791	0.06097
0.4000	0.23763	0.22168	0.20552	0.18916	0.17261	0.15589	0.13901
0.3000	0.51207	0.49927	0.48600	0.47228	0.45812	0.44352	0.42851
0.2000	0.83639	0.83044	0.82377	0.81638	0.80829	0.79950	0.79002
0.1000	1.29178	1.30105	1.30936	1.31671	1.32309	1.32850	1.33294
0.0500	1.67279	1.69971	1.72562	1.75048	1.77428	1.79701	1.81864
0.0400	1.78462	1.81756	1.84949	1.88039	1.91022	1.93896	1.96660
0.0250	2.00688	2.05290	2.09795	2.14202	2.18505	2.22702	2.26790
0.0200	2.10697	2.15935	2.21081	2.26133	2.31084	2.35931	2.40670
0.0100	2.39961	2.47226	2.54421	2.61539	2.68572	2.75514	2.82359
0.0050	2.66965	2.76321	2.85636	2.94900	3.04102	3.13232	3.22281
0.0020	2.99978	3.12169	3.24371	3.36566	3.48737	3.60872	3.72957
0.0010	3.23322	3.37703	3.52139	3.66608	3.81090	3.95567	4.10022
0.0005	3.45513	3.62113	3.78820	3.95605	4.12443	4.29311	4.46189
0.0001	3.93453	4.15301	4.37394	4.59687	4.82141	5.04718	5.27389

Table 13.24 Frequency Factors (K) for the Log-Pearson Type III Distribution (Cont'd)

	Skew						
Prob.	**0.8**	**0.9**	**1.0**	**1.1**	**1.2**	**1.3**	**1.4**
0.9999	2.18448	-2.02891	-1.88410	-1.75053	-1.62838	-1.51752	-1.41753
0.9995	-2.07661	-1.94611	-1.82241	-1.70603	-1.59738	-1.49673	-1.40413
0.9990	-2.01739	-1.89894	-1.78572	-1.67825	-1.57695	-1.48216	-1.39408
0.9980	-1.94806	-1.84244	-1.74062	-1.64305	-1.55016	-1.46232	-1.37981
0.9950	-1.83660	-1.74919	-1.66390	-1.58110	-1.50114	-1.42439	-1.35114
0.9900	-1.73271	-1.66001	-1.58838	-1.51808	-1.44942	-1.38267	-1.31815
0.9800	-1.60604	-1.54886	-1.49188	-1.43529	-1.37929	-1.32412	-1.26999
0.9750	-1.55914	-1.50712	-1.45507	-1.40314	-1.35153	-1.30042	-1.25004
0.9600	-1.44813	-1.40720	-1.36584	-1.32414	-1.28225	-1.24028	-1.19842
0.9500	-1.38855	-1.35299	-1.31684	-1.28019	-1.24313	-1.20578	-1.16827
0.9000	-1.16574	-1.14712	-1.12762	-1.10726	-1.08608	-1.06413	-1.04144
0.8000	-0.85607	-0.85426	-0.85161	-0.84809	-0.84369	-0.83841	-0.83223
0.7000	-0.60412	-0.61146	-0.61815	-0.62415	-0.62944	-0.63400	-0.63779
0.6000	-0.36889	-0.38186	-0.39434	-0.40638	-0.41794	-0.42899	-0.43949
0.5704	-0.29961	-0.31368	-0.32740	-0.34075	-0.35370	-0.36620	-0.37824
0.5000	-0.13199	-0.14807	-0.16397	-0.17968	-0.19517	-0.21040	-0.22535
0.4296	0.04397	0.02693	0.00987	-0.00719	-0.02421	-0.04116	-0.05803
0.4000	0.12199	0.10486	0.08763	0.07032	0.05297	0.03560	0.01824
0.3000	0.41309	0.39729	0.38111	0.36458	0.34772	0.33054	0.31307
0.2000	0.77986	0.76902	0.75752	0.74537	0.73257	0.71915	0.70512
0.1000	1.33640	1.33889	1.34039	1.34092	1.34047	1.33904	1.33665
0.0500	1.83916	1.85856	1.87683	1.89395	1.90992	1.92472	1.93836
0.0400	1.99311	2.01848	2.04269	2.06573	2.08758	2.10823	2.12768
0.0250	2.30764	2.34623	2.38364	2.41984	2.45482	2.48855	2.52102
0.0200	2.45298	2.49811	2.54206	2.58480	2.62631	2.66657	2.70556
0.0100	2.89101	2.95735	3.02256	3.08660	3.14944	3.21103	3.27134
0.0050	3.31243	3.40109	3.48874	3.57530	3.66073	3.74497	3.82798
0.0020	3.84981	3.96932	4.08802	4.20582	4.32263	4.43839	4.55304
0.0010	4.24439	4.38807	4.53112	4.67344	4.81492	4.95549	5.09505
0.0005	4.63057	4.79899	4.96701	5.13449	5.30130	5.46735	5.63252
0.0001	5.50124	5.72899	5.95691	6.18480	6.41249	6.63980	6.86661

Table 13.25 Frequency Factors (K) for the Log-Pearson Type III Distribution (Cont'd)

	Skew					
Prob.	**1.5**	**1.6**	**1.7**	**1.8**	**1.9**	**2.0**
0.9999	-1.32774	-1.24728	-1.17520	-1.11054	-1.05239	-0.99990
0.9995	-1.31944	-1.24235	-1.17240	-1.10901	-1.05159	-0.99950
0.9990	-1.31275	-1.23805	-1.16974	-1.10743	-1.50568	-0.99900
0.9980	-1.30279	-1.23132	-1.16534	-1.10465	-1.04898	-0.99800
0.9950	-1.28167	-1.21618	-1.15477	-1.09749	-1.04427	-0.99499
0.9900	-1.25611	-1.19680	-1.14042	-1.08711	-1.03695	-0.98995
0.9800	-1.21716	-1.16584	-1.11628	-1.06864	-1.02311	-0.97980
0.9750	-1.20059	-1.15229	-1.10537	-1.06001	-1.01640	-0.97468
0.9600	-1.15682	-1.11566	-1.07513	-1.03543	-0.99672	-0.95918
0.9500	-1.13075	-1.09338	-1.05631	-1.01973	-0.98381	-0.94871
0.9000	-1.01810	-0.99418	-0.96977	-0.94496	-0.91988	-0.89464
0.8000	-0.82516	-0.81720	-0.80837	-0.79868	-0.78816	-0.77686
0.7000	-0.64080	-0.64300	-0.64436	-0.64488	-0.64453	-0.64333
0.6000	-0.44942	-0.45873	-0.46739	-0.47538	-0.48265	-0.48917
0.5704	-0.38977	-0.40075	-0.41116	-0.42095	-0.43008	-0.43854
0.5000	-0.23996	-0.25422	-0.26808	-0.28150	-0.29443	-0.30685
0.4296	-0.07476	-0.09132	-0.10769	-0.12381	-0.13964	-0.15516
0.4000	0.00092	-0.01631	-0.03344	-0.05040	-0.06718	-0.08371
0.3000	0.29535	0.27740	0.25925	0.24094	0.22250	0.20397
0.2000	0.69050	0.67532	0.65959	0.64335	0.62662	0.60944
0.1000	1.33330	1.32900	1.32376	1.31760	1.31054	1.30259
0.0500	1.95083	1.96213	1.97227	1.98124	1.98906	1.99573
0.0400	2.14591	2.16293	2.17873	2.19332	2.20670	2.21888
0.0250	2.55222	2.58214	2.61076	2.63810	2.66413	2.68888
0.0200	2.74325	2.77964	2.81472	2.84848	2.88091	2.91202
0.0100	3.33035	3.38804	3.44438	3.49935	3.55295	3.60517
0.0050	3.90973	3.99016	4.06926	4.14700	4.22336	4.29832
0.0020	4.66651	4.77875	4.88971	4.99937	5.10768	5.21461
0.0010	5.23353	5.37087	5.50701	5.64190	5.77549	5.90776
0.0005	5.79673	5.95990	6.12196	6.28285	6.44251	6.60090
0.0001	7.09277	7.31818	7.54272	7.76632	7.98888	8.21034

The following general procedure is used for log-Pearson Type III analyses.

Step 1. Obtain streamflow data.

Step 2. Calculate the maximum discharge for each water year in the period of record.

Step 3. Organize the information in a table form on an Excel spread sheet.

Step 4. Rank the data from the largest discharge to the smallest discharge. Add a column for Rank and number each streamflow value from 1 to n (the total number of values in your dataset).

Step 5. Create a column with the log of each maximum or peak streamflow using the Excel formula {log(Q)} and copy command.

Step 6. Calculate the Average Maximum Q or Peak Q and the Average of the log(Q).

Step 7. Create a column with the Excel formula $\{(\log Q - \text{avg}(\log Q))^2\}$.

Step 8. Create a column with the Excel formula $\{(\log Q - \text{avg}(\log Q))^3\}$.

Step 9. Create a column with the return period (Tr) for each discharge using **Equations (13.35)**, **(13.37)** or **(13.39)**. Weibull plotting position, **Equation (13.35)** has been adopted in this guide.

Step 10. Complete the table with a final column showing the exceedence probability of each discharge using the excel formula {=1/Return Period or 1/T_r} and the copy command.

Step 11. Calculate the sum for the $\{(\log Q\text{-avg}(\log Q))^2\}$ and the $\{\log Q - \text{avg}(\log Q))^3\}$ columns.

Step 12. Calculate the variance, standard deviation, and skew coefficient using **Equations (13.41)**, **(13.42)** and **(13.43)** respectively.

$$Variance\ (C_v) = \frac{\sum(\log Q - \text{avg}(\log Q))^2}{(n-1)} \tag{13.41}$$

$$\text{Standard deviation} = \sigma_{logQ} = \sqrt{C_v} \tag{13.42}$$

$$\text{Skew coefficient } (C_s) = \frac{n \times \sum(\log Q - \text{avg}(\log Q))^3}{(n-1)(n-2)\left(\sigma_{logQ}\right)^3} \quad (13.43)$$

Step 13. Using the frequency factor table and the skew coefficient, find the k values for the 2, 5, 10, 25, 50 and 100 recurrence intervals. If the skew coefficient is between two skew coefficients in the table, linearly extrapolate between the two numbers to get the appropriate k value.

Step 14. Using **Equation (13.40)**, list the discharges associated with each recurrence interval.

Step 15. Create table of discharge values found using the Log-Pearson III distribution analysis.

Example 13.2

GHA has proposed to replace a collapse bridge over a river on the Han-Lawra Road. The replacement bridge is to be designed for the 50-year storm. Determine the peak flow of the design storm. The drainage area upstream of the bridge is approximately 90 km^2. Hydro maintains a gauging station at the location of the bridge and 24 years of stream flow record is available. The table below shows the flow data.

Peak Daily Runoffs for Unnamed River crossing Han-Lawra Road for the Period 1962-1985

Year	Annual Peak Runoff (m^3/s)
1962	64.8
1963	41.6
1964	62.9
1965	84.1
1966	85.5
1967	34.3
1968	70.5
1969	89.8
1970	91.2
1971	49.8
1972	249.2
1973	234.5
1974	37.1
1975	70.8
1976	55.5
1977	60.6
1978	122.9
1979	86.6

Year	Annual Peak Runoff (m^3/s)
1980	50.4
1981	39.1
1982	27.8
1983	29.4
1984	44.7
1985	102.8

Solution: Since the drainage area is so large, it might be time consuming to use NRCS method. With 24 years of stream flow record available, frequency analysis approach is more appropriate and therefore employed.

Using the steps for the Log-Pearson Type III Distribution, complete the analysis as indicated **Table 13.26**:

Table 13.26 Log-Pearson III Annual Peak Flow Frequency Analysis

Year	Annual Peak Runoff (m^3/s)	Rank m	Ranked Runoff Q (m^3/s)	Log Q	(Log Q -(avgLog Q))^2	(Log Q -(avgLog Q))^3	Return Period T_r=(n+1)/m	Exceedance Probability (1/T_r)
1962	64.8	1	249.2	2.397	0.33196	0.19126	25.00	0.0400
1963	41.6	2	234.5	2.370	0.30223	0.16615	12.50	0.0800
1964	62.9	3	122.9	2.090	0.07245	0.01950	8.33	0.1200
1965	84.1	4	102.8	2.012	0.03671	0.00703	6.25	0.1600
1966	85.5	5	91.2	1.960	0.01949	0.00272	5.00	0.2000
1967	34.3	6	89.8	1.953	0.01766	0.00235	4.17	0.2400
1968	70.5	7	86.6	1.938	0.01372	0.00161	3.57	0.2800
1969	89.8	8	85.5	1.932	0.01245	0.00139	3.13	0.3200
1970	91.2	9	84.1	1.925	0.01090	0.00114	2.78	0.3600
1971	49.8	10	70.8	1.850	0.00088	0.00003	2.50	0.4000
1972	249.2	11	70.5	1.848	0.00077	0.00002	2.27	0.4400
1973	234.5	12	64.8	1.812	0.00008	0.00000	2.08	0.4800
1974	37.1	13	62.9	1.799	0.00047	-0.00001	1.92	0.5200
1975	70.8	14	60.6	1.782	0.00144	-0.00005	1.79	0.5600
1976	55.5	15	55.5	1.744	0.00579	-0.00044	1.67	0.6000
1977	60.6	16	50.4	1.702	0.01391	-0.00164	1.56	0.6400
1978	122.9	17	49.8	1.697	0.01517	-0.00187	1.47	0.6800
1979	86.6	18	44.7	1.650	0.02893	-0.00492	1.39	0.7200
1980	50.4	19	41.6	1.619	0.04052	-0.00816	1.32	0.7600
1981	39.1	20	39.1	1.592	0.05208	-0.01189	1.25	0.8000
1982	27.8	21	37.1	1.569	0.06301	-0.01582	1.19	0.8400

Year	Annual Peak Runoff	Rank	Ranked Runoff	Log Q	(Log Q -(avgLog Q))^2	(Log Q -(avgLog Q))^3	Return Period	Exceedance Probability
	(m^3/s)	m	Q (m^3/s)				T_r=(n+1)/m	(1/T_r)
1983	29.4	22	34.3	1.535	0.08128	-0.02317	1.14	0.8800
1984	44.7	23	29.4	1.468	0.12393	-0.04363	1.09	0.9200
1985	102.8	24	27.8	1.444	0.14163	-0.05330	1.04	0.9600
			Average	**Average**	**Sum**	**Sum**		
			78.579	1.820	1.38746	0.22830		

Variance, C_v = 0.06032455

Standard deviation (s.d), σ_{logQ} = 0.24561057

Skew coefficient, C_s = 0.73085919 (lies between K(0.7) and K(0.8))

Return Period T_r	K (0.7) K(i)	K (0.8) K(j)	Slope s	K_{Tr}(0.73085919) t	Log Q_{Tr} Y	Peak Flow Q_{Tr} (m^3/s)
Years	**a**	**b**	$s = \frac{b-a}{K(j)-K(i)}$	$t = s \times (C_s - K(i) + a)$	$Y = \mathrm{Log}Q_{Tr} + (t \times \sigma_{logQ})$	
2	-0.116	-0.132	-0.160	-0.121	1.7907	**61.76**
5	0.790	0.780	-0.100	0.787	2.0137	**103.20**
10	1.333	1.336	0.030	1.334	2.1480	**140.61**
25	1.967	1.993	0.260	1.975	2.3055	**202.06**
50	2.407	2.453	0.460	2.421	2.4151	**260.05**
100	2.824	2.891	0.670	2.845	2.5191	**330.42**

13.3.3.3 TRANSPOSITION OF GAUGE ANALYSIS RESULTS

If gauge data are not available at the design location, discharge values can be estimated by transposition if a peak flow-frequency curve is available at a nearby gauged location. This method is appropriate for hydrologically similar watersheds that differ in area by less than 50 percent, with outlet locations less than 160 km apart. From the research of Asquith and Thompson 2008, an estimate of the desired AEP peak flow at the ungauged site is provided by **Equation (13.44)**.

$$Q_1 = Q_2 \sqrt{\frac{A_1}{A_2}} \tag{13.44}$$

Where,

Q_1 = Estimated AEP discharge at ungauged watershed 1

Q_2 = Known AEP discharge at gauged watershed 2

A_1 = Area of watershed 1

A_2 = Area of watershed 2

If flow-frequency curves are available at multiple gauged sites, **Equation (13.44)** can be used to estimate the desired peak AEP flow from each site. Then, with judgment and knowledge of the watersheds, those estimates could be weighted to provide an estimate of the desired AEP flow at the ungauged location. This process should be well documented.

13.3.4 HYDROGRAPH METHOD

13.3.4.1 APPLICABILITY

The hydrograph method is generally used when analysing larger watersheds. The hydrograph method allows the user to account for hydrologic losses, including evaporation, transpiration, infiltration, surface routing, storage within the watershed, and varying antecedent moisture conditions. In addition, the hydrograph method allows for the analysis of complex drainage facilities, including diversions and detention ponds. In practice, this method allows for the development of a flood hydrograph using a design storm, an appropriate infiltration or loss rate technique, and a synthetic unit hydrograph.

The basic process of the hydrograph method includes:

i. Simulating rainfall from a specified storm event.
ii. Simulating rainfall losses due to interception and infiltration.
iii. Simulating the overland flow into creeks, channels, or pipes to provide a runoff hydrograph at concentrated points.
iv. Routing the hydrograph through creeks, channels, or pipes.
v. Routing the hydrograph through detention basins or reservoirs.

A hydrograph analysis is required for drainage areas greater than 200 acres, or if the drainage area is greater than 50 acres and includes a detention basin or storage reservoir. For areas less than 200 acres or those smaller areas that do not require a detention basin or storage reservoir, the hydrograph method may be applied, but should be compared to the results obtained by application of the Rational Method.

Detailed explanation of the hydrograph method can be found in the Handbook of Applied Hydrology by V. T. Chow, Hydrology for Engineers by Linsley, Kohler, and Paulhus, the ASCE Manual and Report on Engineering Practice No. 28, Hydrology Handbook, and other appropriate references.

13.3.4.2 COMPUTER PROGRAMS

HEC-1 and HEC-HMS are hydrologic modelling computer programs developed by the Hydrologic Engineering Centre (HEC) of the U. S. Army Corps of Engineers. Both programs are designed to compute rainfall-runoff hydrographs and route the hydrographs through channels, pipes, detention basins, and reservoirs.

It is important to recognize that these computer programs are intended only to simulate naturally occurring events. In many instances, the simulated processes or results can vary significantly from those that occur naturally. Whenever possible, the simulated results should be calibrated with verified results, or at a minimum, analysed to determine whether the results are reasonable for the given meteorologic, topographic, and land use conditions.

This guide is not intended to be a user's guide for either HEC-1 or HEC-HMS. It is assumed that the reader is familiar with one or both of these programs. However, if you are unfamiliar with either program, user's manuals for both can be downloaded from HEC's homepage, http://www.hec.usace.army.mil.

13.3.5 DESIGN FOR CLIMATE RESILIENCE

Climate change will affect roads and highways in many different ways. Globally, the accepted characteristics include higher temperatures and more extreme weather events. These are often experienced as higher rainfall, more intense peaks in precipitation, and more sustained periods of both rainfall and drought.

A number of online tools are available that facilitate the process of understanding past trends and anticipating future risks. One such tool is the Climate Change Knowledge Portal at https://climateknowledgeportal.worldbank.org/country/ghana/climate-sector-water. In the case of Ghana, this suggests that, over the past 100 years, annual precipitation levels in Ghana have reduced by about 5%, with (approximately) 10% increases of monthly rainfall in July and August being more than offset by reductions during other months.

Looking ahead, the most compelling feature of the climate projection for Ghana is the continued steady increase in maximum day temperatures. This gives rise to an increased risk of:

- more intense local storms as raising temperatures increase the atmosphere's potential carrying capacity of moisture; and
- drought, as rates of evaporation and transpiration increase, without an associated overall increase in precipitation rates.

In the light of this projection, which is also consistent with recent anecdotal experience in Ghana, one of the simplest and important actions that can be taken is to incorporate a safety factor in the design of drainage structures based on a 15 - 20% flow allowance for a selected recurrence interval.

In addition, there are various other strategies that can help to increase climate resilience. In general, these include:

- identifying the most vulnerable areas and increasing the 'safety factor' inherent in their design;
- ensuring that the drainage systems are well maintained and functioning correctly; and
- local realignment in critical areas or on high priority roads where the consequences of failure and closure are severe. (This is usually only considered as part of a repair and rehabilitation project after storm damage has occurred).

Practical means of increasing the safety factor include:

- adding further protection to culverts that might be blocked by debris;
- better surface drainage so that water is dispersed off the road more readily; and
- reducing water concentration by means of additional cross drains and mitre drains to lower the volume of water that each one needs to deal with.

In areas prone to water shortages, various mitigating measures could potentially be taken as a Complementary Intervention. These include converting used borrow pits into ponds or reservoirs in areas where local development plans have identified a need for increased water storage.

The most significant risks are likely, in most of the country, to relate to increased intensity of local storms. Whatever precautions are taken, the risk will always remain of a storm intensity that exceed what the structure has been designed for. In the case of submersible structures such as vented fords and drifts, this should not be a problem. In the case of larger and higher structures such as culverts and bridges consideration should be given to designing the approach embankments in a manner that ensures that they will overtop and potentially fail sacrificially before the structural integrity of the culvert/bridge itself is compromised.

Erosion is a serious problem in many areas and climate change is likely to make matters worse. There are also likely to be more severe geotechnical problems (such as those related to slope stability) caused by climate change.

Ensuring that the drainage system is working correctly is essentially a maintenance issue although there may be examples of poorly designed culverts with improper alignment or grade relative to the channels and ditch lines that will need to be repaired or replaced, usually after failures have occurred.

13.4 PAVEMENT DRAINAGE

Effective drainage of highway pavements is essential to maintain the levels of service and to traffic safety of roads. Water on the pavement can interrupt traffic, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles. The substructures of a roadway are also highly influenced by intrusion of water. Pavement drainage requires consideration of surface drainage, gutter flow, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of storm water on the pavement surface.

Spread and design frequency are not independent. The implications of the use of a criteria for spread of one-half of a traffic lane is considerably different for one design frequency than for a lesser frequency. It also has different implications for a low-traffic, low-speed highway than for a higher classification highway. These subjects are central to the issue of highway pavement drainage and important to highway safety.

13.4.1 SELECTION OF DESIGN FREQUENCY AND DESIGN SPREAD

The objective of highway storm drainage design is to provide for safe passage of vehicles during the design storm event. The design of a drainage system for a kerbed highway pavement section is to collect runoff in the gutter and convey it to pavement inlets in a manner that provides reasonable safety for traffic and pedestrians at a reasonable cost. As spread from the kerb increases, the risks of traffic accidents and delays, and the nuisance and possible hazard to pedestrian traffic increase.

The process of selecting the recurrence interval and spread for design involves decisions regarding acceptable risks of accidents and traffic delays and acceptable costs for the drainage system. Risks associated with water on traffic lanes are greater with high traffic volumes, high speeds, and higher highway classifications than with lower volumes, speeds, and highway classifications.

Table 13.27 provides suggested minimum design frequencies and spread based on the design class of the highway and traffic speed.

Table 13.27 Suggested Minimum Design Frequency and Spread

DESIGN CLASS		DESIGN FREQUENCY (YEARS)	DESIGN SPREAD*
A, B, C	< 70 km/hr	10	Shoulder + 1 m
	> 70 km/hr	10	Shoulder
	Sag Point	50	Shoulder + 1 m
D	< 70 km/hr	10	1/2 Driving Lane
	> 70 km/hr	10	Shoulder
	Sag Point	10	1/2 Driving Lane
E	Low ADT**	5	1/2 Driving Lane
	High ADT	10	1/2 Driving Lane
	Sag Point	10	1/2 Driving Lane

*A gutter flowing at capacity should not flood more than 1.5m into carriageway

**ADT = Average daily traffic

13.4.2 SELECTION OF CHECK STORM AND SPREAD

A check storm should be used to assess the impact of flooding during less frequent events. Also, inlets should always be evaluated for a check storm when a series of inlets terminate at a sag vertical curve where ponding to hazardous depths could occur.

The frequency selected for the check storm (see **Table 13.2**) should be based on the same considerations used to select the design storm, i.e., the consequences of spread exceeding that chosen for design and the potential for ponding. Where no significant ponding can occur, check storms are normally unnecessary.

13.4.3 SURFACE DRAINAGE

When rain falls on a sloped pavement surface, it forms a thin film of water that increases in thickness as it flows to the edge of the pavement. Factors which influence the depth of water on the pavement are the length of flow path, surface texture, surface slope, and rainfall intensity.

As the depth of water on the pavement increases, the potential for vehicular hydroplaning increases. For the purposes of highway drainage, a discussion of hydroplaning is presented and design guidance for the following drainage elements is presented:

i. kerb and gutter design
ii. roadside and median ditches
iii. bridge decks
iv. median barriers
v. impact attenuators

13.4.3.1 HYDROPLANING

As the depth of water flowing over a roadway surface increases, the potential for hydroplaning increases. When a rolling tire encounters a film of water on the roadway, the water is channelled through the tire tread pattern and through the surface roughness of the pavement. Hydroplaning occurs when the drainage capacity of the tire tread pattern and the pavement surface is exceeded, and the water begins to build up in front of the tire. As the water builds up, a water wedge is created, and this wedge produces a hydrodynamic force which can lift the tire off the pavement surface. This is considered as full dynamic hydroplaning and, since water offers little shear resistance, the tire loses its tractive ability, and the driver has a loss of control of the vehicle. Hydroplaning can occur at speeds of 89 km/h with a water depth of 2 mm.

Hydroplaning is a function of the water depth, roadway geometry, vehicle speed, tread depth, tire inflation pressure, and conditions of the pavement surface.

The hydroplaning potential of a roadway surface can be reduced by the following:

i. Design the highway geometries to reduce the drainage path lengths of the water flowing over the pavement. This will prevent flow build-up.
ii. Increase the pavement surface texture depth by such methods as grooving of Portland cement concrete. An increase of pavement surface texture will increase the drainage capacity at the tire pavement interface.
iii. The use of open graded asphaltic pavements has been shown to greatly reduce the hydroplaning potential of the roadway surface. This reduction is due to the ability of the water to be forced through the pavement under the tire. This releases any hydrodynamic pressures that are created and reduces the potential for the tire to hydroplane.
iv. The use of drainage structures along the roadway to capture the flow of water over the pavement will reduce the thickness of the film of water and reduce the hydroplaning potential of the roadway surface.

13.4.3.2 LONGITUDINAL SLOPE

Experience has shown that the recommended minimum values of roadway longitudinal slope given in **Chapter 7** will provide safe, acceptable pavement drainage. In addition, the following general guidelines are presented:

i. A minimum longitudinal gradient is more important for a kerbed pavement than for an unkerbed pavement since the water is constrained by the kerb. However, flat gradients on unkerbed pavements can lead to a spread problem if vegetation is allowed to build up along the pavement edge.
ii. Desirable gutter grades should not be less than 0.4 percent for kerbed pavements. Minimum grades can be maintained in very flat terrain by use of a rolling profile, or by warping the cross slope to achieve rolling gutter profiles.
iii. To provide adequate drainage in sag vertical curves, a minimum slope of 0.4 percent should be maintained within 30 meters of the low point of the curve. This is accomplished where the length (L) of the curve in meters divided by the algebraic difference in grades (G) in percent (K) is equal to or less than 75 (K=L/G).
   The drainage criterion differs from other criteria in that the length of sag vertical curve determined for it is a maximum, whereas the length for any other criterion is a minimum.
iv. Point (iii) can also be applied to crest vertical curves. However, in the case of crest vertical curves, it is not intended that K of 75m per percent grade be considered a design maximum, but merely a value beyond which drainage should be more carefully designed.

13.4.3.3 KERB AND GUTTER

Kerbs are normally used at the outside edge of pavements for low-speed, highway facilities, and in some instances adjacent to shoulders on moderate to high-speed facilities. They serve the following purposes:

i. contain the surface runoff within the roadway and away from adjacent properties
ii. prevent erosion on fill slopes
iii. provide pavement delineation
iv. enable the orderly development of property adjacent to the roadway.

Gutters formed in combination with kerbs are available in 0.3 through 1.0 m widths. Gutter cross slopes may be the same as that of the pavement or may be designed with a steeper cross slope, usually 80 mm per meter steeper than the shoulder or parking lane (if used). Chapter 7 states that 9% slope is a common maximum cross slope.

A kerb and gutter combination forms a triangular channel that can convey runoff equal to or less than the design flow without interruption of the traffic. When a design flow occurs, there is a spread or widening of the conveyed water surface. The water spreads to include not only the gutter width, but also parking lanes or shoulders, and portions of the travelled surface. **Spread** is what concerns the hydraulic engineer in kerb and gutter flow. The distance of the spread, T, is measured perpendicular to the kerb face to the extent of the water on the roadway and is shown in **Figure 13.5**. Limiting this width becomes a very important design criterion.

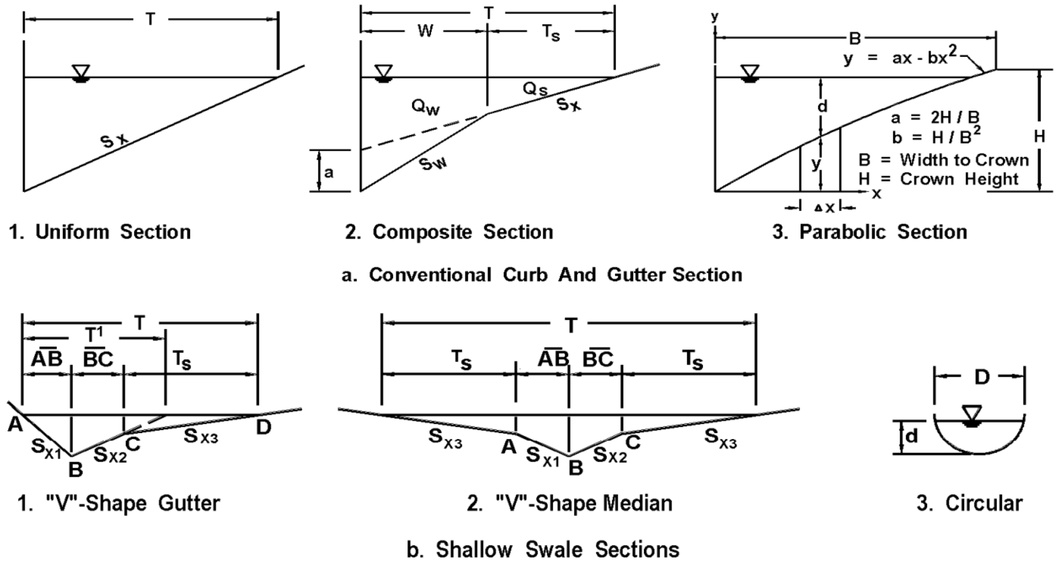


Figure 13.5 Typical gutter sections

Where practical, runoff from cut slopes and other areas draining toward the roadway should be intercepted before it reaches the highway. By doing so, the deposition of sediment and other debris on the roadway as well as the amount of water which must be carried in the gutter section will be minimized. Where kerbs are not needed for traffic control, shallow ditch sections at the edge of the roadway pavement or shoulder offer advantages over kerbed sections by providing less of a hazard to traffic than a near-vertical kerb and by providing hydraulic capacity that is not dependent on spread on the pavement. These ditch sections are particularly appropriate where kerbs have historically been used to prevent water from eroding fill slopes.

13.4.3.4 ROADSIDE AND MEDIAN CHANNELS

Roadside channels are commonly used with unkerbed roadway sections to convey runoff from the highway pavement and from areas which drain toward the highway. Due to right-of-way limitations, roadside channels cannot be used on most urban arterials. They can be used in cut sections, depressed sections, and other locations where sufficient right-of-way is available and driveways or intersections are infrequent. To prevent drainage from the median areas from running across the travel lanes, slope median areas and inside shoulders to a centre swale. This design is particularly important for high-speed facilities and for facilities with more than two lanes of traffic in each direction.

13.4.3.5 BRIDGE DECKS

Bridge deck drainage is similar to that of kerbed roadway sections. Effective bridge deck drainage is important because, hydroplaning often occurs at shallower depths on bridges due to the reduced surface texture of concrete bridge decks.

Bridge deck drainage is often less efficient than roadway sections because cross slopes are flatter, parapets collect large amounts of debris, and drainage inlets or typical bridge scuppers are less hydraulically efficient and more easily clogged by debris. Because of the difficulties in providing for and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. For similar reasons, zero gradients and sag vertical curves should be avoided on bridges. Additionally, runoff from bridge decks should be collected immediately after it flows onto the subsequent roadway section where larger grates and inlet structures can be used.

13.4.3.6 MEDIAN BARRIERS

Slope the shoulder areas adjacent to median barriers to the centre to prevent drainage from running across the travelled pavement. Where median barriers are used, and particularly on horizontal curves with associated superelevations, it is necessary to provide inlets or slotted drains to collect the water accumulated against the barrier. Additionally, a piping system can be used to convey water through the barrier.

13.4.3.7 IMPACT ATTENUATORS

The location of impact attenuator systems should be reviewed to determine the need for drainage structures in these areas. With some impact attenuator systems, it is necessary to have a clear or unobstructed opening as traffic approaches the point of impact to allow a vehicle to impact the system head on. If the impact attenuator is placed in an area where superelevation or other grade separation occurs, grate inlets and/or slotted drains may be needed to prevent water from running through the clear opening and crossing the highway lanes or ramp lanes. kerb, kerb-type structures or swales cannot be used to direct water across this clear opening as vehicle vaulting could occur.

13.4.4 FLOW IN GUTTERS

A pavement gutter is defined as a section of pavement adjacent to the roadway which conveys water during a storm runoff event. It may include a portion or all of a travel lane. Gutter sections can be categorized as conventional or shallow swale type as illustrated in **Figure 13.5**. Conventional kerb and gutter sections usually have a triangular shape with the kerb forming the near-vertical leg of the triangle. Conventional gutters may usually have a straight cross slope (**Figure 13.5-a1**.), or a composite cross slope where the gutter slope varies from the pavement cross slope (**Figure 13.5-a2**).

13.4.5 CAPACITY RELATIONSHIP

Gutter Flow calculations are necessary to establish the spread of water on the shoulder, parking lane, or pavement section. A modification of the Manning's equation can be used for computing flow in triangular channels. The modification is necessary because the hydraulic radius in the equation does not adequately describe the gutter cross section, particularly where the top width of the water surface may be more than 40 times the depth at the kerb. To compute gutter flow, the Manning's equation is integrated for an increment of width across the section. The resulting is **Equations (13.45)** and **(13.46)**.

$$Q = \frac{0.376}{n} S_x^{1.67} S_L^{0.5} T^{2.67} \tag{13.45}$$

or in terms of T

$$T = \left(\frac{Qn}{0.376 S_x^{1.67} S_L^{0.5}} \right)^{0.375} \tag{13.46}$$

Where,

n = Manning's coefficient (**Table 13.9**)
Q = flow rate, m^3/s
T = width of flow (spread), m
S_x = cross slope, m/m
S_L = longitudinal slope, m/m

Equations (13.45) and **(13.46)** neglects the resistance of the kerb face since this resistance is negligible.

Spread on the pavement and flow depth at the kerb are often used as criteria for spacing pavement drainage inlets. Depth of flow is given by **Equation (13.47)**.

$$d = TS_x \tag{13.47}$$

Where,

d = depth of flow, m
T= width of flow (spread), m
S_x = cross slope, m/m

13.4.6 CONVENTIONAL KERB AND GUTTER SECTIONS

Conventional gutters begin at the inside base of the kerb and usually extend from the kerb face toward the roadway centreline a distance of 0.3 to 1 meter. As illustrated in **Figure 13.5**, gutters can have uniform, composite, or curved sections. Uniform gutter sections have a cross-slope which is equal to the cross-slope of the shoulder or travel lane adjacent to the gutter. Gutters having composite sections are depressed in relation to the adjacent pavement slope. That is, the paved gutter has a cross-slope which is steeper than that of the adjacent pavement. Curved gutter sections are sometimes found along older city streets or highways with curved pavement sections. Procedures for computing the capacity of kerb and gutter sections follows.

13.4.6.1 CONVENTIONAL GUTTERS OF UNIFORM CROSS SLOPE

The nomograph in **Chart 1** solves **Equation (13.45)** for gutters having triangular cross sections.

Example 13.3 illustrates its use for the analysis of conventional gutters with uniform cross slope.

Example 13.3

Given: Gutter section illustrated in **Figure 13.5-a1**.

S_L = 0.010 m/m

S_x = 0.020 m/m

n = 0.016

Find: (1) Spread at a flow of 0.05 m^3/s

(2) Gutter flow at a spread of 2.5 m

Solution (1):

Step 1. Compute spread, T, using **Equation (13.45)** or from **Chart 1**.

$$T = \left(\frac{Qn}{0.376 S_x^{1.67} S_L^{0.5}}\right)^{0.375}$$

$$T = \left(\frac{0.05(0.016)}{0.376(0.02^{1.67})(0.01^{0.5})}\right)^{0.375}$$

T = 2.7 m

Solution (2):

Step 1. Using **Equation (13.45)** or **Chart 1** with T = 2.5 m and the information given above, determine Qn.

$$Qn = 0.376 S_x^{1.67} S_L^{0.5} T^{2.67}$$

$$Qn = 0.376(0.02)^{1.67}(0.01)^{0.5}(2.5)^{2.67}$$

$$Qn = 0.00063\ m^3/s$$

Step 2. Compute Q from Qn determined in Step 1.

$$Q = Qn/n$$

$$Q = 0.00063/0.016$$

$$Q = 0.039 m^3/s$$

13.4.6.2 COMPOSITE GUTTER SECTIONS

The design of composite gutter sections requires consideration of flow in the depressed segment of the gutter, Q_w. **Equation (13.48)**, is provided for use with **Equations (13.49)** and **(13.50)** to determine the flow in a width of gutter in a composite cross section, W, less than the total spread, T.

$$E_O = 1/\left\{1 + \frac{\frac{S_W}{S_x}}{\left[1 + \frac{\frac{S_W}{S_x}}{\frac{T}{W} - 1}\right]^{2.67} - 1}\right\} \tag{13.48}$$

$$Q_w = Q - Q_s \tag{13.49}$$

$$Q = \frac{Q_s}{(1 - E_o)} \tag{13.50}$$

Where,

Q_w = flow rate in the depressed section of the gutter, m^3/s

Q = gutter flow rate, m^3/s

Q_s = flow capacity of the gutter section above the depressed section, m^3/s

E_o = ratio of flow in a chosen width (usually the width of a grate) to total gutter flow (Q_w/Q) $S_w = S_x + a/W$ (**Figure 13.5-a.2**)

Figure 13.6 illustrates a design chart for a composite gutter with a 0.60 m wide gutter section with a 50 mm depression at the kerb that begins at the projection of the uniform cross slope at the kerb face.

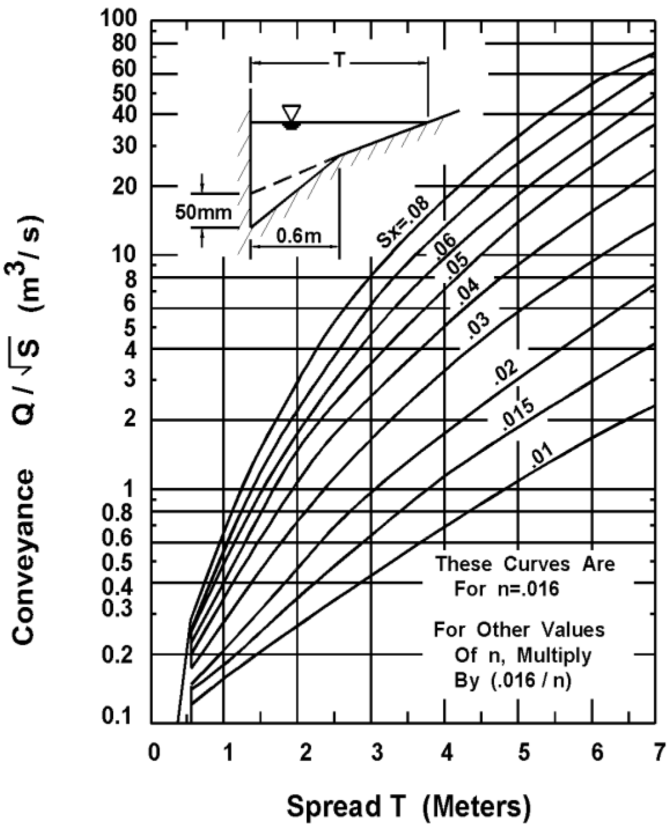


Figure 13.6 Conveyance - spread curves for a composite gutter section

Example 13.4

Given: Gutter section illustrated in **Figure 13.5-a.2** with

W = 0.6 m

S_L = 0.01

S_x = 0.020

n = 0.016

Gutter depression, a = 50 mm

Find: (1) Gutter flow at a spread, T, of 2.5 m

(2) Spread at a flow of 0.12 m^3/s

Solution (1):

Step 1. Compute the cross slope of the depressed gutter, S_w, and the width of spread from the junction of the gutter and the road to the limit of the spread, T_s.

$S_w = a / W + S_x$

$S_w = [(50)/(1000)]/(0.6) + (0.020)$

S_w = 0.103 m/m

T_s = T - W = 2.5 m - 0.6 m

T_s = 1.9 m

Step 2. From **Equation (13.45)** or from **Chart 1** (using T_s)

$$\mathrm{Q}_s\mathrm{n} = 0.376\mathrm{S}_\mathrm{x}^{1.67}\mathrm{S}_\mathrm{L}^{0.5}T_s^{2.67}$$

$$\mathrm{Q}_s\mathrm{n} = 0.376(0.02)^{1.67}(0.01)^{0.5}(1.9)^{2.67}$$

$$\mathrm{Q}_s\mathrm{n} = 0.00031\ \mathrm{m}^3/s \text{ , and}$$

$$\mathrm{Q}_s = (\mathrm{Q}_s\mathrm{n})/\mathrm{n} \ \ = 0.00031/0.016$$

$$\mathrm{Q}_s = 0.019\ \mathrm{m}^3/s$$

Step 3. Determine the gutter flow, Q, using **Equation (13.48)** or **Chart 2**

T / W = 2.5 / 0.6 = 4.17

S_w / S_x = 0.103 / 0.020 = 5.15

$$\mathrm{E_O} = 1/\left\{1 + \frac{\mathrm{S_W/S_x}}{\left[1 + \frac{\mathrm{S_W/S_x}}{\frac{\mathrm{T}}{\mathrm{W}} - 1}\right]^{2.67} - 1}\right\}$$

$$E_O = 1/\left\{1 + \frac{5.15}{\left[1 + \frac{5.15}{4.17 - 1}\right]^{2.67} - 1}\right\}$$

$E_O = 0.70$

Or from **Chart 2**, for W/T = 0.6/2.5 = 0.24

$E_o = Q_w / Q = 0.70$

$Q = Q_s / (1 - E_o)$

Q = 0.019 / (1 - 0.70)

Q = 0.06 m^3/s

Solution (2):

Since the spread cannot be determined by a direct solution, an iterative approach must be used.

Step 1. Try $Q_s = 0.04\ m^3/s$

Step 2. Compute Q_w

$Q_w = Q - Q_s = 0.12 - 0.04$

$Q_w = 0.08\ m^3/s$

Step 3. Using **Equation (13.48)** or from **Chart 2**, determine W/T ratio

$E_o = Q_w / Q = 0.08 / 0.12 = 0.67$

$S_w/S_x = 0.103 / 0.020 = 5.15$

W/T = 0.23 from **Chart 2**

Step 4. Compute spread based on the assumed Q_s

T = W / (W/T) = 0.6 / 0.23

T = 2.6 m

Step 5. Compute Ts based on assumed Q_s

$T_s = T - W = 2.6 - 0.6 = 2.0$ m

Step 6. Use **Equation (13.45)** or **Chart 1** to determine Q_s for computed T_s

$Q_s n = 0.376 S_x^{1.67} S_L^{0.5} T_s^{2.67}$

$Q_s n = 0.376(0.02)^{1.67}(0.01)^{0.5}(2.0)^{2.67}$

$Q_s n = 0.00035\ m^3/s$

$Q_s = Q_s n/n = 0.00035/0.016$

$Q_s = 0.022\ m^3/s$

Step 7. Compare computed Q_s with assumed Q_s.

Q_s assumed, 0.04 > Q_s computed, 0.022

Not close - try again

Step 8. Try a new assumed Q_s and repeat Steps 2 through 7.

Assume $Q_s = 0.058\ m^3/s$

$Q_w = 0.12 - 0.058 = 0.062\ m^3/s$

$E_o = Q_w / Q = 0.062 / 0.12 = 0.52$

$S_w / S_x = 5.15$

$W / T = 0.17$

$T = 0.60 / 0.17 = 3.5\ m$

$T_s = 3.5 - 0.6 = 2.9\ m$

$Q_s n = 0.00094\ m^3/s$

$Q_s = 0.00094 / 0.016 = 0.059\ m^3/s$

Q_s assumed = 0.058 m^3/s close to 0.059 m^3/s = Qs computed

13.4.7 SHALLOW SWALE SECTIONS

Where kerbs are not needed for traffic control, a small swale section of circular or V-shape may be used to convey runoff from the pavement. As an example, the control of pavement runoff on fills may be needed to protect the embankment from erosion. Small swale sections may have sufficient capacity to convey the flow to a location suitable for interception.

13.4.7.1 V-SECTIONS

Chart 1 can be used to compute the flow in a shallow V-shaped section. When using **Chart 1** for V-shaped channels, the cross slope, S_x is determined by **Equation (13.51)**.

$$S_x = \frac{S_{x1} S_{x2}}{(S_{x1} + S_{x2})} \tag{13.51}$$

Example 13.5 demonstrates the use of **Chart 1** to analyse a V-shaped shoulder gutter. Analysis of a V-shaped gutter resulting from a roadway with an inverted crown section is illustrated in **Example 13.6**.

Example 13.5

Given: V-shaped roadside gutter (**Figure 13.5-b.1**) with

$S_L = 0.01$ $\quad S_{x1} = 0.25$ $\quad S_{x3} = 0.02$

$n = 0.016$ $\quad S_{x2} = 0.04$ $\quad \overline{BC} = 0.6\ m$

Find: Spread at a flow of 0.05 m^3/s

Solution:

Step 1. Calculate S_x using **Equation (13.51)** assuming all flow is contained entirely in the V-shaped

gutter section defined by S_{x1} and S_{x2}.

$S_x = S_{x1}\ S_{x2} / (S_{x1} + S_{x2}) = (0.25)\ (0.04) / (0.25 + 0.04)$

$S_x = 0.0345$

Step 2. Using **Equation (13.45)** or **Chart 1** find the hypothetical spread, T', assuming all flow contained entirely in the V-shaped gutter.

$T' = [(Qn)/(K_u\ S_x^{1.67}\ S_L^{0.5})]^{0.375}$

$T' = [(0.05)(0.016)/\{(0.376)(0.0345)^{1.67}(0.01)^{0.5}\}]^{0.375}$

$T' = 1.94$ m

Step 3. To determine if T' is within S_{x1} and S_{x2}, compute the depth at point B in the V-shaped gutter knowing and S_{x2} $\overline{BC}$. Then knowing the depth at B, the distance can be $\overline{AB}$ computed.

$d_B = \overline{BC}S_{x2}$
$= (0.6)(0.04)$
$= 0.024$ m

$\overline{AB} = d_B/S_{x1}$
$= (0.024) / (0.25)$
$= 0.096$ m

$\overline{AC} = \overline{AB} + \overline{BC}$
$= 0.096 + 0.60$
$= 0.7$m

0.7 m < T' therefore, spread falls outside V-shaped gutter section. An iterative solution technique must be used to solve for the section spread, T, as illustrated in the following steps.

Step 4. Solve for the depth at point C, d_c, and compute an initial estimate of the spread, along, $T_{\overline{BD}}$ along $\overline{BD}$,

$d_c = d_B - \overline{BC}\,(S_{x2})$

From the geometry of the triangle formed by the gutter, an initial estimate for d_B is determined as

$(d_B/0.25) + (d_B/0.04) = 1.94$

$d_B = 0.067$ m

$d_c = 0.067 - (0.60)(0.04) = 0.043$ m

$T_s = dc / S_{x3} = 0.043 / 0.02 = 2.15$ m

$T_{\overline{BD}} = T_s + \overline{BC} = 2.15 + 0.6 = 2.75$ m

Step 5. Using a spread along $\overline{BD}$ equal to 2.75 m and develop a weighted slope for S_{x2} and S_{x3}.

0.6 m at S_{x2} (0.04) and 2.15 m at S_{x3} (0.02)

$$\frac{(0.6)(0.04) + (2.15)\,(0.02)}{2.75} = 0.0243$$

Use this slope along with S_{x1}, find S_x using **Equation (13.51)**

$$S_x = \frac{S_{x1}S_{x2}}{(S_{x1} + S_{x2})}$$

$$= \frac{(0.25)\,(0.0243)}{(0.25 + 0.0243)} = 0.0221$$

Step 6. Using **Equation (13.45)** or **Chart 1**, compute the gutter spread using the composite cross

slope, S_x.

$T = [(Q\ n)/(K_u\ S_x^{1.67}\ S_L^{0.5})]^{0.375}$

$T = [(0.05)(0.016)/\{(0.376)(0.0221)^{1.67}(0.01)^{0.5}\}]^{0.375}$

T = 2.57 m

This (2.57 m) is lower than the assumed value of 2.75 m.

Therefore, assume $T_{\overline{BD}}$ = 2.50 m and repeat Step 5 and Step 6.

Step 5. 0.6 m at S_{x2} (0.04) and 1.90 m at S_{x3} (0.02)

$$\frac{(0.6)(0.04) + (1.90)\ (0.02)}{2.50} = 0.0248$$

Use this slope along with S_{x1}, find S_x using **Equation (13.51)**

$$S_x = \frac{S_{x1} S_{x2}}{(S_{x1} + S_{x2})}$$

$$= \frac{(0.25)\ (0.0248)}{(0.25 + 0.0248)} = 0.0226$$

Step 6. Using **Equation (13.45)** or **Chart 1**, compute the gutter spread using the composite cross

slope, Sx.

$T = [(Q\ n)/(K_u\ S_x^{1.67}\ S_L^{0.5})]^{0.375}$

$T = [(0.05)(0.016)/\{(0.376)(0.0226)^{1.67}(0.01)^{0.5}\}]^{0.375}$

T = 2.53 m

This value of T = 2.53 m is close to the assumed value of 2.50 m, therefore, OK.

Example 13.6

Given: V-shaped gutter as illustrated in **Figure 13.5-b.2** with

$\overline{AB}$ = 1 m

$\overline{BC}$ = 1 m

SL = 0.01

n = 0.016

$S_{x1} = S_{x2} = 0.25$

$S_{x3} = 0.04$

Find: (1) Spread at a flow of 0.7 m^3/s

(2) Flow at a spread of 7 m

Solution (1):

Step 1. Assume spread remains within middle "V" (A to C) and compute S_x

$S_x = (S_{x1}\ S_{x2}) / (S_{x1} + S_{x2})$

$S_x = (0.25)\ (0.25) / (0.25 + 0.25)$

$S_x = 0.125$

Step 2. From **Equation (13.45)** or **Chart 1**

$T = [(Q\ n)/(K_u\ S_x^{1.67}\ SL^{0.5})]^{0.375}$

$T = [(0.70)(0.016)/\{(0.376)\ (0.125)^{1.67}\ (0.01)^{0.5}\}]^{0.375}$

$T = 2.34$ m

Since "T" is outside S_{x1} and S_{x2} an iterative approach (as illustrated in **Example 13.5**) must be used to compute the spread.

Step 3. Treat one-half of the median gutter as a composite section and solve for T' equal to one-half of the total spread.

Q' for T' = ½ Q = 0.5 (0.7) = 0.35 m^3/s

Step 4. Try Q's = 0.05 m3/s

$Q'_w = Q' - Q'_s = 0.35 - 0.05 = 0.30\ m^3/s$

Step 5. Using equation 4-4 or **Chart 2** determine the W/T' ratio

$E'_o = Q'_w/Q' = 0.30/0.35 = 0.86$

$S_w / S_x = S_{x2} / S_{x3} = 0.25 / 0.04 = 6.25$

W/T' = 0.33 from **Chart 2**

Step 6. Compute spread based on assumed Q's

T' = W / (W/T') = 1 / 0.33 = 3.03 m

Step 7. Compute Ts based on assumed Q's

Ts = T' - W = 3.03 - 1.0 = 2.03 m

Step 8. Use **Equation (13.45)** or **Chart 1** to determine Q's for Ts

$Q'_s\ n = K_u\ S_{x3}^{1.67}\ S_L^{0.5}\ T_s^{2.67} = (0.376)\ (0.04)^{1.67}\ (0.01)^{0.5}\ (2.03)^{2.67}$

$Q'_s\ n = 0.00115$

$Q'_s = 0.00115 / 0.016 = 0.072\ m^3/s$

Step 9. Check computed Q'_s with assumed Q'_s

Q'_s assumed = 0.05 < 0.072 = Q's computed

therefore, try a new assumed Q's and repeat steps 4 through 9.

Assume $Q'_s = 0.01$

$Q'_w = 0.34$

$E'_o = 0.97$

$S_w/S_x = 6.25$

$W/T' = 0.50$

$T' = 2.0$ m

$T_s = 1.0$

$Q_s\ n = 0.00017$

$Q_s = 0.01$

Q_s computed = 0.01 = 0.01 = Q_s assumed

$T = 2\ T' = 2\ (2.0) = 4.0$ m

Solution (2):

Analyse in half-section using composite section techniques. Double the computed half-width flow rate to get the total discharge:

Step 1. Compute half-section top width

$T' = T/2 = 7.0 / 2 = 3.5$ m

$T_s = T' - 1.0 = 2.5$ m

Step 2. From equation 4-2 or **Chart 1** determine Qs

$Q_s\ n = K_u\ S_x^{1.67}\ S_L^{0.5}\ T_s^{2.67}$

$Q_s\ n = (0.376)\ (0.04)^{1.67}\ (0.01)^{0.5}\ (2.5)^{2.67}$

$Q_s\ n = 0.0020$

$Q_s = 0.0020 / 0.016 = 0.126$

Step 3. Determine flow in half-section using **Equation (13.48)** or **Chart 2**

$T'/W = 3.5 / 1.0 = 3.5$

$S_w / S_x = 0.25 / 0.04 = 6.25$

$E_o = 1 / \{1 + (S_w/S_x) / [(1 + (S_w/S_x) / (T'/W - 1))^{2.67} - 1]\}$

$E_o = 1 / \{1 + (6.25) / [(1 + (6.25) / (3.5 - 1))^{2.67} - 1]\}$

$E_o = 0.814 = Q'_w / Q = 1 - Q'_s / Q'$

$Q' = Q'_s / (1 - 0.814) = 0.126 / (1 - 0.814)$

$Q' = 0.68\ m^3/s$

$Q = 2\ Q' = 2\ (0.68) = 1.36\ m^3/s$

13.4.7.2 CIRCULAR SECTIONS

Flow in shallow circular gutter sections can be represented by **Equation (13.52)** which is displayed on **Chart 3**. The width of circular gutter section Tw is represented by the chord of the arc which can be computed using **Equation (13.53)**.

$$\frac{d}{D} = K_u \left[\frac{Qn}{D^{2.67} S_L^{0.5}}\right]^{0.488} \qquad (13.52)$$

Where,

d = depth of flow in circular gutter, m
D = diameter of circular gutter, m
K_u = 1.179

$$T_W = 2(r^2 - (r - d)^2)^{0.5} \tag{13.53}$$

Where,

T_w = width of circular gutter section, m

r = radius of flow in circular gutter, m

Example 13.7 illustrates the use of **Chart 3**.

Example 13.7

Given: A circular gutter swale as illustrated in **Figure 13.5-b.3** with a 1.5 m diameter and

$S_L = 0.01$ m/m

$n = 0.016$

$Q = 0.5\ m^3/s$

Find: Flow depth and top width

Solution:

Step 1. Determine the value of

$Q\ n / (D^{2.67}\ S_L^{0.5})$

$= (0.5)(0.016)/[(1.5)^{2.67}\ (0.01)^{0.5}]$

$= 0.027$

Step 2. Using **Equation (13.52)** or **Chart 3**, determine d/D

$d/D = K_u\ [(Q\ n)/\ (D^{2.67}\ S_L^{0.5})]^{0.488}$

$d/D = (1.179)\ [0.027]^{0.488}$

$d/D = 0.20$

$d\ = D\ (d/D) = 1.5\ (0.20)$

$= 0.30$ m

Step 3. Using **Equation (13.53)**, determine T_w

$T_w\ = 2\ [r^2 - (r - d)^2]^{0.5}$

$= 2\ [(0.75)^2 - (0.75 - 0.3)^2]^{0.5}$

$= 1.2$ m

13.4.8 FLOW IN SAG VERTICAL CURVES

As gutter flow approaches the low point in a sag vertical curve the flow can exceed the allowable design spread values as a result of the continually decreasing gutter slope. The spread in these areas should be checked to insure it remains within allowable limits. If the computed spread exceeds design values, additional inlets should be provided to reduce the flow as it approaches the low point.

13.4.9 RELATIVE FLOW CAPACITIES

Examples 4-1 and 4-2 illustrate the advantage of a composite gutter section. The capacity of the section with a depressed gutter in the examples is 70 percent greater than that of the section with a straight cross slope with all other parameters held constant.

Equation (13.45) can be used to examine the relative effects of changing the values of spread, cross slope, and longitudinal slope on the capacity of a section with a straight cross slope.

To examine the effects of cross slope on gutter capacity, **Equation (13.45)** can be transformed as follows into a relationship between S_x and Q as follows (**Equation (13.54)**):

Let

$$K_1 = \frac{n}{K_c S_L^{0.5} T^{2.67}}$$

then

$$S_x^{1.67} = K_1 Q$$

and

$$\left(\frac{S_{x1}}{S_{x2}}\right)^{1.67} = \frac{K_1 Q_1}{K_1 Q_2} = \frac{Q_1}{Q_2} \tag{13.54}$$

Similar transformations can be performed to evaluate the effects of changing longitudinal slope and width of spread on gutter capacity resulting in **Equations (13.55)** and **(13.56)** respectively.

$$\left(\frac{S_{L1}}{S_{L2}}\right)^{0.5} = \frac{Q_1}{Q_2} \tag{13.55}$$

$$\left(\frac{T_1}{T_2}\right)^{2.67} = \frac{Q_1}{Q_2} \tag{13.56}$$

Equations (13.54), **(13.55) and (13.56)** are illustrated in **Figure 13.7**. As illustrated, the effects of spread on gutter capacity are greater than the effects of cross slope and longitudinal slope, as would be expected due to the larger exponent of the spread term. The magnitude of the effect is demonstrated when gutter capacity with a 3 m spread is 18.8 times greater than with a 1 m spread, and 3 times greater than a spread of 2 m.

The effects of cross slope are also relatively great as illustrated by a comparison of gutter capacities with different cross slopes. At a cross slope of 4 percent, a gutter has 10 times the capacity of a gutter of 1 percent cross slope. A gutter at 4 percent cross slope has 3.2 times the capacity of a gutter at 2 percent cross slope.

Little latitude is generally available to vary longitudinal slope in order to increase gutter capacity, but slope changes which change gutter capacity are frequent. **Figure 13.7** shows that a change from S = 0.04 to 0.02 will reduce gutter capacity to 71 percent of the capacity at S = 0.04.

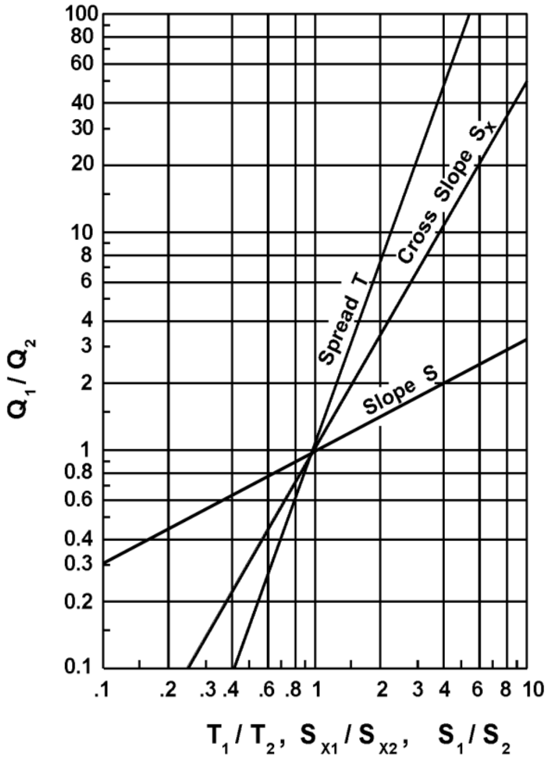


Figure 13.7 Relative effects of spread, cross slope, and longitudinal slope on gutter capacity.

13.4.10 GUTTER FLOW TIME

The Flow time in gutters is an important component of the time of concentration for the contributing drainage area to an inlet. To find the gutter flow component of the time of concentration, a method for estimating the average velocity in a reach of gutter is needed. The velocity in a gutter varies with the flow rate and the flow rate varies with the distance along the gutter, i.e., both the velocity and flow rate in a gutter are spatially varied. The time of flow can be estimated by use of an average velocity obtained by integration of the Manning's equation for the gutter section with respect to time. **Table 13.28** and **Chart 4** can be used to determine the average velocity in triangular gutter sections.

In **Table 13.28**, T_1 and T_2 are the spread at the upstream and downstream ends of the gutter section respectively. T_a is the spread at the average velocity. **Chart 4** is a nomograph to solve **Equation (13.57)** for the velocity in a triangular channel with known cross slope, gutter slope, and spread.

Table 13.28 Spread at Average Velocity in a Reach of Triangular Gutter.

T_1/T_2	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8
T_a/T_2	0.65	0.66	0.68	0.70	0.74	0.77	0.82	0.86	0.90

$$V = \frac{K_u}{n} S_L S_x^{0.67} T^{0.67} \qquad (13.57)$$

Where,

$K_u = 0.752$

V = velocity in the triangular channel, m/s

Example 13.8 illustrates the use of **Table 13.28** and **Chart 4** to determine the average gutter velocity.

Example 13.8

Given: A triangular gutter section with the following characteristics:

$T_1 = 1$ m

$T_2 = 3$ m

$S_L = 0.03$ m/m

$S_x = 0.02$ m/m

$n = 0.016$

Inlet Spacing anticipated to be 100 m.

Find: Time of flow in gutter

Solution:

Step 1. Compute the upstream to downstream spread ratio.

$T_1 / T_2 = 1 / 3 = 0.33$

Step 2. Determine the spread at average velocity interpolating between values in **Table 13.28**.

$(0.30 - 0.33) / (0.3 - 0.4) = X / (0.74-0.70)$

$X = 0.01$

$T_a / T_2 = 0.70 + 0.01$

$= 0.71$

$T_a = (0.71)(3) = 2.13$ m

Step 3. Using **Equation (13.57)** or **Chart 4**, determine the average velocity

$V_a = (K_u / n) S_L^{0.5} S_x^{0.67} T^{0.67}$

$V_a = [0.752/(0.016)](0.03)^{0.5}(0.02)^{0.67}(2.13)^{0.67}$

$V_a = 0.98$ m/s

Step 4. Compute the travel time in the gutter.

$t = L / V = (100) / (0.98) / 60$

$= 1.7$ minutes

13.4.11 DRAINAGE INLET DESIGN

The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Inadequate inlet capacity or poor inlet location may cause flooding on the roadway resulting in a hazard to the traveling public.

13.4.11.1 INLET TYPES

Storm drain inlets are used to collect runoff and discharge it to a storm drainage system. Inlets are typically located in gutter sections, paved medians, and roadside and median ditches. Inlets used for the drainage of highway surfaces can be divided into the following four classes:

i. Grate inlets
ii. Kerb-opening inlets
iii. Slotted inlets
iv. Combination inlets

Grate inlets consist of an opening in the gutter or ditch covered by a grate. Kerb-opening inlets are vertical openings in the kerb covered by a top slab. Slotted inlets consist of a pipe cut along the longitudinal axis with bars perpendicular to the opening to maintain the slotted opening. Combination inlets consist of both a kerb-opening inlet and a grate inlet placed in a side-by-side configuration, but the kerb opening may be located in part upstream of the grate. **Figure 13.8** illustrates each class of inlets. Slotted drains may also be used with grates and each type of inlet may be installed with or without a depression of the gutter.

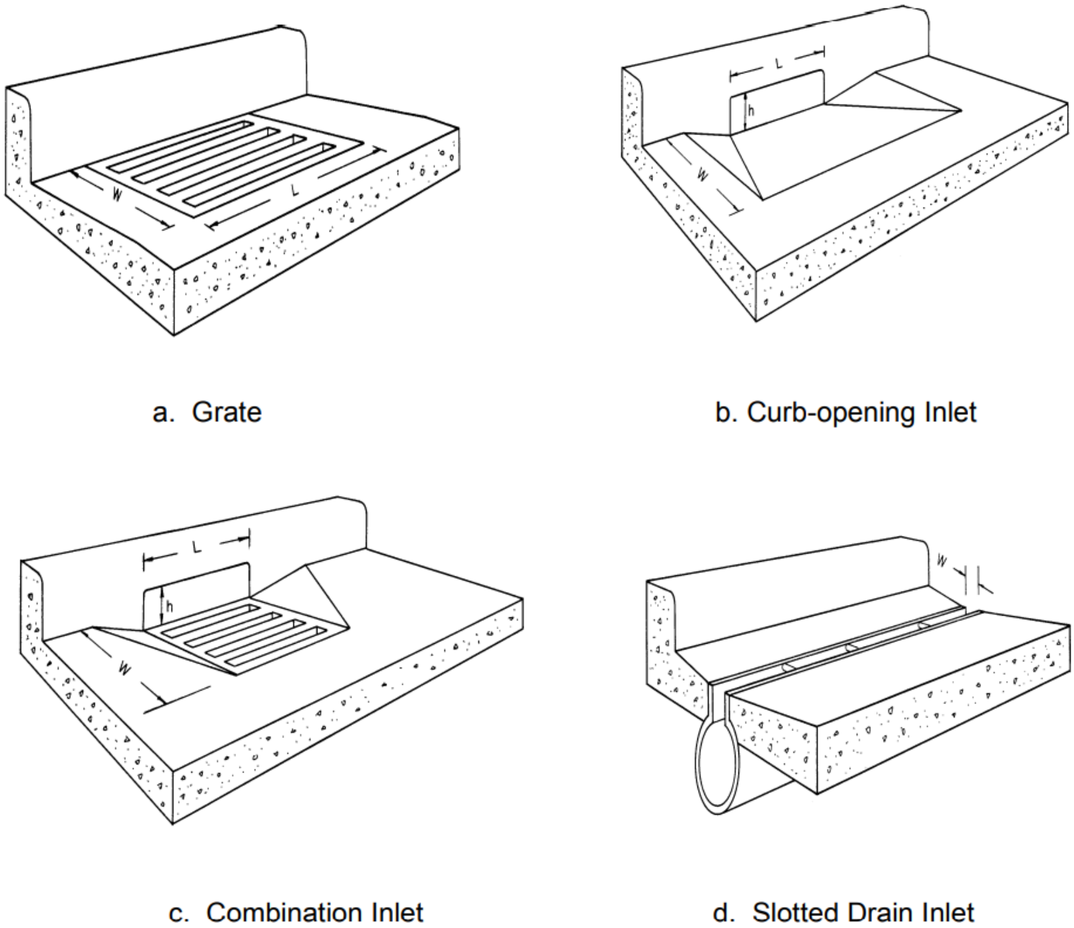


Figure 13.8 Classes of storm drain inlets

13.4.11.2 CHARACTERISTICS AND USES OF INLETS

i. **Grate inlets**, as a class, perform satisfactorily over a wide range of gutter grades. Grate inlets generally lose capacity with increase in grade, but to a lesser degree than kerb opening inlets. The principal advantage of grate inlets is that they are installed along the roadway where the water is flowing. Their principal disadvantage is that they may be clogged by floating trash or debris. For safety reasons, preference should be given to grate inlets where out-of-control vehicles might be involved. Additionally, where bicycle traffic occurs, grates should be bicycle safe.
ii. **Kerb-opening inlets** are most effective on flatter slopes, in sags, and with flows which

typically carry significant amounts of floating debris. The interception capacity of kerb-opening inlets decreases as the gutter grade steepens. Consequently, the use of kerb-opening inlets is recommended in sags and on grades less than 3%. Of course, they are bicycle safe as well.

iii. **Combination inlets** provide the advantages of both kerb opening and grate inlets. This combination results in a high-capacity inlet which offers the advantages of both grate and kerb-opening inlets. When the kerb opening precedes the grate in a "Sweeper" configuration, the kerb-opening inlet acts as a trash interceptor during the initial phases of a storm. Used in a sag configuration, the sweeper inlet can have a kerb opening on both sides of the grate.

iv. **Slotted drain inlets** can be used in areas where it is desirable to intercept sheet flow before it crosses onto a section of roadway. Their principal advantage is their ability to intercept flow over a wide section. However, slotted inlets are very susceptible to clogging from sediments and debris and are not recommended for use in environments where significant sediment or debris loads may be present. Slotted inlets on a longitudinal grade do have the same hydraulic capacity as kerb openings when debris is not a factor.

13.4.11.3 FACTORS AFFECTING INLET INTERCEPTION CAPACITY AND EFFICIENCY ON CONTINUOUS GRADES

Inlet interception capacity, Q_i , is the flow intercepted by an inlet under a given set of conditions. The efficiency of an inlet, E, is the percent of total flow that the inlet will intercept for those conditions. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and, to a lesser extent, pavement roughness. In mathematical form, efficiency, E, is defined by **Equation (13.58)**.

$$E = \frac{Q_i}{Q} \tag{13.58}$$

Where,

E = inlet efficiency
Q = total gutter flow, m^3/s
Q_i = intercepted flow, m^3/s

Flow that is not intercepted by an inlet is termed carryover or bypass and is defined by **Equation (13.59)**.

$$Q_b = Q - Q_i \tag{13.59}$$

Where,

Q_b = bypass flow, m^3/s

The interception capacity of all inlet configurations increases with increasing flow rates, and inlet efficiency generally decreases with increasing flow rates. Factors affecting gutter flow also affect inlet interception capacity. The depth of water next to the kerb is the major factor in the interception capacity of both grate inlets and kerb-opening inlets. The interception capacity of a grate inlet depends on the amount of water flowing over the grate, the size and configuration of the grate and the velocity of flow in the gutter. The efficiency of a grate is dependent on the same factors and total flow in the gutter.

Interception capacity of a kerb-opening inlet is largely dependent on flow depth at the kerb and kerb opening length. Flow depth at the kerb and consequently, kerb-opening inlet interception capacity and efficiency, is increased by the use of a local gutter depression at the kerb-opening or a continuously depressed gutter to increase the proportion of the total flow adjacent to the kerb. Top slab supports placed flush with the kerb line can substantially reduce the interception capacity of kerb openings. Tests have shown that such supports reduce the effectiveness of openings downstream of the support by as much as 50 percent and, if debris is caught at the support, interception by the downstream portion of the opening may be reduced to near zero. If intermediate top slab supports are used, they should be recessed several inches from the kerb line and rounded in shape.

Slotted inlets function in essentially the same manner as kerb opening inlets, i.e., as weirs with flow entering from the side. Interception capacity is dependent on flow depth and inlet length. Efficiency is dependent on flow depth, inlet length and total gutter flow.

The interception capacity of an equal length combination inlet consisting of a grate placed alongside a kerb opening on a grade does not differ materially from that of a grate only. Interception capacity and efficiency are dependent on the same factors which affect grate capacity and efficiency. A combination inlet consisting of a kerb-opening inlet placed upstream of a grate inlet has a capacity equal to that of the kerb-opening length upstream of the grate plus that of the grate, taking into account the reduced spread and depth of flow over the grate because of the interception by the kerb opening. This inlet configuration has the added advantage of intercepting debris that might otherwise clog the grate and deflect water away from the inlet.

13.4.11.4 FACTORS AFFECTING INLET INTERCEPTION CAPACITY IN SAG LOCATIONS

Grate inlets in sag vertical curves operate as weirs for shallow ponding depths and as orifices at greater depths. Between weir and orifice flow depths, a transition from weir to orifice flow occurs. The perimeter and clear opening area of the grate and the depth of water at the kerb affect inlet capacity. The capacity at a given depth can be severely affected if debris collects on the grate and reduces the effective perimeter or clear opening area.

Kerb-opening inlets operate as weirs in sag vertical curve locations up to a ponding depth equal to the opening height. At depths above 1.4 times the opening height, the inlet operates as an orifice and between these depths, transition between weir and orifice flow occurs. The kerb-opening height and length, and water depth at the kerb affect inlet capacity. At a given flow rate, the effective water depth at the kerb can be increased by the use of a continuously depressed gutter, by use of a locally depressed kerb opening, or by use of an increased cross slope, thus decreasing the width of spread at the inlet.

Slotted inlets operate as weirs for depths below approximately 50 mm and orifices in locations where the depth at the upstream edge of the slot is greater than about 120 mm. Transition flow exists between these depths. For orifice flow, an empirical equation derived from experimental data can be used to compute interception capacity. Interception capacity varies with flow depth, slope, width, and length at a given spread. Slotted drains are not recommended in sag locations because they are susceptible to clogging from debris.

13.4.11.5 INTERCEPTION CAPACITY OF INLETS ON GRADE

The capacity of an inlet depends upon its geometry, the cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the kerb is the major factor in the interception capacity of both gutter inlets and kerb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs.

13.4.11.5.1 GRATE INLETS

Grates are effective highway pavement drainage inlets where clogging with debris is not a problem. A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. They are efficient in debris handling. In certain locations where leaves may create constant maintenance problems, the parallel bar grate may be used more efficiently if bicycle traffic is prohibited. Where debris is a problem, consideration shall be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was made to qualitatively simulate field conditions. **Table 13.29** presents the results of debris handling efficiencies of several grates.

Table 13.29 Grate Debris Handling Efficiencies

Rank	Grate	Longitudinal Slope	
		(0.005)	(0.04)
1	Curved Vane	46	61
2	30°- 85 Tilt Bar	44	55
3	45°- 85 Tilt Bar	43	48
4	P - 50	32	32
5	P - 50xl00	18	28
6	45°- 60 Tilt Bar	16	23
7	Recticuline	12	16
8	P - 30	9	20

Curved Vane	Curved vane grate with 83 mm longitudinal bar and 108 mm transverse bar spacing on centre (**Figure 13.9**).
30^0- 85 Tilt Bar	30^0 tilt-bar grate with 83 mm longitudinal bar and 102 mm transverse bar spacing on centre (**Figure 13.10**).
45^0- 85 Tilt Bar	45^0 tilt-bar grate with 83 mm longitudinal bar and 102 mm transverse bar spacing on centre (**Figure 13.11**).
P-50	Parallel bar grate with bar spacing 48 mm on centre (**Figure 13.12**).
P-50x100	Parallel bar grate with bar spacing 48 mm on centre and 10 mm diameter lateral rods spaced at 102 mm on centre (**Figure 13.12**8).
45^0- 60 Tilt Bar	45^0 tilt-bar grate with 57 mm longitudinal bar and 102 mm transverse bar spacing on centre (**Figure 13.11**).
Reticuline	"Honeycomb" pattern of lateral bars and longitudinal bearing bars (**Figure 13.13**).
P-30	Parallel bar grate with 29 mm on centre bar spacing (**Figure 13.14**).

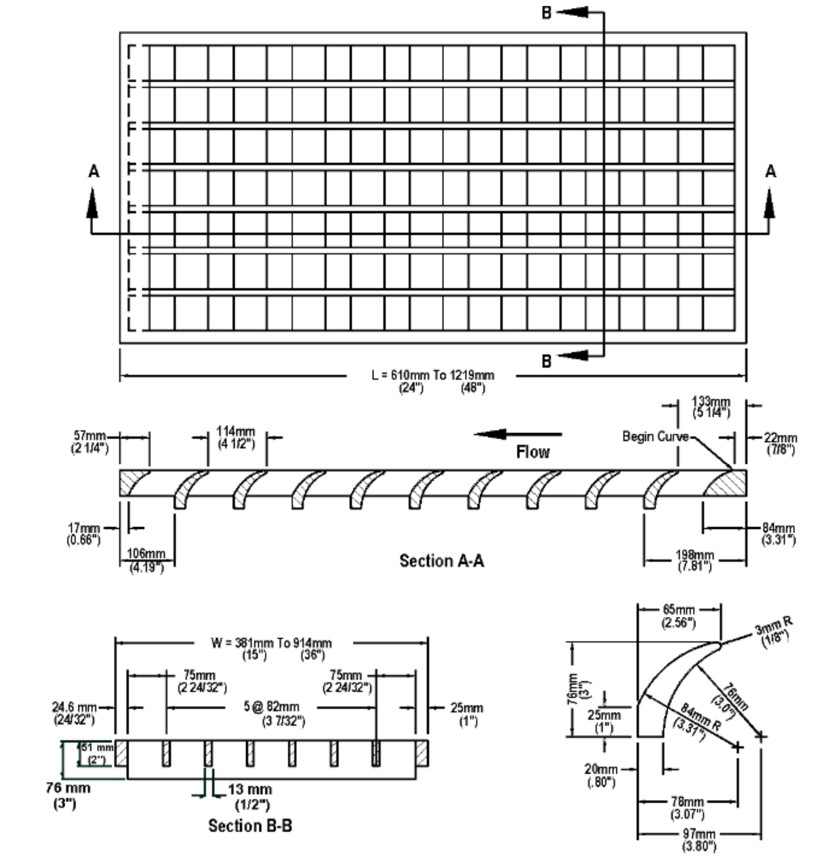


Figure 13.9 Curved vane grate.

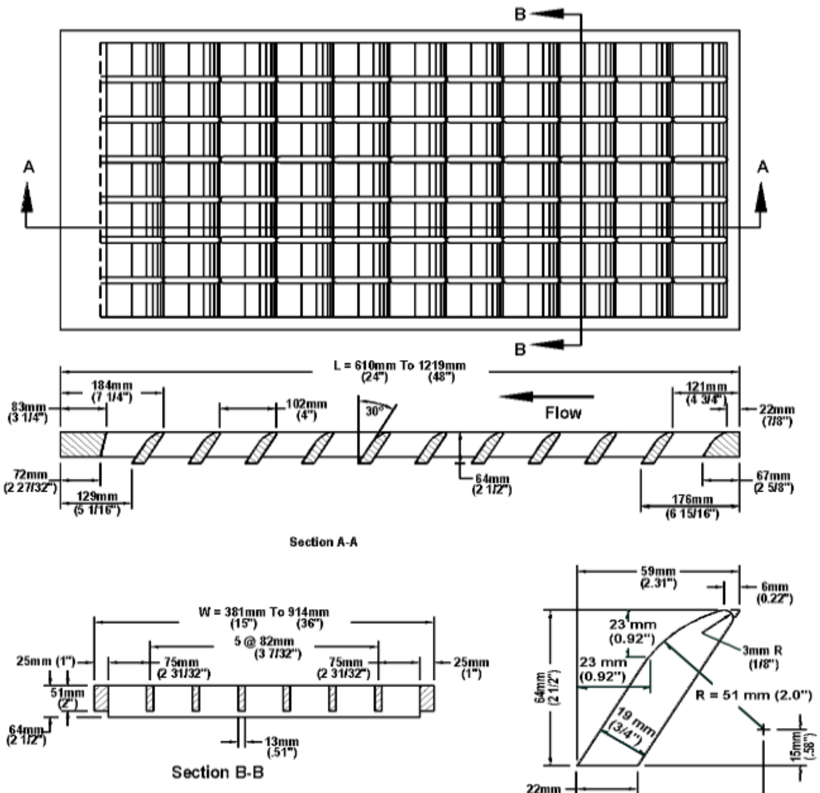


Figure 13.10 30^0- 85 tilt-bar grates

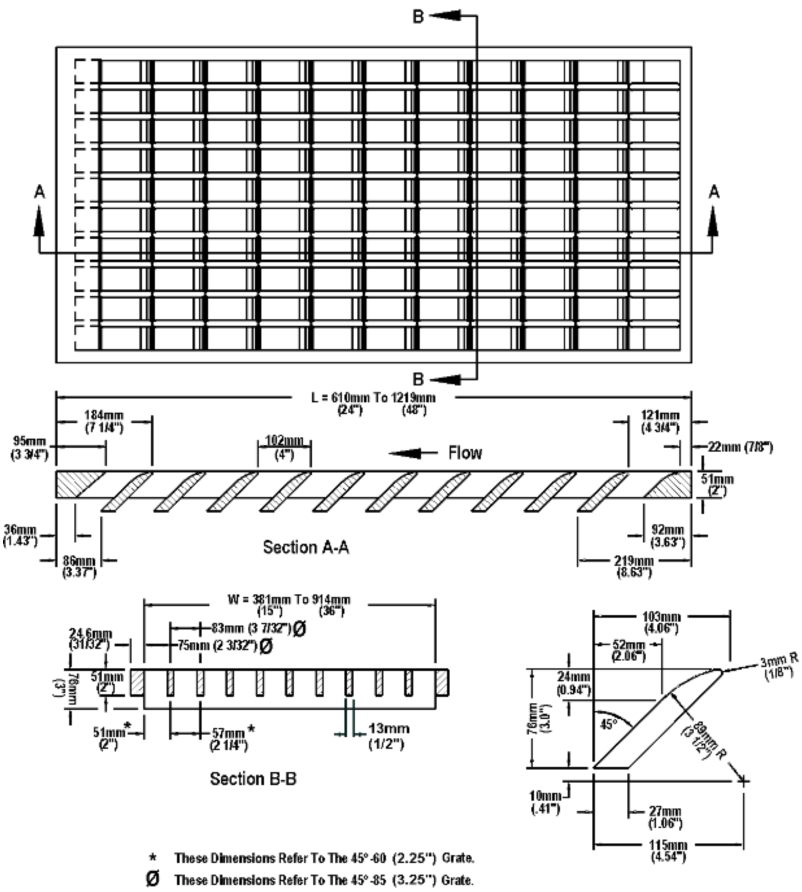


Figure 13.11 45^{0}- 60 and 45^{0}- 85 tilt-bar grates

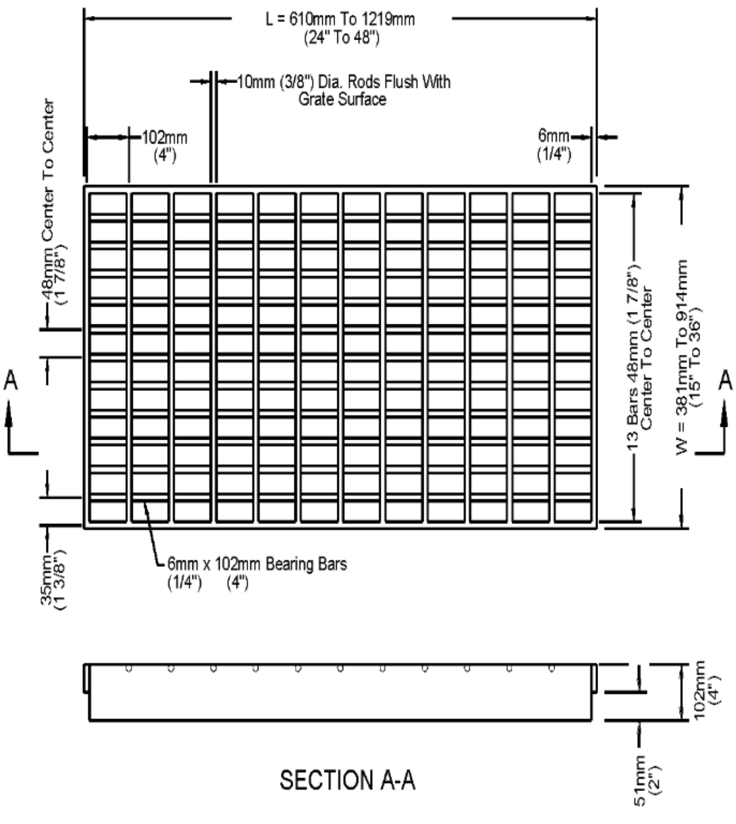


Figure 13.12 P-50 and P-50 x 100 Grate (P-50 is this grate without 10mm transverse rods)

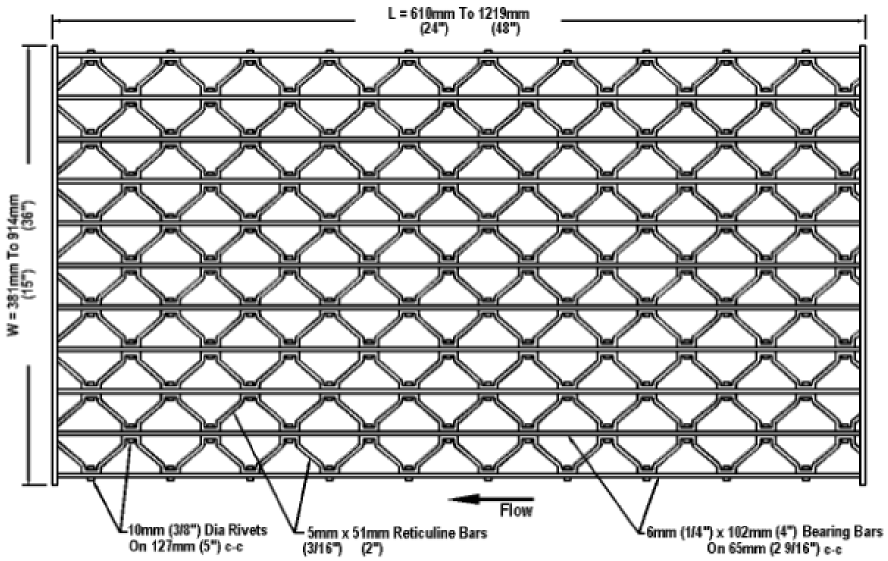


Figure 13.13 Reticuline grate

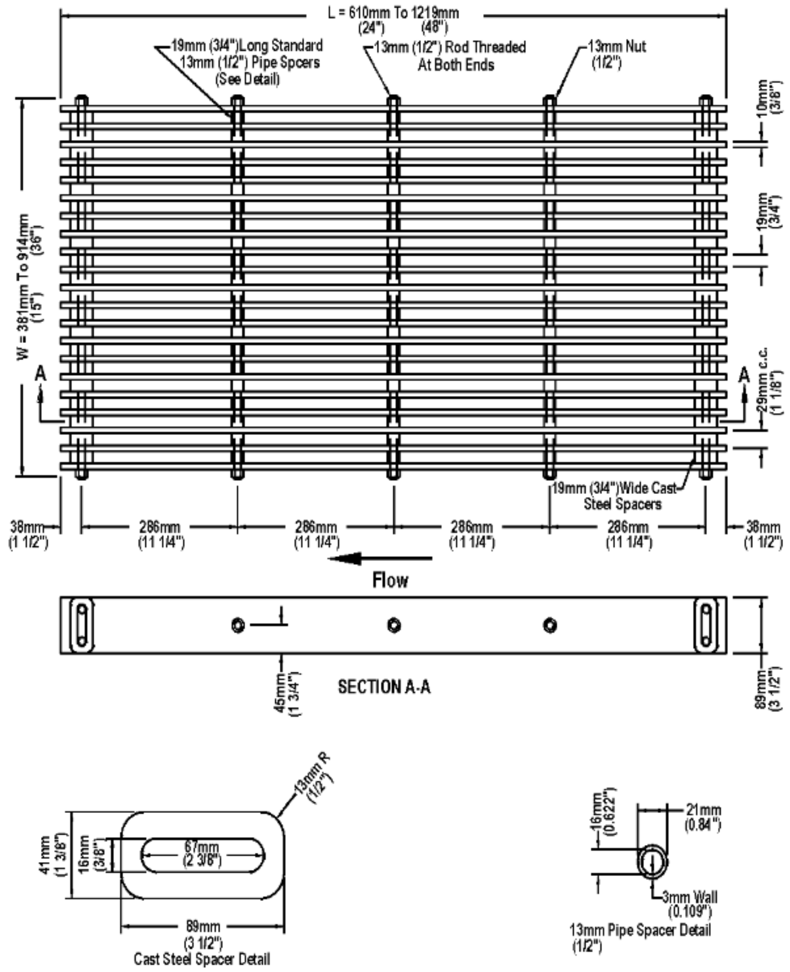


Figure 13.14 P-30 grate

The ratio of frontal flow to total gutter flow, E_o, for a uniform cross slope is expressed by **Equation (13.60)**.

$$E_O = \frac{Q_W}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \qquad (13.60)$$

Where,

Q = total gutter flow, m^3/s

Q_w = flow in width W, m^3/s

W = width of depressed gutter or grate, m

T = total spread of water, m

The ratio of side flow, Q_s, to total gutter flow is given by **Equation (13.61)**.

$$\frac{Q_S}{Q} = 1 - \frac{Q_W}{Q} = 1 - E_O \qquad (13.61)$$

The ratio of frontal flow intercepted to total frontal flow, R_f , is expressed by **Equation (13.62)**.

$$R_f = 1 - 0.295(V - V_o) \qquad (13.62)$$

Where,

V = velocity of flow in the gutter, m/s

V_o = gutter velocity where splash-over first occurs, m/s

(Note: R_f cannot exceed 1.0)

This ratio is equivalent to frontal flow interception efficiency. **Chart 5** provides a solution for **Equation (13.62)** which takes into account grate length, bar configuration, and gutter velocity at which splash-over occurs. The average gutter velocity (total gutter flow divided by the area of flow) is needed to use **Chart 5**. This velocity can also be obtained from **Chart 4**. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by **Equation (13.63)**. **Chart 6** provides a solution to **Equation (13.63)**.

$$R_s = 1/\left(1 + \frac{0.0828V^{1.8}}{S_x L^{2.3}}\right) \qquad (13.63)$$

The efficiency, E, of a grate is expressed as provided in **Equation (13.64)**.

$$E = R_f E_o + R_s(1 - E_o) \qquad (13.64)$$

It is important to recognize that the frontal flow to total gutter flow ratio, E_o, for composite gutter sections assumes by definition a frontal flow width equal to the depressed gutter section width. The use of this ratio when determining a grate's efficiency requires that the grate width be equal to the width of the depressed gutter section, W. If a grate having a width less than W is specified, the gutter flow ratio, E_o, must be modified to accurately evaluate the grate's efficiency. Because an average velocity has been assumed for the entire width of gutter flow, the grate's frontal flow ratio, E'_o, can be calculated by multiplying E_o by a flow area ratio. The area ratio is defined as the gutter flow area in a width equal to the grate width divided by the total flow area in the depressed gutter section. This adjustment is represented in **Equation (13.65)**.

$$E'_o = E_o \left(\frac{A'_W}{A_W} \right) \tag{13.65}$$

Where,

E'_o = adjusted frontal flow area ratio for grates in composite cross sections

A'_w = gutter flow area in a width equal to the grate width, m^2

A_w = flow area in depressed gutter width, m^2

The interception capacity (Q_i) of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow as represented in **Equation (13.66)**. Note that E'_o should be used in place of E_o in **Equation (13.66)** when appropriate.

$$Q_i = EQ = Q[R_f E_o + R_s(1 - E_o)] \tag{13.66}$$

The use of **Charts 5** and **6** are illustrated in the following examples.

Example 13.9

Given: Given the gutter section from **Example 13.4** (illustrated in **Figure 13.5-a.2**) with

T = 2.5 m

S_L = 0.010

W = 0.6 m

S_x = 0.02

n = 0.016

Continuous Gutter depression, a = 50 mm

Find: The interception capacity of a curved vane grate 0.6 m by 0.6 m

Solution: From **Example 13.4**,

S_w = 0.103 m/m

E_o = 0.70

Q = 0.06 m^3/s

Step 1. Compute the average gutter velocity

$V = Q / A = 0.06 / A$

$A = 0.5\ T^2\ S_x + 0.5\ a\ W$

$A = 0.5\ (2.5)^2\ (0.02) + 0.5(0.050)(0.6)$

$A = 0.08\ m^2$

$V = 0.06 / 0.08 = 0.75\ m/s$

Step 2. Determine the frontal flow efficiency using **Chart 5**.

$R_f = 1.0$

Step 3. Determine the side flow efficiency using **Equation (13.63)** or **Chart 6**.

$$R_s = 1/\left(1 + \frac{0.0828V^{1.8}}{S_x L^{2.3}}\right)$$

$$R_s = 1/\left(1 + \frac{0.0828(0.75)^{1.8}}{(0.02)(0.6)^{2.3}}\right)$$

$$R_s = 0.11$$

Step 4. Compute the interception capacity using **Equation (13.66)**.

$$Q_i = Q[R_f E_o + R_s(1 - E_o)$$

$$Q_i = (0.06)[(1.0)(0.70) + (0.11)(1 - 0.70)]$$

$$Q_i = 0.044 m^3/s$$

Example 13.10

Given: Given the gutter section illustrated in **Figure 13.5-a.1** with

$T = 3\ m$

$S_L = 0.04\ m/m$

$S_x = 0.025\ m/m$

$n = 0.016$

Bicycle traffic not permitted

Find: The interception capacity of the following grates:

a. P-50; 0.6 m x 0.6 m

b. Reticuline; 0.6 m x 0.6 m

c. Grates in a. and b. with a length of 1.2 m

Solution:

Step 1. Using **Equation (13.45)** or **Chart 1** determine Q.

$Q = (0.376/n)S_x^{1.67}\ S_L^{0.5}\ T^{2.67}$

$Q = \{(0.376)/(0.016)\}(0.025)^{1.67}(0.04)^{0.5}\ (3)^{2.67}$

$Q = 0.19\ m^3/s$

Step 2. Determine E_o from **Equation (13.48)** or **Chart 2**.

$W/T = 0.6/3$

$= 0.2$

$E_o = Q_w/Q$

$E_o = 1\text{-}(1\text{-}W/T)^{2.67}$

$= 1 - (1 - 0.2)^{2.67}$

$E_o = 0.45$

Step 3. Using **Equation (13.57)** or **Chart 4** compute the gutter flow velocity.

$V = (0.752/n)S_L^{0.5}\ S_x^{0.67}\ T^{0.67}$

$V = \{0.752/(0.016)\}\ (0.04)^{0.5}\ (0.025)^{0.67}\ (3)\ 0.67$

$V = 1.66\ m/s$

Step 4. Using **Equation (13.62)** or chart 5, determine the frontal flow efficiency for each grate.

Using **Equation (13.63)**or **Chart 6**, determine the side flow efficiency for each grate.

Using **Equation (13.66)**, compute the interception capacity of each grate.

Table 13.30 summarizes the results.

Table 13.30 Interception capacities of various types of grating

Grate	Size (Width by length, m)	Frontal Flow Efficiency, R_f	Side Flow Efficiency, R_s	Interception Capacity, Q_i, m^3/s
P - 50	0.6 by 0.6	1.0	0.036	0.091
Reticuline	0.6 by 0.6	0.9	0.036	0.082
P - 50	0.6 by 1.2	1.0	0.155	0.103
Reticuline	0.6 by 1.2	1.0	0.155	0.103

The P-50 parallel bar grate will intercept about 14 percent more flow than the reticuline grate or 48 percent of the total flow as opposed to 42 percent for the reticuline grate. Increasing the length of the grates would not be cost-effective, because the increase in side flow interception is small.

13.4.11.5.2 KERB-OPENING INLETS

Kerb-opening inlets are effective in the drainage of highway pavements where flow depth at the kerb is sufficient for the inlet to perform efficiently. Kerb openings are less susceptible to clogging and offer little interference to traffic operation. They are a viable alternative to grates on flatter grades where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

Kerb opening heights vary in dimension, however, a typical maximum height is approximately 100 to 150 mm. The length of the kerb-opening inlet required for total interception of gutter flow on a pavement section with a uniform cross slope is expressed by **Equation (13.67)**.

$$L_T = 0.817Q^{0.42}S_L^{0.3}\left(\frac{1}{nS_x}\right)^{0.6} \quad (13.67)$$

Where,

L_T = kerb opening length required to intercept 100 percent of the gutter flow, m

S_L = longitudinal slope

Q = gutter flow, m^3/s

n= Manning’s roughness

The efficiency of kerb-opening inlets shorter than the length required for total interception is expressed by **Equation (13.68)**.

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} \quad (13.68)$$

Where,

L = kerb-opening length, m

Chart 7 is a nomograph for the solution of **Equation (13.67)**, and **Chart 8** provides a solution of **Equation (13.68)**.

The length of inlet required for total interception by depressed kerb-opening inlets or kerb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in **Equation (13.67)** in place of S_x. S_e can be computed using **Equation (13.69)**.

$$S_e = S_x + S'_W E_o \quad (13.69)$$

Where,

S'_w = cross slope of the gutter measured from the cross slope of the pavement, S_x, m/m

S'_w = a / [1000 W]W, for W in m; or = S_w - S_x

a = gutter depression, mm

E_o = ratio of flow in the depressed section to total gutter flow determined by the gutter configuration upstream of the inlet

Figure 13.15 shows the depressed kerb inlet for **Equation (13.69)**. E_o is the same ratio as used to compute the frontal flow interception of a grate inlet.

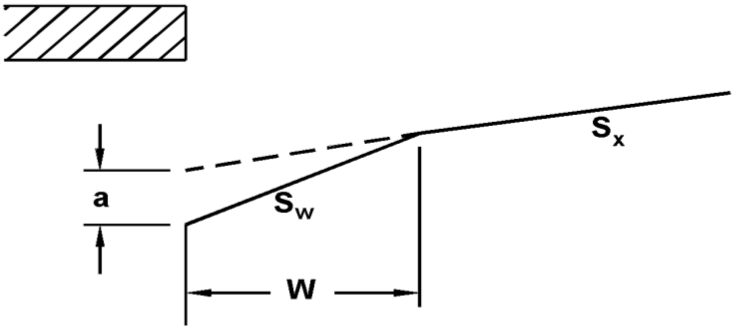


Figure 13.15 Depressed kerb opening inlet

As seen from **Chart 7**, the length of kerb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

Using the equivalent cross slope, S_e, **Equation (13.70)** can be derived from **Equation (13.67)**.

$$L_T = 0.817 Q^{0.42} S_L^{0.3} \left(\frac{1}{nS_e}\right)^{0.6} \tag{13.70}$$

Equation (13.68) is applicable with either straight cross slopes or composite cross slopes. **Charts 7** and **8** are applicable to depressed kerb-opening inlets using S_e rather than S_x.

Equation (13.69) uses the ratio, E_o, in the computation of the equivalent cross slope, S_e. **Example 13.11** demonstrates the procedure to determine spread and then the example uses **Chart 2** to determine E_o. **Example 13.12** demonstrates the use of these relationships to design length of a kerb opening inlet.

Example 13.11

Given: A kerb-opening inlet with the following characteristics:

S_L = 0.01 m/m

S_x = 0.02 m/m

Q= 0.05 m3/s

n = 0.016

Find:

(1) Q_i for a 3 m kerb-opening.

(2) Q_i for a depressed 3 m kerb opening inlet with a continuously depressed kerb section.

a = 25 mm

W = 0.6 m

Solution (1):

Step 1. Determine the length of kerb

opening required for total interception of gutter flow using **Equation (13.67)** or **Chart 7**.

$L_T = 0.817Q^{0.42}\ S_L^{0.3}\ (1/(n\ S_x))^{0.6}$

$L_T = 0.817(0.05)^{0.42}(0.01)^{0.3}(1/[(0.016)(0.02)])^{0.6}$

$L_T = 7.29$ m

Step 2. Compute the kerb-opening efficiency using **Equation (13.68)** or **Chart 8**.

$L / L_T = 3 / 7.29 = 0.41$

$E = 1 - (1 - L / L_T)^{1.8}$

$E = 1 - (1 - 0.41)^{1.8}$

$E = 0.61$

Step 3. Compute the interception capacity.

$Q_i = E\ Q$

$= (0.61)(0.05)$

$Q_i = 0.031\ m^3/s$

Solution (2):

Step 1. Use **Equation (13.48)** (**Chart 2**) and **Equation (13.45)** (**Chart 1**) to determine the W/T ratio.

Determine spread, T, (Procedure from **Example 13.4**, solution 2)

Assume $Q_s = 0.018\ m^3/s$

$Q_w = Q - Q_s$

$= 0.05 - 0.018$

$= 0.032\ m^3/s$

$E_o = Q_w / Q$

$= 0.032 / 0.05$

$= 0.64$

$S_w = S_x + a/W$

$= 0.02 + (25/1000)/0.6$

$S_w = 0.062$

$S_w/S_x = 0.062/0.02 = 3.1$

Use **Equation (13.48)** or **Chart 2** to determine W/T

$W/T = 0.24$

$T = W / (W/T)$

$= 0.6 / 0.24$

$= 2.5$ m

$T_s = T - W$

$= 2.5 - 0.6$

$= 1.9$ m

Use **Equation (13.45)** or **Chart 1** to obtain Q_s

$Q_s = (0.376/n)\ S_x^{1.67}\ S_L^{0.5}\ T_s^{2.67}$

$Q_s = \{(0.376)/(0.016)\}\ (0.02)^{1.67}(0.01)^{0.5}\ (1.9)^{2.67}$

$Qs = 0.019\ m^3/s$

(equals Q_s assumed)

Step 2. Determine efficiency of kerb opening

$S_e = S_x + S'_w E_o = S_x + (a/W)E_o$

$= 0.02 +[(25/1000)/(0.6)](0.64)$

$S_e = 0.047$

Using **Equation (13.70)** or **Chart 7**

$L_T = 0.817\ Q^{0.42}\ S_L^{0.3}\ [1/(n\ S_e)]^{0.6}$

$L_T = (0.817)\ (0.05)^{0.42}\ (0.01)^{0.3}[1/((0.016)(0.047))]^{0.6}$

$L_T = 4.37$ m

Using **Equation (13.68)** or **Chart 8** to obtain kerb inlet efficiency

$L/L_T = 3/4.37 = 0.69$

$E = 1 - (1 - L/L_T)^{1.8}$

$E = 1 - (1 - 0.69)^{1.8}$

$E = 0.88$

Step 3. Compute kerb opening inflow using **Equation (13.58)**

$Q_i = Q\,E$

$= (0.05)\,(0.88)$

$Q_i = 0.044\ m^3/s$

The depressed kerb-opening inlet will intercept 1.5 times the flow intercepted by the undepressed kerb opening.

Example 13.12

Given: From **Example 13.9**, the following information is given:

$S_L = 0.01$ m/m

$S_x = 0.02$ m/m

$T = 2.5$ m

$Q = 0.064\ m^3/s$

$n = 0.016$

$W = 0.6$ m

$a = 50$ mm

$E_O = 0.70$

Find: The minimum length of a locally depressed kerb opening inlet required to intercept 100 percent of the gutter flow.

Solution:

Step 1. Compute the composite cross slope for the gutter section using **Equation (13.69)**.

$S_e = S_x + S'_w\,E_o$

$S_e = 0.02 + (50 / 1000 / 0.6)\ 0.60$

$S_e = 0.07$

Step 2. Compute the length of kerb opening inlet required from **Equation (13.70)**.

$L_T = 0.817\ Q^{0.42}\ S_L^{0.3}\ (1 / n\ S_e)^{0.6}$

$L_T = (0.817)(0.064)^{0.42}(0.01)^{0.3}[1/ (0.016)(0.07)]^{0.6}$

$L_T = 3.81$ m

13.4.11.5.3 SLOTTED INLETS

Wide experience with the debris handling capabilities of slotted inlets is not available. Deposition in the pipe is the problem most commonly encountered. The configuration of slotted inlets makes them accessible for cleaning with a high-pressure water jet.

Slotted inlets are effective pavement drainage inlets which have a variety of applications. They can be used on kerbed or unkerbed sections and offer little interference to traffic operations. An installation is illustrated in **Plate 13.2**. Flow interception by slotted inlets and kerb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement.

Analysis of data from the Federal Highway Administration tests of slotted inlets with slot widths ≥ 45 mm indicates that the length of slotted inlet required for total interception can be computed by **Equation (13.67)**. **Chart 7** is therefore applicable for both kerb-opening inlets and slotted inlets. Similarly, **Equation (13.68)** is also applicable to slotted inlets and **Chart 8** can be used to obtain the inlet efficiency for the selected length of inlet.

When slotted drains are used to capture overland flow, research has indicated that with water depths ranging from 9.7 mm to 14.2 mm the 25, 44, and 63 mm wide slots can accommodate 0.0007 $m^3/s/m$ with no splash over for slopes from 0.005 to 0.09 m/m.

At a test system capacity of 0.0011 $m^3/s/m$, a small amount of splash over occurred.

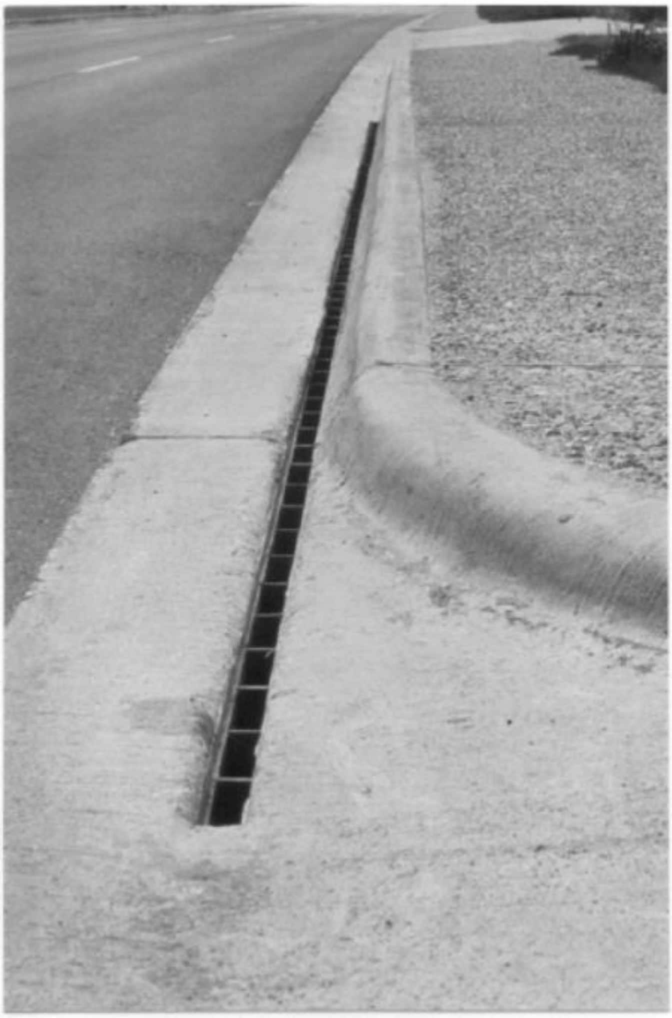


Plate 13.2 Slotted drain inlet at an intersection.

13.4.11.5.4 COMBINATION INLETS

The interception capacity of a combination inlet consisting of a kerb opening and grate placed side-by-side, as shown in **Plate 13.3**, is no greater than that of the grate alone. Capacity is computed by neglecting the kerb opening. A combination inlet is sometimes used with a part of the kerb opening placed upstream of the grate as illustrated in **Plate 13.4**. The kerb opening in such an installation intercepts debris which might otherwise clog the grate and is called a "sweeper" inlet. A sweeper combination inlet has an interception capacity equal to the sum of the kerb opening upstream of the grate plus the grate capacity, except that the frontal flow and thus the interception capacity of the grate is reduced by interception by the kerb opening.

Plate 13.3 Combination kerb-opening, 45 degree tilt-bar grate lnlet.

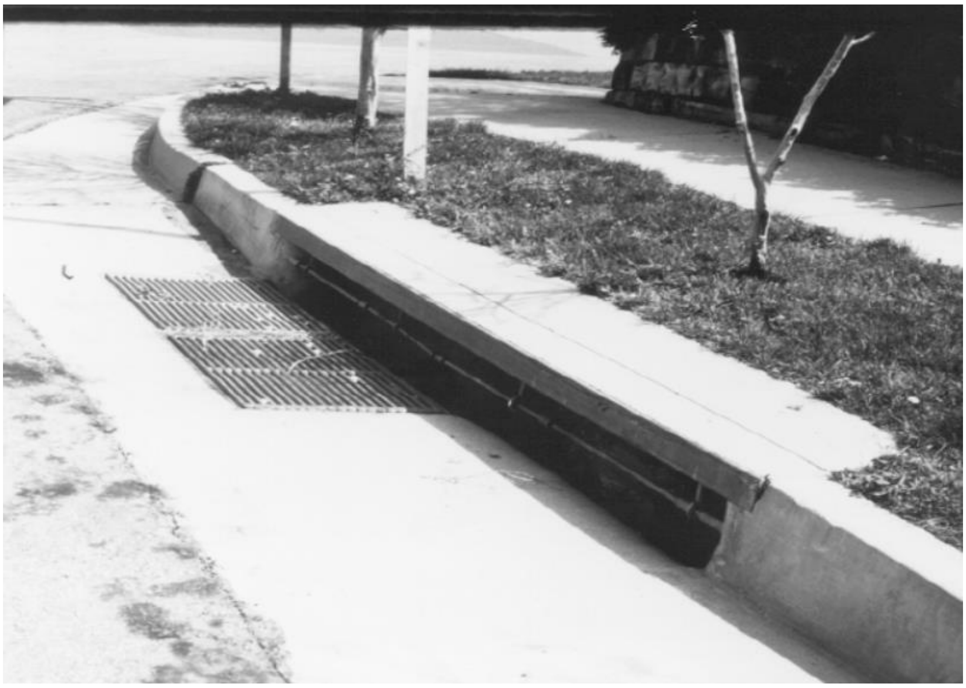


Plate 13.4 Sweeper combination inlet.

The following example illustrate computation of the interception capacity of a combination kerb opening grate inlet with a portion of the kerb opening upstream of the grate.

Example 13.13

Given: A combination kerb-opening grate inlet with a 3 m kerb opening, 0.6 m by 0.6m

curved vane grate placed adjacent to the downstream 0.6 m of the kerb opening. This inlet is located in a gutter section having the following characteristics:

W = 0.6 m

$Q = 0.05\ m^3/s$

$S_L = 0.01\ m/m$

$S_x = 0.02\ m/m$

$S_W = 0.062\ m/m$

$n = 0.016$

Find: Interception capacity, Q_i

Solution:

Step 1. Compute the interception capacity of the kerb-opening upstream of the grate, Q_{ic}.

$L = 3m - 0.6\ m = 2.4\ m$

From **Example 13.11**, Solution 2, Step 2

$L_T = 4.37\ m$

$L / L_T = 2.4 / 4.37 = 0.55$

Using **Equation (13.68)** or **Chart 8**

$E = 1 - (1 - L / L_T)^{1.8}$

$E = 1 - (1 - 0.55)^{1.8}$

$E = 0.76$

$Q_{ic} = E\ Q$

$= (0.76)(0.05)$

$= 0.038\ m^3/s$

Step 2. Compute the interception capacity of

the grate.

Flow at grate

$Q_g = Q - Q_{ic}$

$= 0.05 - 0.038$

$Q_g = 0.012\ m^3/s$

Determine Spread, T (Procedure from **Equation (13.45)**, Solution 2)

Assume $Q_s = 0.0003\ m^3/s$

$Q_w = Q - Qs$

$= 0.0120 - 0.0003 = 0.0117 m^3/s$

$E_o = Q_w / Q$

$= 0.0117 / 0.0120$

$= 0.97$

$S_w / S_x = 0.062 / 0.02 = 3.1$

From **Equation (13.48)** or **Chart 2**

$W/T = 1/\{(1/[(1/(1/E_o - 1))(S_w/S_x)+1]^{0.375} -1)(S_w / S_x) +1\}$

$W/T = 1 / \{(1/[(1/(1/0.97 - 1))(3.1)+1]^{0.375} - 1)(3.1)+1\}$

$W/T = 0.62$

$T \quad = W / (W/T) = 0.6 / 0.62$

$\quad = 0.97$ m

$T_s = T - W = 0.97 - 0.60$

$= 0.37$ m

From **Chart 1** or **Equation (13.45)**

$Q_s = 0.0003$ m^3/s

Q_s Assumed = Q_s calculated

Determine velocity, V

$V = Q / A = Q / [0.5T^2 S_x + 0.5 \text{ a } W]$

$V = 0.012/[(0.5)(0.97)^2(0.02) + (0.5)(25/1000)(0.6)]$

$V = 0.68$ m/s

From **Chart 5**

$R_f = 1.0$

From **Equation (13.63)** or **Chart 6**

$R_s = 1 / (1 + (0.0828\ V^{1.8})/(S_x\ L^{2.3}))$

$R_s = 1 / (1 + [(0.0828)\ (0.68)^{1.8}]/[(0.02)\ (0.6)^{2.3}]$

$R_s = 0.13$

From **Equation (13.66)**

$Q_{ig} = Q_g\ [R_f\ E_o + R_s\ (1-E_o)]$

$Q_{ig} = 0.012\ [\ (1.0)(0.97) + (0.13)(1 - 0.97)]$

$Q_{ig} = 0.011$ m^3/s

Step 3. Compute the total interception capacity. (Note: Interception capacity of kerb opening adjacent to grate was neglected.)

$Q_i = Q_{ic} + Q_{ig} = 0.038 + 0.011$

$Q_i = 0.049$ m^3/s (approximately 100% of the total initial flow)

The use of depressed inlets and combination inlets enhances the interception capacity of the inlet. **Example 13.9** determined the interception capacity of a depressed curved vane grate, 0.6m by 0.6m, **Example 13.11** and **Example 13.12** for an undepressed kerb opening inlet, length = 3.0 m and a depressed kerb opening inlet, length = 3.0 m, and **Example 13.13** for a combination of 0.6 m by 0.6 m depressed curve vane grate located at the downstream end of 3.0 m long depressed kerb opening inlet. The geometries of the inlets and the gutter slopes were consistent in the examples and **Table 13.31** summarizes a comparison of the intercepted flow of the various configurations.

Table 13.31 Comparison of Inlet Interception Capacities.

Inlet Type	Intercepted Flow, Q_i
Curved Vane - Depressed	0.033 m^3/s (**Example 13.9**)
Kerb Opening - Undepressed	0.031 m^3/s (**Example 13.11**)
Kerb Opening - Depressed	0.045 m^3/s (**Example 13.12**)
Combination - Depressed	0.049 m^3/s (**Example 13.13**)

From **Table 13.31**, it can be seen that the combination inlet intercepted approximately 100 percent of the total flow whereas the curved vane grate alone only intercepted 66 percent of the total flow. The depressed kerb opening intercepted 90 percent of the total flow. However, if the kerb opening was undepressed, it would have only intercepted 62 percent of the total flow.

13.4.11.6 INTERCEPTION CAPACITY OF INLETS IN SAG LOCATIONS

Inlets in sag locations operate as weirs under low head conditions and as orifices at greater depths. Orifice flow begins at depths dependent on the grate size, the kerb opening height, or the slot width of the inlet. At depths between those at which weir flow definitely prevails and those at which orifice flow prevails, flow is in a transition stage. At these depths, control is ill-defined, and flow may fluctuate between weir and orifice control. Design procedures presented here are based on a conservative approach to estimating the capacity of inlets in sump locations.

The efficiency of inlets in passing debris is critical in sag locations because all runoff which enters the sag must be passed through the inlet. Total or partial clogging of inlets in these locations can result in hazardous ponded conditions. Grate inlets alone are not recommended for use in sag locations because of the tendencies of grates to become clogged. Combination inlets or kerb opening inlets are recommended for use in these locations.

13.4.11.6.1 GRATE INLETS IN SAGS

A grate inlet in a sag location operates as a weir to depths dependent on the size of the grate and as an orifice at greater depths. Grates of larger dimension will operate as weirs to greater depths than smaller grates.

The capacity of grate inlets operating as weirs is given by **Equation (13.71)**.

$$Q_i = C_w P d^{1.5} \tag{13.71}$$

Where,

C_w = 1.66
P = perimeter of the grate in m disregarding the side against the kerb
d = average depth across the grate; 0.5 (d_1 + d_2), m (refer to **Figure 13.16**)

The capacity of a grate inlet operating as an orifice is given by **Equation (13.72)**.

$$Q_i = C_o A_g (2gd)^{0.5} \tag{13.72}$$

Where,

C_O = orifice coefficient = 0.67
A_g = clear opening area of the grate, m^2
g = 9.81 m/s^2

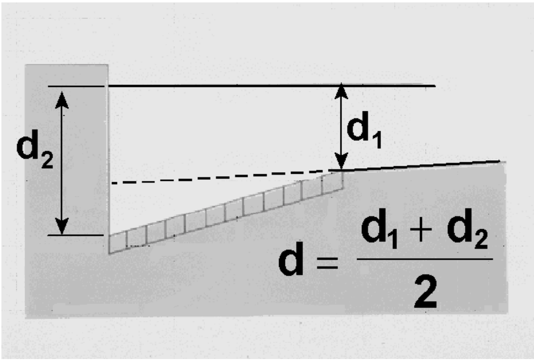


Figure 13.16 Definition of depth

Example 13.14 illustrates use of **Equation (13.71)** and **Equation (13.72)** and **Chart 9**.

Example 13.14

Given: Under design storm conditions a flow to the sag inlet is 0.19 m^3/s . Also,

S_x = S_W = 0.05 m/m

n = 0.016

$T_{allowable}$ = 3 m

Find: The grate size required and depth at kerb for the sag inlet assuming 50% clogging where the width of the grate, W, is 0.6 m.

Solution:

Step 1. Determine the required grate perimeter.

Depth at kerb, d_2

$d_2 = T\ S_x = (3.0)\ (0.05)$

$d_2 = 0.15$ m

Average depth over grate

$d = d_2 - (W/2)\ S_W$

$d = 0.15 - (0.6/2)(.05)$

$d = 0.135$ m

From **Equation (13.71)** or **Chart 9**

$P = Q_i / [C_w\ d^{1.5}]$

$P = (0.19)/[(1.66)(0.135)^{1.5}]$

$P = 2.31$ m

Some assumptions must be made regarding the nature of the clogging in order to compute the capacity of a partially clogged grate. If the area of a grate is 50 percent covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50 percent. For example, if a 0.6 m by 1.2 m grate is clogged so that the effective width is 0.3 m, then the perimeter, P = 0.3 + 1.2 + 0.3 = 1.8 m, rather than 2.31 m, the total perimeter, or 1.2 m, half of the total perimeter.

The area of the opening would be reduced by 50 percent and the perimeter by 25 percent.

Therefore, assuming 50 percent clogging along the length of the grate, a 1.2 m by 1.2 m, 0.6 m by 1.8 m, or a .9 m by 1.5 m grate would meet requirements of a 2.31 m perimeter 50 percent clogged.

Assuming 50 percent clogging along the grate length,

$P_{effective} = 2.4\ m = (0.5)\ (2)\ W + L$

if $W = 0.6$ m then $L \geq 1.8$ m

if $W = 0.9$ m then $L \geq 1.5$ m

Select a double 0.6 m by 0.9 m grate.

$P_{effective} = (0.5)\ (2)\ (0.6) + (1.8)$

$P_{effective} = 2.4$ m

Step 2. Check depth of flow at kerb using **Equation (13.71)** or **Chart 9**.

$d = [Q/(C_w\ P)]^{0.67}$

$d = [0.19/((1.66)\ (2.4))]^{0.67}$

$d = 0.130$ m

Therefore, ok

Conclusion:

A double 0.6 m by 0.9 m grate 50 percent clogged is adequate to intercept the design storm flow at a spread which does not exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet or kerb-opening inlet in a sag where ponding can occur, and flanking inlets in long flat vertical curves.

13.4.11.6.2 KERB-OPENING INLETS

The capacity of a kerb-opening inlet in a sag depends on water depth at the kerb, the kerb opening length, and the height of the kerb opening. The inlet operates as a weir to depths equal to the kerb opening height and as an orifice at depths greater than 1.4 times the opening height.

At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

Spread on the pavement is the usual criterion for judging the adequacy of a pavement drainage inlet design. It is also convenient and practical in the laboratory to measure depth at the kerb upstream of the inlet at the point of maximum spread on the pavement. Therefore, depth at the kerb measurements from experiments coincide with the depth at kerb of interest to designers. The weir coefficient for a kerb-opening inlet is less than the usual weir coefficient for several reasons, the most obvious of which is that depth measurements from experimental tests were not taken at the weir, and drawdown occurs between the point where measurement were made and the weir.

The weir location for a depressed kerb-opening inlet is at the edge of the gutter, and the effective weir length is dependent on the width of the depressed gutter and the length of the kerb opening. The weir location for a kerb-opening inlet that is not depressed is at the lip of the kerb opening, and its length is equal to that of the inlet, as shown in **Chart 10**.

The interception capacity of a depressed kerb-opening inlet operating as a weir is given by **Equation (13.73)**.

$$Q_i = C_w(L + 1.8W)d^{1.5} \qquad (13.73)$$

Where,

C_w = 1.25
L = length of kerb opening, m
W = lateral width of depression, m
d = depth at kerb measured from the normal cross slope, m, i.e., d = T S_x

The weir equation is applicable to depths at the kerb approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of **Equation (13.73)** for a depressed kerb-opening inlet is (**Equation (13.74)**):

$$d \leq h + a\,/\,(1000) \qquad (13.74)$$

Where,

h = height of kerb-opening inlet, m
a = depth of depression, mm

The weir equation for kerb-opening inlets without depression is given by **Equation (13.75)**.

$$Q_i = C_w d^{1.5} \tag{13.75}$$

Without depression of the gutter section, the weir coefficient, C_w, becomes 1.60. The depth limitation for operation as a weir becomes $d \leq h$.

At kerb-opening lengths greater than 3.6m, **Equation (13.75)** for non-depressed inlet produces intercepted flows which exceed the values for depressed inlets computed using **Equation (13.73)**. Since depressed inlets will perform at least as well as non-depressed inlets of the same length, **Equation (13.73)** should be used for all kerb opening inlets having lengths greater than 3.6 m.

Kerb-opening inlets operate as orifices at depths greater than approximately 1.4 times the opening height. The interception capacity can be computed by **Equation (13.76)** and **Equation (13.77)**. These equations are applicable to depressed and undepressed kerb-opening inlets. The depth at the inlet includes any gutter depression.

$$Q_i = C_o h L (2 g d_o)^{0.5} \tag{13.76a}$$

or

$$Q_i = C_o A_o \left[2g\left(d_i - \frac{h}{2}\right)\right]^{0.5} \tag{13.76b}$$

Where:

C_o = orifice coefficient (0.67)

d_o = effective head on the centre of the orifice throat, m

L = length of orifice opening, m

A_g = clear area of opening, m^2

d_i = depth at lip of kerb opening, m

h = height of kerb-opening orifice, m

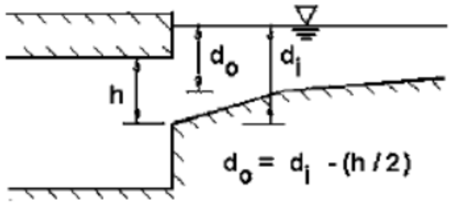
The height of the orifice in **Equation (13.76a)** and **Equation (13.76b)** assumes a vertical orifice opening. As illustrated in **Figure 13.17**, other orifice throat locations can change the effective depth on the orifice and the dimension (d_i - h/2). A limited throat width could reduce the capacity of the kerb-opening inlet by causing the inlet to go into orifice flow at depths less than the height of the opening.

For kerb-opening inlets with other than vertical faces (see **Figure 13.17**), **Equation (13.76a)** can be used with:

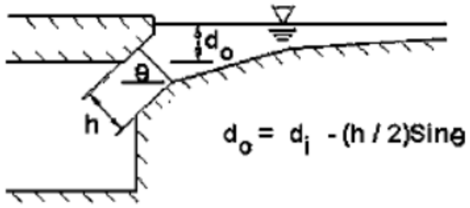
h = orifice throat width, m

d_o = effective head on the centre of the orifice throat, m

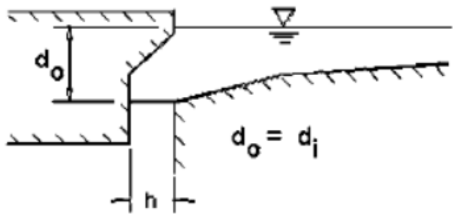
Chart 10 provides solutions for **Equation (13.73)** and **Equation (13.76)** for depressed kerb-opening inlets, and **Chart 11** provides solutions for **Equation (13.75)** and **Equation (13.76)** for kerb-opening inlets without depression. **Chart 12** is provided for use for kerb openings with other than vertical orifice openings.



a. Horizontal Throat



b. Inclined Throat



c. Vertical Throat

Figure 13.17 Kerb-opening inlets

Example 13.15 illustrates the use of **Charts 11** and **12**.

Example 13.15

Given: Kerb opening inlet in a sump location with

L = 2.5 m

h = 0.13 m

(1) Undepressed kerb opening

S_x = 0.02

T = 2.5 m

(2) Depressed kerb opening

$S_x = 0.02$

a = 25 mm local

W = 0.6 m

T = 2.5 m

Find: Q_i

Solution (1): Undepressed

Step 1. Determine depth at kerb.

$d = T\ S_x = (2.5)\ (0.02)$

d = 0.05 m

$d = 0.05 \text{ m} \leq h = 0.13 \text{ m}$,

therefore weir flow controls

Step 2. Use **Equation (13.75)** or **Chart 11** to find Q_i.

$Q_i = C_w\ L\ d^{1.5}$

$Q_i = (1.60)\ (2.5)\ (0.05)^{1.5}$

$= 0.045 \text{ m}^3/\text{s}$

Solution (2): Depressed

Step 1. Determine depth at kerb, d_i

$d_i = d + a$

$d_i = S_x\ T + a$

$d_i = (0.02)(2.5) + 25/1000$

$d_i = 0.075$ m

$d_i = 0.075 \text{ m} < h = 0.13 \text{ m}$, therefore weir flow controls

Step 2. Use **Equation (13.73)** or **Chart 10** to find Q_i.

P = L + 1.8 W

P = 2.5 m + (1.8)(0.6)

P = 3.58 m

$Q_i = C_w\ (L + 1.8\ W)\ d^{1.5}$

$Q_i = (1.25)\ (3.58)\ (0.05)^{1.5}$

$Q_i = 0.048 \text{ m}^3/\text{s}$

The depressed kerb-opening inlet has 10 percent more capacity than an inlet without depression.

13.4.11.6.3 SLOTTED INLETS

Slotted inlets in sag locations perform as weirs to depths of about 0.06 m, dependent on slot width. At depths greater than about 0.12 m, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as a weir can be computed by **Equation (13.77)**.

$$Q_i = C_w L d^{1.5} \qquad (13.77)$$

Where,

C_w = weir coefficient; varies with flow depth and slot length; typical value is approximately 1.4

L = length of slot, m

d = depth at kerb measured from the normal cross slope, m

The interception capacity of a slotted inlet operating as an orifice can be computed by **Equation (13.78)**.

$$Q_i = 0.8LW(2gd)^{0.5} \qquad (13.78)$$

Where,

W= width of slot, m

L = length of slot, m

d = depth of water at slot for d > 0.12 m

g = 9.81 m/s^2

For a slot width of 45 mm, **Equation (13.78)** becomes **Equation (13.79)**.

$$Q_i = C_D L d^{0.5} \qquad (13.79)$$

where:

C_D = 0.16

Chart 13 provides solutions for weir and orifice flow conditions as represented by **Equation (13.77)** and **Equation (13.78)**. As indicated in **Chart 13**, the transition between weir and orifice flow occurs at different depths. To conservatively compute the interception capacity of slotted inlets in sump conditions in the transition area, orifice conditions should be assumed. Due to clogging characteristics, slotted drains are not recommended in sag locations.

Example 13.16

Given: A slotted inlet located along a kerb having a slot width of 45 mm. The gutter flow at the upstream end of the inlet is 0.14 m^3/s.

Find: The length of slotted inlet required to limit maximum depth at the kerb to 0.09 m assuming no clogging.

Solution:

From **Chart 13** with Q = 0.14 m^3/s and d = 0.09, L = 3.66 m say 4.0 m

Note: Since the point defined by Q and d on **Chart 13** falls in the weir flow range, **Equation (13.77)** defines the flow condition. However, **Equation (13.77)** cannot be directly applied since C_w varies with both flow depth and slot length.

13.4.11.6.4 COMBINATION INLETS

Combination inlets consisting of a grate and a kerb opening are considered advisable for use in sags where hazardous ponding can occur. Equal length inlets refer to a grate inlet placed alongside a kerb opening inlet, both of which have the same length. A sweeper inlet refers to a grate inlet placed at the downstream end of a kerb opening inlet. The kerb opening inlet is longer than the grate inlet and intercepts the flow before the flow reaches the grate. The sweeper inlet is more efficient than the equal length combination inlet and the kerb opening has the ability to intercept any debris which may clog the grate inlet. The interception capacity of the equal length combination inlet is essentially equal to that of a grate alone in weir flow. In orifice flow, the capacity of the equal length combination inlet is equal to the capacity of the grate plus the capacity of the kerb opening.

Equation (13.71) and **Chart 9** can be used for grates in weir flow or combination inlets in sag locations. Assuming complete clogging of the grate, **Equation (13.73)**, **Equation (13.75)**, and **Equation (13.78)** and **Charts 10**, **11** and **12** for kerb-opening inlets are applicable.

Where depth at the kerb is such that orifice flow occurs, the interception capacity of the inlet is computed by adding **Equation (13.72)** and **Equation (13.76a)** to obtain **Equation (13.80)**.

$$Q_i = 0.67A_g(2gd)^{0.5} + 0.67hL(2gd_o)^{0.5} \tag{13.80}$$

Where,

- A_g = clear area of the grate, m^2
- g = 9.81 m/s^2
- d = average depth over the grate, m
- h = height of kerb opening orifice, m
- L = length of kerb opening, m
- d_o = effective depth at the centre of the kerb opening orifice, m

Trial and error solutions are necessary for determining the depth at the kerb for a given flow rate using **Charts 9**, **10**, and **11** for orifice flow. Different assumptions for clogging of the grate can also be examined using these charts as illustrated by the following example.

Example 13.17

Given: A combination inlet in a sag location with the following characteristics:

Grate - 0.6 m by 1.2 m P-50

Kerb opening -

$L = 1.2$ m

$h = 0.1$ m

$Q = 0.15\ m^3/s$

$S_x = 0.03$ m/m

Find: Depth at kerb and spread for:

(1) Grate clear of clogging

(2) Grate 100 percent clogged

Solution (1):

Step 1. Compute depth at kerb.

Assuming grate controls interception:

$P = 2W + L = 2(0.6) + 1.2$

$P = 2.4$ m

From **Equation (13.71)** or **Chart 9**

$d_{avg} = [Q_i / (C_w\ P)]^{0.67}$

$d_{avg} = [(0.15) / \{(1.66)(2.4)\}]^{0.67} = 0.11$ m

Step 2. Compute associated spread.

$d = d_{avg} + S_x W/2$

$d = 0.11 + 0.03\ (0.6)/2 = 0.119$ m

$T = d / S_x = (0.119) / (0.03)$

$T = 3.97$ m

Solution (2):

Step 1. Compute depth at kerb.

Assuming grate clogged. Using **Chart** 11 or **Equation (13.76b)** with

$Q = 0.15\ m^3/s$

$d = \{Q_i/(C_o\ h\ L)\}^2 / (2g) + h/2$

$d = \{(0.15)/[(0.67)(0.10)(1.2)]\}^2 / [(2)(9.81)] + (0.1/2)$

$d = 0.24$ m

Step 2. Compute associated spread.

$T = d / S_x$

$T = (0.24) / (0.03)$

$T = 8.0$ m

Interception by the kerb-opening only will be in a transition stage between weir and orifice flow with a depth at the kerb of about 0.24 m. Depth at the kerb and spread on the pavement would be almost twice as great if the grate should become completely clogged.

13.4.11.7 INLET LOCATIONS

The location of inlets is determined by geometric controls which require inlets at specific locations, the use and location of flanking inlets in sag vertical curves, and the criterion of spread on the pavement. In order to adequately design the location of the inlets for a given project, the following information is needed:

i. Layout or plan sheet suitable for outlining drainage areas
ii. Road profiles
iii. Typical cross sections
iv. Grading cross sections
v. Superelevation diagrams
vi. Contour maps

13.4.11.7.1 GEOMETRIC CONTROLS

There are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or flow bypass. Examples of such locations follow.

At all low points in the gutter grade

i. Immediately upstream of median breaks, entrance/exit ramp gores, cross walks, and street intersections., i.e., at any location where water could flow onto the travelway
ii. Immediately upgrade of bridges (to prevent pavement drainage from flowing onto bridge decks)
iii. Immediately downstream of bridges (to intercept bridge deck drainage)
iv. Immediately upgrade of cross slope reversals
v. Immediately upgrade from pedestrian cross walks
vi. At the end of channels in cut sections
vii. On side streets immediately upgrade from intersections
viii. Behind kerbs, shoulders or sidewalks to drain low area

In addition to the areas identified above, runoff from areas draining towards the highway pavement should be intercepted by roadside channels or inlets before it reaches the roadway. This applies to drainage from cut slopes, side streets, and other areas alongside the pavement. Kerbed pavement sections and pavement drainage inlets are inefficient means for handling extraneous drainage.

13.4.11.7.2 INLET SPACING ON CONTINUOUS GRADES

Design spread is the criterion used for locating storm drain inlets between those required by geometric or other controls. The interception capacity of the upstream inlet will define the initial spread. As flow is contributed to the gutter section in the downstream direction, spread increases. The next downstream inlet is located at the point where the spread in the gutter reaches the design spread. Therefore, the spacing of inlets on a continuous grade is a function of the amount of upstream bypass flow, the tributary drainage area, and the gutter geometry.

For a continuous slope, the designer may establish the uniform design spacing between inlets of a given design if the drainage area consists of pavement only or has reasonably uniform runoff characteristics and is rectangular in shape. In this case, the time of concentration is assumed to be the same for all inlets. The following procedure and example illustrates the effects of inlet efficiency on inlet spacing.

In order to design the location of inlets on a continuous grade, the computation sheet shown in

Figure 13.18 may be used to document the analysis. A step by step procedure for the use of **Figure 13.18** follows.

Step 1. Complete the blanks at the top of the sheet to identify the job by state project number, route, date, and your initials.

Step 2. Mark on a plan, the location of inlets which are necessary even without considering any specific drainage area, such as the locations described in **Section 13.5.7.7.1**.

Step 3. Start at a high point, at one end of the job if possible, and work towards the low point. Then begin at the next high point and work backwards toward the same low point.

Step 4. To begin the process, select a trial drainage area approximately 90 m to 150 m long below the high point and outline the area on the plan. Include any area that may drain over the kerb, onto the roadway. However, where practical, drainage from large areas behind the kerb should be intercepted before it reaches the roadway or gutter.

Step 5. Col. 1 Describe the location of the proposed inlet by number and station and record this Col. 2 information in columns 1 and 2. Identify the kerb and gutter type in column 19, Col. 19 remarks. A sketch of the cross section should be prepared.

Step 6. Col. 3 Compute the drainage area in km^2 and record in column 3.

Step 7. Col. 4 Determine the runoff coefficient, C, for the drainage area or determine a weighted C and record the value in column 4.

Step 8. Col. 5 Compute the time of concentration, t_c, in minutes, for the first inlet and record in column 5. The minimum time of concentration is 5 minutes.

Step 9. Col. 6 Using the time of concentration, determine the rainfall intensity from the Intensity-Duration-Frequency (IDF) curve for the design frequency. Enter the value in column 6.

Step10. Col. 7 Calculate the flow in the gutter using Q=CIA/3.6. The flow is calculated Enter the flow value in column 7.

Step 11. Col. 8 From the roadway profile, enter in column 8 the gutter longitudinal slope,

S_L, at the inlet, taking into account any superelevation.

Step12. Col. 9 From the cross section, enter the cross slope, S_x, in column 9 and the Col.13 grate or gutter width, W, in column 13.

Step13. Col. 11 For the first inlet in a series, enter the value from column 7 into column11, since there was no previous bypass flow. Additionally, if the inlet is the first in a series, enter 0 into column 10.

Step14. Col. 14 Determine the spread, T, by using equations 4-2 and 4-4 or **Chart 1** and Col. 12 2 and enter the value in column 14. Also, determine the depth at the kerb, d, by multiplying the spread by the appropriate cross slope, and enter the value in column 12. Compare the calculated spread with the allowable spread as determined by the design criteria outlined in **Section 13.4.1**. Additionally, compare the depth at the kerb with the actual kerb height in column 19. If the calculated spread, column 14, is near the allowable spread and the depth at the kerb is less than the actual kerb height, continue on to step 15. Else, expand or decrease the drainage area up to the first inlet to increase or decrease the spread, respectively. The drainage area can be expanded by increasing the length to the inlet and it can be decreased by decreasing the distance to the inlet. Then, repeat steps 6 through 14 until appropriate values are obtained.

Step 15. Col. 15 Calculate W/T and enter the value in column 15.

Step 16. Col. 16 Select the inlet type and dimensions and enter the values in column 16.

Step 17. Col. 17 Calculate the flow intercepted by the grate, Qi, and enter the value in column 17. Use **Equations (13.60)** and **(13.57)** or **Charts 2** and **4** to define the gutter flow. Use **Chart** 5 and **Equation (13.63)** or **Chart 6** to define the flow intercepted by the grate. Use **Equations (13.67)** and **(13.68)** or **Charts 7** and **8** for kerb opening inlets. Finally, use **Equation (13.66)** to determine the intercepted flow.

Step 18. Col. 18 Determine the bypass flow, Q_b, and enter into column 18. The bypass flow is column 11 minus column 17.

Step 19. Col. 1-4 Proceed to the next inlet down the grade. To begin the procedure, select a drainage area approximately 90 m to 120 m below the previous inlet for a first trial. Repeat steps 5 through 7 considering only the area between the inlets.

Step 20. Col. 5 Compute the time of concentration for the next inlet based upon the area between the consecutive inlets and record this value in column 5.

Step 21. Col. 6 Determine the rainfall intensity from the IDF curve based upon the time of concentration determined in step 19 and record the value in column 6.

Step 22. Col. 7 Determine the flow in the gutter by using **Equation (13.3)** and record the value in column 7.

Step 23. Col 11 Record the value from column 18 of the previous line into column 10 of the current line. Determine the total gutter flow by adding column 7 and column 10 and record in column 11.

Step 24. Col. 12 Determine the spread and the depth at the kerb as outlined in step 14.

Col. 14 Repeat steps 18 through 24 until the spread and the depth at the kerb are within the design criteria.

Step 25. Col. 16 Select the inlet type and record in column 16.

Step 26. Col. 17 Determine the intercepted flow in accordance with step 17.

Step 27. Col. 18 Calculate the bypass flow by subtracting column 17 from column 11. This completes the spacing design for the inlet.

Step 28. Repeat steps 19 through 27 for each subsequent inlet down to the low point.

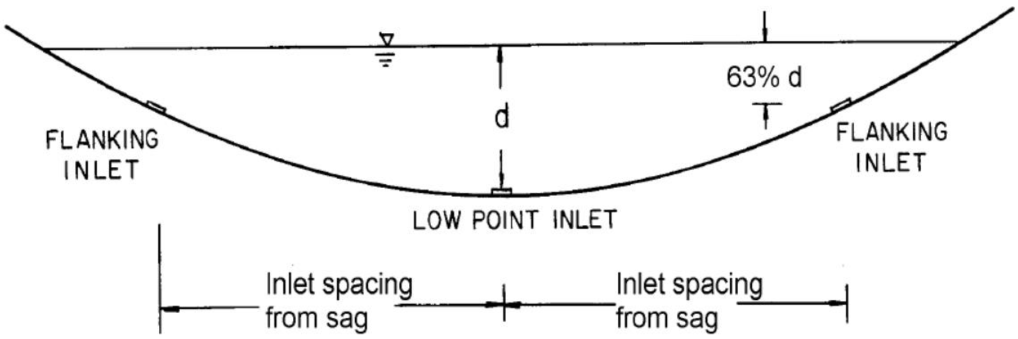
INLET SPACING COMPUTATION SHEET												Date_____ SP_____ ROUTE_____ Computed By:_____ Sheet __ of___						
INLET		GUTTER DISCHARGE Design Frequency_____					GUTTER DISCHARGE Allowable Spread_____								INLET DISCHARGE			RMK
No.	Stat.	Drain. Area A ( )	Run-off Coeff. C	Time of Conc. t_c (min)	Rain. Inten. I (/hr)	Q = CIA/K_c (^{3}/s)	Long. Slope S_L (/)	Cross Slope S_x or S_w (/)	Prev. By-pass Flow (^{3}/s)	Total Gutter Flow (^{3}/s)	Depth d ( )	Grate or Gutter Width W ( )	Spread T ( )	W/T	Inlet Type	Inter-cept Flow Q_i (^{3}/s)	By-pass Flow Q_b (^{3}/s)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)

Figure 13.18 Inlet spacing computation sheet

For inlet spacing in areas with changing grades, the spacing will vary as the grade changes. If the grade becomes flatter, inlets may be spaced at closer intervals because the spread will exceed the allowable. Conversely, for an increase in slope, the inlet spacing will become longer because of increased capacity in the gutter sections.

13.4.11.7.3 FLANKING INLETS

As discussed in the previous section, inlets should always be located at the low or sag points in the gutter profile. In addition, it is good engineering practice to place flanking inlets on each side of the low point inlet when in a depressed area that has no outlet except through the system. This is illustrated in **Figure 13.19**. The purpose of the flanking inlets is to act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.



d=depth at kerb at design spread

Figure 13.19 Example of flanking inlets.

Flanking inlets can be located so they will function before water spread exceeds the allowable spread at the sump location. The flanking inlets should be located so that they will receive all of the flow when the primary inlet at the bottom of the sag is clogged. They should do this without exceeding the allowable spread at the bottom of the sag. If the flanking inlets are the same dimension as the primary inlet, they will each intercept one-half the design flow when they are located so that the depth of ponding at the flanking inlets is 63 percent of the depth of ponding at the low point.

If the flanker inlets are not the same size as the primary inlet, it will be necessary to either develop a new factor or do a trial and error solution using assumed depths with the weir equation to determine the capacity of the flanker inlet at the given depths.

Table 13.32 shows the spacing required for various depth at kerb criteria and vertical curve lengths defined by $K = L / (G_2 - G_1)$, where L is the length of the vertical curve in meters and G_1 and G_2 are the approach grades in percent. While the Chapter 7 specifies minimum K values for various design speeds, a maximum sag K of 75 shall be selected for kerbed storm drainage.

Table 13.32 Distance to Flanking Inlets in Sag Vertical Curve Using Depth at kerb Criteria

d (m)	K (m/%)														
	2	4	6	7	10	15	18	24	28	38	40	52	60	70	75
	X (m)														
0.01	2.0	2.8	3.5	3.7	4.5	5.5	6.0	6.9	7.5	8.7	8.9	10.2	11.0	11.8	12.2
0.02	2.8	4.0	4.9	5.3	6.3	7.7	8.5	9.8	10.6	12.3	12.6	14.4	15.5	16.7	17.3
0.03	3.5	4.9	6.0	6.5	7.7	9.5	10.4	12.0	13.0	15.1	15.5	17.7	19.0	20.5	21.2
0.06	4.9	6.9	8.5	9.2	11.0	13.4	14.7	17.0	18.3	21.4	21.9	25.0	26.8	29.0	30.0
0.09	6.0	8.5	10.4	11.2	13.4	16.4	18.0	20.8	22.4	26.2	26.8	30.6	32.9	35.5	36.7
0.12	6.9	9.8	12.0	13.0	15.5	19.0	20.8	24.0	25.9	30.2	31.0	35.3	37.9	41.0	42.4
0.15	7.7	11.0	13.4	14.5	17.3	21.2	23.2	26.8	29.0	33.8	34.6	39.5	42.4	45.8	47.4
0.18	8.5	12.0	14.7	15.9	19.0	23.2	25.5	29.4	31.7	37.0	37.9	43.3	46.5	50.2	52.0
0.21	9.2	13.0	15.9	17.1	20.5	25.1	27.5	31.7	34.3	39.9	41.0	46.7	50.2	54.2	56.1
0.24	9.8	13.9	17.0	18.3	21.9	26.8	29.4	33.9	36.7	42.7	43.8	50.0	53.7	58.0	60.0

NOTES: 1. $x = (200\ dK)^{0.5}$, where x = distance from sag point.

2. $d = Y - Y_f$ where Y = depth of ponding and Y_f = depth at the flanker inlet

3. Drainage maximum K = 75

Example 13.18

Given: A 150 m (L) sag vertical curve at an underpass on a 4-lane divided highway with begin and end slopes of -2.5% and +2.5% respectively. The spread at design Q is not to exceed the shoulder width of 3.0 m.

$S_x = 0.02$

Find: The location of the flanking inlets if located to function in relief of the inlet at the low point when the inlet at the low point is clogged.

Solution:

Step 1. Find the rate of vertical curvature,

K.

$K = L/(G_2 – G_1)$

K= 150 m/(2.5% - (-2.5%))

K= 30 m/ %

Step 2. Determine depth at design spread.

$d = S_x\ T = (0.02)\ (3.0)$

d = 0.06 m

Step 3. Determine the depth for the flanker locations.

d = 63% of depth over inlet at bottom of sag

= 0.63 (.06)

= 0.04 m

Step 4. For use with **Table 13.32**;

d = 0.06 - 0.04 = 0.02 m

X = $(200\ d\ K)^{0.5}$

= $\{(200)(0.02)(30)\}^{0.5}$ = 10.95 m

Inlet spacing = 10.95 m from the sag point.

13.4.11.8 MEDIAN AND ROADSIDE DITCH INLETS

Median and roadside ditches may be drained by drop inlets similar to those used for pavement drainage, by pipe culverts under one roadway, or by cross drainage culverts which are not continuous across the median. Inlets, pipes, and discontinuous cross drainage culverts should be designed so as not to detract from a safe roadside. Drop inlets should be flush with the ditch bottom and traffic-safe bar grates should be placed on the ends of pipes used to drain medians that would be a hazard to errant vehicles, although this may cause a plugging potential. Cross drainage structures should be continuous across the median unless the median width makes this impractical. Ditches tend to erode at drop inlets; paving around the inlets helps to prevent erosion and may increase the interception capacity of the inlet marginally by acceleration of the flow.

Pipe drains for medians operate as culverts and generally require more water depth to intercept median flow than drop inlets. No test results are available on which to base design procedures for estimating the effects of placing grates on culvert inlets. However, little effect is expected.

The interception capacity of drop inlets in median ditches on continuous grades can be estimated by use of **Equation (13.81)** or **Charts 14** and **15** to estimate flow depth and the ratio of frontal flow to total flow in the ditch.

$$Q = \frac{A}{n} R^{2/3} S^{1/2} \qquad (13.81)$$

Where,

Q = discharge rate, m^3/s

n = Manning's coefficient

A = cross sectional area of flow, m^2

R = hydraulic radius = area/wetted perimeter, m

S = bed slope, m/m

The ratio of frontal flow (over grate) to total flow is expressed given by **Equation (13.82)**.

$$E_O = W/(B + dz) \qquad (13.82)$$

Where,

E_o = ratio of frontal flow

W = width of grate, m

B = bottom width, m

z = horizontal distance of side slope to a rise of 1.0m vertical, m

13.4.12 GULLY

Gullies are small chambers that form the top part of the surface water drainage system. Rain water from the road surface flows through the gratings into the gullies and then into the underground drainage system. Gully pots actively trap sediments transported by urban runoff to prevent in-pipe blockages and surface flooding.

Gulley distributions are based on the following considerations:

i. The recommended interval is generally between 20-30 m to make maintenance easy. For roads with small longitudinal gradients around 0.5%, 25m may be considered the minimum value.
ii. In sag curves 3 gulley pots may be necessary. One at the bottom and two between 3-5m on both sides of the bottom one.
iii. Gulley pots are distributed uniformly about each catchment area.

Examples of positions of gulley at intersections and along transition curves are shown in **Figure 13.20**.

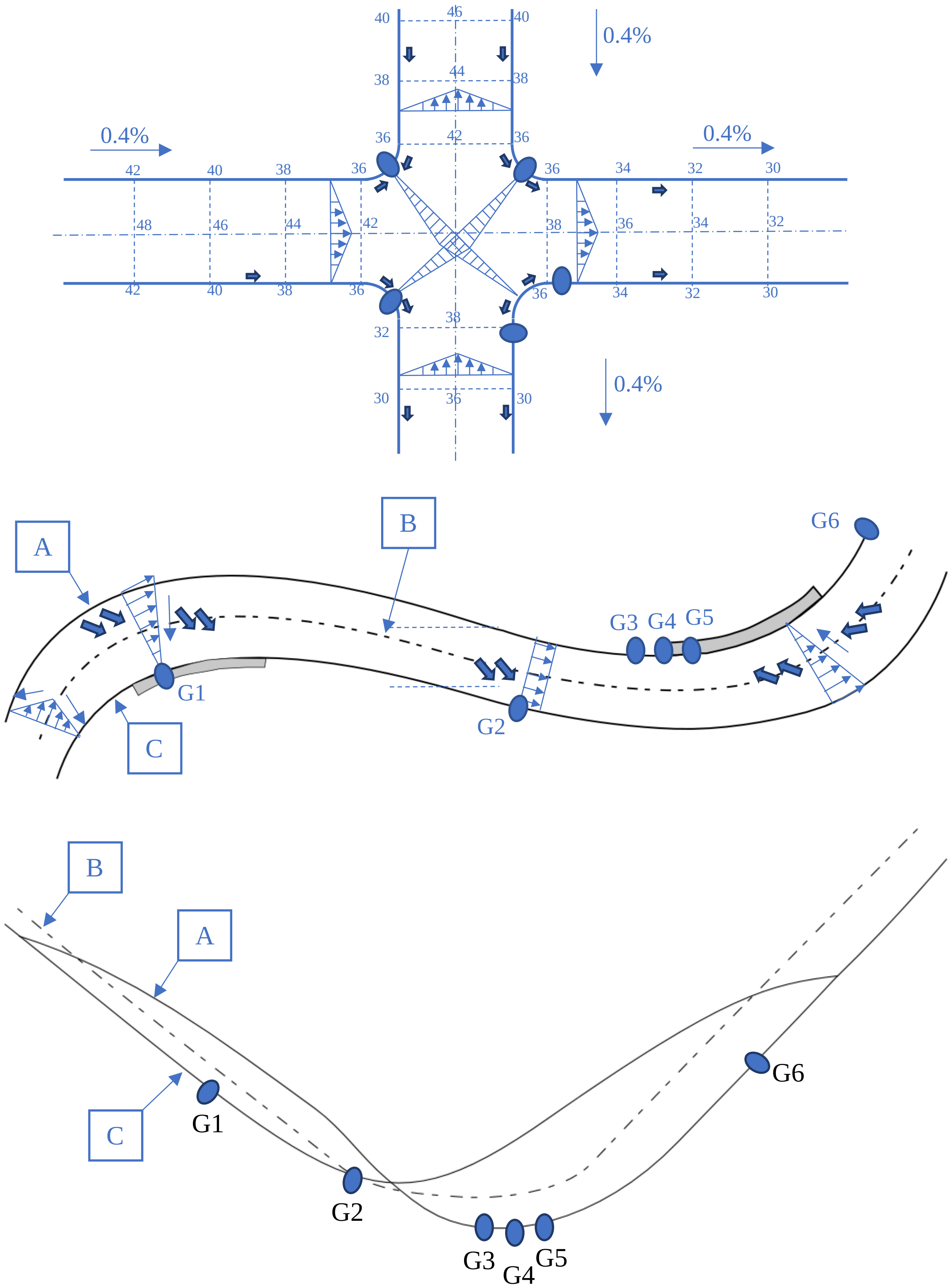


Figure 13.20 Example of position of gulley at intersection

13.4.13 DESIGN OF GULLEY INTERVALS

(1) Flow chart of design of gulley intervals

For the design of gully intervals, see the flow chart in **Figure 13.21**. **Table 13.33** provides information on gully drop ratio.

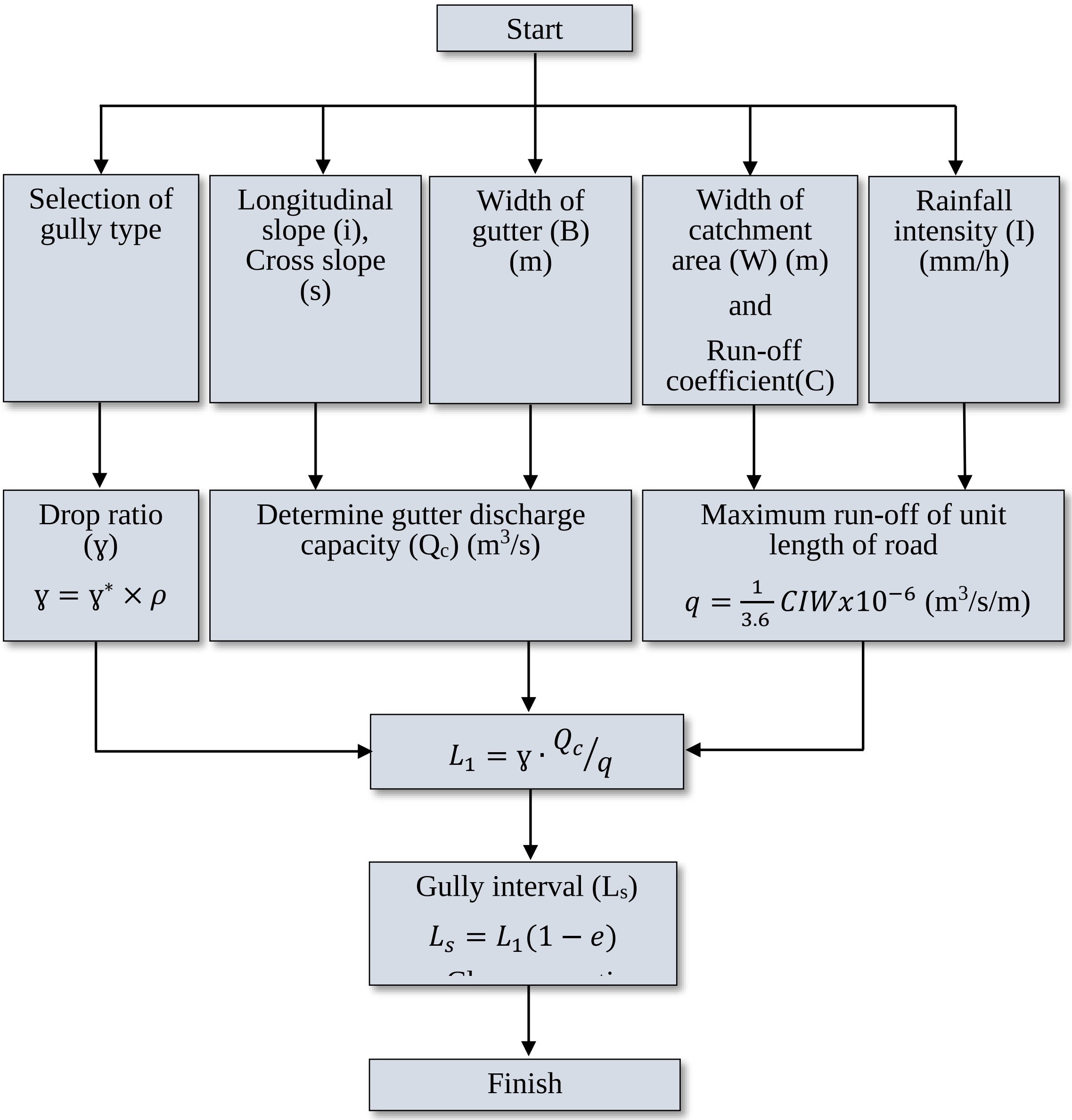
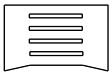
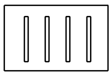
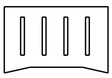
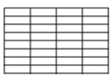


Figure 13.21 Flow chart for the design of gulley intervals

Table 13.33 Drop ratio

Type of gully	Drop area(A) cm²	Apparent drop ratio (ɣ *)				S (%)	Correction coefficient (ρ)			Range of drop ratio (ɣ)
		$Q_c \leq 1$	$Q_c=2$	$Q_c=4$	$Q_c \geq 8$		i ≤ 0.1%	i = 1%	i ≥ 10%	
A-Type	100	0.70	0.60	0.46	0.45					0.45-0.70
	200	0.85	0.85	0.73	0.65	10	1.0	1.0	1.0	0.65-0.85
	300	0.96	0.92	0.84	0.80					0.80-0.96
	100	0.10	0.08	0.05	0.04					0.04-0.11
	200	0.30	0.20	0.15	0.10	1.5	0.9	1.0	1.1	0.09-0.33
	300	0.40	0.38	0.32	0.30					0.27-0.44
B-Type	100	0.60	0.50	0.40	0.30					0.18-0.72
	200	0.80	0.80	0.70	0.50	10	1.2	1.0	0.6	0.30-0.96
	300	0.90	0.85	0.75	0.65					0.39-1.00
	100	0.50	0.40	0.30	0.20					0.19-0.53
	200	0.60	0.52	0.42	0.28	1.5	0.95	1.0	1.05	0.27-0.63
	300	0.70	0.60	0.50	0.38					0.36-0.74
C-Type	100	0.60	0.50	0.40	0.36					0.33-0.62
	200	0.85	0.83	0.70	0.62	10	1.04	1.0	0.92	0.57-0.88
	300	0.95	0.90	0.80	0.75					0.69-0.99
	100	0.45	0.38	0.30	0.18					0.17-0.47
	200	0.50	0.50	0.40	0.28	1.5	0.92	1.0	1.05	0.26-0.58
	300	0.55	0.55	0.45	0.38					0.35-0.68
D-Type	200	0.85	0.85	0.76	0.65					0.62-0.86
	400	1.00	0.95	0.85	0.75	10	1.01	1.0	0.95	0.71-1.00
	600	1.00	1.00	0.85	0.80					0.76-1.00
	200	0.50	0.40	0.28	0.22					0.17-0.70
	400	0.60	0.50	0.36	0.32	1.5	0.78	1.0	1.4	0.25-0.84
	600	0.98	0.55	0.42	0.36					0.28-0.95

(2) Calculation of drop ratio

The calculation of the drop ratio shall be given by the following procedure. The condition of the calculation shall be as follows.

> **<Condition>**
>
> Gully type : D, Drop Area 400 cm^2, Gutter discharge capacity: 5.5m^3/s, Gutter cross slope 10%, Gradient 5%

i. Obtain r* by interpolation from Table A.13.

$$ɣ *= 0.75 + \frac{5.5 - 4}{8 - 4} \times (0.85 - 0.75) = 0.79$$

ii. Obtain ρ also by interpolation

$$\rho = 0.95 + \frac{5 - 1}{10 - 1} \times (1.0 - 0.95) = 0.97$$

iii. Obtain ɣ

$$ɣ = ɣ * \times \rho$$

$$ɣ = 0.79 \times 0.97 = 0.77$$

(3) Calculation of maximum gulley intervals

The calculation of the maximum gully shall be given by the following procedure. The condition of the calculation shall be as follows.

<Condition>

Width of gulley drainage area (R): 1.0m, Catchment width (w); 6.0m, Gutter cross slope; 10%, Gradient 1%, Rainfall Intensity; 120 mm/h, Gully type and drop area; B and 200m^2 from **Table 13.33**, Coefficient of roughness: 0.013 from **Table 13.9**, Run-off coefficient : 0.9 from **Table 13.4**.

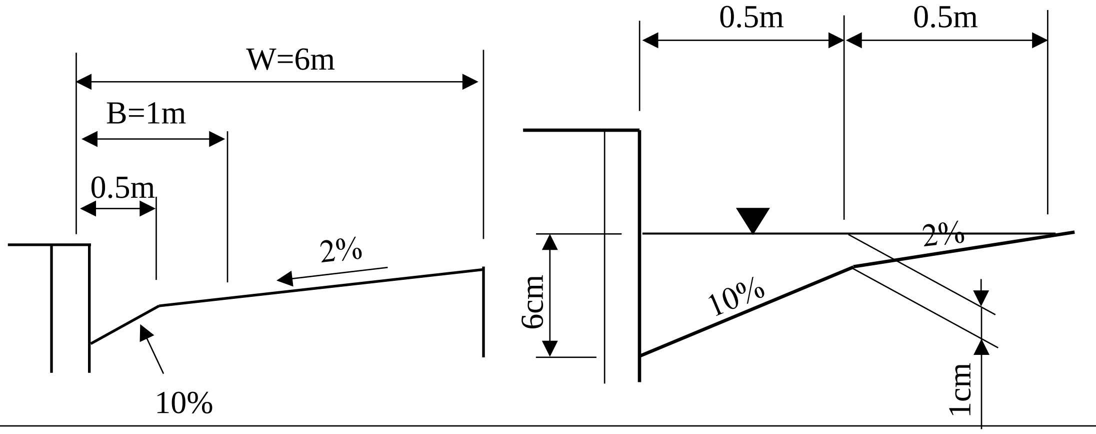


Figure 13.22 Sectional area (A) of gutter

i. **Determine gutter sectional area (A)**
$$A = \{^1/_2 \times (0.01 + 0.06) \times 0.5\} + (^1/_2 \times 0.01 \times 0.5) = 0.02m^2$$

ii. **Determine length of wetted perimeter (p)**
$$P = (0.5 \times \sqrt{(1^2 + 0.01^2} + (0.5 \times \sqrt{1^2 + 0.02^2}) + 0.06 = 1.062m$$

iii. **Determine hydraulic mean depth (R)**
$$R = {}^A/_P \quad R = {}^{0.02}/_{1.062} = 0.019m$$

iv. **Determine maximum gutter discharge capacity (Qc)**
$$\mathrm{Q}_c = \frac{1}{0.013} \times 0.019^{2/3} \times 1/100^{1/2} \times 0.02 = 10.9\ \mathrm{m}^3/\mathrm{s}$$

v. **Determine maximum run-off of road unit length of road**
$$q = \frac{1}{3600} \times 0.9 \times 120 \times 6 = 0.18\ \mathrm{m}^3/s\,/m$$

vi. **Determine drop ratio**
$$ɣ = ɣ^* \times \rho = 0.5 \times 1.0 = 0.5$$

vii. **Determine gulley interval**

$$L_1 = 0.5 \times 10.9/0.18 = 30.3m$$

Clearance ratio of 10% is assumed as the section is small.

$$L_s = 30.3 \times (1 - 0.1) = 27.3m$$

Therefore, the gulley interval shall be 27m.

13.5 HYDRAULIC DESIGN OF OPEN CHANNELS

An open channel is a conduit in which water is conveyed with a free surface. Although closed conduits such as culverts and storm drains are open channels when flowing partially full, the term is generally applied to natural and improved watercourses, gutters, ditches, and channels. While the hydraulic principles discussed in this section are valid for all drainage structures, the primary consideration is given to channels along, across, approaching and leaving the highway.

The most efficient shape of channel is that of a semi-circle, but hydraulic efficiency is not the sole criterion. In addition to performing its hydraulic function, the drainage channel should be economical to construct and maintain. Open channels should be reasonably safe for vehicles accidentally leaving the travelled way, pleasing in appearance, convey collected water without damage to the highway or adjacent property and minimize the environmental impacts.

These considerations are usually so interrelated that optimum conditions cannot be met for one without compromising one or more of the others. The objective is to achieve a reasonable balance, but the importance of traffic safety must not be underrated.

There are various types of open channels encountered by the road drainage designer of highway facilities including:

i. Stream channels
ii. Chutes
iii. Roadside channels or ditches
iv. Irrigation channels and
v. Drainage ditches.

The principles of open channel flow hydraulics are applicable to all drainage systems including culverts.

Stream channels are usually:

i. Natural channels with their size and shape determined by natural forces (morphology);
ii. Compound in cross section with a main channel for conveying low flows and a floodplain to transport extreme flood flows; and
iii. Shaped geomorphologically by the long-term history of the sediment load and water discharge that they have experienced.

Artificial channels include roadside channels, irrigation channels, and drainage ditches that are man-made with regular geometric cross sections, and unlined, or lined with artificial or natural material to protect against erosion.

13.5.1 HYDRAULIC CONSIDERATIONS

An evaluation of hydraulic considerations for channel design alternatives should be made early in the project development process. The extent of the hydrologic and hydraulic analysis should be commensurate with the type of highway, complexity of the drainage facility, and associated costs, risks, and environmental impacts.

The hydraulic design of an open channel consists of developing a channel section to carry the design discharge under the controlling conditions, adding freeboard as needed and determining the type of channel protection required to prevent erosion. In addition to erosion protection, channel linings can be used to increase the hydraulic capacity of the channel by reducing the channel roughness.

The hydraulic capacity of a drainage channel is dependent on the size, shape, slope and roughness of the channel section. For a given channel, the hydraulic capacity becomes greater as the grade or depth of flow increases. The channel capacity decreases as the channel surface becomes rougher.

A rough channel can sometimes be an advantage on steep slopes where it is desirable to keep flow velocities from becoming excessively high. A good open channel design minimizes the effect on existing water surface profiles.

Open channel designs, which lower the water surface elevation, can result in excessive flow velocities and cause erosion problems. A planned rise in water surface elevation can cause:

- Objectionable flooding of the roadbed and adjacent properties; and
- An environmental and maintenance problem with sedimentation due to reduced flow velocities.

Additional hydraulic considerations include those of channel and floodwater characteristics such as movable beds, heavy bed loads and bulking during flood discharges.

13.5.2 SAFETY CONSIDERATION

An important aspect of highway drainage design is that of traffic safety. The shape of a roadside channel section should minimize vehicular impact and provide a traversable section for vehicles leaving the travelled way. The ideal channel section, from a safety standpoint, will have flattened side slopes and a curved transition to the channel bottom.

13.5.3 MAINTENANCE CONSIDERATION

The design of open channels and roadside ditches should recognize that periodic maintenance inspection and repair is required. Provisions should be incorporated into the design for access to a channel by maintenance personnel and equipment.

When assessing the need for permanent or temporary access easements, entrance ramps and gates through the right of way fences, consideration should be given to the size and type of maintenance equipment required. Damaged channels can be expensive to repair and interfere with the safe and orderly movement of traffic. Minor erosion damage within the right of way should be repaired immediately after it occurs, and action taken to prevent the recurrence.

Conditions, which require extensive repair or frequently recurring maintenance, may require a complete redesign rather than repetitive or extensive reconstruction. The advice of an Expert Drainage Engineer should be sought when evaluating the need for major restoration.

The growth of weeds, brush, and trees in a drainage channel can effectively reduce its hydraulic efficiency. The result being that a portion of the design flow may overflow the channel banks causing flooding and possible erosion.

Channel work on some projects may be completed several months before total project completion. During this interim period, the contractor must provide interim protection measures and possibly advance the planned erosion control program to assure that minor erosion will not develop into major damage.

13.5.4 ALIGNMENT AND GRADE

The highway drainage channel must be located where it will best serve its intended purpose, using the grade and alignment obtainable at the site. However, abrupt changes in alignment and grade should be avoided as much as possible. A sharp change in alignment presents a point of attack for flowing water, and abrupt changes in grade can result in possible scour when the grade is steepened or deposition of transported material when the grade is flattened.

Changes in alignment should be made as gradually as possible to introduce the least amount of unstable flow. When possible, locate horizontal curves where the profile is flattest. Open channels should be graded to a desirable minimum gradient of 0.004 m/m so that some slight settlement will not create large areas of standing water. Maximum slopes should reflect the type of soil and linings to be used especially for grass-lined channels.

In principle, a drainage channel should have a flow velocity that neither erodes nor cause deposition in the channel (refer to **Table 13.36**). This optimum velocity is dependent on the size and slope of channel, the quantity of flowing water, the material used to line the channel, the nature of the bedding soil and the sediment being transported by the flow.

The point of discharge into a natural watercourse requires special attention. Water entering a natural watercourse from a highway drainage channel should not cause eddies with attendant scour of the natural watercourse. In erodible embankment soils, if the flow line of the drainage channel is appreciably higher than that of the watercourse at the point of discharge, then the use of a spillway is advisable to prevent erosion of the channel.

13.5.5 CHANNEL DESIGN

Hydraulic design associated with natural channels and roadway drains is a process that selects and evaluates alternatives according to established criteria. These criteria are the standards established to ensure that a highway drainage system meets its intended purpose without endangering the structural integrity of the facility itself and without undue adverse effects on the environment or the public welfare.

Side drains are essential for the performance of the road, and they should be properly designed. Any savings in design cost will be far outweighed by increased maintenance costs over the life of the road.

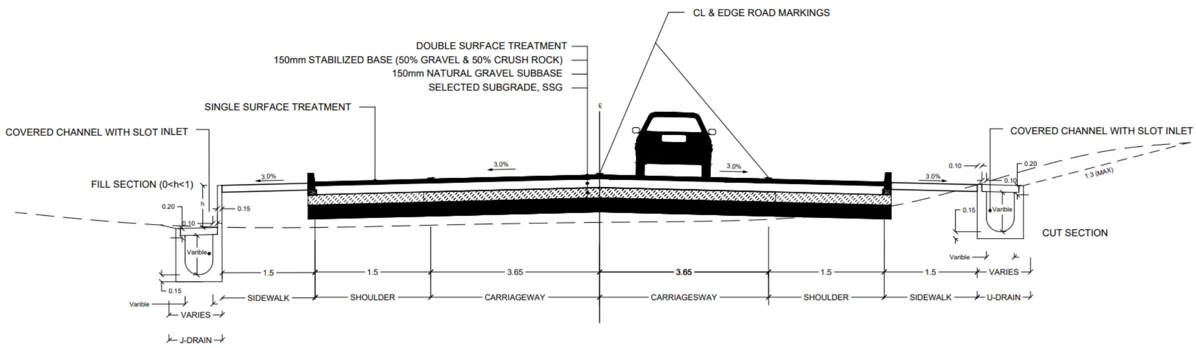
The design of side drains should consider:

i. Whether the drain serves the whole width of the road or just half the width.
ii. Does the drain serve just the road or does it also provide drainage to the adjoining areas (refer to **Plate 13.5**). Typical placement of side drains to intercept runoff from both the road and adjoining ground is illustrated in **Figure 13.23**.
iii. The gradient of the road; and
iv. The nature of the materials the road is crossing: are they easily eroded like silts and sands or erosion resistant like stiff clays or rock

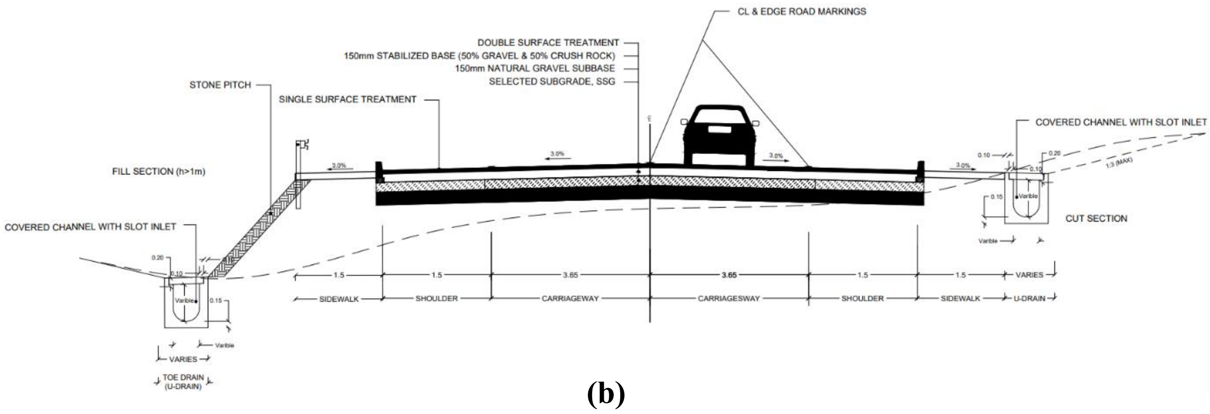
The designer controls a range of variables to fit the side drains to their environment:

i. The channel shape and size.
ii. Whether or not the channel is lined.
iii. Whether scour checks are provided and if so their spacing.
iv. The spacing of turnouts or side drain relief culverts.

Plate 13.5 Roadside drain collecting lateral flows



(a)



(b)

Figure 13.23 Typical placement of side drains to intercept runoff from both the road and adjoining ground

13.5.6 DESIGN CRITERIA OF OPEN CHANNELS

Design criteria establish the standards by which an open channel shall be constructed. They form the basis for the selection of the final design configuration. Listed below are examples of design criteria that shall be considered for channel design.

13.5.6.1 STREAM CHANNELS

The following design criteria apply to natural channels: `

i. The hydraulic effects of flood plain encroachments shall be evaluated over a full range of frequency-based peak discharges from the design frequency to the check/review recurrence intervals on any major highway facility as deemed necessary by the designer.
ii. If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions as much as practicable. Some form of energy dissipation may be necessary when existing conditions cannot be duplicated.
iii. Stream bank stabilization shall be provided, when appropriate, to any stream disturbance such as encroachment and should include both upstream and downstream banks and the local site.
iv. Features, such as dikes and levees, associated with natural channel modifications should have a 5m minimum top width with access for maintenance equipment. Vehicle turning points shall be provided no further than 500m apart and at the end of any such feature.

13.5.6.2 ROADSIDE AND MEDIAN DRAINS

Side drains keep water off the surface of the road and keep the foundations of the road dry. Effective side drains will reduce the need for maintenance by preventing deterioration of the surface and will provide a drier and hence safer road.

If the side drains are missing or not working then, water running along or across the road may lead to gully erosion. The foundations may get wet and soft leading to rutting. Where the batter slope is steep due to restricted width and erosion of the slope is likely, a kerb should be provided. However, on high-speed carriageways, kerbs or steep batter slopes should be avoided for safety reasons.

A common reason that side drains stop working is that people crossing the drain block them: either vehicles driving across the drain and damaging it or pedestrians trying to make walking over it easier.

i. Water Disposal

Side drains collect runoff water. That water then has to be discharged from the drain. This can be either:

- To the adjoining ground by means of a turnout; or
- Across the road to the side drain on the downstream side of the road via a side drain relief culvert.

The position and number of turnouts should be indicated on the design drawings. The final location should be determined by site inspection so they are provided where they will work.

ii. Erosion Control

Side drains channel water and concentrate flows, especially where water is discharged via turnouts. Scour of the side drains, if not controlled, can lead to the formation of gullies that eventually can become so deep that the road may have to be abandoned. The construction of simple scour checks and check dams in the side drains will reduce velocity, remove silt, and allow vegetation to become established thus controlling erosion. Erosion downstream of turnouts can affect not only the road but also the adjoining land. To prevent erosion, provide sufficient turnouts to disperse the flow and provide erosion protection where necessary.

iii. Channel Location and Type

Assuming adequate functional design, the next most important design consideration is channel location. Locations that avoid poorly drained areas, unstable soil conditions, and frequently flooded areas can greatly reduce drainage related problems.

Often, drainage and open channel considerations are not considered the primary decision factors in the roadway location; however, they are factors, which will often directly or indirectly affect many other considerations. Often, minor alignment adjustments can avoid serious drainage problems.

If a channel can be located far enough away from the highway, the concerns of traffic safety and aesthetics can be somewhat mitigated. The cost of additional right of way may be offset somewhat by the reduced cost of erosion control, traffic protection, and landscaping.

Ditches should be located where they can fully intercept the flow from the natural catchments adjacent to the road. The location of ditches is mainly dependent on the space available. Possible locations are:

- Along the edge of the road.
- Along the top of cuttings.
- At the toe of embankments.

The main limitation on median drains relates to safe slopes for errant vehicles. Median drains are usually flat bottomed and 2.0 m to 2.5 m wide at the bottom to accommodate maintenance machinery. For very wide medians it is desirable from a road safety perspective to have batter slopes of about 1:10 (to cater for trucks) and grated pits and underground pipes. However, the side slope of 1:10 or flatter severely restricts the capacity of such drains unless the median is very wide. It may therefore be necessary in many instances to provide a steeper batter slope up to 1:6 (satisfactory for cars) or greater in which case a safety barrier may be required.

13.5.6.2.1 SHAPE AND MATERIAL OF ROADSIDE DRAINS

The shape and material of a drain section is generally determined by considering the intended purpose, terrain, flow velocity and quantity of flow to be conveyed.

A. Shape

i. **V-ditch**. The Triangular V-ditch is intended primarily for low flow conditions such as in median and roadside ditches. V-shaped ditches are susceptible to erosion and will require lining when flow velocities exceed the permissible flow velocities. The use of ‘V’ drains is to be limited and generally confined to situations where the width available for a drain is constrained.

ii. **Trapezoidal drain/table drain**. The most common drain shape for large flows is the trapezoidal section. Trapezoidal drains are easily constructed by machinery and are often the most economical to construct. When a wide trapezoidal section is proposed, both traffic safety and aesthetics can be improved by rounding all angles of the channel cross section with vertical curves. The trapezoidal shape is the preferred type or shape, the bottom being wide enough to accommodate maintenance machinery.

iii. **Rectangular and U-shaped drains**. Rectangular and U-shaped channels are often used to convey large flows in areas with limited right of way and at town sections. At some locations, guardrail or other types of positive traffic barrier may be necessary between the travelled way and the channel. Although rectangular channels are relatively expensive to construct, since the walls must be designed as earth retaining structures, the construction costs can be somewhat offset by the reduced costs associated with right of way, materials, and channel excavation.

The minimum recommended width of the bottom of the side drain is 500 mm.

B. Material

i. **Earth ditch.** The use of earth ditch may not be suitable in high rainfall areas because of erosion and its attendant high maintenance costs. Despite these it is still popular because the construction cost is cheap. It is however not recommended on steep sections and loose soils (See **Figure 13.24)**.

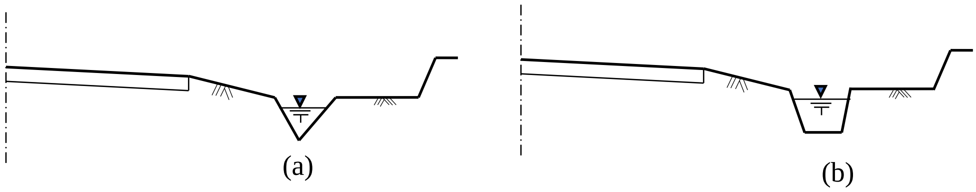


Figure 13.24 Earth ditch

ii. **Stone pitched drain.** The earth surface may be reinforced with stone pitching to prevent scouring. They may have curved surfaces (See **Figure 13.25)**.

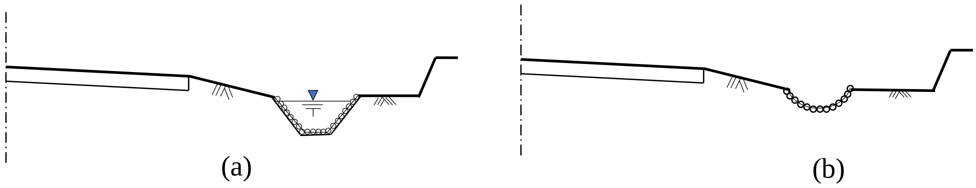


Figure 13.25 Stone pitched drain

iii. **Block masonry drain.** Block pitching may be resorted to instead of stone pitching. The bottom can also be protected by a concrete bed (See **Figure 13.26**).

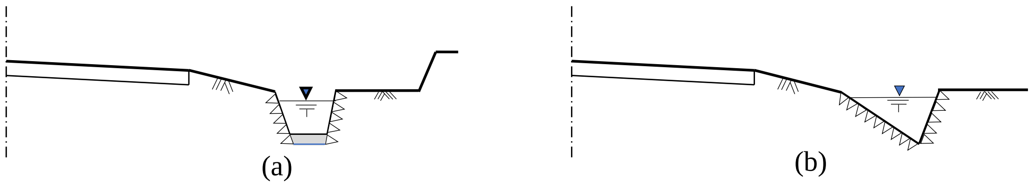


Figure 13.26 Block/masonry drain

iv. **Cement concrete drain.** They can take different forms as illustrated in **Figure 13.27**.

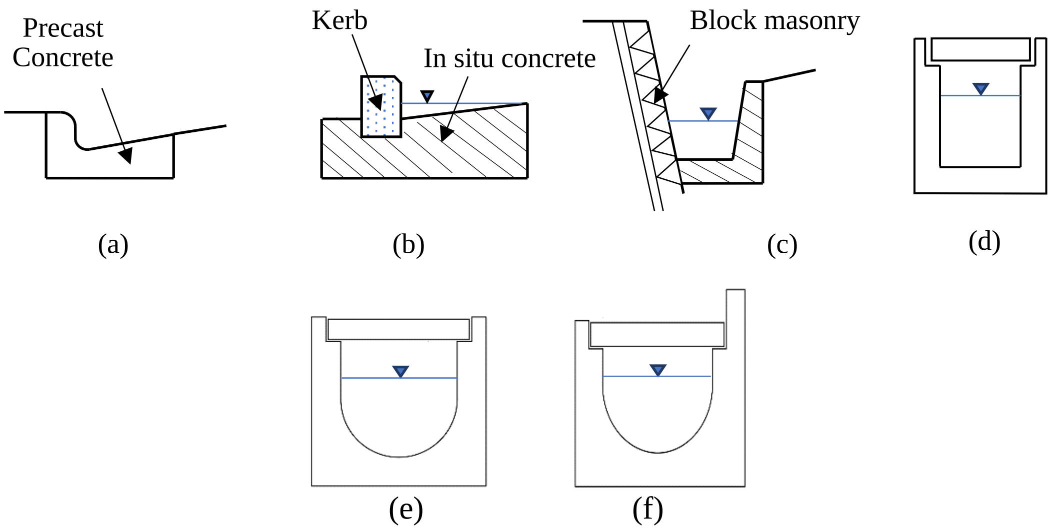


Figure 13.27 Cement concrete drain

v. **Asphalt concrete drain.** The different forms of this are shown in **Figure 13.28**.

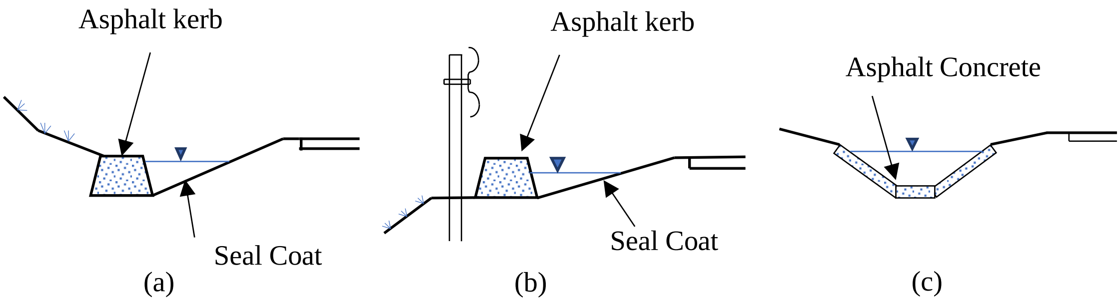


Figure 13.28 Asphalt concrete drain

13.5.6.2.2 TURNOUTS

Turnouts (mitre drain) are designed to take water from the side drain into the natural drainage of the surrounding environment. Turnouts are designed in the same way as side drains. The minimum width of turnouts should be 0.60m and the cross-section should have at least the same capacity as side drain. When possible, they should have a steeper longitudinal slope than the side drains. The angle between the turnouts and the side drain should never be greater than 45°. An angle of 30° is ideal. Some excavated soil should be used to block the downhill side of drain to ensure water flows into mitre drain. Alternatively, when the side drain is constructed, gaps should be left in the drains immediately downhill of each turnout location. **Figure 13.29** shows a typical turnout/mitre drain.

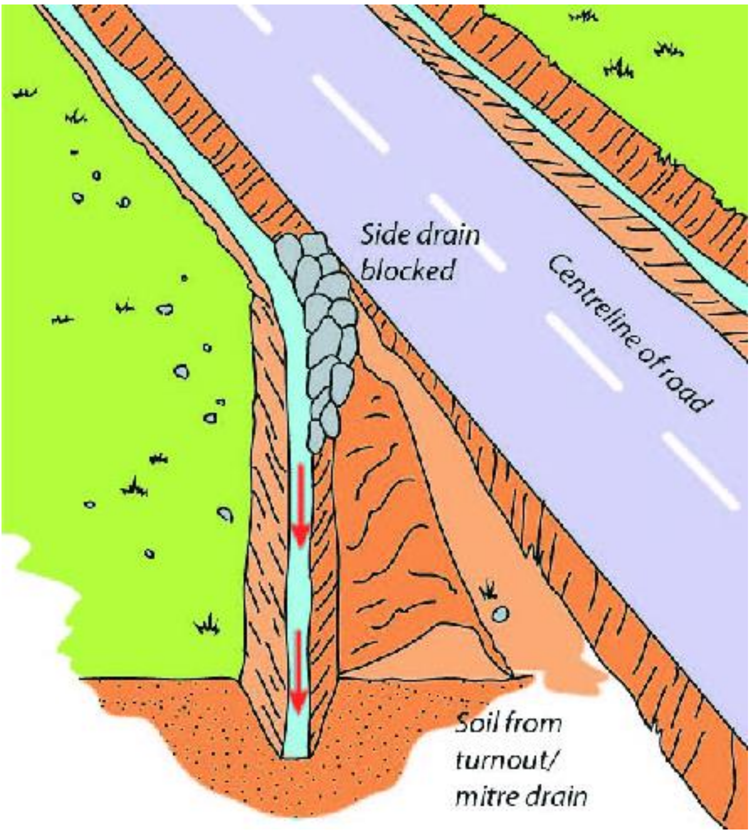


Figure 13.29 Turnout/mitre drain

The frequency of turnouts is controlled by various factors. On flat ground, the capacity of the side drain is critical: turnouts are required to prevent the side drain from overflowing. On sloping ground concentration of flows is critical: as the ground and hence the side drain becomes steeper more turnouts are required to prevent the flow concentration leading to erosion at the end of the turnout.

The maximum spacing for turnouts is given in **Table 13.34**. This ensures that the quantity and velocity of water being discharged at each mitre drain outlet is limited and reduces the risk of erosion damage to the drainage system or on the adjoining land.

Table 13.34 Maximum spacing of turnouts

Side drain gradient, S (%)	Spacing (m)
1 - 2	200
4	100
6	500
8	400
10	25
12	20

13.5.6.2.3 BATTER DRAINS AND CHUTES

Batter drains or chutes are structures that are designed to convey run-off from the tops of embankments down the slope of a cut or fill batter and discharge to the natural ground surface or channel at either non-erosive velocities or onto a non-erodible surface. Occasionally, in steep terrain they may be used to convey run-off from the tops of cuttings down the batter to the roadway table drain or piped drainage system. **Plate 13.6** is an example of a batter drain.

Plate 13.6 Batter drain

Batter drains can be permanent or temporary. Temporary batter drains are used during the construction period to control water flow and to protect mulched or newly seeded batters from the erosive forces of concentrated flow.

On traversable batter slopes, the permanent batter drain or chute should be shaped to a channel which is traversable by a vehicle.

Care needs to be taken, if batter drains and chutes are proposed in highly dispersive or erodible soils, as their use is not generally desirable for these conditions.

A major problem with this type of drain is that of the flow overtopping or running parallel to the channel and eroding the surrounding soil. The problem is prevalent with slope drains constructed from half-round metal or concrete pipes, pre-cast concrete units or cast-in-situ concrete channels. This situation may be aggravated by poor installation or by not providing sufficient slope drains, or not satisfactorily locating them, to cater for the predicted flow. It is preferable to use rock mattresses, as they encourage a wider, less concentrated flow, are flexible enough to settle and allow water to enter the channel from the side.

The slope drains used on embankments may be constructed progressively as the height increases. Care should be taken to ensure the efficient collection of water at the top of the batter. It may be desirable to have the slope drains more closely spaced during construction to minimise the size of flows.

The inlet to the slope drain should ensure that all the run-off is collected and should not allow water to flow down the slope adjacent to the channel. The bottom of the drain should have some type of energy dissipating, or erosion control device to prevent scouring at the base of the batter. The opposite bank of the drain at the bottom of the batter drain or chute should also provide an erosion control device to protect the bank and turn the flow, if required.
A four-step process is provided for the design of batter drains.

Step 1 – Dimensions

Batter drains should have a minimum depth of 300 mm. Hydraulic capacity of a batter drain is normally defined by the allowable head water height upstream of the drain's inlet.

Step 2 – Foundations

The lining of the batter drain should be adequately anchored to the foundations to avoid slippage or separation, with a maximum distance of 3 m between anchorage points.

In cases where prefabricated units need to be bolted together, it is important that all bolt holes are sealed with a flexible sealant to allow for flexural movement.

Step 3 – Inlet design

The inlet area should be protected against possible scour resulting from accelerating flow velocities (usually more important on temporary batter drains). This protection is necessary to prevent water from either undermining the top of the batter drain or being diverted along the edge of the lining (the most common cause of failure).

For temporary batter drains (i.e. drainage chutes) and during the early revegetation stage of permanent batter drains, sand or gravel bags can be used to direct inflow towards the centre of the chute.

Step 4 – Outlet design

Typically, an energy dissipator will be required.

The outlet may consist of a bed of nominal 150 mm rock (minimum) placed with a minimum bed thickness of 250 mm or at least 1.5 times the maximum rock size. Typical dimensions of the rock bed are (**Equation (13.81)**):

$$L = 6D_e \text{ metres long (minimum)} \tag{13.81}$$

W =T + 0.6 metres wide at the batter drain outlet, expanding to T + 0.5L + 0.3 metres at the end of the dissipator.

Where,

D_e = equivalent pipe diameter (m) of the batter drain flow area

T = top width (m) of flow in the batter drain

Batter drains should be lined each side with a minimum 300 mm wide (turf) grassed filter strip or rock to control side erosion caused by splash. In areas where the total disturbance is to be minimised, or where introducing turf is undesirable, other forms of erosion control such as geotextiles or concrete may be preferred.

13.5.6.2.4 BENCH DRAINS

A bench drain is provided for the bench (i.e., ledge) that is constructed on a batter, or natural slope. The purpose of benches is to reduce erosion to the batter faces, reduce the amount of water in cuttings to be carried by table drains and in some cases to also improve sight distance on horizontal curves. Bench drains carry water from the bench to suitable drainage outlets. An example of a bench drain is shown in **Figure 13.30**.

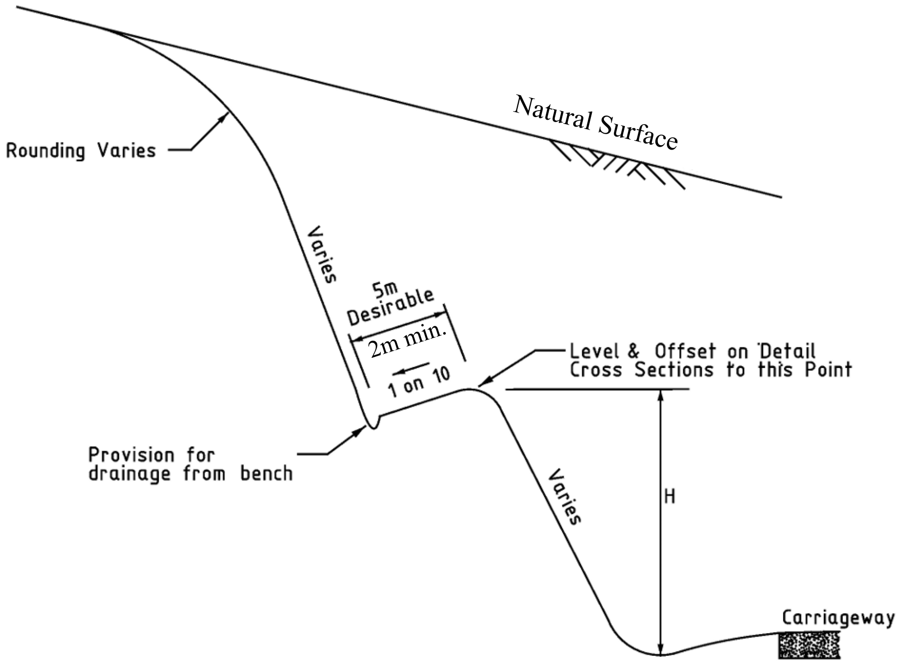


Figure 13.30 Bench drain

13.5.6.2.5 CATCH DRAINS AND CATCH BANKS

Catch drains (also known as cut-off drains) and catch banks are separate devices but can be used together. They are generally located on the high side of cuttings clear of the top of batters to intercept the flow of surface water and upper soil seepage water (**Figure 13.31**). Their purpose is to prevent overloading of the table drain and drilling and erosion or scour of the batter face.

Catch drains and/or catch banks can also protect embankments, disturbed areas and stockpile sites from surface water. Alternatively, catch drains placed at the bottom of fill slopes intercept water from adjacent properties as well as convey road drainage to an outlet.

These devices are generally located no closer than 2.0 m from the edge of the cuttings in order to minimise possible undercutting of the top of the batter.

The type of catch drain shown in **Figure 13.31** is usually about 0.3 m deep and is a function of the required capacity. The width of the flat bottom drain will depend on the space available within the road reservation and the likely run-off. The width and depth should be sufficient to carry the design flow, a width in the range 1.2 m to 2.5 m being typical.

In erodible soils, it is preferable for the catch drain in-cut to take the form of a low mound along the top of the batter (**Figure 13.31**) as a drain, cut into the surface, may rapidly erode and enlarge itself, or cause local slips in the batter by piping. V-shaped drains are not preferred and should not be used in erodible soils.

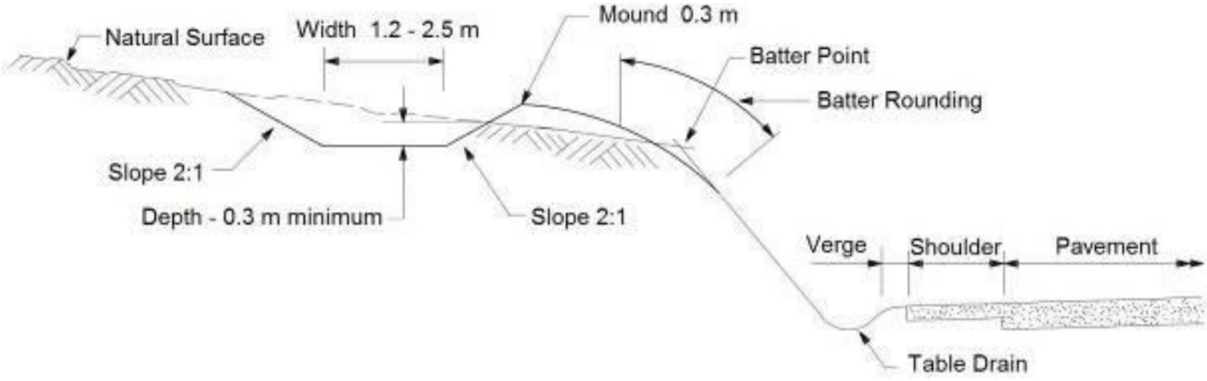


Figure 13.31 Typical catch drain with flat bottom

Catch banks (**Figure 13.32**) are sometimes used instead of drains to reduce effects of seepage on stability of the batter slopes and also to minimise disturbed ground and hence scour potential. However, use of catch banks alone may necessitate importation of suitable embankment material as in situ material may not be suitable.

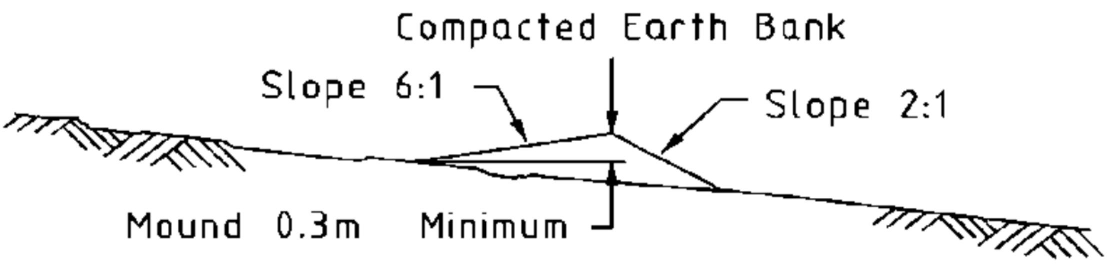


Figure 13.32 Catch bank diagram

The design of catch drains and banks is based on the same methodology as for open channels. A freeboard of at least 150 mm should be used. Where site conditions or some other constraint restricts the construction of a channel with suitable dimensions, supplementary channel treatments such as synthetic channel linings, riprap or concrete inverts should be utilised to withstand the higher velocities, **Plate 13.7** shows an example of a concrete lined catch drain.

Flow discharging from the catch drain should not be allowed to cause or aggravate erosion. Flow from a catch drain may be discharged to an existing drain, watercourse, or to a chute.

Plate 13.7 Concrete-lined catch drain

13.5.6.2.6 RELIEF CULVERTS

Relief culverts are different from watercourse culverts. Their purpose is to relieve the flow from long stretches of side drain ditches located on the uphill side of a road where turnouts cannot be provided. These culverts should be used where possible instead of using a long side ditch without relief culverts.

The maximum spacing of relief culverts is set out in **Table 13.35**. If it is not possible to meet this requirement, erosion control measures such as side drain lining are required in addition to possible widening of the side drain.

Table 13.35 Recommended spacing between relief culverts

Longitudinal gradient of road and drain (%)	Recommended interval of cross drainage (m)
2	200
3-4	150
5	135
6	120
7-8	100
9-10	80
11-12	60

13.5.6.2.7 ACCESS TYPE CULVERTS AT AN INTERSECTION

The provision of access across drainage channels is often overlooked at the design stage. If access across drainage channels is not provided, people will block the drain if it presents an obstruction, causing water to flood the road. Therefore, access needs to be considered carefully at a design stage.

The purpose of an access culvert is to allow water in roadside drains to flow under a minor road where it branches off from the main road at an intersection.

- An access culvert will be required if the minor road slopes towards the main road;
- But if the minor road slopes away from the main road the side drains can be connected.

Access to plots or property is usually provided in the form of access slabs over lined drains (**Plate 13.8**).

Plate 13.8 Access slab

Intersection with other roads may also need access culverts to maintain continuity of drainage. The same covered drain detail can be used to avoid raising the road locally to accommodate larger structures.

Access culverts should be of just sufficient capacity/size to accommodate the maximum side drain flow whilst ensuring that they can be easily maintained.

The invert level of an access culvert should be the same as or slightly lower than that of the uphill side drain that needs to be passed under the road.

It is common to provide a bell mouth to allow longer turning vehicles to negotiate the intersection. For reasons of economy, the access culvert is often sited away from the main road edge before the bell mouth where the minor road is of normal width.

13.5.6.2.8 CATCH PIT INLETS

Side drain relief culverts are normally provided with a catch pit inlet structure (**Plate 13.9**). This is designed to capture the water flowing along the side drain and turn that flow through 90º. It then flows under the road and is discharged through an outfall channel.

Plate 13.9 Catch pit inlet structure

13.5.7 DESIGN ANALYSIS OF OPEN CHANNEL FLOW

Design analysis of both natural and artificial channels proceeds according to the basic principles of open channel flow. The basic principles of fluid mechanics continuity, momentum, and energy can be applied to open channel flow with the additional complication that the position of the free surface is usually one of the unknown variables.

The determination of this unknown is one of the principal problems of open channel flow analysis and it depends on quantification of the flow resistance. Natural channels display a much wider range of roughness values than artificial channels. The drainage designer is expected to posses the knowledge of theoretical principles of open channel flow; hence most of the theoretical equation are not included in this chapter. The designer should refer to open channel flow text books for theoretical background information.

Steady, uniform flow is an idealized concept of open channel flow which seldom occurs in natural channels. However, for most practical highway applications, the flow is steady and changes in width, depth or direction (resulting in non-uniform flow) are sufficiently small so that flow can be considered uniform.

The following equations are those most commonly used to analyse open channel flow and are included here.

Continuity Equation – The continuity equation is the statement of mass in fluid mechanics. For the special case of one dimensional, steady flow of an incompressible fluid, it is given by **Equation (13.82)**.

$$Q = A_1 \times V_1 = A_2 \times V_2 \quad (13.82)$$

Where,

Q = discharge, m^3/s

A = cross-sectional area of flow, m^2

V = mean cross-sectional velocity, m/s (which is perpendicular to the cross section). Subscripts 1 and 2 refer to successive cross sections along the flow path.

Manning's Equation –For a given depth of flow in a channel with a steady, uniform flow, the mean velocity, V, can be computed with Manning's equation (**Equation (13.83)**).

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{13.83}$$

Where,

V = velocity, m/s

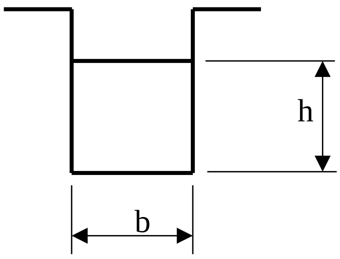
n = Manning's roughness coefficient

R = hydraulic mean depth (hydraulic radius) = A/P, m

P = wetted perimeter, m

S = slope of the energy gradeline, m/m (note: for steady uniform flow, S = channel slope, m/m)

Hydraulic mean depth (R) is derived from dividing the flowing water section area (m^2) by the length of wetted perimeter (m). See the following example (**Figure 13.33**). Mathematical formula of hydraulic mean depth by various sections of drainage facilities are given in **Table 13.37** and **Table 13.37**.



$$R = \frac{A}{P} = \frac{b \times h}{b + 2h}$$

A: Flowing water section area (m^2)

P: Length of wetted perimeter (m)

Figure 13.33 Example of hydraulic mean depth calculation

The selection of Manning's 'n' is generally based on observation; however, considerable experience is essential in selecting appropriate 'n' values. The range of 'n' values for various types of channels and floodplains is given in **Table 13.9**.

For a given channel geometry, slope, and roughness, and a specified value of discharge Q, a unique value of depth occurs in steady uniform flow. It is called the normal depth. The normal depth is used to design artificial channels in steady, uniform flow and is computed from Manning's Equation (**Equation (13.84)**).

$$Q = \frac{A}{n} R^{2/3} S^{1/2} \tag{13.84}$$

If the normal depth computed from Manning's Equation is greater than critical depth, the slope is classified as a mild slope. Conversely, if the normal depth is less than critical depth, the slope is a steep slope. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

The actual sectional area of discharge includes a clearance of 20 to 30% to take account of possible pile up sand or silt which reduces the effective area of flow. Average velocity of flow is adopted in design as a compromise between maximum and minimum velocities, since the former is responsible for gutter scouring and the latter sedimentation. **Table 13.36** gives recommended ranges of average velocities for different surface conditions.

Table 13.36 Average velocity of flow

Surface condition	Average velocity of flow (m/s)
Cement concrete	0.6 - 3.0
Asphalt concrete	0.6 - 1.5
Stone or block pitching	0.6 - 1.8
Hard gravel or clay	0.6 - 1.0
Sand or silt	0.1 - 0.2

Channel Conveyance - In channel analysis, it is often convenient to group the channel properties in a single term called the channel conveyance K given by **Equation (13.85)**.

$$K = \frac{AR^{2/3}}{n} \qquad (13.85)$$

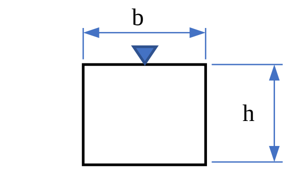
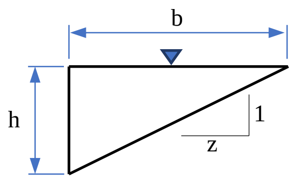
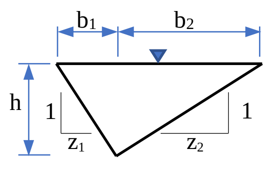
and then Manning's Equation can be written as **Equation (13.86)**:

$$Q = KS^{1/2} \qquad (13.86)$$

The conveyance represents the carrying capacity of a stream cross-section based upon its geometry and roughness characteristics alone and is independent of the streambed slope.

The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross section and the flow distribution through the opening in a proposed stream crossing. Manning's Equation should not be used for determining high water elevations in a bridge opening.

Table 13.37 Flowing water section area and hydraulic mean depth of section

<table>
<tr><th>Section</th><th>Flowing water section area (A), m^2</th><th>Hydraulic mean depth (R), m</th></tr>
<tr><td>
</td><td>bh</td><td>$\frac{bh}{2h+b}$</td></tr>
<tr><td rowspan="2">
</td><td rowspan="2">$\frac{1}{2}bh$</td><td>$\frac{1}{2}\cdot\frac{bh}{h+\sqrt{b^2+h^2}}$</td></tr>
<tr><td>$\frac{1}{2}\cdot\frac{bh}{h+h\sqrt{(1+z^2)}}$</td></tr>
<tr><td rowspan="2">
</td><td>$\frac{1}{2}h(b_1+b_2)$</td><td>$\frac{1}{2}\cdot\frac{h(b_1+b_2)}{\sqrt{h^2+b_1^2}+\sqrt{h^2+b_2^2}}$</td></tr>
<tr><td>$\frac{h^2}{2}(z_1+z_2)$</td><td>$\frac{1}{2}\cdot\frac{h(z_1+z_2)}{\left(\sqrt{(1+z_1^2)}+\sqrt{(1+z_2^2)}\right)}$</td></tr>
</table>

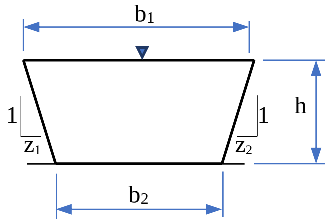
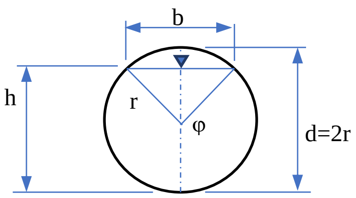
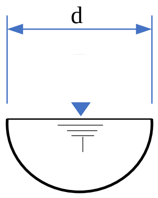
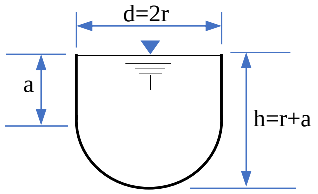
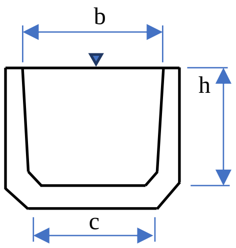
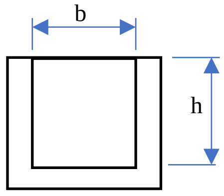
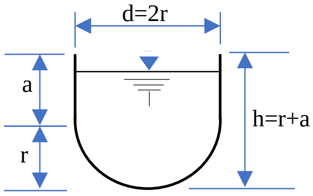
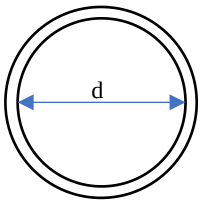
Section	Flowing water section area (A), m²	Hydraulic mean depth (R), m
	$\frac{1}{2}h(b_1+b_2)$	$\frac{h}{2}\cdot\frac{b_1+b_2}{b_2+\sqrt{4h^2+(b_1-b_2)^2}}$
	$\frac{h^2}{2}(z_1+z_2)+hb_2$	$\frac{\frac{h^2}{2}(z_1+z_2)+hb_2}{\left(b_2+h\left(\sqrt{(1+z_1^2)}+\sqrt{(1+z_2^2)}\right)\right)}$
 φ: Radian (=π/180×φ°)	$\frac{d^2}{2}\left(\varphi-\frac{1}{2}\sin 2\varphi\right)$ Full case $\frac{\pi d^2}{4}$	$\frac{d}{4}\left(1-\frac{1}{2\varphi}\sin 2\varphi\right)$ Full case $R=\frac{d}{4}$
	$\frac{\pi d^2}{8}$	$\frac{d}{4}$
	$ad+\frac{\pi d^2}{8}$	$2a+\frac{d}{4}$

Table 13.38 Calculation of Flowing water section area and hydraulic mean depth

Section	Section size			Flowing water section area A(m²)	Hydraulic mean depth R(m)
	a/b	c/d	h		
	0.18	0.17	0.18	0.033(0.025)	0.050(0.055)
	0.24	0.22	0.24	0.055(0.044)	0.079(0.073)
	0.30	0.26	0.24	0.067(0.054)	0.091(0.084)
	0.30	0.26	0.30	0.084(0.067)	0.098(0.091)
	0.30	0.26	0.36	0.101(0.081)	0.103(0.097)
	0.36	0.31	0.30	0.101(0.070)	0.110(0.089)
	0.36	0.31	0.36	0.121(0.080)	0.117(0.090)
	0.45	0.40	0.45	0.191(0.153)	0.147(0.137)
	0.60	0.54	0.50	0.342(0.228)	0.196(0.170)

Section	Section size			Flowing water section area $A(m^2)$	Hydraulic mean depth R(m)
	a/b	c/d	h		
	0.25		0.25	0.063(0.05)	0.083(0.067)
	0.30		0.30	0.090(0.072)	0.100(0.08)
	0.35		0.35	0.123(0.098)	0.117(0.093)
	0.45		0.45	0.203(0.162)	0.150(0.120)
	0.60		0.60	0.360(0.288)	0.200(0.160)
	0.80		0.80	0.640(0.512)	0.267(0.213)
	1.00		1.00	1.000(0.800)	0.333(0.267)
	0.225	0.45	0.45	0.181(0.145)	0.563(0.45)
	0.375	0.45	0.60	0.248(0.199)	0.863(0.69)
	0.150	0.60	0.45	0.231(0.185)	0.45(0.36)
	0.300	0.60	0.60	0.321(0.257)	0.75(0.6)
	0.450	0.60	0.75	0.411(0.329)	1.05(0.84)
	0.600	0.60	0.90	0.501(0.401)	1.35(1.08)
	0.375	0.75	0.75	0.502(0.402)	0.938(0.75)
	0.525	0.75	0.90	0.615(0.492)	1.238(0.99)
	0.150	0.90	0.60	0.453(0.363)	0.525(0.42)
	0.450	0.90	0.90	0.723(0.579)	1.125(0.9)
	0.750	0.90	1.20	0.993(0.795)	1.725(1.38)
	0.300	1.20	0.90	0.926(0.740)	0.9(0.72)
	0.600	1.20	1.20	1.286(1.028)	1.5(1.2)
		0.20		0.031(0.025)	0.05(0.04)
		0.30		0.071(0.057)	0.075(0.06)
		0.40		0.126(0.101)	0.1(0.08)
		0.50		0.196(0.157)	0.125(0.1)
		0.60		0.283(0.226)	0.15(0.12)
		0.70		0.385(0.308)	0.175(0.14)
		0.80		0.503(0.402)	0.2(0.16)
		0.90		0.636(0.509)	0.225(0.18)
		1.00		0.786(0.628)	0.25(0.2)
		1.10		0.950(0.760)	0.275(0.22)
		1.20		1.131(0.905)	0.3(0.24)
		1.35		1.432(1.145)	0.338(0.27)
		1.50		1.767(1.414)	0.375(0.3)
		1.65		2.139(1.711)	0.413(0.33)
		1.80		2.545(2.036)	0.45(0.36)

Note: Values in ( ) indicates the case of 20% clearance (freeboard)

13.5.7.1 DESIGN PROCEDURE

This section presents a generalized procedure for the design of roadside and median channels. Although each project will be unique, the design steps outlined below will normally be applicable.

Step 1. Establish a Preliminary Drainage Plan

For proposed median or roadside channels, the following preliminary action should be taken:

A. Prepare existing and proposed plan and profile of the proposed channels. Include any constraints on design such as highway and road locations, culverts, utilities, etc.
B. Determine and plot on the plan the locations of natural basin divides and channel outlet points.
C. Collect any available site data such as soil types and topographic information.

Step 2. Obtain or Establish Cross Section Data

Establish preliminary cross section geometric parameters and controlling physical features considering the following guides:

A. Adequate channel depth should be provided to drain the subbase.
B. Channel side slopes based on geometric design criteria including safety, economics, soil, aesthetics, and access should be chosen.

Step 3. Determine Initial Channel Grades

Plot initial grades on the plan and profile. Note that slopes on roadside channels in cuts are usually controlled by highway grades. Use the following guides when establishing initial grades:

A. Provide a channel slope with sufficient grade to minimize ponding and sediment accumulation.
B. Where possible, avoid features which may influence or restrict grade, such as utility structures.

Step 4. Check flow Capacities and Adjust Sections as Necessary

A. Compute the design discharge at the downstream end of channel segments
B. Set preliminary values for channel size, roughness, and slope, based on long term conditions and considering maintenance.
C. Determine the maximum allowable depth of channel including freeboard.
D. Check the flow capacity using Manning's equation.
E. If the capacity is not adequate, possible considerations for increasing the capacity are provided below.
   i. Increase bottom width
   ii. Make channel side slopes flatter
   iii. Make channel slope steeper
   iv. Provide smoother channel lining
   v. Install drop inlets and a parallel storm drain pipe beneath the channel to supplement channel capacity
F. Provide smooth transitions at changes in channel cross sections
G. Provide extra channel storage where needed to replace floodplain storage and/or to reduce peak discharge.
   Figure 13.34 is a flow chart for determining the capacity of an open channel.

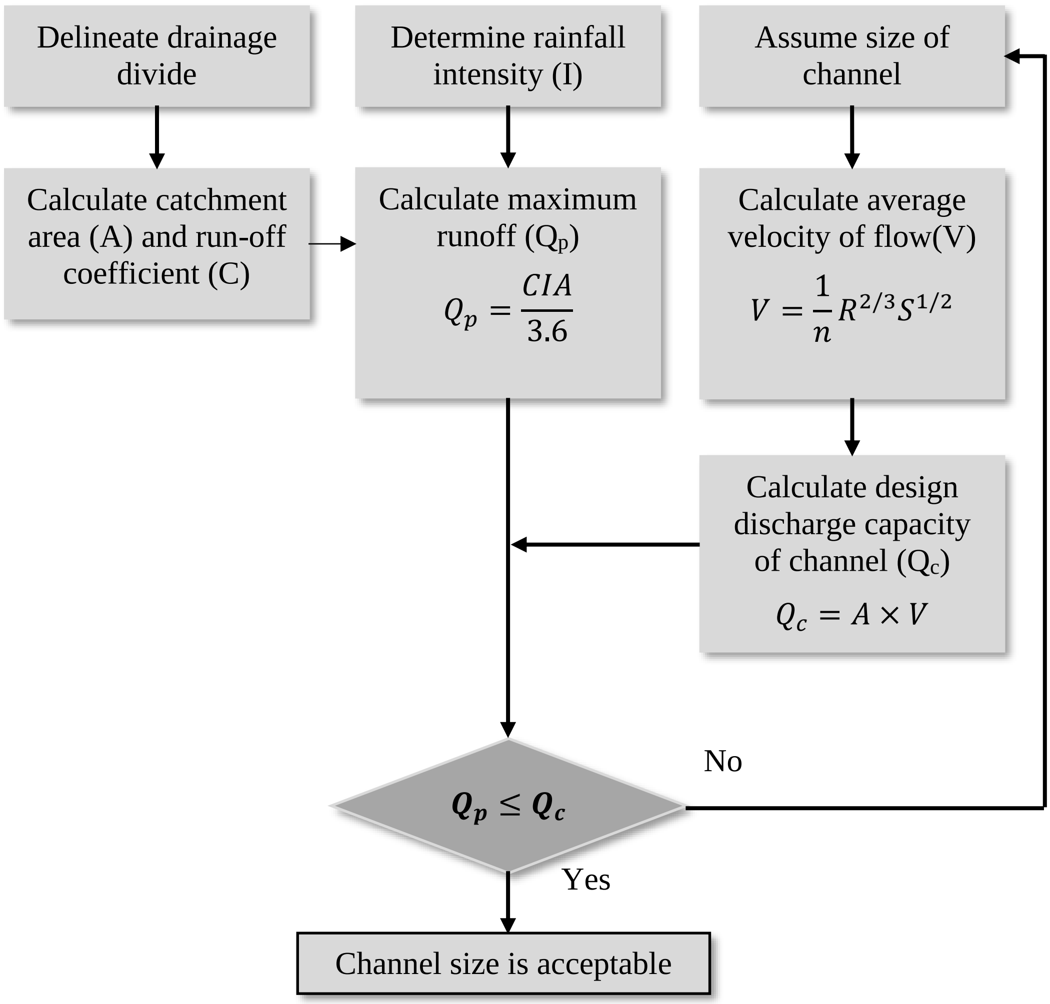


Figure 13.34 Flow chart for determining the capacity an open channel

Step 5. Step 5 Determine channel lining/protection needed (HEC-15)

A. Select a lining and determine the permissible shear stress τ_p in Pascals (N/m^2) from Table **13.39** and/or **Table 13.40**.
B. Estimate the flow depth and choose an initial Manning's n value from **Table 13.41**.
C. Calculate the normal flow depth at design discharge using Manning's equation and compare with the estimated depth. If the flow depth is acceptable, continue with the design procedure. If the depth is not acceptable, modify the channel size.
D. Compute the maximum shear stress at normal depth using **Equation (13.87)**.

$$\tau_d = \gamma \, d \, S \qquad (13.87)$$

Where,

τ_d = maximum shear stress, Pa
d = maximum depth of flow, m
γ = unit weight of water, 9810 N/m^3
S = average bed slope or energy slope, m/m

E. If the maximum shear stress (step 5D) is less than the permissible shear stress (step 5A), then the lining is acceptable. Otherwise consider the following options:
   i. Choose a more resistant lining
   ii. Use concrete, gabions, or other more rigid lining either as full lining or composite (keeping in consideration the possible deficiencies of rigid linings)
   iii. Decrease channel slope

iv. Decrease slope in combination with drop structures
v. Increase channel width and/or flatten side slopes

Step 6 Analyse outlet points and downstream effects

A. Identify any adverse impacts to downstream properties that may result from one of the following at the channel outlet:
   i. Increase or decrease in discharge,
   ii. Increase in velocity of flow,
   iii. Confinement of sheet flow,
   iv. Change in outlet water quality, or
   v. Diversion of flow from another catchment area
B. Mitigate any adverse impacts identified in 6A, possibilities include:
   i. Enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel,
   ii. Install velocity control structures,
   iii. Increase capacity and/or improve lining of downstream channel,
   iv. Install sedimentation/infiltration basins,
   v. Install sophisticated weirs or other outlet devices to redistribute concentrated channel flow, and
   vi. Eliminate diversions that result in downstream damage, and which cannot be mitigated in a less expensive manner.

Table 13.39 Classification of Vegetal Covers as to Degrees of Retardancy

Retardance	Cover	Condition
A	Native grass	Excellent stand, tall >750 mm
B	Native grass	Good stand, tall (average 300 – 600 mm)
C	Native grass	Good stand, uncut 150 – 300 mm
D	Native grass	Good stand, uncut 50 – 150 mm
E	Native grass	Good stand, cut to 40 mm, Stubble

Table 13.40 Summary of Shear Stress for Various Protection Measures

Protective Cover	Category	τ_p (Pa)
Vegetation	Class A	177
	Class B	101
	Class C	48
	Class D	29
	Class E	17
	Woven Paper	7
	Jute Net	22
Temporary	Straw W/Net	69
	Curled Wood Mat	74
	Synthetic Mat	96
Gravel: D_{50} = 25 mm		19
D_{50} = 50 mm		38
Rock: D_{50} = 150 mm		120
D_{50} = 300 mm		239
Gabions		1676
Geoweb		479
Soil Cement	(8%)	>2155
Concrete construction blocks, granular filter under layer		>958
Wedge-shaped blocks with drainage slot		>1197

Table 13.41 Manning's Roughness Coefficients (HEC-15)

Lining Category	Lining Type	n – value: Depth Ranges		
		0-0.15m	**0.15–0.06m**	**> 0.6m**
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	25 mm D_{50}	0.044	0.033	0.030
	50 mm D_{50}	0.066	0.041	0.034
Rock Riprap	(Class AI) 150 mm D_{50}	0.104	0.069	0.035
	(Class I) 300 mm D_{50}	--	0.078	0.040

Note: Values listed are representative values for the respective coefficients, n, vary with the flow depth.

*Some "temporary" linings become permanent when buried.

13.5.8 DESIGN OF DRAINAGE PIPES

13.5.8.1 PIPE DISCHARGE CAPACITY

The pipe discharge capacity is obtained from **Equation (13.84)**. The slope of the pipe may normally approximate to slope of the road. When the terrain is relatively flat, the recommended slope for the different sizes of pipe is indicted in **Table 13.42**.

Table 13.42 Standard slope, velocity, discharge capacity (n=0.013)

Diameter (mm)	Standard slope	Sectional area (m^2)	Velocity (m/s)	Discharge capacity (m^3/s)
200	1/100	0.031	1.04	0.032
250	1/150	0.049	0.99	0.048
300	1/150	0.071	1.12	0.079
350	1/200	0.096	1.08	0.103
400	1/250	0.126	1.05	0.132
450	1/300	0.159	1.04	0.165
500	1/350	0.196	1.03	0.201
600	1/450	0.283	1.02	0.288
700	1/550	0.385	1.03	0.392
800	1/650	0.503	1.04	0.518
900	1/750	0.636	1.05	0.661
1000	1/850	0.785	1.05	0.824
1100	1/950	0.950	1.05	0.997
1200	1/1000	1.131	1.09	1.232
1350	1/1100	1.431	1.13	1.617
1500	1/1200	1.767	1.16	2.049
1650	1/1200	2.138	1.23	2.629
1800	1/1300	2.545	1.25	3.181
2000	1/1400	3.142	1.29	4.053

13.5.9 PIPE DESIGN

Other considerations for the design of pipes are indicated below.

i. Design velocity of 1.0 -1.8 m/s is recommended because this velocity increases towards the lower end of the pipe.
ii. Minimum velocity is 0.6 m/s and maximum velocity is 3.0 m/s.
iii. It is recommended that pipes be laid at a minimum depth of 1.2m to withstand traffic load where they cross roads.
iv. Where drainage pipe is to be laid parallel or across other pipes such as electric pipes, water pipes etc. enough spacing should be provided between them. A minimum of 50cm is recommended for parallel laying and 30cm for those crossing. See **Figure 13.35**.

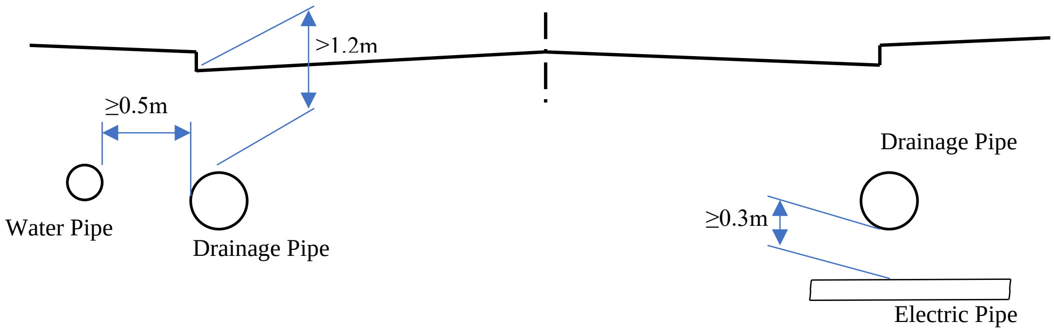


Figure 13.35 Recommendable minimum space between pipes

v. Interval of manholes are specified in **Table 13.43** and **Table 13.44**.

Table 13.43 Absolute minimum interval of manholes

Diameter (mm)	≤300	400-700	800-1800	≥2000
Interval (m)	30	50	60	100

Table 13.44 Maximum interval of manholes

Diameter (mm)	≤300	400-600	700-1000	1100-1500	≥1650
Interval (m)	50	75	100	150	200

Example 13.19

Determine an appropriate gutter section given the following conditions.

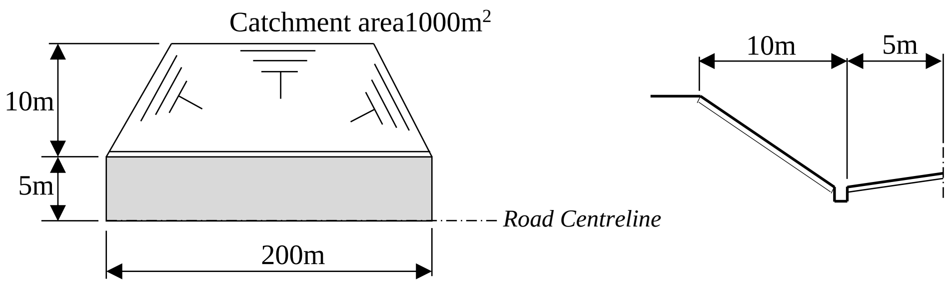
Gutter slope: 1/200

Slope area run-off coefficient: 0.6

Road area run-off coefficient: 0.85

Gutter surface (cement concrete) coefficient of roughness: 0.015

Rainfall intensity: 110mm/h



Step 1: Determine maximum run-off

From the condition given above, the area (a), weighted run-off coefficient (c), and peak discharge (Q) shall be given as follows:

$$a = (200 \times 5) + 1000 = 2000\ m^2$$

$$c = \frac{(0.6 \times 1000) + (0.95 \times 5 \times 200)}{2000} = 0.78$$

$$Q = \frac{1}{3.6 \times 10^6} \times 0.78 \times 110 \times 2000 = 0.048\ m^3/s$$

Step 2: Assume gutter section

Rectangle type with 30% clearance shall be applied.

First, assume the dimension of the gutter as b=0.3m, h=0.25m. Then, area (A), hydraulic mean depth (R), velocity of flow (V) and capacity (Q_c) shall be given as:

$$A = 0.3 \times 0.25 \times 70\% = 0.0525\ m^2$$

$$R = \frac{0.0525}{2 \times 0.25 \times 70\% + 0.3} = 0.081\ m$$

$$V = \frac{1}{0.015} \times 0.081^{2/3} \times 1/200^{1/2} = 0.882 \text{ m/s} \ \ (0.6\text{-}3.0 \text{ m/s}) \ \ \text{OK}$$

$$Q_c = A \times V = 0.0525 \times 0.882 = 0.046\ m^3/s < Q(0.048\ m^3/s) \quad Fail$$

Try with a bigger gutter size (b=0.3m, h=0.30) to satisfy the peak discharge (Q). Same procedure as above shall be followed.

$$A = 0.3 \times 0.3 \times 70\% = 0.063\ m^2$$

$$R = \frac{0.063}{2 \times 0.30 \times 70\% + 0.3} = 0.088m$$

$$V = \frac{1}{0.015} \times 0.088^{2/3} \times 1/200^{1/2} = 0.931 \text{ m/s} \ \ (0.6\text{-}3.0 \text{ m/s}) \ \ \text{OK}$$

$$Q_c = A \times V = 0.063 \times 0.931 = 0.058\ m^3/s > Q(0.048\ m^3/s) \quad OK$$

Example 13.20

Determine the interval (L) of batter drains given the following conditions

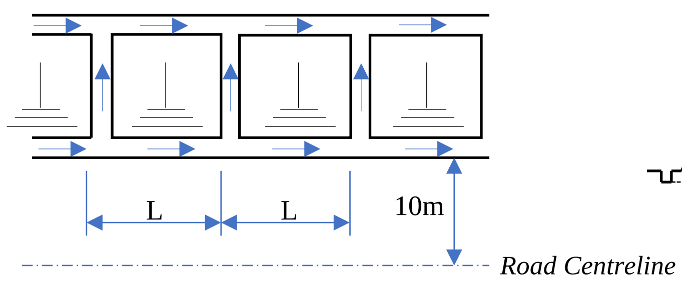
Gutter section: 0.25m ×0.25m

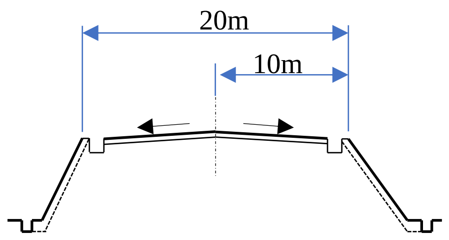
Gutter slope: 1/300

Road area run-off coefficient: 0.95

Gutter surface (cement concrete) coefficient of roughness: 0.015

Rainfall intensity: 85mm/h





Step 1: Determine maximum run-off

$$\text{Q} = \frac{1}{3.6 \times 10^6} \times 0.95 \times 85 \times (10 \times L) = 0.000224 \times L\ \ m^2/s$$

Step 2: Determine discharge capacity (Apply 30% clearance)

$$A = 0.25 \times (0.25 \times 70\%) = 0.044\ m^2$$

$$R = \frac{0.044}{2 \times (0.25 \times 70\%) + 0.25} = 0.073\ m$$

$$V = \frac{1}{0.015} \times 0.073^{2/3} \times 1/300^{1/2} = 0.671\ m/s \quad \text{(0.6-3.0m/s)} \quad \text{OK}$$

$$Qc = 0.044\ m^2 \times 0.671 \frac{m}{s} = 0.0295\ m^3/s$$

$$Q \le Qc\ 0.00224 \times L\ {m^3}/{s} \le 0.0295\ m^3/s$$

$\therefore L \le 131.6$ m   Interval of batter drain (chute) shall be 132m

Example 13.21

Determine an appropriate gutter section given the following conditions.

Gutter slope: 1/200

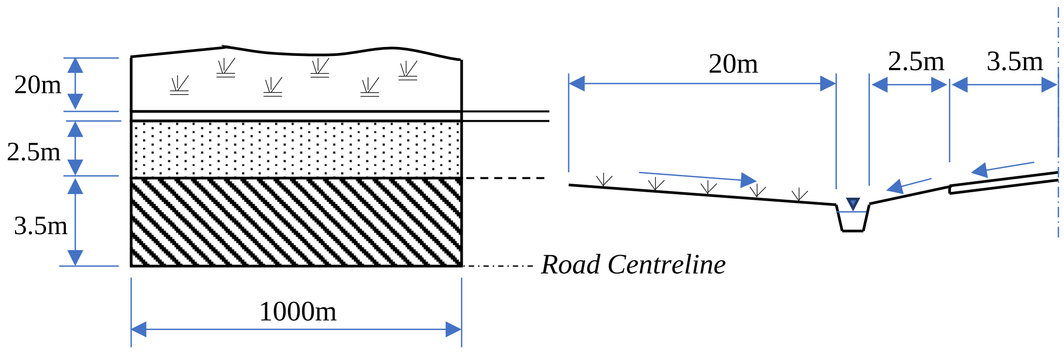
Road area run-off coefficient; 0.95

Shoulder area run-off coefficient: 0.70

Flat area run-off coefficient: 0.30

Gutter surface (hard clay) coefficient of roughness: 0.077

Rainfall intensity: 130mm/h



Step 1: Determine maximum run-off

$$a = 20 \times 1000 + (2.5 + 3.5) \times 1000 = 26{,}000\text{m}^2$$

$$C = \frac{(20 \times 1000 \times 0.3) + (2.5 \times 1000 \times 0.7) + (3.5 \times 1000 \times 0.95)}{26000} = 0.43$$

$$Q = \frac{1}{3.6 \times 10^6} \times 0.43 \times 130 \times 26000 = 0.407\ m^3/s$$

Step 2: Assume gutter section (Apply 20% clearance)

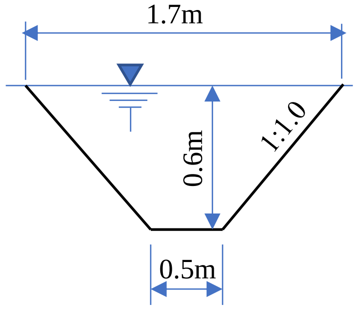
<Section-1>

$$A = \frac{1}{2} \times (0.6 \times 80\%) \times (1.7 + 0.5) = 0.528\text{m}^2$$

$$R = \frac{0.528}{0.5 + \sqrt{4 \times (0.6 \times 80\%)^2 + (1.7 - 0.5)^2}}$$
$$= 0.259m$$

$$V = \frac{1}{0.027} \times 0.259^{2/3} \times {}^{1}\!/_{200}{}^{1/2} = 1.067\ m/s$$

Out of range (0.6-1.0m/s) **Fail**



$$Qc = 0.528m^2 \times 1.067m/s = 0.563m^3/s$$
$$Qc\left(0.563\,{}^{m^3}\!/_{s}\right) > Q(0.407\,{}^{m^3}\!/_{s})$$ **OK**

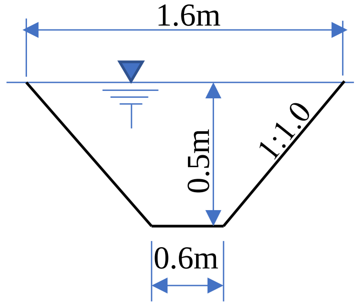
<Section-2>

$$A = \frac{1}{2} \times (0.5 \times 80\%) \times (1.6 + 0.5) = 0.44\text{m}^2$$

$$R = \frac{0.44}{0.6 + \sqrt{4 \times (0.8 \times 80\%)^2 + (1.6 - 0.6)^2}}$$
$$= 0.234m$$

$$V = \frac{1}{0.027} \times 0.234^{2/3} \times {}^{1}\!/_{200}{}^{1/2} = 0.999\ m/s$$

Within range of (0.6-1.0m/s) **OK**



$$Q_c = 0.44m^2 \times 0.999m/s = 0.440m^3/s$$
$$Q_c(0.440m^3/s) > Q(0.407m^3/s)$$

OK (Size is appropriate)

Example 13.22

Determine gutter section given the following condition (**Figure 13.36**):

Rainfall intensity: 100mm/h

Roadside (town) area run-off coefficient: 0.7

Road (Asphalt concrete) area run-off coefficient: 0.9

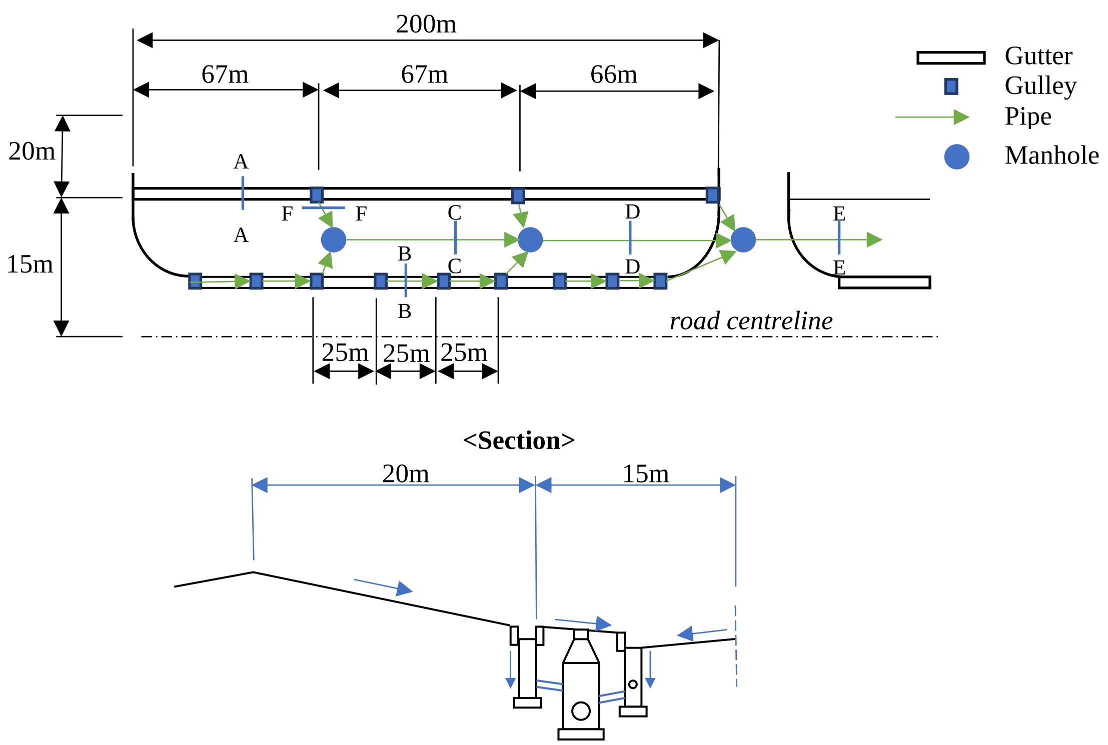


Figure 13.36 Example of surface water discharge

Step 1: Determine maximum run-off

Assuming 200m total length of catchment area and setting up 3 gulley in the gutter, length of each catchment area becomes 67m or 66m.

$$\mathrm{a} = 20\mathrm{m} \times 67\mathrm{m} = 1{,}240\mathrm{m}^2$$

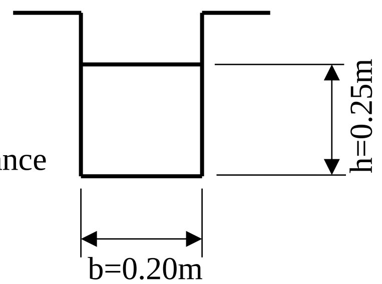
$$\mathrm{Q} = \frac{1}{3.6 \times 10^6} \times 0.7 \times 100\ {}^{\mathrm{mm}}\!/_{\mathrm{h}} \times 1{,}340\mathrm{m}^2 = 0.026 m^3/s$$

Step 2: Determine gutter section

Gutter surface: Precast concrete, Gutter slope: 1/200

Coefficient of roughness: 0.013

Gutter section: b=0.2m, h=0.25m, Apply 30% clearance



$$A = 0.2 \times (0.25 \times 70\%) = 0.035 m^2$$

$$R = \frac{0.035}{2 \times (0.25 \times 70\%) + 0.2} = 0.063 m$$

$$V = \frac{1}{0.013} \times 0.063^{2/3} \times {}^{1}\!/_{200}{}^{1/2} = 0.86\mathrm{m/s}$$

$$Q_c = 0.086 \times 0.035 = 0030 m^3/s$$

$$Q_c(0.030 m^3/s) > Q(0.026 m^3/s) \quad OK$$

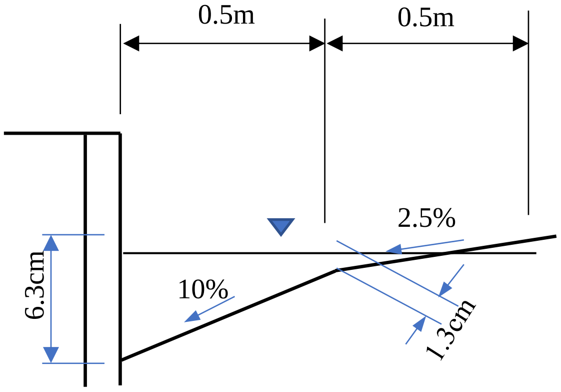
Step 4: Determine maximum gutter discharge capacity

Width of gulley discharge area (R): 1m

Catchment area width (W): 15m

Gulley type and drop area: D and 600 cm^2

Coefficient of roughness: 0.013



A=1/2×(0.013+0.063)×0.5+1/2×0.013×0.5=0.022m^2

P=0.5×(1^2+0.1^2)$^{1/2}$+0.5×(1^2+0.025^2)$^{1/2}$+0.063=1.065m

R=0.022/1.065=0.021m

$$Q = \frac{1}{0.013} \times 0.021^{2/3} \times \frac{1}{100^{1/2}} \times 0.022 = 0.013\, m^3/s$$

Step 5: Determine maximum run-off of unit length of road (q)

$$q = \frac{1}{3600} \times 0.9 \times 100 \times 15 = 0.000375 m^3/s\ /\mathrm{m}$$

Step 6: Determine drop ratio (r)

γ *:0.8, ρ :1/0

$\gamma = 0.8 \times 1.0 = 0.8$

Step 7: Determine gulley interval (L_s)

$$L_1 = 0.8 \times \frac{13}{0.375} = 27.7\mathrm{m}$$

Clearance ratio is 10%

$L_s = 27.7 \times (1 - 0.1) = 24.9m$

Gulley interval is 25m

Example 13.23

Calculate drainage pipe section (Figure 13.36)

i. Determine drainage pipe of section B-B
$a = 25\mathrm{m} \times 15\mathrm{m} = 375\mathrm{m}^2$

$$Q = \frac{1}{3.6 \times 10^6} \times 0.9 \times 100\, mm/h \times 375 m^2 = 0.009\, m^3/s$$

Diameter of pipe is 200mm. However, diameter of 300mm is selected for easy maintenance.

ii. Determine drainage pipe of section C-C
Maximum run-off of gulley is 0.009 m^3/s/point. (Road area)
Maximum run-off of gutter is 0.026 m^3/s/point (Roadside area)
(Road area) (Roadside area)
Q=(0.009m^3/s/point × 3point) + (0.026m^3/s/point × 1point)=0.053m^3/s
Diameter of pipe is 300mm.

iii. Determine drainage pipe of section D-D
(Road area) (Roadside area)
Q= (0.009m^3/s/point × 6 point) + (0.026m^3/s/point × 2 point) =0.106 m^3/s
Diameter of pipe is 400 mm.

iv. Determine drainage pipe of section E-E
(Road area) (Roadside area)
Q= (0.009m^3/s/point × 9 point) + (0.026m^3/s/point × 3 point) =0.159 m^3/s
Diameter of pipe is 450 mm.

v. Determine drainage pipe of section F-F
Q=0.026 m^3/s
Diameter of pipe is 200mm.
Pipe length is short, less than 10m, therefore maintenance is not difficult.

13.5.10 SCOUR CHECKS

Scour checks (sometimes called check dams) serve to reduce the speed of water in a drain, remove suspended material and thus help prevent erosion of the road structure. Scour checks must be provided in longitudinal drains with gradient steeper than 4%. They should not be provided on flatter slopes as they can give rise to excessive sedimentation in the drain. The top level of the scour check should be a minimum of 200 mm below the edge of carriageway. The distance between the scour checks depends on the road gradient and the erosion potential of the soils.

The distance between the scour checks depends on the road gradient and the erosion potential of the soils.

Table 13.45 provides a guide for scour check spacing. In very weak soils, or areas of very heavy rainfall, it may be necessary to increase this basic provision.

Table 13.45 Scour check spacing

Drain gradient, S (%)	Scour check spacing (m)
< 5	not required
5	15
7	10
10	7
12	5

The scour check acts as a small dam and, when naturally silted up on the upstream side, effectively reduces the gradient of the drain on that side, and therefore the velocity of the water. The energy of the water flowing over the dam is dissipated by allowing it to fall onto an apron of stones. Scour checks are usually constructed with natural stone, masonry, concrete or with wooden or bamboo stakes. By using natural building materials available along the road side, they can be constructed at low cost and be easily maintained.

After the basic scour check has been constructed, an apron should be built immediately downstream using stones. The apron will help resist the forces of the waterfall created by the scour check. Sods of grass should be placed against the upstream face of the scour check wall to prevent water seeping through it and to encourage silting to commence on the upstream side. The goal is to establish a complete grass covering over the silted scour checks to stabilise them.

An example of a scour check is shown **Plate 13.10**. Typical design details for scour checks are given in **Figure 13.37**.

Plate 13.10 Scour check made from wooden stakes

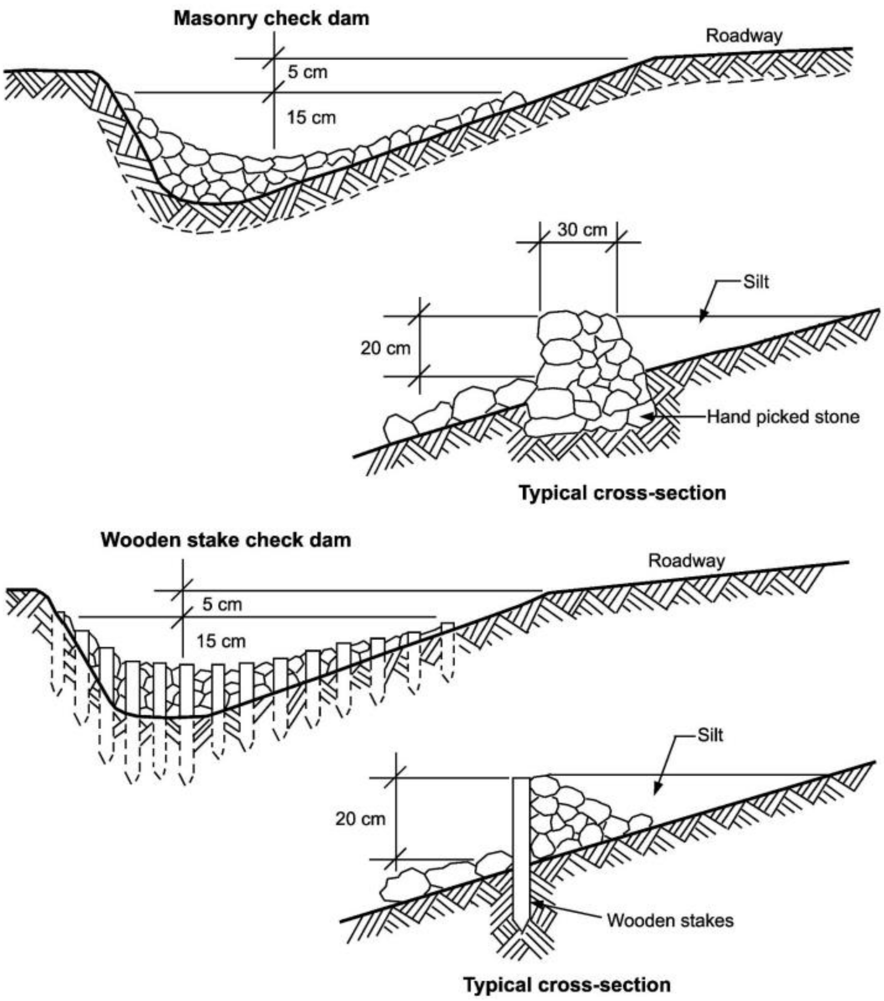


Figure 13.37 Typical design of scour checks

13.5.11 RIPRAP DESIGN

Once it has been determined that the natural or backfill material is unstable, using either tractive force procedure, a table of allowable velocities for specific soil types, or engineering judgment, select a trial riprap size and proceed as follows:

1. Determine the permissible shear stress (τ_p) for riprap size selected (Use **Table 13.46**). For larger stones not shown in the table, use **Equation (13.88)**.

$$\tau_p = 628.3D_{50} \tag{13.88}$$

Where,

D_{50} = median riprap/gravel size, m

Table 13.46 Permissible Shear Stresses for Lining Material

Lining Category	Lining Type (D_{50})	Permissible Unit Shear Stress (N/m^2)
Gravel Riprap	50 mm	31.6
Rock Riprap (Class AI)	150 mm	95.8
Rock Riprap (Class I)	300 mm	191.5

For non-cohesive soil (D_{75} (soil size where 75% of the material is finer, mm) particle diameter), use **Figure 13.38**.

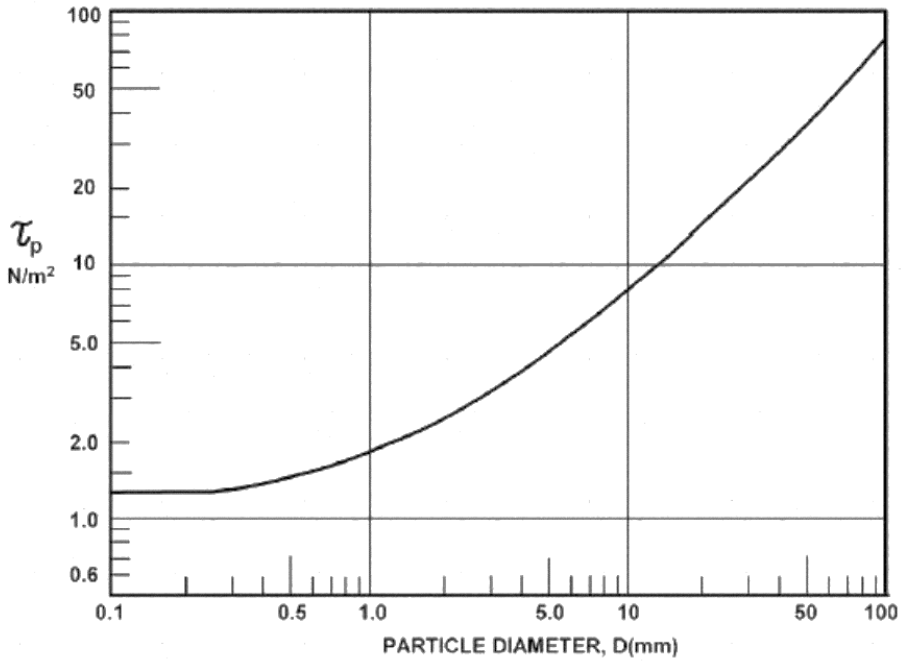


Figure 13.38 Permissible Shear Stress for Non-Cohesive Soils

2. Select a trial flow depth range for the ditch configuration using **Table 13.41**.
3. Using the n-value for the selected trial flow depth range, calculate the actual depth of flow for the ditch configuration using the actual design discharge and ditch slope using an appropriate method or software such as Flow Pro. If the calculated depth is within the trial depth range that was selected, proceed to the next step. If it is not, select another trial depth range and try again.
4. Calculate the actual shear stress (τ_o) using **Equation (13.89)**.

   $$\tau_o = \gamma d S_o \qquad (13.89)$$

   Where,

   γ = specific weight of water (9810 N/m^3)
   d = depth of flow, m
   S_o = average ditch slope, m/m

   If $\tau_o > \tau_p$, the selected riprap size is too small. Choose the next larger riprap size and try again. If the ditch side slopes are steeper than 3:1, it is necessary to perform the additional calculation shown below.
5. Determine the angle of repose for the riprap size determined above. It should be noted that all standard riprap sizes are assumed to have angle of repose of 42^o.
6. Determine the ratio of maximum side shear to maximum bottom shear (K_1) using **Equation (13.90)**.

   $$\begin{aligned} K_1 &= 0.77 && Z \leq 1.5 \\ K_1 &= 0.066Z + 0.67 && 1.5 < Z < 5 \qquad (13.90) \\ K_1 &= 1.0 && 5 \leq 5 \end{aligned}$$

   The Z value represents the horizontal dimension 1:Z (V:H). Use of side slopes steeper than 1:3 (V:H) is not encouraged for flexible linings other than riprap or gabions because of the potential for erosion of the side slopes. Steep side slopes are allowable within a channel if cohesive soil conditions exist. Channels with steep slopes should not be allowed if the channel is constructed in non-cohesive soils.

7. Determine tractive force ratio (K_2) from **Equation (13.91)**.

$$K_2 = \sqrt{\left(1 - \left(\frac{\sin\theta}{\sin\Phi}\right)^2\right)} \tag{13.91}$$

Where,

K_2 = tractive force ratio

Φ = angle of side slope

θ = angle of repose (use 42°)

8. Calculate required D_{50} for the side slope ($D_{50\ side\ slope}$) from **Equation (13.92)**.

$$D_{50\ side} = (K_1/K_2)D_{50\ bottom} \tag{13.92}$$

Where,

K_1 = ratio of maximum side shear to maximum bottom shear

$D_{50\ side}$ = median particle size on channel side slope

$D_{50\ bottom}$ = median particle size on channel bottom

From practical standpoint, whatever riprap size is indicated for the ditch side slope should be used on the bottom as well.

13.5.12 RIPRAP LINING DESIGN METHOD FOR MAJOR CHANNELS (HEC-11)

If it is determined that riprap lining protection is required for a major channel, the design procedure is based on U.S. FHWA publication, "Design of Riprap Revetment" (HEC-11). The design is based on the **Equation (13.93)**.

$$D_{50} = C\left(\frac{0.00594V_a^3}{K_1^{1.5}{d_{avg}}^{0.5}}\right) \tag{13.93}$$

Where,

D_{50} = median riprap particle size, m

C = stone size correction factor (**Equation (13.95)**)

V_a = average velocity in the main channel, m^3/s

d_{avg} = average flow depth in the main flow channel, m

In HEC-11, K_1 is given by the **Equation (13.94)**.

$$K_1 = \sqrt{\left(1 - \left(\frac{\sin\theta}{\sin\Phi}\right)^2\right)} \qquad 13.94)$$

where,

θ = side slope angle (measured from the horizontal), deg.

Φ = natural angle of repose of material under consideration (measured from the horizontal), deg (refer to **Figure 13.39**).

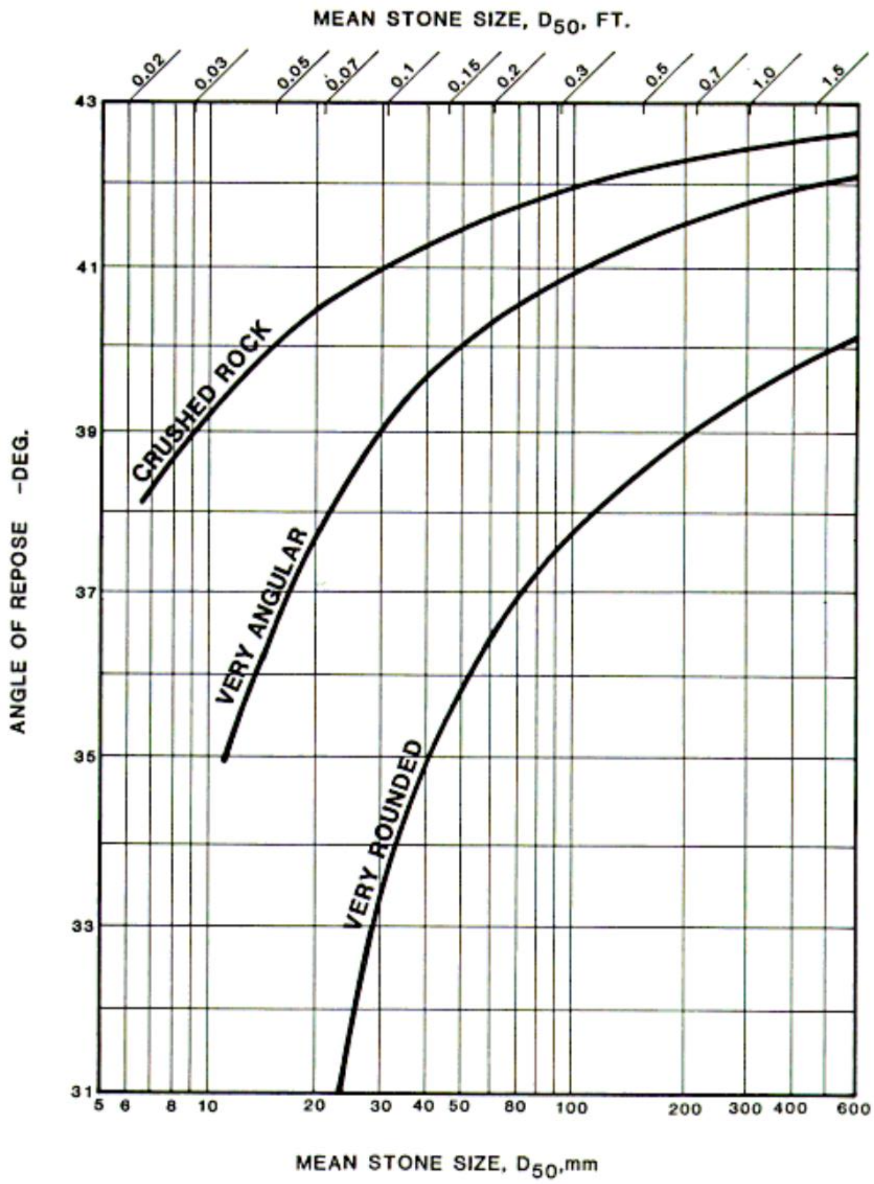


Figure 13.39 Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone

$$C = \frac{1.61 SF^{1.5}}{(S_s - 1)^{1.5}} \tag{13.95}$$

where,

SF = stability factor, see **Table *13.47***, (usually 1.2)

S_s = specific gravity of the rock riprap (assumed 2.65 for all standard riprap sizes.)

Note:

1. K_1 in HEC-11 is the same variable as K_2 in HEC-15.

Table 13.47 Stability Factor

CONDITION	STABILITY FACTOR RANGE
Uniform flow; straight or mildly curving reach [curve radius(R)/ channel width (W) >30]; impact from wave action and floating debris is minimal; little or no uncertainty in design parameters.	1.0 -1.2
Gradually varying flow; moderate bend curvature (30 > curve radius/channel width > 10); impact from waves or floating debris is moderate.	1.3 - 1.6
Approaching rapidly varying flow; sharp bend curvature (30 > curve radius/channel width >10); significant impact potential from floating debris; significant wind and/or bore generated waves (0.3-0.6 m); high flow turbulence; mixing flow at bridge abutments; significant uncertainty in design parameters.	1.6 - 2.0
Channel bends when ratio of curve radius to channel width (R/W) > 30.	1.2
Channel bends when 30 > R/W > 10.	1.3 -1.6
Channel bends when R/W < 10.	1.7

13.5.13 ROCK RIPRAP

Sizes of ripraps used by are shown in **Table 13.48**. **Figure 13.40** shows a typical Rock Riprap. Riprap may be constructed in several ways. Two types of riprap placement dumped rock riprap and hand-placed riprap are recommended. However, hand-placed riprap is more applicable around the outlet end of culvert to protect against the erosive action of the water.

Riprap should be placed on a 0.3 m thick filter material graded from sand to 150-mm gravel to protect the original bank material from scour or sloughing. The filter should be placed in layers from fine to coarse, out to the riprap. Filter fabric or geotextiles may also be used. Riprap should be placed 0.30 m above the design depth of the water.

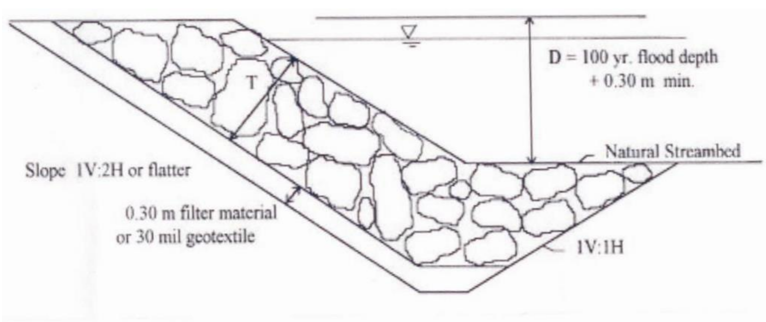


Figure 13.40 Typical Rock Riprap

Table 13.48 Standard Riprap Classification, Weights, and Blanket Thickness

Classification	D_{50} (mm)	Weight (kg)	Thickness, T (mm)
Class AI	245	23.0	500
Class I	335	45.0	660
Class II	490	136.0	965
Class III	670	453.0	1345

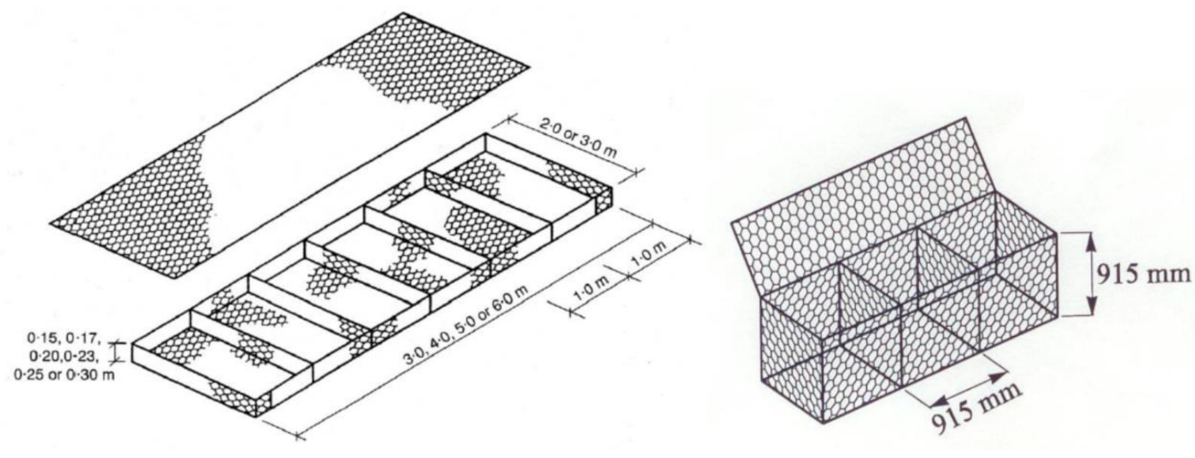
13.5.14 GABIONS

Gabions are heavy steel wire mesh containers, rectangular in shape. They vary in size from cubes to mattresses and are filled with clean stone larger than the mesh opening.

Gabions are generally used when a source of large riprap rock is unavailable, when velocities are extremely high, or when space is extremely limited due to steep narrow channel banks. Gabions can be constructed by hand and placed accordingly.

Although gabions are an attractive alternative to using riprap bank protection in situations mentioned above, they have their drawbacks. Gabions are very labour intensive and require a lot of maintenance if there is a structural breakdown in the steel mesh from abrasive forces of moving rocks, or from corrosive soils. They are also not aesthetically very pleasing. Gabion should be used only when it is impractical to use rock riprap.

Figure 13.41 shows standard gabion types and sizes.



(a) Gabion mattresses (b) Gabion baskets

Figure 13.41 Gabions

13.5.14.1.1 SAMPLE RIPRAP CALCULATIONS

1) It has been determined that a special ditch connected to culvert cross drain pipe will need a riprap lining. The ditch has a bottom width (B) of 0.6 m and 2:1 side slope (z=2), and a slope along the ditch line (S) of 0.004 m/m. The design discharge is 0.60 m3 /s. Determine what size of standard riprap will be required?

Solution:

Try Class A1 standard riprap, D_{50} = 0.25 m

τ_p = 153.22 pa (from **Equation (13.88)**)

Assume a depth range of from 0.15 – 0.60 m with an n-value of 0.069.

From **Table 13.37**, with S=0.005 m/m, B =0.6 m, z=2 and Q=0.6 m^3/s, computed d=0.468 m. (0.15<0.468<0.6),

Therefore, the assumed n-value of 0.069 is acceptable.

Maximum shear stress, τ_{max}

$\tau_{max} = \gamma d S_o$

$= 9810(0.468)(0.005)$

$= 22.96$ pa

$\tau_{max} < \tau_p$

Therefore, Class A1 riprap is acceptable for the ditch bottom and side slopes of 3:1 of flatter. However, since the side slope is 2:1, check the side slope stability.

Angle of repose for riprap, θ

$\theta = 42^o$

Determine K_1

B/d = 0.6/0.468 = 1.28

From **Equation (13.90)**, with z = 2, K_1 =0.9

Determine K_2

z = 2

Angle of side slope, Φ

$\tan \Phi = 1/z$

$= 0.5$

$\Phi = 29.52^o$

$K_2 = (1 - (\sin\theta / \sin\Phi)^2)^{0.5}$

$= (1 - (\sin 29.52/\sin 42)^2)^{0.5}$

$= 0.683$

D_{50} for side slopes

D_{50} side = $(K_1/K_2)D_{50\ bottom}$

= (0.9/0.683)0.25 m

= **0.329** m

Therefore, it would probably be best to use Class I riprap (D_{50} = 0.335 m) for both the channel bottom and side slopes in lieu of the originally proposed Class AI (D_{50} = 0.25 m).

2) A natural channel has bed and banks of native materials composed of cobbles and pebbles. Mean diameter is 30 mm for the D_{75} size stone. The channel bottom width (B) is 3.0 m. The longitudinal slope is 0.008 m/m. Its side slope is 2:1 (z=2).
The flow (Q) is 4.2 m^3/s at a depth (d) of 0.6 m. Determine whether the channel is stable for the indicated condition.

Solution:

Permissible shear stress, (τ_p)

From **Figure 13.38**, for a D_{75} particle diameter of 30 mm, read a permissible tractive force (τ_p) on the channel bottom of 22.14 pa.

Side slope angle, θ

$\tan\theta = 1/z = 0.5$

$\sin\theta = 0.447$

Angle of repose, Φ

From **Figure 13.39**, for a particle diameter of 30mm, read an angle of repose (Φ) of 40°.

The sine of 40°= 0.588

$K_1 = (1 - (\sin\theta / \sin\Phi)^2)^{0.5}$

$= (1-(0.447/0.588)^2)^{0.5}$

$= 0.65$

Permissible tractive force on the side slopes (τ_s)

$\tau_s = K_1 \times \tau_p$

$= 0.65(22.14)$

$= 14.391\ N/m^3$

Flow cross-sectional area

$A = Bd + zd^2$

$= 3(0.6) + 2(0.6)^2$

= 2.52 m2

Wetted perimeter

$P = B + 2d(1 + z^2)^{0.5}$

$= 3 + 2(0.6)(1 + 22)^{0.5}$

= 5.683 m

Hydraulic radius

R = A/P

= 2.52/5.683 m

= 0.443 m

Average tractive force

τ_o = 9810RS_o

= 9810(0.443)(0.008)

= 34.77 N/m^3

Compare the average tractive force of the channel to the allowable tractive force of the channel sides and bottom τ_o of 34.77 N/m^3> $\tau_{p\ (bottom)}$ of 22.14 N/m^3 or τ_s (side slope) of 14.391N/m^3.Native material is unsuitable. Channel protection is required.

3) A major channel at Dwowulu area in Accra has a serious erosion problem. DUR has determined that the most cost-effective measure to mitigate the problem is to riprap the channel. The average velocity in the channel is 1.65 m/s. The average depth of flow is 0.60 m. The side slope ratios are 2:1 (29.52^o). Angle of repose for the riprap is 42^o and the specific gravity of the riprap is 2.65. What size of standard riprap is required?

Solution:

From **Table 13.47**, select a stability factor (SF) of 1.2

C = 1.61$SF^{1.5}$/(S_s-1)$^{1.5}$

= 1.61(1.2)$^{1.5}$/(2.65-1)$^{1.5}$

= 0.99855

K_1 = (1- (sinθ/sinΦ)^2)$^{0.5}$

= (1- $\sin^2$(29.52)/$\sin^2$42^o)$^{0.5}$

= 0.684

D_{50} = C(0.00594Va^3/($K_1$1.5davg$^{0.5}$)

= 0.99855(0.00594 (1.65)^3/0.684$^{1.5}$ (0.6$^{0.5}$))

= 0.061 m

Closest standard riprap size = Class AI (D_{50} = 0.245 m)

13.6 HYDRAULIC DESIGN OF CULVERTS

13.6.1 INTRODUCTION

A culvert is a conduit that conveys flow through a roadway embankment or past some other type of flow obstruction. In addition to this hydraulic function, it must also carry construction and highway traffic and earth loads; therefore, culvert design involves both hydraulic and structural design. The hydraulic and structural designs must be such that risks to traffic, of property damage, and of failure from floods are consistent with good engineering practice and economics. This section is concerned with the hydraulic aspects of culvert design and makes reference to structural aspects only as they are related to the hydraulic design.

Flow conditions in a culvert may occur as open-channel flow, gravity full flow or pressure flow, or in some combination of these conditions. A complete theoretical analysis of the hydraulics of culvert flow is time-consuming and difficult. Flow conditions depend on a complex interaction of a variety of factors created by upstream and downstream conditions, barrel characteristics and inlet geometry. For purposes of design, standard procedures and nomographs have been developed to simplify the analysis of culvert flow.

Culverts, as distinguished from bridges, are usually covered with embankment and are composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert. Bridges are not designed to take advantage of submergence to increase hydraulic capacity even though some are designed to be inundated under flood conditions. For economy and hydraulic efficiency, culverts should be designed to operate with the inlet submerged during flood flows, if conditions permit. At many locations, either a bridge or a culvert will fulfill both the structural and hydraulic requirements for the stream crossing. Structure choice at these locations should be based on construction and maintenance costs, risk of failure, risk of property damage, traffic safety, and environmental and aesthetic considerations. Some of the advantages of culverts are better traffic safety and lower maintenance costs than bridges. Culverts do not have bridge railing, which can be a hazard, or a bridge deck, which is subject to deterioration.

Numerous cross-sectional shapes are available. The most commonly used shapes, depicted in **Figure 13.42**, include circular, box (rectangular), elliptical, pipe-arch, arch, and U. The shape selection is based on the cost of construction, the limitation on upstream water surface elevation, roadway embankment height, and hydraulic performance.

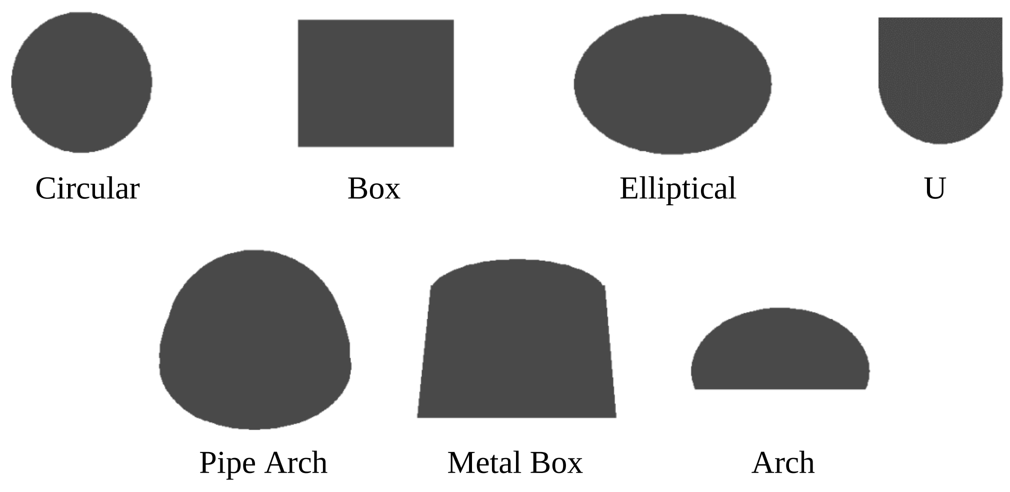


Figure 13.42 Commonly used culvert shapes

The selection of a culvert material may depend upon structural strength, hydraulic roughness, durability, and corrosion and abrasion resistance. The three most common culvert materials are concrete (reinforced and nonreinforced), corrugated aluminium, and corrugated steel. Culverts may also be lined with other materials to inhibit corrosion and abrasion, or to reduce hydraulic resistance. For example, corrugated metal culverts may be lined with asphaltic concrete.

In designing culverts, a number of issues must be considered including:

i. Economy (e.g., cost of the project and available budget)
ii. Road immunity – the extent to which flows are passed through culverts under the road rather than allowed to overflow the road
iii. Stream characteristics such as channel shape, slope and natural stream flow (also consider possible stream morphology over the life of the culvert)
iv. The configuration of culverts including size and number; alternative culvert types and materials
v. Afflux (that is the increase in water level caused by the road and its culvert)
vi. Tailwater and presence of possible backwater effects
vii. The culvert's outlet velocity and flow energy
viii. Groundwater effects (including longitudinal seepage, piping and erosion – considering the need for cut-off collars)
ix. Groundwater conditions (including pipe strength, culvert settlement and culvert anchorage when located on steep slopes)
x. The special needs of culverts which are to be used as fishways, for the passage of terrestrial fauna, or as cattle creeps (larger culverts will often be required for fauna or fish passage than for hydraulic reasons)
xi. Safety and access (that is catering for the needs of pedestrians, cyclists, or maintenance crews)
xii. Durability – design life of the structure
xiii. Environmental issues (minimising the potential for unacceptable environmental damage).

13.6.2 INFORMATION REQUIRED

It is important to gather all required information before design commences. Some information should be gathered/verified by site inspection. Sources of data include aerial or field survey; interviews; water resource, fish and wildlife, and planning agencies; newspapers; and floodplain zoning studies. Complete and accurate survey information is necessary to design a culvert to best serve the requirements of a site. The individual in charge of the drainage survey should have a general knowledge of drainage design and coordinate the data collection with the hydraulics engineer. The amount of survey data gathered should be commensurate with the importance and cost of the proposed structure.

13.6.2.1 TOPOGRAPHIC FEATURES

The survey should provide the designer with sufficient data for locating the culvert and may aid in determining the hydraulic design controls. All significant physical features and culture in the vicinity of the culvert site should be located by the survey, and especially those features that could be affected by the installation or operation of the culvert. Such features as residences, commercial buildings, croplands, roadways, and utilities can influence a culvert design; therefore, their elevation and location should be obtained.

The extent of survey coverage required for culvert design is related to topography and stream slope. In streams with relatively flat slopes, the effects of structures may be reflected a considerable distance upstream and require extensive surveys to locate features that may be affected by the culvert installation.

13.6.2.2 DRAINAGE AREA

Drainage area is an important factor in estimating the flood potential; therefore, the area of the watershed should be carefully defined by means of survey, photogrammetric maps, Geological survey topographic maps or a combination of these.

In locations where accurate definition of drainage areas from maps is difficult, the map information should be supplemented by survey. Non-contributing areas may need to be defined. The survey should note land usage, type and density of vegetation, and any constructed changes or developments (e.g., dams) which could significantly alter runoff characteristics.

13.6.2.3 CHANNEL CHARACTERISTICS

The physical characteristics of the existing stream channel should be described by the survey. For purposes of documentation and design analysis, sufficient channel cross sections, a streambed profile and the horizontal alignment should be obtained to provide an accurate representation of the channel, including the floodplain area. The channel profile should extend beyond the proposed culvert location far enough to define the slope and locate any large streambed irregularities (e.g., headcutting).

General characteristics helpful in making design decisions should be noted. These include the type of soil or rock in the streambed, the bank conditions, type and extent of vegetal cover, amount of drift and debris, and any other factors that could affect the sizing of the culvert and the durability of culvert materials. Photographs of the channel and the adjoining area can be a valuable aid to the designer and serve as excellent documentation of existing conditions.

13.6.2.4 FISH LIFE

Survey data should include information regarding the value of the stream to fish life and the type of fish found in the stream. The necessity to protect fish life and to provide for fish passage can affect many decisions regarding culvert, channel change and riprap designs and construction requirements for protection of the stream environment.

13.6.2.5 HIGHWATER INFORMATION

Reliable, documented highwater data, when available, can be a valuable design aid. Often, the designer must rely upon highwater marks as the only basis on which to document past floods. Highwater marks can also be used to check results of flood-estimating procedures, establish highway grade lines and locate hydraulic controls, but considerable experience is necessary to properly evaluate highwater information.

Data related to highwater should be taken in the vicinity of the proposed structure, but it is sometimes necessary to use highwater marks from upstream or downstream points. The location of the highwater mark with respect to the proposed structure should be recorded. Highwater elevations should be referenced to the project data.

If highwater information is obtained from residents, the individuals should be identified, and the length of residency indicated. Other sources for data include commercial drivers, law enforcement officers, road maintenance personnel, or other persons who have frequently travelled through the area over a long period of time.

Unusual highwater elevations should be examined to ascertain whether irregularities existed during the flood, such as blockage of the channel from drift, or backwater from stream confluences.

13.6.2.6 EXISTING STRUCTURES

Considerable importance should be placed on the hydraulic performance of existing structures, and all information available should be gathered in the survey. The performance of structures some distance either upstream or downstream from the culvert site can be helpful in the design. Local residents, road maintenance personnel, or others can furnish important highwater data and dates of flood occurrences at such structures.

Data at existing structures should include the following, if available:

i. date of construction
ii. major flood events since construction and dates of occurrence
iii. performance during past floods
iv. scour indicated near the structure
v. type of material in streambeds and banks
vi. alignment and general description of structure, including condition of structure, especially noting abrasion, corrosion, or deterioration
vii. alignment and general description of structure, including dimensions, shape, and material and flowline invert elevations
viii. highwater elevations with data and dates of occurrence
ix. location and description of overflow areas
x. photographs
xi. silt and drift accumulation
xii. evidence of headcutting in stream
xiii. appurtenant structures (e.g., energy dissipators, debris control structures, stream grade control devices)
xiv. as-built plan of structure.

Culverts that are to be retained in a project need to be checked for both hydraulic capacity and structural durability. Construction staging and final design cover should also be reviewed.

13.6.2.7 FIELD REVIEW

The engineer designing drainage structures should be thoroughly familiar with the watershed site under consideration. Much can be learned from the survey notes, but the most complete survey cannot adequately depict all watershed site considerations or substitute for a personal inspection by the designer. Often, a plans-in-hand inspection by the designer and the construction engineer will prove mutually beneficial by improving the drainage design and reducing construction problems.

13.6.3 CULVERT LOCATION

Culvert location deals with the horizontal and vertical alignment of the culvert with respect to both the stream and the highway. It is important to the hydraulic performance of the culvert, to stream stability, to construction and maintenance costs, and to the safety and integrity of the highway.

The horizontal and vertical alignment are important in maintaining a sediment-free culvert. Deposition occurs in culverts because the sediment transport capacity of flow within the culvert is often less than in the stream. The following factors contribute to deposition in culverts:

i. at moderate flow rates, the culvert cross section is larger than that of the stream, thus the flow depth and sediment transport capacity is reduced;
ii. point bars form on the inside of stream bends, and culvert inlets placed at bends in the stream will be subjected to deposition in the same manner. This effect is most pronounced in multiple-barrel culverts with the barrel on the inside of the curve often

becoming almost totally plugged with sediment deposits; and

iii. abrupt changes to a flatter grade in the culvert or in the channel adjacent to the culvert will induce deposition. Gravel and cobble deposits are common downstream from the break in grade because of the reduced transport capacity in the flatter section.

Deposition usually occurs at flow rates smaller than the design flow rate. The deposits may be removed during larger floods, dependent upon the relative transport capacity of flow in the stream and in the culvert, compaction and composition of the deposits, flow duration, ponding depth above the culvert, and other factors.

13.6.3.1 PLAN

Plan location deals basically with the route the flow will take in crossing the right-of-way. Regardless of the degree of sinuosity of the natural channel within the right-of-way, a crossing is generally accomplished by using a straight culvert either normal to or skewed with the roadway centreline.

Ideally, a culvert should be placed in the natural channel (see **Figure 13.43**). This location usually provides good alignment of the natural flow with the culvert entrance and outlet, and little structural excavation and channel work are required.

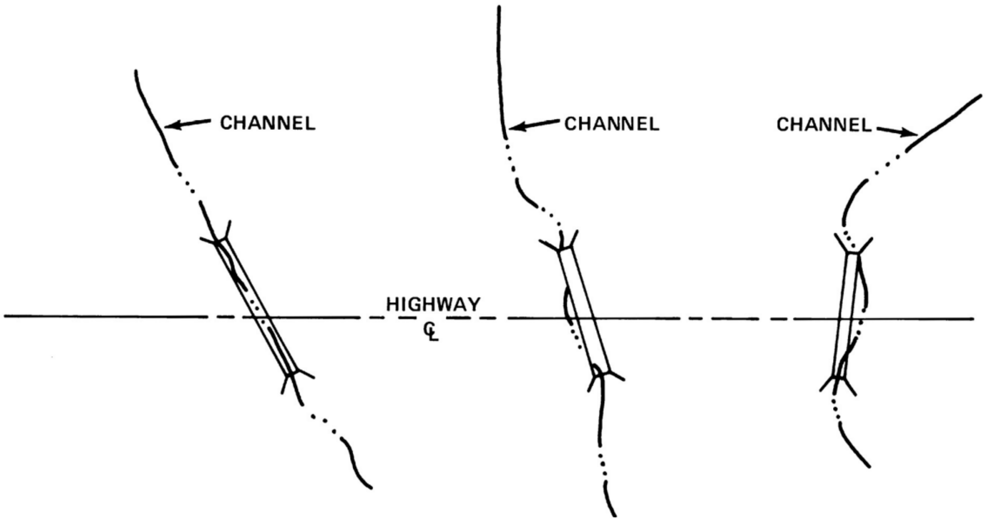


Figure 13.43 Culvert Located in Natural Channel

Where location in the natural channel would require an inordinately long culvert, some stream modification may be in order (see **Figure 13.44**). Such modifications to reduce skew and shorten culverts should be carefully designed to avoid erosion and siltation problems.

Culvert locations normal to the roadway centreline are not recommended where severe or abrupt changes in channel alignment are required upstream or downstream of the culvert. Short radius bends are subject to erosion on the concave bank and deposition on the inside of the bend. Such changes upstream of the culvert result in poor alignment of the approach flow to the culvert, subject the road fill to erosion and increase the probability of deposition in the culvert barrel. Abrupt changes in channel alignment downstream of culverts may cause erosion on adjacent properties.

In flat terrain, drainage is often provided by excavated channels. Highway planning should be coordinated with the drainage authority where drainage improvements are planned. Where planned channels are not at the location of natural drainage swales, concurrent channel and road construction is desirable. If concurrent construction is not possible, it will be necessary to provide road culverts for the existing drainage pattern. The drainage authority may contribute toward modifications to accommodate future channel construction, revise drainage plans to conform with road culvert locations, or make the necessary changes in road drainage at the time of channel construction.

Culvert construction in live stream environments frequently necessitates the installation of temporary diversion channels to carry the stream around the work site. The temporary diversion channels need protective linings to prevent erosion. At times, it may also be necessary to develop a staged construction sequence that will permit a portion of the work to be done; stream flow is then diverted through the completed portion of the culvert while the remainder of the culvert installation is constructed.

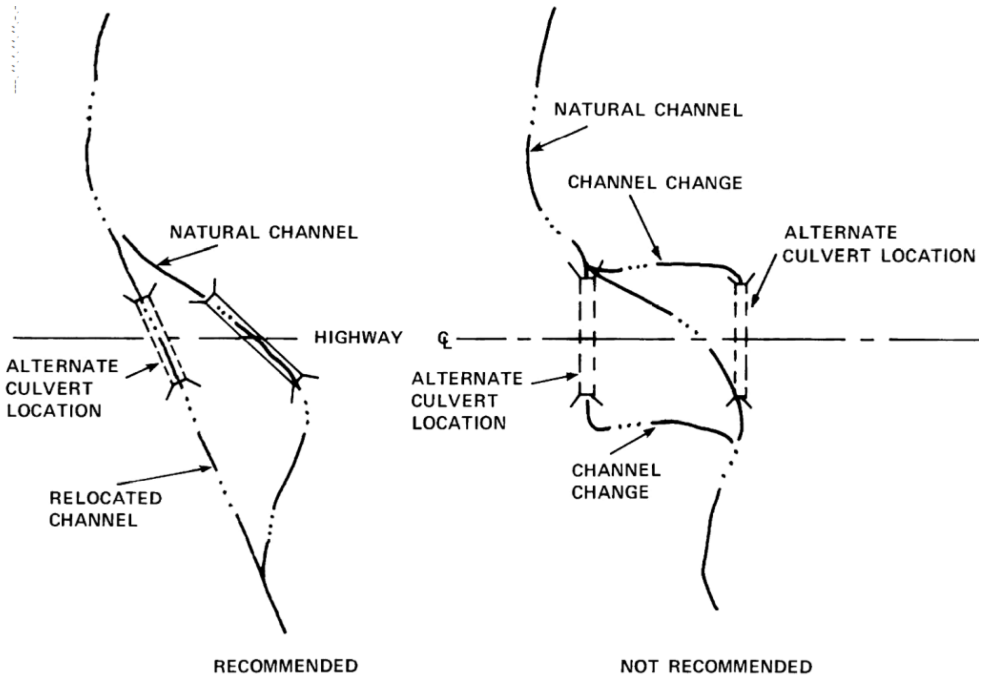


Figure 13.44 Methods of Culvert Location Where Location in the Natural Channel Would Involve an Inordinately Long Culvert

13.6.3.2 PROFILE

Most culvert locations approximate the natural streambed, though other locations may be chosen for economy in the total cost to construct and maintain. Modified culvert slopes, or slopes other than that of the natural stream, can be used to arrest stream degradation, induce sedimentation, improve the hydraulic performance of the culvert, shorten the culvert or reduce structural requirements. Modified slopes can also cause stream erosion and deposition; therefore, slope alterations should be given special attention to ensure that detrimental effects do not result from the change.

Channel changes often are shorter and steeper than the natural channel. A modified culvert slope can be used to achieve a flatter gradient in the channel so that degradation will not occur.

Figure 13.45 illustrates some possible culvert profiles.

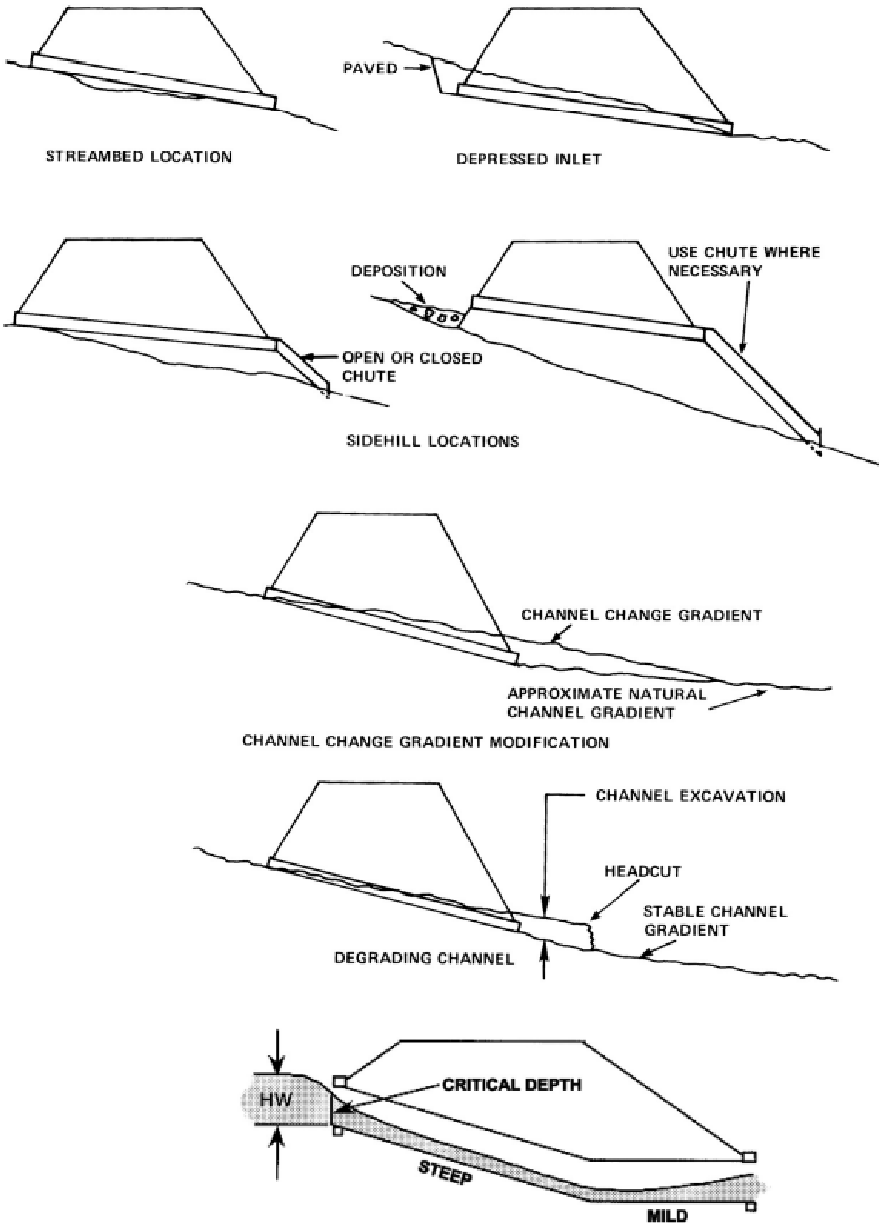


Figure 13.45 Possible Culvert Profiles

13.6.4 CULVERT TYPE

Selection of culvert type includes the choice of material, shape, and cross section, and the number of culvert barrels. Total culvert cost can vary considerably depending upon the culvert type selected. Fill height, terrain, foundation condition, fish passage, shape of the existing channel, roadway profile, allowable headwater, stream stage-discharge and frequency-discharge relationships, cost and service life are some of the factors that influence culvert-type selection.

The most common culvert type is the pipe because it is hydraulically and structurally efficient in most cases, and is cheaper to produce than a comparable box shaped culvert.

A culvert of rectangular cross section can be designed to pass large floods and to fit nearly any site condition. They are suitable for locations with minimal embankment depth (boxes require less material cover). A rectangular culvert lends itself more readily than other shapes to low allowable headwater situations, because the height may be decreased and the total span increased to satisfy the location requirement. The required total span can consist of one or multiple cells. Modified box shapes in the form of hexagons or octagons have been used and proved economical under certain construction situations. The longer construction time required for cast-in-place boxes can be an important consideration in the selection of this type of culvert. Precast concrete and metal box sections have been used to overcome this disadvantage.

13.6.5 CULVERTS IN FLAT TERRAIN

In flat terrain, drainage channels are often ill-defined or non-existent and culverts should be located and designed for least disruption of the existing flow conditions. In these locations multiple culverts can be considered to have a common headwater elevation, although this will not be precise.

13.6.6 MULTIPLE BARRELS

Culverts consisting of more than one barrel are useful in wide channels where the constriction or concentration of flow is to be kept to a minimum. Low roadway embankments offering limited cover may require the use of a series of small openings. The barrels may be separated by a considerable distance to maintain flood flow distribution. The practice of altering channel geometry to accommodate a wide culvert will generally result in deposition in the widened channel and in the culvert. Where overbank flood flow occurs, relief culverts with inverts at the floodplain elevation should be used to avoid the need for channel alteration.

In the case of box culverts, it is usually more economical to use a multiple structure than a wide single span. In some locations, multiple barrels have a tendency to catch debris, which clogs the waterway. They are also susceptible to deposition of silt in one or more barrels. Alignment of the culvert face normal to the approach flow and installation of debris control structures can help to alleviate these problems. To avoid widening of the natural channel, provide overflow (flood) relief, support environmental preservation, and reduce sedimentation and debris problems, it is good practice to install one barrel of the multiple-barrel culverts at the flow line of the stream, while the other barrels are set at a slightly higher invert elevation.

13.6.7 SILTATION/BLOCKAGE

The likelihood of blockage should be considered for all culverts. Blockage can occur through siltation or vegetation, although blockage by siltation is more likely to be temporary in nature. This is because during flood events, silt deposits can be removed by high velocity flows. To prevent siltation the desirable minimum velocity in the culvert should be above 0.6 m/s. A check of velocities should be undertaken as part of design process.

Where debris blockage is considered likely, larger culvert sizes may be required, in accordance with the extent of adverse impacts that could occur to the roadway or to surrounding properties. Blockage by debris is more likely to occur where the catchment contains significant woody riparian vegetation. In this case detailed assessment of the catchment is required.

13.6.8 MINIMUM CULVERT SIZE ALLOWABLE

All culverts shall be designed hydraulically except where difficult geometry dictates otherwise. The minimum diameter of pipe culverts should be 1.2m (Design Class A and B, and multilane Design Class C, D and E roads) and 0.9m (single carriageway Design Class C, D & E roads) in order to reduce the likelihood of blockage and enhance easy inspection and maintenance. The recommended minimum height of any box culvert is 1.5m.

13.6.9 OTHER SIZING CONSIDERATIONS

Considerations for the selection of culvert size are:

i. A culvert flowing full makes the most use of the waterway opening available. The culvert opening height should desirably match the tailwater depth; however, it is recommended that the height not be greater than 1.25 times the depth of the tailwater. Typically, this results in an outlet control culvert which gives a near full flow with lower outlet velocities. Culverts will generally not flow full if operating under inlet control.
ii. The larger the opening, the less prone to blocking of the waterway by debris. The minimum culvert size required for debris passage should be assessed including the likely debris (type and size), the risk of blockage and likely impacts of subsequent failure of the culvert.

13.6.10 COVER OVER CULVERT

The desirable minimum cover to be adopted for design purposes for box culverts is 0.1m below the subgrade height, to allow for continuous usage of paving machinery. However, if necessary, the absolute minimum cover can be 0.1 metres to finished surface height, that is, the culvert can be laid in the pavement layers. Advice should be sought from experienced structural and pavement engineers before adopting the absolute minimum condition to ensure appropriate structural integrity is provided and that cracking induced at the culvert joint does not reflect to the pavement surface.

For box culverts the maximum economic height of fill or depth of trench is about 1.5m. Boxes or crown units required to carry greater depths may be specially designed by an accredited structural designer.

Minimum cover for all pipes shall be 0.60m. For pipes under railroads, the minimum cover shall be 1.2m.

13.6.11 TYPES OF CULVERT INLETS AND OUTLETS

A culvert typically represents a significant contraction of flow over conditions in the upstream and downstream channels, and often is a hydraulic control point in the channel. The provision of a more gradual flow transition at the inlet of a culvert can improve the discharge capacity of the culvert by reducing the energy losses associated with flow contraction. Culvert inlets are available in a variety of configurations and may be prefabricated or constructed in place. Commonly used inlet configurations include projecting culvert barrels, cast-in-place concrete headwalls, precast or prefabricated end sections, and culvert ends mitered to conform to the fill slope (**Plate 13.11**). Structural stability, aesthetics, erosion control, fill retention, economics, safety, and hydraulic performance are considerations in the selection of an inlet.

	
Projecting Barrel	Cast-in-place headwall/wingwalls
	
Standard end section	Mitered to the slope

Plate 13.11 Four standard inlet types

Hydraulic performance is improved by use of bevelled edges rather than square edges, as illustrated in **Figure 13.46**.

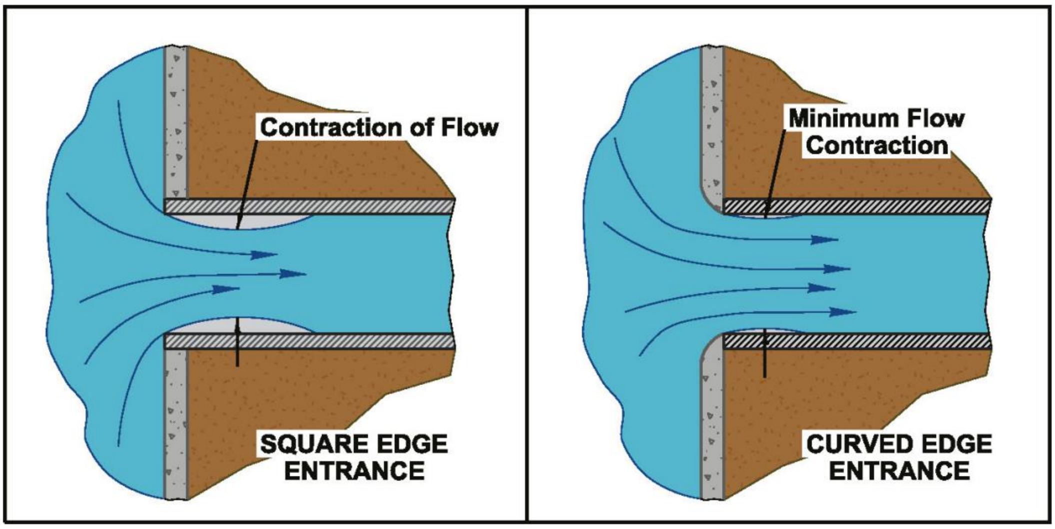


Figure 13.46 Entrance contraction

Side-tapered and slope-tapered inlets, commonly referred to as improved inlets, can significantly increase culvert capacity. **Figure 13.47** illustrates side-tapered and slope-tapered inlet conditions.

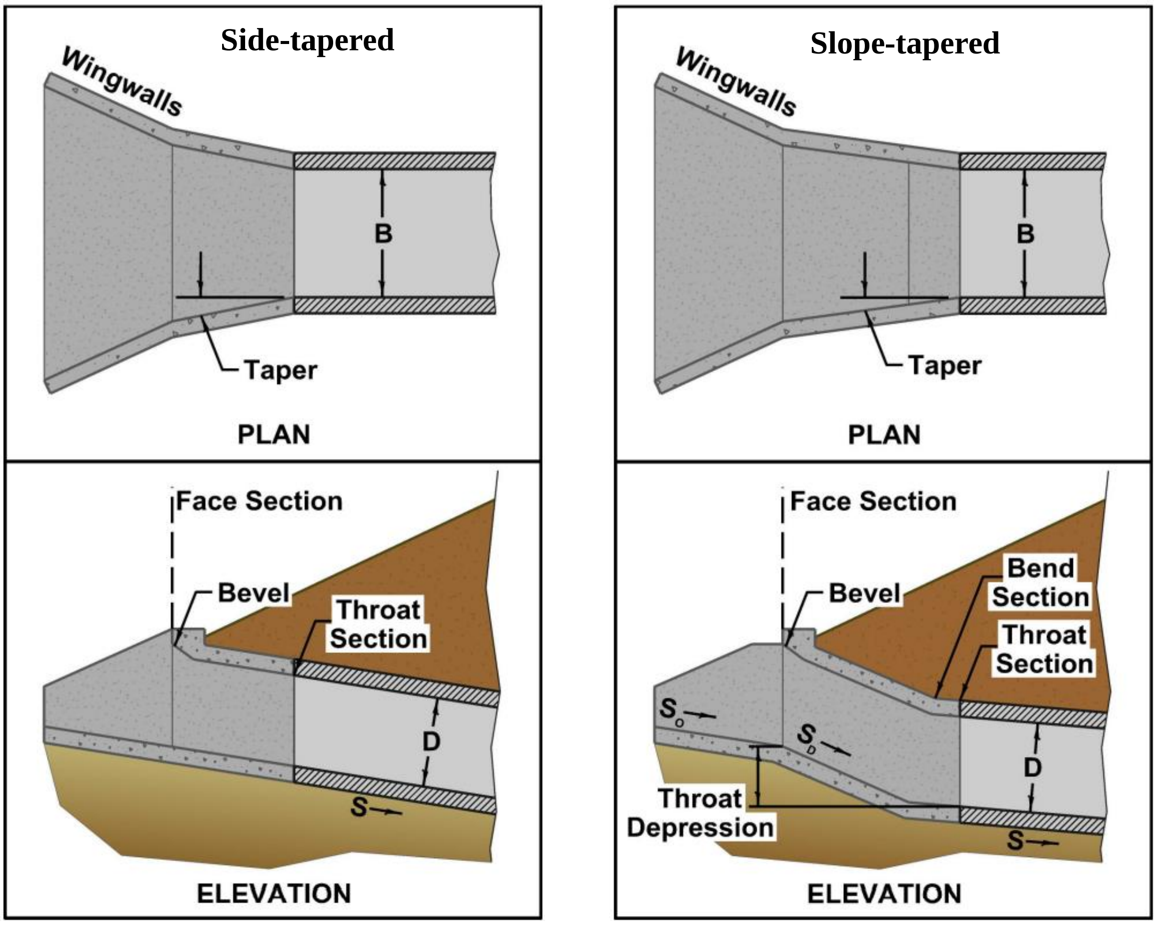


Figure 13.47 Side- and slope-tapered inlets

A side-tapered inlet provides a more gradual contraction of flow and reduces energy losses. A slope-tapered inlet, or depressed inlet, increases the effective head on the control section and improves culvert efficiency. Culvert outlet configuration can be similar to any of the typical inlet configurations; however, hydraulic performance of a culvert is influenced more by tailwater conditions in the downstream channel than by the type of outlet. Outlet design is important for transitioning flow back into the natural channel, since outlet velocities are typically high and can cause scour of the downstream streambed and bank.

13.6.12 HYDRAULIC DESIGN CONSIDERATIONS

The hydraulic design of a culvert consists of an analysis of the performance of the culvert in conveying flow from one side of the roadway to the other. To meet this conveyance function adequately, the design must include consideration of the variables discussed in the following sections.

13.6.12.1 DESIGN FLOOD DISCHARGE

The design discharge for a culvert is the discharge the culvert is designed to pass based on an accepted ARI (Refer to Section **13.2**). Typically, this discharge equals the catchment run–off as calculated by the methods set out in Section **13.3**.

Recognizing that floods cannot be estimated precisely and that it is seldom economically feasible to design for the very rare flood, all designs should be reviewed using a larger review (check) flood for the extent of probable damage should the design flood be exceeded. The performance curve should include this larger review flood.

13.6.12.2 HEADWATER ELEVATION

Culverts generally constrict the natural stream flow, which causes a rise in the upstream water surface. The elevation of this water surface is termed headwater elevation, and the total flow depth in the stream measured from the culvert inlet invert is termed headwater depth.

Two types of headwater are considered in culvert design:

i. Design headwater or Allowable headwater (AHW)
ii. Review headwater (RHW)

A. **Allowable headwater,** (Design headwater) depth is determined by the maximum permissible elevation of the headwater (HW) pool at the culvert inlet for the design discharge. It is based on the following considerations:
   i. Damage to adjacent property. Potential damage to adjacent property or inconvenience to owners should be of primary concern in the design of all culverts. In urban areas, the potential for damage to adjacent property is greater because of the number and value of properties that can be affected.
   ii. Damage to the culvert and the roadway.
   iii. Traffic interruption. If roadway embankments are low, flooding of the roadway and delay to traffic are usually of primary concern, especially on highly travelled routes.
   iv. Hazard to human life and safety.
   v. Damage to stream and floodplain environment.
   vi. Not higher than 0.5m below the edge of the shoulder.
   vii. Not higher than the low point in the road grade; and/or equal to the elevation where flow diverts around the culvert.
   viii. An HW/D not greater than 1.5. If deep ponding is considered, the possibility of catastrophic failure should be investigated because a breach in the highway fill could be quite similar to a dam failure. When headwater depths will exceed, say 6 to 8 m for the estimated design flood, the roadway embankment will function as a dam, and an appropriate investigation should be made to evaluate the risk in case of the occurrence of a larger flood or blockage of the culvert by debris. In some instances, design of the highway fill as a dam and use of emergency facilities (e.g., spillways, relief culverts) should be considered as alternative designs to the construction of larger structures or changes in the roadway profile.
   ix. In the event of conditions permitting a high headwater, the associated outlet velocity may be intolerably high. In this case,
      a. the allowable headwater may have to be reduced to limit the outlet velocity to an acceptable value, that is, one that does not cause unacceptable scouring or
      b. adequate outlet protection should be provided.
   x. Where floodwaters are expected to remain on or over the road embankment for some time, the road embankment design should allow for infiltration of floodwater depending on the anticipated duration of inundation.

B. **Review Headwater:** The headwater should be checked for the review flood frequency based on the following considerations:
   i. Compliance with flood plain management criteria. If the increase in damage over the design flood is minor, the facility is satisfactory as designed. If the amount of

damage increases substantially, the designer should consider increasing the hydraulic opening to reduce the damage.

ii. Overtopping of the highway. The review flood shall not be permitted to flow over the highway. It is desirable to provide a freeboard of at least 0.3m between the review upstream floodwater surface and the upstream shoulder edge. Where this is not economically acceptable (applicable to design class D & E Roads), pavement design should make allowance for higher water heights, and the likely duration of inundation.

13.6.12.3 TAILWATER

Tailwater is the flow depth in the downstream channel measured from the invert at the culvert outlet. It can be an important factor in culvert hydraulic design because a submerged outlet may cause the culvert to flow full rather than partially full.

A field inspection of the downstream channel should be made to determine whether there are obstructions that will influence the tailwater depth. Tailwater depth may be controlled by the stage in a contributing stream, headwater from structures downstream of the culvert, reservoir water surface elevations, or other downstream features.

13.6.12.4 OUTLET VELOCITY

The outlet velocity of a highway culvert is the velocity measured at the downstream end of the culvert, and it is usually higher than the maximum natural stream velocity. This higher velocity can cause streambed scour and bank erosion for a limited distance downstream from the culvert outlet. Permissible velocities at the outlet will depend upon streambed type, and the kind of energy dissipation (outlet protection) that is provided.

If the outlet velocity of a culvert is too high, it may be reduced by changing the barrel roughness. If this does not give a satisfactory reduction, it may be necessary to use some type of outlet protection or energy dissipation device (e.g., riprap, gabion, masonry slabs etc.).

13.6.13 CULVERT HYDRAULICS

The culvert size and type can be selected after the determination of the design discharge, culvert location, tailwater, and controlling design headwater. The hydraulic performance of culverts is complex, and the flow characteristics for each site should be analysed carefully to select an economical installation, which will perform satisfactorily over a range of flow rates. Headwater and capacity computations can be made by using mathematical equations, electronic computer programs or nomographs.

Flood routing through a culvert is an alternative culvert-sizing practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. In some instances, a culvert should be sized on the basis of the flood routing concept, depending on the amount of temporary storage involved and the degree of environmental concern and flood hazard. The flood-routing procedure requires three basic data inputs:

i. an inflow hydrograph,
ii. an elevation versus storage relationship, and
iii. an elevation versus discharge relationship.

A complete inflow hydrograph, not just the peak discharge, must be generated. Elevation, often denoted as stage, is the parameter that relates storage to discharge providing the key to the flood routing solution.

13.6.13.1 CONDITIONS OF FLOW

A culvert may flow full over all its length or partially full. Full flow throughout a culvert is rare, and generally some portion of the barrel flows partly full. A water surface profile analysis is the only way to accurately determine how much of the barrel flows full.

i. **Full Flow**. The hydraulic condition in a culvert flowing full is called pressure flow. If the cross-sectional area of the culvert in pressure flow were increased, the flow area would expand. One condition which can create pressure flow in a culvert is the back pressure caused by a high downstream water surface elevation. A high upstream water surface elevation may also produce full flow. Regardless of the cause, the capacity of a culvert operating under pressure flow is affected by upstream and downstream conditions and by the hydraulic characteristics of the culvert.

ii. **Partly Full (Free Surface) Flow**. Free surface flow or open channel flow may be categorized as subcritical, critical, or supercritical. A determination of the appropriate flow regime is accomplished by evaluating the dimensionless number, F_r, called the Froude number using **Equation (13.95)**.

$$F_r = \frac{V}{(gy_h)^{0.5}} \tag{13.95}$$

Where,

V= average velocity of flow, m/s
g= gravitational acceleration, m/s^2
y_h= hydraulic depth, m (The hydraulic depth is calculated by dividing the cross-sectional flow area by the width of the free water surface)

When Fr > 1.0, the flow is supercritical and is characterized as swift.
When Fr < 1.0, the flow is subcritical and characterized as smooth and tranquil.
If Fr = 1.0, the flow is said to be critical.

The three flow regimes are illustrated in the depiction of a small dam in **Figure 13.48**. Subcritical flow occurs upstream of the dam crest where the water is deep, and the velocity is low. Supercritical flow occurs downstream of the dam crest where the water is shallow, and the velocity is high. Critical flow occurs at the dam crest and represents the dividing point between the subcritical and supercritical flow regimes. To analyse free surface flow conditions, a point of known depth and flow (control section) must first be identified. A definable relationship exists between critical depth and critical flow at the dam crest, making it a convenient control section.

Identification of subcritical or supercritical flow is required to continue the analysis of free surface flow conditions. The example using the dam of **Figure 13.48** depicts both flow regimes. Subcritical flow characteristics, such as depth and velocity, can be affected by downstream disturbances or restrictions. For example, if an obstruction is placed on the dam crest (control section); the water level upstream will rise. In the supercritical flow regime, flow characteristics are not affected by downstream disturbances. For example, an obstruction placed at the toe of the dam does not affect upstream water levels.

The same type of flow illustrated by the small dam may occur in a steep culvert flowing partly full (**Figure 13.49**). In this situation, critical depth occurs at the culvert inlet, subcritical flow exists in the upstream channel, and supercritical flow exists in the culvert barrel. A special type of free surface flow is called "just-full flow." This is a special condition where a pipe flows full with no pressure. The water surface just touches the crown of the pipe. The analysis of this type of flow is the same as for free surface flow.

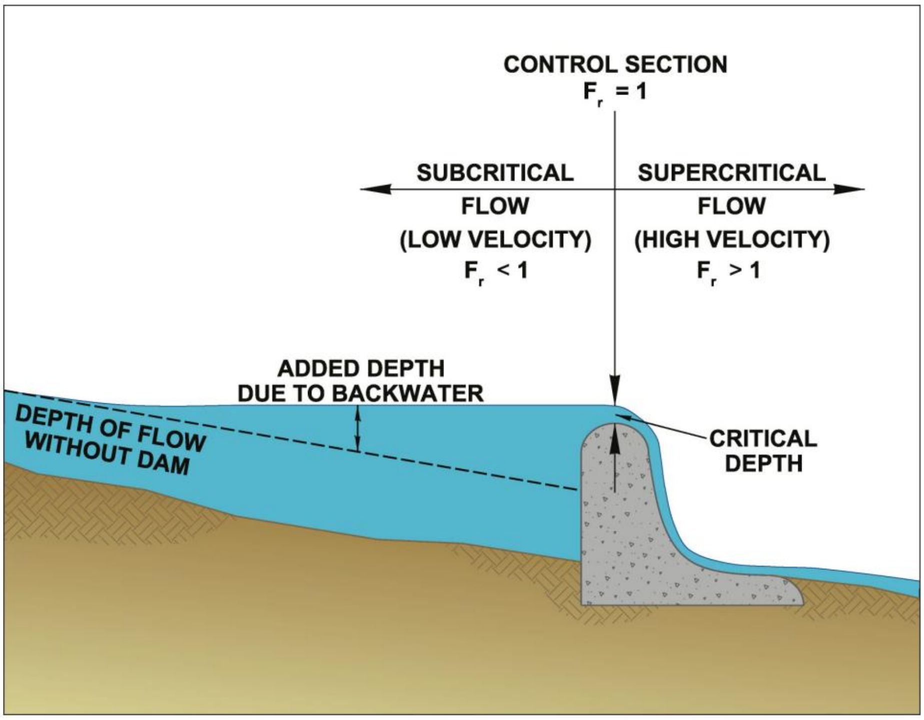


Figure 13.48 Flow conditions over a small dam

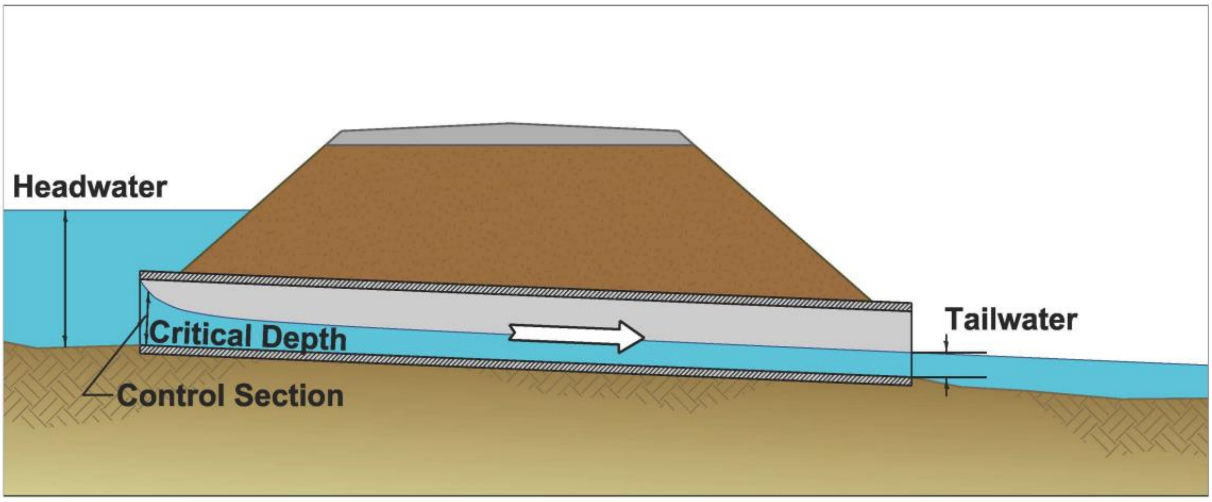


Figure 13.49 Typical inlet control flow section

13.6.13.1.1 FLOW CONTROLS

An exact theoretical analysis of culvert flow is extremely complex because the flow is usually nonuniform with regions of both gradually varying and rapidly varying flow. An exact analysis involves backwater and drawdown calculations, energy and momentum balance, and application of the results of hydraulic model studies. Often, hydraulic jumps form inside or downstream of the culvert barrel. The U.S. Geological Survey has defined 18 different culvert flow types based on inlet and outlet submergence, the flow regime in the barrel, and the downstream brink depth (USGS 1968). The flow type can change in a given culvert as the flow rate and tailwater elevations change.

Culvert analysis is based on the various types of flow and the location of the control section. A control section is a location where there is a unique relationship between the flow rate and the upstream water surface elevation. Many different flow conditions exist over time, but at a given time the flow is either governed by:

i. inlet control or
ii. outlet control.

For each type of control, a different combination of factors is used to determine the hydraulic capacity of a culvert.

Figure 13.50 shows the range of USGS flow types commonly encountered in culverts.

For inlet control, two distinct regimes exist, depending on whether the inlet is submerged or not submerged. Outlet control occurs in long culverts, culverts laid on flat grades and with high tailwater depths. In designing culverts, the type of control is determined by adopting the greater of the headwater depths calculated for both inlet control and outlet control.

With the aid of a computer analysis program (e.g., HY-8), it is possible to analyse both inlet and outlet flow conditions easily to determine which condition should prevail.

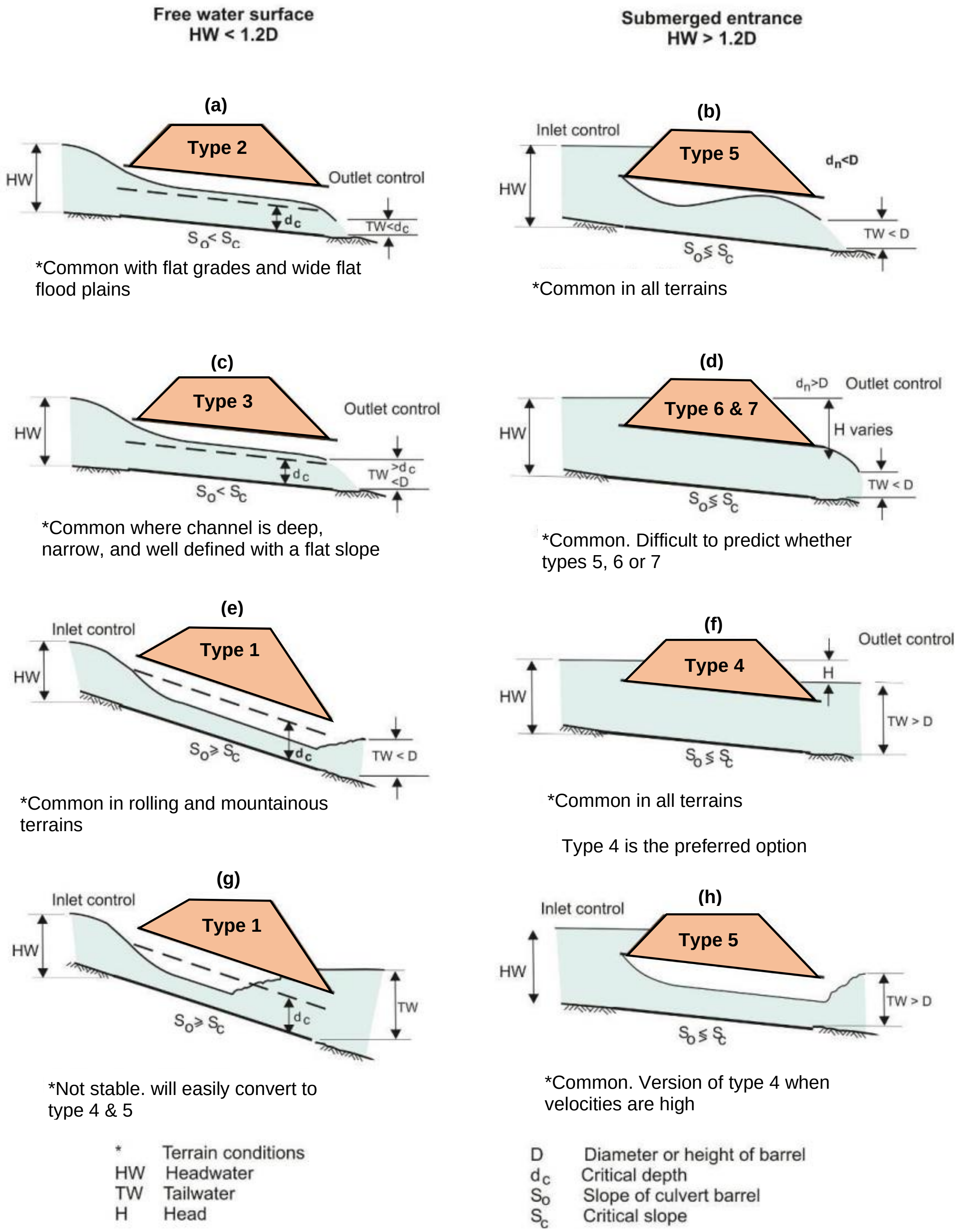


Figure 13.50 Typical USGS flow types under which standard culverts operate

13.6.13.1.2 INLET CONTROL

Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. A culvert operates with inlet control when the flow capacity is controlled at the entrance by these factors:

i. Depth of headwater
ii. Inlet area (Cross-sectional area of the face of the culvert)
iii. Inlet edge configuration (see **Plate 13.12**)
iv. Barrel shape

(A) Thin Edge Projecting – The culvert barrel projects out of the embankment.

(B) Mitered entrance – The culvert barrel is cut so it is flush with the embankment slope.

(C) Square edge in headwall - The end of the culvert barrel is flush with the headwall.

(D) Grove edge projecting - A concrete pipe culvert section extends beyond the fill or headwall.

Plate 13.12 Typical inlet configurations.

When a culvert operates under inlet control, headwater depth and the inlet edge configuration determine the culvert capacity with the culvert barrel usually flowing only partially full.

Inlet control for culverts may occur in two ways. The least common occurs when the headwater depth is not sufficient to submerge the top of the culvert and the culvert invert slope is supercritical as shown in **Figure 13.50 (c)** and **(d)**.

The most common occurrence of inlet control is when the headwater submerges the top of the culvert [**Figure 13.50 (e)** and **(h)**], and the pipe does not flow full. A culvert flowing under inlet control is defined as a hydraulically short culvert.

For a culvert operating with inlet control, the roughness, slope, and length of the culvert barrel and outlet conditions (including tailwater) are not factors in determining culvert hydraulic performance. The entrance edge and the overall entrance geometry have much to do with culvert performance in this type of flow; therefore, special entrance designs can improve hydraulic performance and result in a more efficient and economical culvert.

For one-dimensional flow, the relationship between the discharge and the upstream energy can be computed by an iterative process or by the use of nomographs (**Charts 16**, **17**, **18** and **23**).

13.6.13.1.3 OUTLET CONTROL

Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

If the headwater is high enough, the culvert slope sufficiently flat, and the culvert sufficiently long, the control will shift to the outlet. In outlet control, the discharge is a function of the inlet losses, the headwater depth, the culvert roughness, the culvert length, the barrel diameter, the culvert slope, and sometimes the tailwater elevation.

In outlet control, culvert hydraulic performance is determined by these factors:

i. Depth of headwater
ii. Cross-sectional area
iii. Inlet edge configuration
iv. Culvert shape
v. Barrel slope
vi. Barrel length
vii. Barrel roughness
viii. Depth of tailwater

Outlet control will exist under two conditions. The first and least common is that where the headwater is insufficient to submerge the top of the culvert, and the culvert slope is subcritical [**Figure 13.50 (a)** and **(b)**]. The most common condition exists when the culvert is flowing full [**Figure 13.50 (f)** and **(g)**]. A culvert flowing under outlet control is defined as a hydraulically long culvert.

Flow under outlet control can be calculated from the formulae below, the parameters for which are illustrated in **Figure 13.51**. The total head (H) required to convey water through a culvert flowing under outlet control is determined by **Equation (13.96)**:

$$\mathrm{H} = \mathrm{H_e} + \mathrm{H_f} + \mathrm{H_v} \tag{13.96}$$

Where,

$$\mathrm{H_e} = \text{Entrance loss (m)} = \mathrm{K}_e \frac{V^2}{2g}$$

$$\mathrm{H_f} = \text{Friction loss (m)} = \frac{19.6 n^2 L}{R^{1.33}} \times \frac{V^2}{2g}$$

$$\mathrm{H_V} = \text{Velocity head (m)} = \frac{V^2}{2g}$$

and

V = Mean velocity of flow in the culvert barrel (m/s)

g = Acceleration due to gravity (assume 9.81 m/s^2)

k_e = Entrance loss coefficient (see **Table 13.49**)

n = Manning's roughness coefficient

L = Length of culvert barrel (m)

R = Hydraulic radius (m)

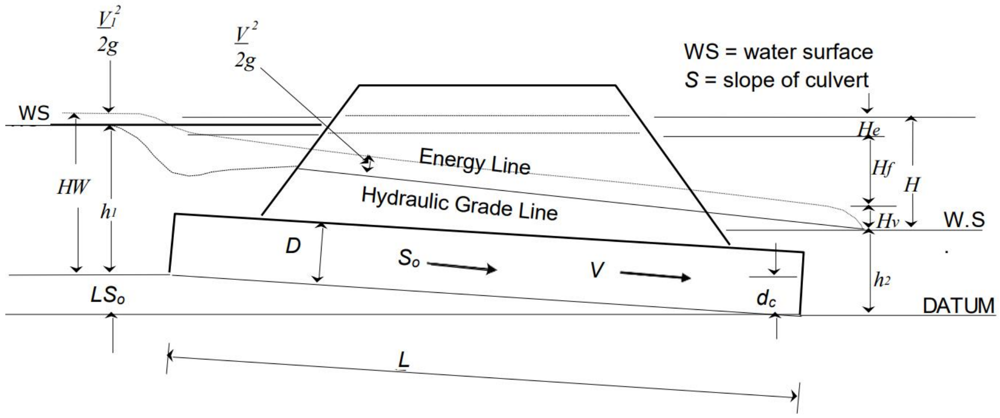


Figure 13.51 Hydraulics of culvert flowing full under outlet control

Substituting in the first equation [**Equation (13.96)**] and simplifying gives **Equation (13.97)**:

$$H = \left(1 + K_e + \frac{19.6n^2L}{R^{1.33}}\right) \times \frac{V^2}{2g} \tag{13.97}$$

This equation can be solved for H by the use of the full flow nomographs in **Charts 20**, **21**, **22** and **25**.

From the development of this energy equation and **Figure 13.52**, H is the difference between the elevation of the hydraulic grade line at the outlet and the energy line at the inlet. Since the velocity head in the entrance pool usually is small when ponded conditions occur ($V^2/2g = 0$), the water surface of the headwater pool elevation can be assumed to equal the elevation of the energy line.

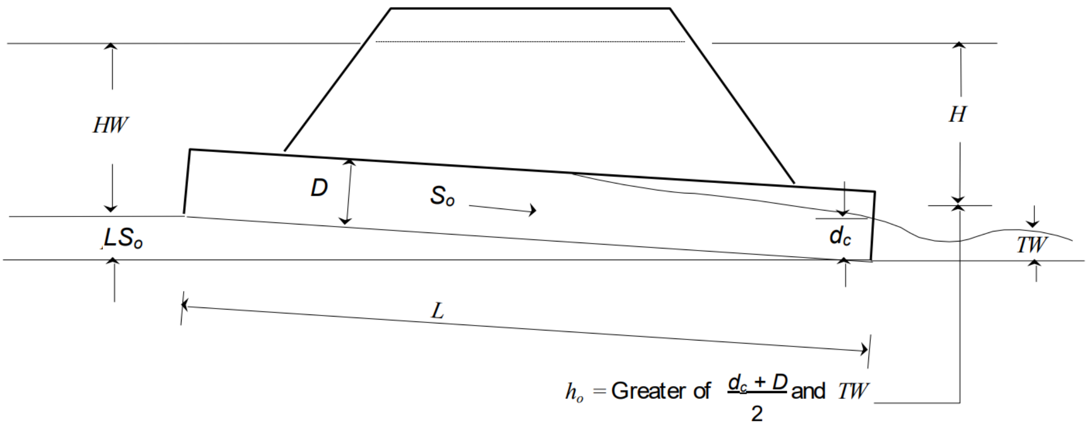


Figure 13.52 Determination of adopted outlet depth (h_o)

Headwater depth under outlet control is calculated according to **Equation (13.98)**:

$$HW = H + h_o - LS_o \tag{13.98}$$

Where,

H = Total head (m) determined from Equation 23

h_o = Adopted outlet depth (m)

L = Length of culvert (m)

S_o = Slope of culvert barrel (m/m)

The various components of this equation are illustrated in **Figure 13.52**.

The adopted outlet height (h_o) equals TW if TW > D, otherwise it is the greater of TW or **Equation (13.99)**:

$$\text{TW or } \frac{d_c + D}{2} \tag{13.99}$$

Where,

d_c = Critical depth (m)

D = Diameter or height of culvert (m)

Table 13.49 Entrance Loss Coefficients

Type of structure and design of entrance	**Coefficient k_e**
Concrete/fibre reinforced/poly pipe	
Projecting from fill, socket end	0.2
Projecting from fill, square cut end	0.5
Headwall, with or without wingwalls:	
Socket end	0.2
Rounded edge (radius = *D*/12)	0.2
Square edge	0.5
End section conforming to fill slope (precast end unit)	0.5
Mitred/cut to conform to fill slope (field cut)	0.7
Bevelled edges, 33.7^0 or 45^0 bevels	0.2
Side- or slope-tapered inlet	0.2
Corrugated metal pipe or pipe arch	
Headwall rounded edge	0.2
Headwall or headwall and wingwalls: square edged	0.5
End section conforming to fill slope (manufacturer end unit)	0.5
Mitred/cut to conform to fill slope	0.7
Projecting from fill (no headwall)	0.9
Side- or slope-tapered inlet	0.2
Reinforced concrete box	
Headwall parallel to embankment (no wingwalls):	
Rounded on 3 edges (radius of 1/12 cell dimension)	0.2
Square on 3 edges	0.5

Type of structure and design of entrance	**Coefficient k_e**
Wingwalls at 30° to 75° to cell:	0.5
Crown edge rounded (radius of 1/12 cell dimension)	0.2
Crown edge square	0.4
Wingwalls at 10° to 30° to cell: square edged at crown	0.5
Wingwalls parallel (extension of sides) square edged at crown	0.7

Notes:

The effect of wing walls reduces with multi-cell culverts.

For 3–6 cell culverts, assume entrance loss for wing walls 10° to 25° to barrel.

13.6.13.2 PERFORMANCE CURVES

Performance curves are plots of discharge versus culvert headwater depth or elevation. A culvert may operate with outlet or inlet control over the entire range of flow rates, or control may shift from the inlet to the outlet. For this reason, it is necessary to plot both inlet and outlet control curves to develop the culvert performance curve. In culvert design, the designer usually selects a design flood frequency, estimates the design discharge for that frequency, and sets an allowable headwater elevation based on the selected design flood. There are, however, uncertainties in estimating flood peaks for any desired recurrence interval and a probability that the design frequency flood will be exceeded during the life of the project. Because of the uncertainties, it is necessary for the designer to develop information from which he can evaluate the culvert performance or headwater capacity relationship over a range of flow rates. With this information on culvert performance, the risks involved in the event of large floods can be evaluated. This evaluation should include the probability of occurrence, the possibility of traffic interruption by flow over the highway, and damages that would occur to the highway and other property.

Performance curves aid in the selection of the culvert type, including size, shape, material and inlet geometry, which fulfils site requirements at the least annual cost. The curves also may reveal opportunities for increasing the factor of safety and improving the hydraulic capacity at little or no increase in cost. A typical culvert performance curve is shown in **Figure 13.53**. Flood frequency has been added to the abscissa to aid in evaluating the risk of exceeding the design headwater with the selected culvert design.

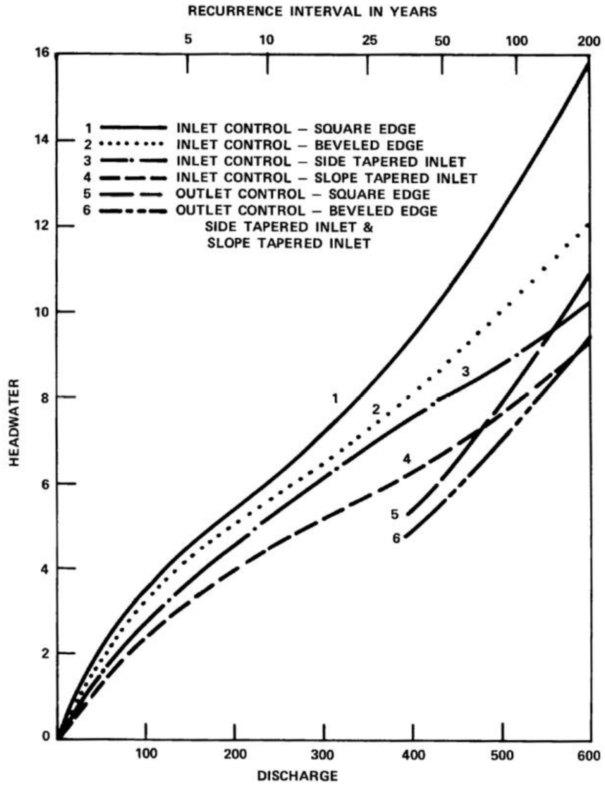


Figure 13.53 Performance Curves for Single Box Culvert 90° Wingwall

13.6.14 HYDRAULIC DESIGN

The standard culvert design procedure is illustrated in the flow chart in **Figure 13.55** and the following subsections align with this figure.

For this method, calculations should be recorded on a form similar to the one shown in **Figure 13.54**. The following procedure provides direction in completing this form.

13.6.14.1 COLLECT DESIGN DATA

The data to be collected/determined, and then recorded in the design workings includes:

i. required flood immunity (ARI)
ii. design discharge (Q), check discharge (if required) and extreme event discharge (if available/applicable)
iii. tailwater height (TW), stream velocity (V) and Froude Number (Fr) for each discharge flow
iv. road shoulder height and any freeboard requirements
v. maximum/allowable headwater height (AHW)
vi. proposed culvert slope (S_o)
vii. proposed culvert length (L)
viii. inlet/outlet invert heights
ix. maximum allowable stream velocity (V_{max}) for outlet channel.

13.6.14.2 SELECT A TRIAL CULVERT

To select an initial trial culvert or culverts, first determine an initial trial culvert waterway area (A) using **Equation (13.100)**.

$$A = Q/V_{max} \tag{13.100}$$

Where,

Q = Design discharge (m^3/s)

V_{max} = Maximum allowable outlet velocity (m/s)

Maximum allowable outlet velocity should be based on site conditions or if no data is available, assume an outlet velocity of 2.0 to 2.5 m/s depending on channel conditions.

Alternatively, if the allowable head loss (H) through the culvert is known (from the allowable afflux to limit flooding height impacts on upstream development), then the initial trial culvert area may be estimated using **Equation (13.101)**.

$$A = Q/V_{max} \tag{13.101}$$

Choose culvert material, shape (pipe or box) and entrance type, allowing for minimum and maximum allowable cover heights over the culvert.

Select a culvert trial size/configuration using the waterway area determined above. For example, the initial trial culvert waterway area may have been determined as 1.25 m^2. A single barrel 900mm concrete pipe (PC) has a waterway area of 0.64 m^2. Two barrels of 900mm PC would have a waterway area of 1.27 m^2, which is slightly larger than the required waterway area and therefore suitable as an initial trial.

If possible, select a culvert size so that the obvert of the outlet is at, or just below, the tailwater height. This ensures the probability of the culvert running full, which is usually desirable. This is not always practicable in wide shallow flood plains, or in steep terrain. If the trial size is too large because of limited embankment height or availability of size, multiple culverts may be used by dividing the discharge equally between the number of cells used.

A further method is the use of culvert capacity charts (**Charts 16 - 25**), which can be used to determine a preliminary culvert size and corresponding headwater depth for a known discharge. To use these charts the value of $L/(30S_o)$, where L is the culvert length (m) and S_o is the culvert slope (m/m). The value of $L/(30S_o)$ will dictate whether to adopt the inlet control line, outlet control line, or interpolate between the two control lines drawn on the charts, to determine the headwater depth.

In locations where fauna passage is an issue, compare trial culvert size with the minimum fauna requirements.

Several initial trial culverts of different size/configuration may be selected to start the design process.

13.6.14.3 DESIGN DISCHARGE FOR TRIALS

Where a single barrel culvert is not sufficient, multiple barrels or cells will be required and these configurations constitute a parallel system.

Provided that all barrels of a multi-cell culvert are the same type, size and roughness (equal conveyance), and also have the same invert levels/bed slope, flow will distribute evenly.

Design nomographs as shown in **Charts 16 - 18**, **20 - 23** and **25**, are based on a single barrel installation, therefore divide the design discharge (Q) by the number of barrels for the trial pipe culvert.

For box culverts, a ratio (Q/B) is also required. This ratio is determined by dividing Q per cell by the nominal box width (B).

Any proposed, multi-cell culvert that does not have all cells the same will require specialist advice as the methodology presented in this Guide does not allow for these types of configurations. These culverts must be referred to specialist hydraulic engineers.

13.6.14.4 DETERMINE INLET CONTROL HEADWATER DEPTH

Using the trial culvert(s) from the previous section, find the HW/D value by use of the appropriate inlet control nomograph (**Charts 16 - 18** and **23**). In this case, tailwater conditions are to be neglected.

Three lines are presented in the nomographs for HW/D and the designer needs to select the appropriate line based on:

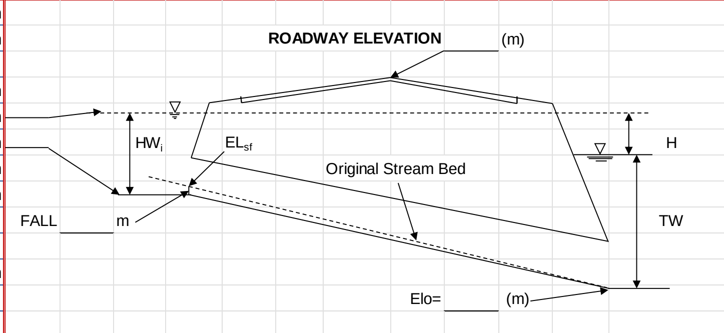
- entrance type for pipe culverts
- wingwall flare angle for box culverts.

Headwater under inlet control conditions (HW_i) is found by multiplying HW/D by the height of the culvert, D.

Check HW_i against AHW and if HW_i is greater (or much less) than AHW, try another size until HW_i is acceptable for inlet control before computing headwater for outlet control.

PROJECT TITLE:	STATION	**CULVERT DESIGN FORM**	
		DESIGNER:	DATE:
	SHEET OF	REVIEWER:	DATE:

HYDROLOGICAL DATA				Allowable Headwater:	m	
Method:				Review Headwater	m	
Drainage Area:	km^2					
Channel Shape:				Shldr. Elev. LHS:	m	
Routing:				El_{hd}:	m	
				El_i:	m	
DESIGN FLOWS/TAILWATER					m	
Q_1:	m^3/s	ARI: yrs	TW_1: m	L:	m	
REVIEW FLOWS/TAILWATER						
Q_2:	m^3/s	ARI: yrs	TW_2: m	S:	m/m	
				SKEW(°):		



Shldr. Elev. RHS=		(m)
		(m)
TW Depth		(m)
Maximim allowable stream velocity (V_{max}):		m/s
Min. Inv.		
(Out) Elev.=		(m)

CULVERT DESCRIPTION:							**HEADWATER CALCULATIONS**												**CONTROL**		**COMMENTS**
					TOTAL FLOW	**FLOW PER BARREL**	**INLET CONTROL**				**OUTLET CONTROL**								**HEADWATER**	**OUTLET**	
					Q	Q/N	HW_i/D	HW_i	FALL	EL_{hi}	TW	d_c	$(d_c+D)/2$	h_o	k_e	H	EL_{ho}		**ELEVELATION**	**VELOCITY**	
					(m^3/s)	(m^3/s)	-	(m)	(m)	(m)	(m)	(m)	(m)	(m)		(m)	(m)		**(m)**	**(m/s)**	
MATERIAL	**TYPE**	**SPAN (m)**	**RISE(m)**	**N**		(1)	(2)		(3)	(4)	(5)			(6)		(7)	(8)				

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) HW_i/D = HW/D OR HW_i/D FROM DESIGN CHARTS

(3) FALL = HW_i - (EL_{hd} - EL_{sf}); FALL IS ZERO FOR CULVERTS ON GRADE

(4) ELhi = HWi + ELi (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWNSTREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) h_o = TW or (d_c + D/2) (WHICHEVER IS GREATER)

(7) $H = [1 + k_e + (19.62n^2L)/R^{1.33}]v^2/2g$

(8) $EL_{ho} = El_o + H + h_o$

SUBSCRIPT DEFINITIONS:				**COMMENTS / DISCUSSION:**	**CULVERT BARREL RECOMMENDED**	
hd	DESIGN HEADWATER	sf	STREAMBED AT CULVERT FACE			
hi	HW IN INLET CONTROL				SIZE:	n:
ho	HW IN OUTLET CONTROL				TYPE:	ENTRANCE:
i	INLET				MATERIAL:	
o	OUTLET				DOWNSTREAM PROTECTION REQUIRED:	

Figure 13.54 Form for culvert design calculations

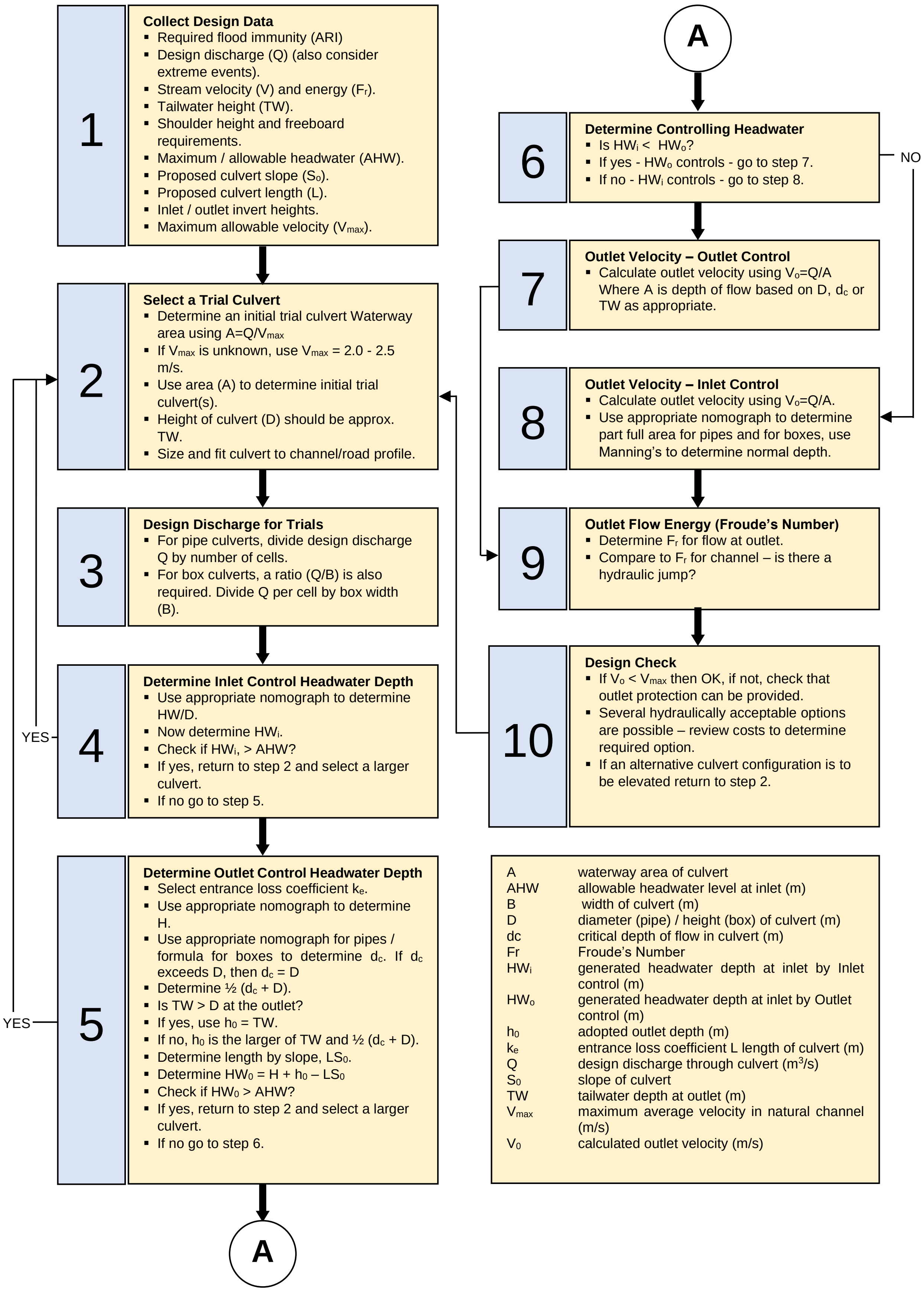


Figure 13.55 Culvert design flow chart in steps 1 to 10

13.6.14.5 DETERMINE OUTLET CONTROL HEADWATER DEPTH

Several steps are required to determine the headwater under outlet control conditions (HW_o).

Firstly, determine the entrance loss coefficient, k_e from **Table 13.49**.

Calculate the losses through the culvert, H, using the outlet control nomographs, **Charts 20 - 22** and **25**. In using these nomographs, some interpolation can be used for k_e and L.

The next step is to determine the critical depth (d_c) for the culvert. If d_c exceeds D, then take d_c as D. For pipe culverts, use nomographs as shown in **Chart 19** and for box culverts, use **Equation (13.102)** or **Chart 24**.

$$d_c = 0.467\left(\frac{Q}{B}\right)^{2/3} \tag{13.102}$$

where Q = Q per cell

Calculate (dc + D)/2.

Tailwater (TW) is required for determination of the next variable.

The next step is to establish the adopted outlet depth (h_o) for design.

Determine if TW > D:

- if yes, h_o = TW
- if no, h_o = the larger of TW and (d_c + D)/2.

Multiply the proposed culvert length (L) and slope (S_o) to calculate LS_o.

Headwater under outlet control conditions (HW_o) is calculated using **Equation (13.103)**:

$$HW_0 = H + h_0 - LS \tag{13.103}$$

Now check HW_o against AHW and if HW_o is greater than AHW, try another size until both HW_i and HW_o are acceptable (< AHW).

13.6.14.6 DETERMINE THE CONTROLLING HEADWATER

Compare the values of HW_i and HW_o. The higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected.

- If $HW_i > HW_o$ the culvert is under inlet control and the controlling HW = HW_i.
- If $HW_i < HW_o$ the culvert is under outlet control and the controlling HW = HW_o.

The controlling headway for trial should be clearly shown in design workings (i.e. design form).

There are now two checks that should be undertaken before completing trial design:

i. If controlling HW is less than 1.2D, then the culvert is most likely not operating with a submerged inlet and therefore may not be operating efficiently. Design should be revised using a different (slightly smaller) culvert size/configuration.
ii. If controlling HW is less than 0.75D and the culvert is under outlet control, then the culvert may be flowing only part-full and using (d_c+D)/2 to calculate h_o may not be giving accurate results. Design should be revised using a different culvert size/configuration, where D ≈ TW.

Sometimes, calculations show HW_i and HW_o to be equal or nearly equal. In this instance, it is not clear whether or not the culvert will perform under inlet control or outlet condition. In reality, the culvert could also operate under both conditions (swap from one to the other) during the same rainfall event. Therefore, outlet velocity calculations should be done for both inlet control and outlet control conditions with the higher velocity (and associated control condition) being adopted.

13.6.14.7 ROADWAY OVERTOPPING

Overtopping will begin when the headwater rises to the elevation of the roadway (**Figure 13.56**). The overtopping will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad crested weir.

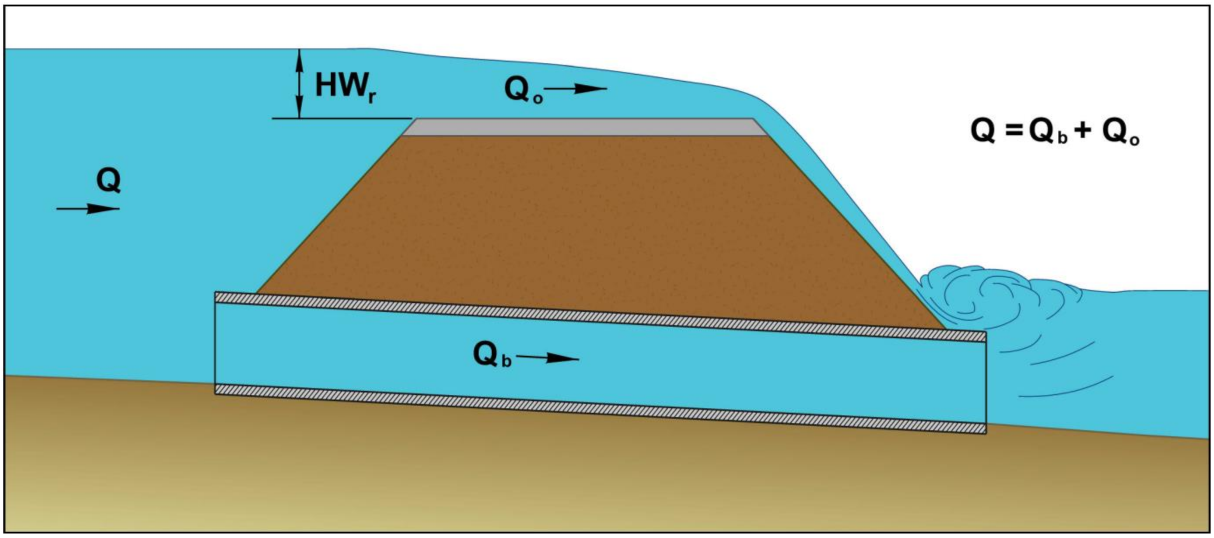


Figure 13.56 Roadway overtopping

Flow coefficients for flow overtopping roadway embankments are found in HDS 1, Hydraulics of Bridge Waterways (FHWA 1978), as well as in the documentation of Curves from HY-7, Bridge Waterways Analysis Model (FHWA 1986a) are shown in **Figure 13.57**:

- **Figure 13.57A** is for shallow overtopping
- **Figure 13.57B** is for deep overtopping
- **Figure 13.57C** is a correction factor for downstream submergence. Submergence occurs as the tailwater begins to encroach on the free overfall from the weir.

Equation (13.104) defines the flow across the roadway.

$$Q_o = C_f L H^{1.5}\left(\frac{C_s}{C_f}\right) \qquad (13.104)$$

Where,

Q_o	=	Overtopping flow rate in m³/s
C_f	=	Coefficient of discharge ‘free’ flow
L	=	Length of roadway crest, m
H	=	Specific head or specific energy (HW_r), m

With reference to **Figure 13.57**

h = Height difference between the floodway crown and the upstream water surface

V = Approach velocity of the stream

l = Top width of road formation

The flow over the roadway may be calculated by means of the following design procedure:

i. Calculate the Stage-Discharge curve (height versus discharge) for the unrestricted section, from open channel hydraulics.
ii. Select a tailwater height and a headwater height (given h and V) from the Stage-Discharge curve.
iii. Calculate H/l with **Equation (13.105)**.

$$\frac{H}{l} \tag{13.105}$$

Where,

$$H = h + \frac{V^2}{2g} = specific\ head$$

iv. From **Figure 13.57**

- For H/l < 0.15 use curve A to obtain value of C_f.
- For H/l > 0.15 (usual case) use curve B to obtain the free flow coefficient of discharge C_f.
- Calculate D/H x 100 and use curve C to obtain the submergence factor C_s/C_f
- Calculate the discharge over the road using the broad crested weir formula, **Equation (13.104)**.

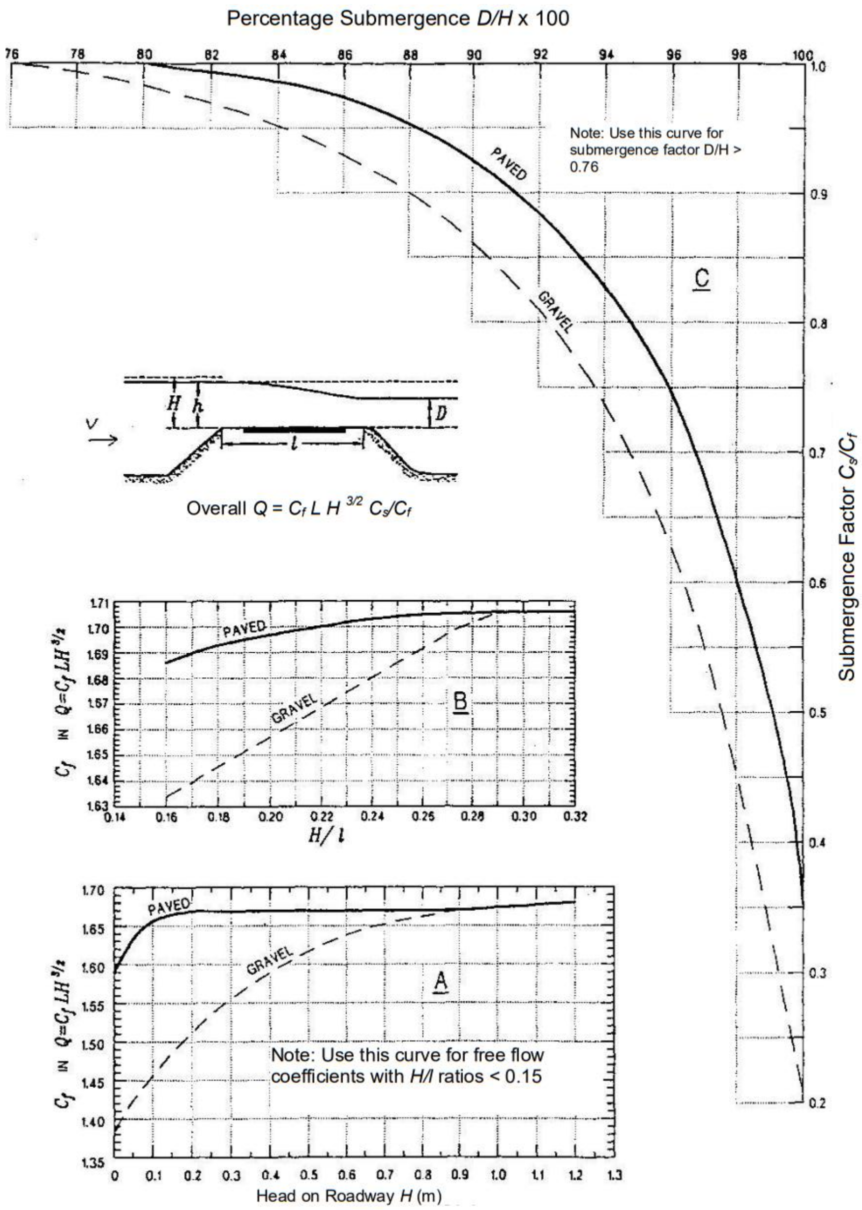


Figure 13.57 Discharge coefficients for roadway overtopping

The length and elevation of the roadway crest are difficult to determine when the crest is defined by a roadway sag vertical curve. The sag vertical curve can be broken into a series of horizontal segments as shown in **Figure 13.58A**. Using **Equation (13.104)**, the flow over each segment is calculated for a given headwater. Then, the incremental flows for each segment are added together, resulting in the total flow across the roadway.

Representing the sag vertical curve by a single horizontal line (one segment) is often adequate for culvert design (**Figure 13.58B**). Using this approach, the length of the weir (L) can be represented by the top width of the overflow area in the sag, the upstream depth (H) by the hydraulic depth (overflow area in the sag divided by the top width of flow), and the elevation of the weir crest defined from the lowest point in the sag.

It is a simple matter to calculate the flow across the roadway for a given upstream water surface elevation using **Equation (13.104)**. The problem is that the roadway overflow plus the culvert flow must equal the total design flow. A trial and error process is necessary to determine the amount of the total flow passing through the culvert and the amount flowing across the roadway. Performance curves may also be superimposed for the culvert flow and the road overflow to yield an overall solution.

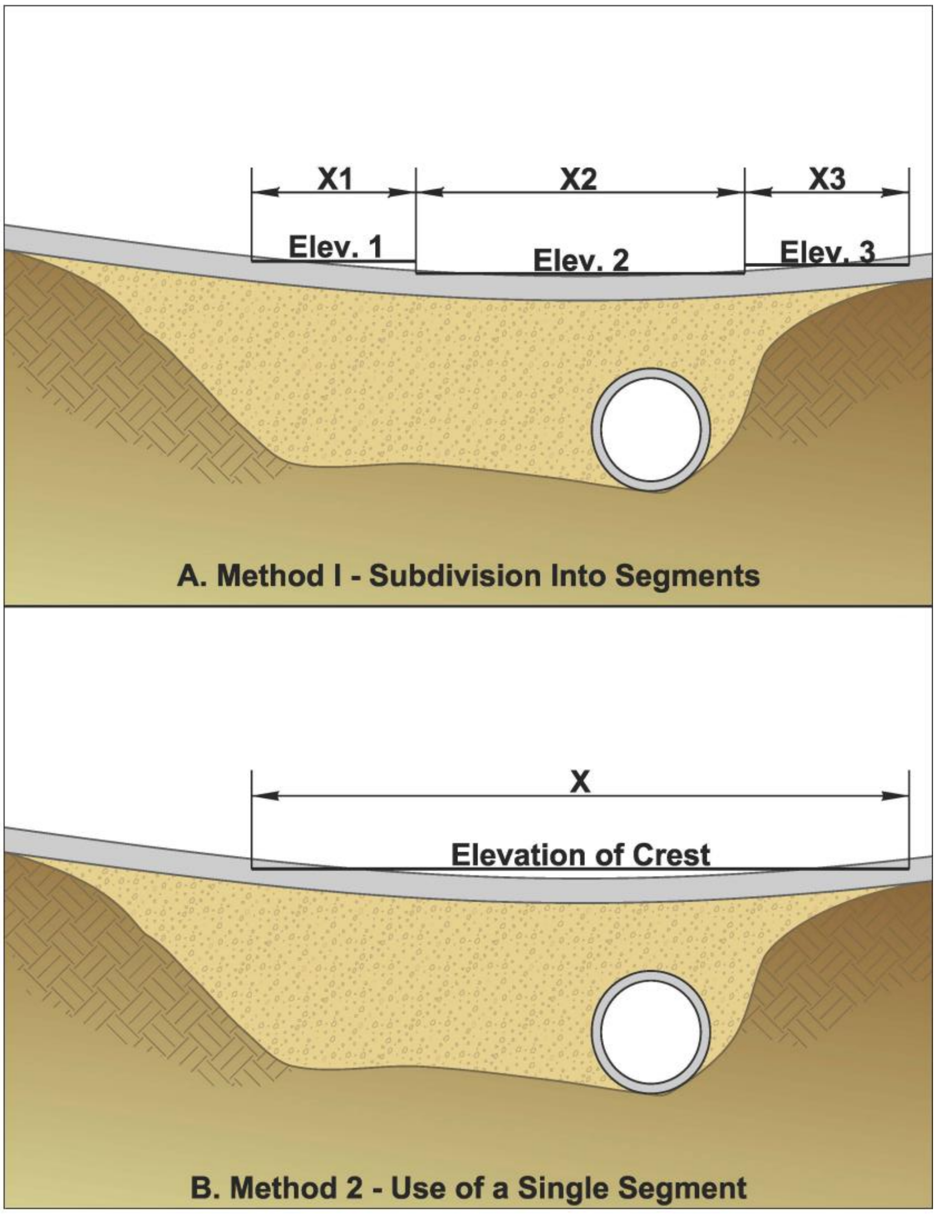


Figure 13.58 Weir crest length determinations for roadway overtopping.

13.6.14.8 OUTLET VELOCITY – OUTLET CONTROL

The average outlet velocity for all culvert types can be calculated using **Equation (13.106)**.

$$V_o = Q/A \tag{13.106}$$

Where,

Q = Design discharge per culvert barrel/cell (m^3/s)

A = Cross-sectional area of flow from culvert barrel/cell (m^2)

The cross-sectional area of flow (A) depends on the flow depth at the outlet.

Flow depth will be one of the following:

- critical depth (d_c) if the tailwater is below critical depth
- tailwater depth (TW) if the tailwater is between critical depth and the top of the barrel
- the height of the barrel (D) if the tailwater is above the top of the barrel.

See **Figure 13.59** for guidance in determining the flow area for box culverts.

$$Area = B \times y$$

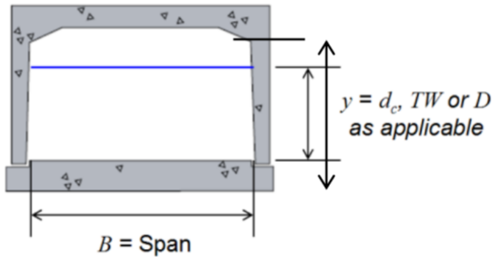


Figure 13.59 Flow area for box culverts

Determination of flow area for pipes is a little more difficult. The area can be determined using an approved CADD package or calculated using **Equation (13.107)**.

$Area = \pi R^2$ 28

when y = D

Or Equation 29

$$Area = \pi R^2 - \frac{1}{2} R^2(\theta - \sin\theta) \qquad (13.107)$$

Where,

R = Internal radius of pipe (m)

θ = Angle in radians

Figure 13.60 provides guidance in determining the flow area for pipes.

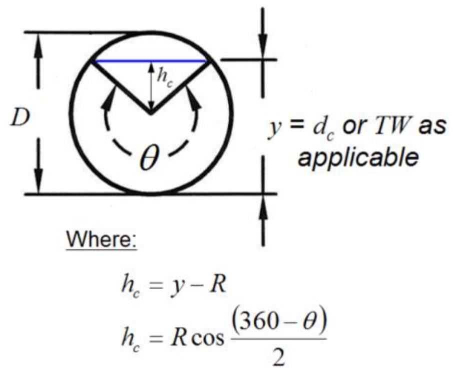


Figure 13.60 Flow area for pipe culverts

13.6.14.9 OUTLET VELOCITY – INLET CONTROL

The average outlet velocity, V_o, for all culvert types can be calculated using **Equation (13.106)**.

The cross-sectional area of flow (A) depends on the flow depth at the outlet which can be approximated by the normal depth of open-channel flow in the barrel, computed by Manning's

Equation for the discharge flow, barrel size, roughness and slope of culvert selected.

13.6.14.9.1 PIPE CULVERTS

For a pipe culvert, the culvert will not be flowing full at the outlet when under inlet control, meaning only a part of the full waterway area will be used. To determine this 'Part Area', the designer needs to firstly establish the relationship between 'Full Flow' (Q_f) and 'Part Flow' (Q_p) for the culvert trial where:

- 'Part Flow' (Q_p) is the design discharge per cell
- 'Full Flow' (Q_f) is the capacity of the trial culvert and its velocity can be determined using **Chart 26**

Now determine the ratio Q_p/Q_f.

Using this ratio and **Chart 27** determine the percentage factors for velocity (V_o), depth of flow (y), area of flow (A) and hydraulic radius (R).

Chart 27 is used as follows:

- Step 1 – plot the Q_p/Q_f ratio on the x-axis.
- Step 2 – draw a line up to the Discharge line.
- Step 3 – draw a line left and right to both edges (the left edge being the y-axis).
- Step 4 – read off '% Depth of Flow' from the y-axis.
- Step 5 – drop lines to the x-axis from each intersect between the horizontal line drawn in Step 3 and the Velocity, Area curves.
- Step 6 – read off % Values from the x-axis for each of these hydraulic elements.

Now draw a table as shown in **Table 13.50** and enter values as follows:

- A is Q_f
- B is Q_p
- C is the outlet velocity determined using **Chart 26**
- D is the nominal diameter of pipe
- E is the waterway area of pipe
- F is the hydraulic element values determined using **Chart 27**.

The remaining spaces of the table, including the determination of V_o, are calculated by multiplying the 'Full' values by the relevant 'Factor'.

Table 13.50 Full flow/part flow table

Variable	Pipe full	Applied factor	Pipe part
Q	*A*		*B*
V_o	*C*	*F*	
y	*D*		
A	*E*		

13.6.14.9.2 BOX CULVERTS

As for a pipe culvert, a box culvert will not be flowing full at the outlet when under inlet control. To determine the normal flow conditions, depth (y) and velocity (V_o), use Manning's Equation to develop a Modified Stage-Discharge Curve for the culvert cell.

Using the Modified Stage-Discharge Curve, the flow depth (y) and outlet velocity (V_o) can be read directly.

13.6.14.10 OUTLET FLOW ENERGY

To complete the hydraulic calculations for a trial culvert, the designer must determine Froude's Number (F_r) for the flow at the outlet. This is important as the designer can check this against Froude's Number for the channel flow and determine if a hydraulic jump will occur.

Froude's Number can be determined using **Equation (13.108)**.

$$F_r = \frac{V}{\sqrt{g\left({}^{A}/_{B}\right)}}$$

$$\text{and } F_r = Q\sqrt{{}^{B}/_{(gA^3)}} \qquad (13.108)$$

Where,

A = Cross-sectional area of flow normal to direction of flow (m^2)
B = Width of flow at surface (m)
V = Velocity of flow (m/s)
Q = Flow rate (m^3/s)
g = Acceleration due to gravity (9.81 m/s^2)

B is derived from **Equation (13.109)**.

$$B = 2\sqrt{y(D-y)} \qquad (13.109)$$

Figure 13.61 illustrates D, y and B.

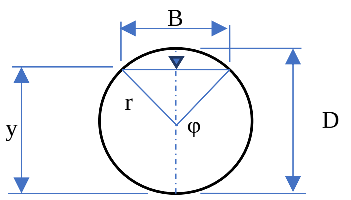


Figure 13.61 Flow width in pipes

When a pipe or box culvert is flowing full, there is some pressure in the system and there is no water surface (no. B). Strictly, Froude's Number does not apply, however for simplicity and that pressure is minimal, it is considered suitable to calculate F_r at the moment just before the water surface touches the obvert of the culvert and adopt this F_r value for the trial culvert. Therefore:

- for box culverts, use full waterway area and B = nominal width of box section
- for pipe culverts, use full waterway area, but assume B as 10% of D (≈ 99.7% of A).

13.6.14.11 DESIGN CHECK

At this stage, the outlet velocity (V_o) should be checked against the maximum allowable stream velocity (V_{max}) for the outlet channel. Preferably, V_o should be less than V_{max}. If V_o exceeds V_{max}, then the designer must include suitable outlet protection for the culvert if the trial is to be kept.

Where outlet protection is not suitable (size/cost/V_o > 5 m/s/F_r > 1.7) then a larger/wider culvert should be trialled, or specialist advice/input obtained.

13.6.14.12 PRACTICAL DESIGN

If a culvert trial design is considered unacceptable, the designer is required to redesign the culvert by trialling another culvert size/configuration. The choices the designer has in determining a new trial culvert are:

i. add another barrel or cell if channel width permits
ii. increase barrel or cell height if vertical clearance permits
iii. alter culvert slope (Note: desirable minimum is 0.4%)
iv. a combination of the above.

Where a designer decides to alter invert heights, it is preferable to lower inlets and leave outlets as close to the natural surface as possible. Drop inlets (and structures) are better than hanging or buried outlets.

13.6.15 BLOCKAGE OF CULVERTS

The likelihood of blockage should be considered for all culverts. Blockage can occur through siltation or by debris (such as vegetation). To assist in preventing and/or minimising blockage, the culvert grades should match the stream grade.

During the times of flood, the water contains silt, vegetation and other debris from the catchment. The designer should therefore consider the impacts on the effects of the culvert becoming blocked. Allowance for blockage is commonly provided in the sizing of the culvert (see **Section 13.6.7**).

Blockage reduces the waterway area of the culvert and therefore adversely affects the capacity/performance of the culvert. The result of blockage is typically:

i. an increase in upstream peak water heights/flooding
ii. an increased potential for water to overtop the road
iii. an increased risk of failure to the road embankment/culvert.

Silt deposits and some debris can be detected and removed during normal maintenance processes at times outside of rainfall events and therefore these deposits/debris would not impede any flows; however, the effectiveness of this measure is highly dependent on the efficiency of maintenance in the area.

Where debris blockage during an event is considered likely (typical in catchments that contain significant woody riparian vegetation), larger culvert sizes may be required, in accordance with the extent of adverse impacts that could occur to the roadway or to surrounding properties.

Where large or long branches and/or tree trunks are a possibility, sloped extensions to piers, as shown in **Plate 13.13** can be used to 'turn' long objects into the culvert barrel.

Designers must consider the potential for and impacts of blockage for each catchment/culvert installation and, where impacts are considered unacceptable, include appropriate mitigating treatments in the design.

Plate 13.13 Flood water flowing into box culverts

13.6.16 CULVERT OUTLET PROTECTION

Outlet protection is required in situations where:

i. Outlet velocity exceeds the scour velocity of the bed or bank material.
ii. An unprotected channel bend exists within a short distance of the culvert outlet.
iii. The outlet channel and banks are actively eroding.
iv. If an erodible channel bank exists less than 10 to 13 times the pipe diameter downstream of the outlet and this bank is in line with the outlet jet (i.e., likely to be eroded by the outlet jet), the bank should be adequately protected to control any undesirable damage as a result of the outlet jetting.

The most appropriate outlet protection is determined by considering the hydraulic performance of the outlet in the prevailing stream environment.

At outlet structures, the best hydraulic performance is obtained when the confining sidewalls are parallel and the distribution of flow across the channel is uniform.

Culverts, however, are generally narrower than the natural waterway and a transition section is required to return the flow to the natural channel. When culvert outlet velocities are high, additional measures at the outlets may prove to be necessary for energy dissipation.

To check whether standard inlet and outlet structures with headwalls, wingwalls, aprons and cut-off walls are adequate, the outlet velocity for the culvert requires examination with respect to:

i. natural environment (soil and vegetation cover)
ii. size of peak flow
iii. duration of large flows.

If outlet velocities exceed the acceptable limits, it may be necessary to check for potential bed scour problems. **Table 13.51** indicates the required culvert end treatment based on outlet velocity. Most culverts require adequate outlet protection, and this is a frequently overlooked issue during design.

Table 13.51 Required Culvert End Treatment Based on Outlet Velocity

Outlet Velocity (m/s)	Culvert End Treatment, CE (Type)*	
	End sections	Endwalls or Headwalls
0-1.80	None	None
1.80 – 3.00	CE – 1 Class I	None
3.00 – 4.30	CE – 1 Class I	CE – 1 Class I
4.30 – 5.80	CE – 1 Class II	CE – 1 Class II
5.80 +	Special Design	Special Design

*Refer to **Table 13.48** for riprap classes.

Variations in shape and size of a culvert seldom have a significant effect on the outlet velocity. Slope and roughness of the culvert barrel are the principal factors affecting the outlet velocity.

Maximum water flow velocities that can be tolerated without channel protection related to the type of bed material are shown in **Table 13.52**.

Table 13.52 Maximum water velocities

Bed material	Maximum water velocities without channel protection	
	Clear water (m/s)	Water carrying silt (m/s)
Stiff clay	1.0	1.5
Volcanic ash	0.7	1.0
Silty soil / sandy clay	0.6	0.9
Fine sand / coarse silt	0.4	0.7
Sandy soil	0.5	0.7
Firm soil / coarse sand	0.7	1.0
Graded sand and gravel	1.2	1.5
Firm soil with silt and gravel	1.0	1.5
Gravel (5 mm)	1.1	1.2
Gravel (10 mm)	1.2	1.5
Course gravel (25 mm)	1.5	1.9
Cobbles (50 mm)	2.0	2.4
Cobbles (100 mm)	3.0	3.5
Well established grass in good soil	1.8	2.4
Grass with exposed soil	1.0	1.8

The type and length of the riprap lined apron is related to the outlet flow rate and the tailwater level and whether there is a defined channel downstream. If the tailwater depth is less than half the outlet culvert rise, it shall be classified as a Minimum Tailwater Condition. If the tailwater depth is greater than or equal to half the outlet culvert rise, it shall be classified as a Maximum

Tailwater Condition.

The length of an apron (L_a) shown in **Figure 13.62** is determined using **Equations (13.110)** and **(13.111)** or 3 times the rise of the culvert, whichever is greater. The minimum width of the riprap is computed from **Equation (13.112)**.

$$L_a = \frac{3.26(Q - 0.142)}{S_p{}^{1.5}} + 3.05 \text{ or } 3R_p \text{ for TW} < 0.5R_p \tag{13.110}$$

$$L_a = \frac{5.44(Q - 0.142)}{S_p{}^{1.5}} + 3.05 \text{ or } 3R_p \text{ for TW} \geq 0.5R_p \tag{13.111}$$

$$W = 3S_p \tag{13.112}$$

Where,

L_a = length of apron measured from end section or face of end wall, m
S_p = inside diameter/span of culvert, m
Q = design discharge, m^3/s
TW = tailwater depth, m
R_p = maximum inside culvert rise, m
W = width of apron, m
$S_p = R_p$ = inside diameter/span of culvert, m

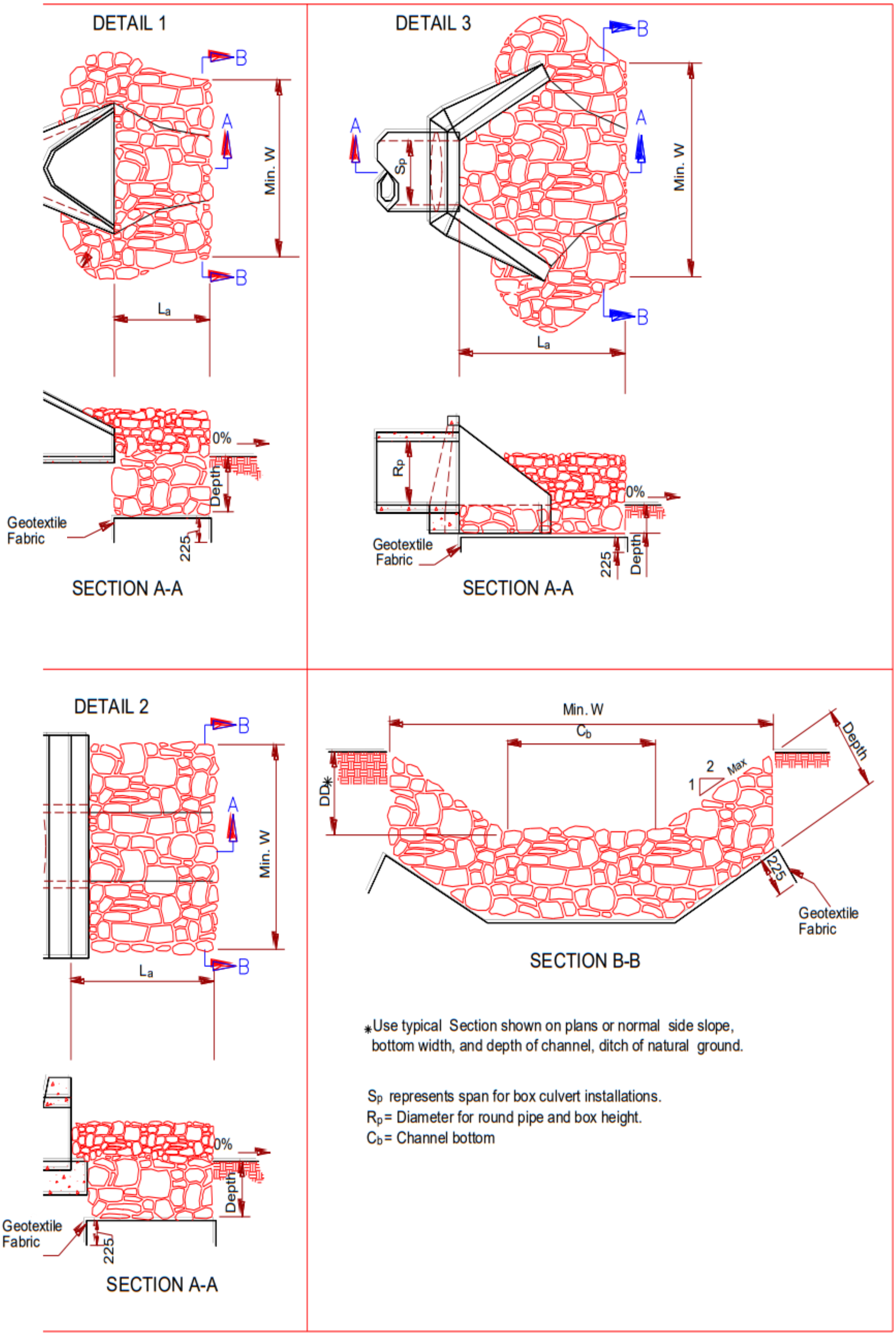


Figure 13.62 Culvert End Treatment

Example 13.24 Culvert Design Procedure

This example describes a culvert design procedure for a road crossing of a small channel (see **Figure 13.63** and **Figure 13.64**), based on the following requirements and data:

Design discharge (Q) is 6.25 m^3/s for a 50 year ARI event.

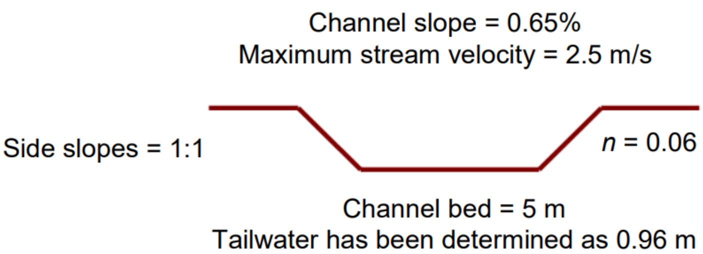


Figure 13.63 Channel cross-section

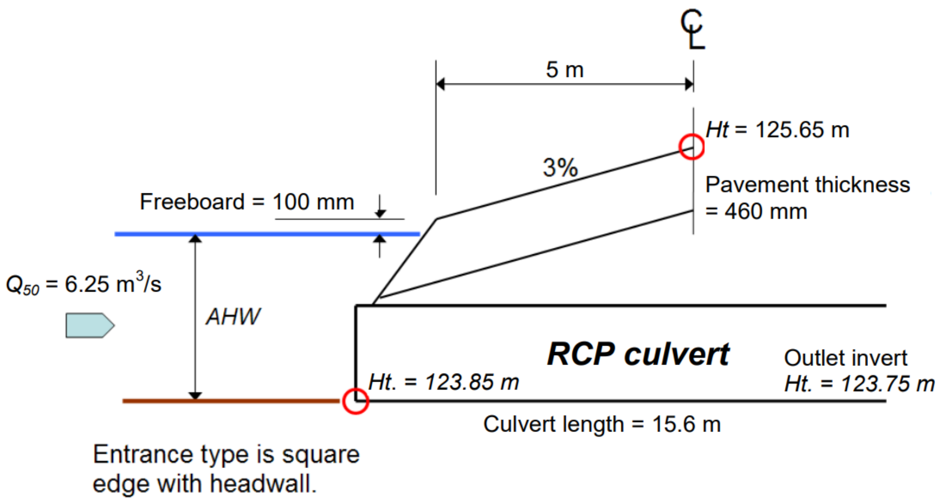


Figure 13.64 Road cross-section for worked example

Solution

Step 1 – Collect design data

Use the hydraulic calculation form (**Figure 13.54**) and fill out the known information.

The shoulder height can be calculated based on the centreline height, road crossfall and traffic lane width.

Shoulder height = Centreline height – road crossfall

= 125.65 m – 5 m x 3%

= 125.5 m

A 100 mm freeboard below shoulder height is required, therefore the Allowable Headwater depth (AHW) will be:

AHW = Shoulder height – invert height at outlet – freeboard

AHW = 125.50 – 123.85 – 0.100 = 1.55 m

The tailwater depth can be obtained from the Stage-Discharge curve shown in **Figure 13.65**.

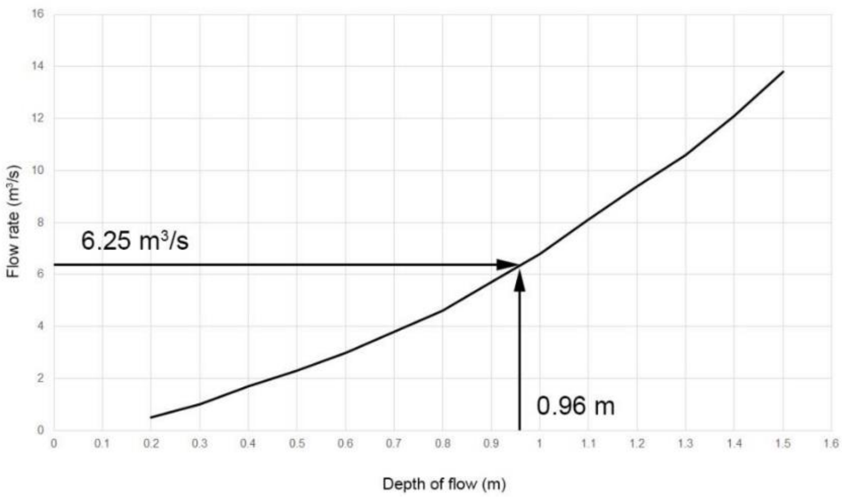


Figure 13.65 Stage-Discharge curve for culvert design example

Complete top section of the design form as follows:

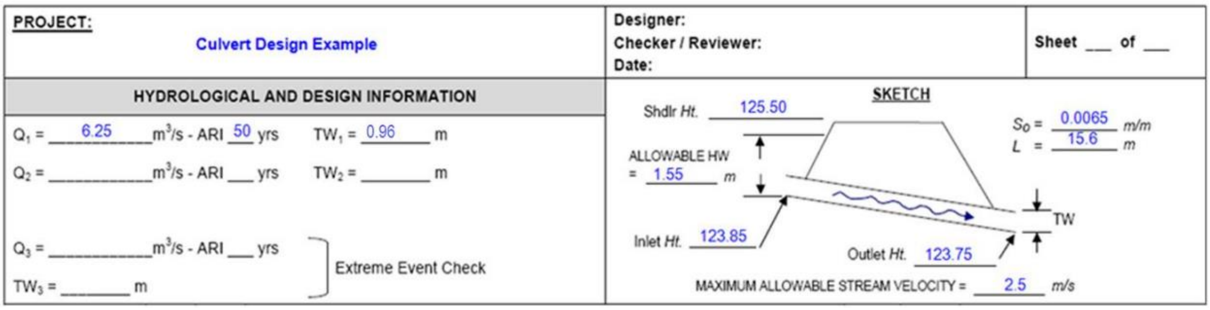


Figure 13.66 Geometric detail of key parameters for worked example

Step 2 – Select a trial culvert

Firstly, determine an indicative waterway area of culvert(s) based on Q and maximum allowable velocity.

A = Q/V

= 6.25/2.5

= 2.50 m^2

Use area to select an initial culvert configuration that meets the area required. Size and fit the culvert to the channel/road.

At this stage, and based on preliminary considerations given above, a Concrete Pipe Culvert ≥ 1200mm will not fit as the height of the barrel will be within the pavement. That is:

- Maximum barrel height = 125.5 – 0.460 – 123.85 = 1.19 m
- Initial trial size selection area = 2.50 m^2
- Try 4/900mm PC, Area = 2.54 m^2
- (Pipe D ≤ 1.19 – OK).

Step 3 – Design discharge for trials

For multiple cells culvert determine Q per cell:

Q per cell = 6.25/4 = 1.56 m^3/s

Step 4 – Determine inlet control headwater depth

- Analyse the culvert assuming inlet control.
- Use the nomograph **Chart 16** to determine HW/D and then calculate HW_i.
- Q = 1.56 m^3/s, D = 900 mm, from nomograph, HW/D = 1.46.
- HW_i = HW/D x D = 1.46 x 1.05 = HW_i = 1.53 m < AHW (1.55 m).

Step 5 – Determine outlet control headwater depth

Analyse the culvert assuming outlet control.

- Select the entrance loss coefficient in accordance with the project description (see **Figure 13.64**) above.
  The culvert entrance type is a square edge with headwall, therefore k_e = 0.5 (see **Table 13.49**).
- Determine head (H) for pipe culvert flowing full from **Chart 20**.
- Plot L = 15.6 m on k_e = 0.5, plot D = 900mm and draw line.
- Locate pivot on turning line.
- Plot Q = 1.56 m^3/s and draw line crossing pivot on turning line to Head line.
- Read H = 0.53 m.
- Determine the critical depth in a pipe, d_c from the nomograph **Chart 19**.
- d_c = 0.73 m.
- Calculate $(d_c + D)/2$ = (0.73 + 0.9)/2 = 0.815 m.
- Determine the tailwater. TW was earlier determined as 0.96 m (**Figure 13.65**).
- Determine h_o, which is the greater of TW and (dc + D)/2, therefore h_o = 0.96 m.
- Calculate length x slope = 15.6 m x 0.0065 m/m = 0.101 m.
- Determine $HW_O = H + h_o - LS_o$
  $= 0.53 + 0.96 - 0.101$
  = 1.39 m < AHW (1.55 m).

Step 6 – Determine the controlling headwater (HW)

- The controlling headwater is determined from the larger of
  HW_i = 1.53 m and HW_O = 1.39 m
- Therefore, the 4/900 mm diameter concrete pipe culverts are operating under inlet control.

Step 7 – Calculate outlet velocity – inlet control pipe

- Determine relationship of part full pipe (Q_p) and full flow pipe (Q_f) = (Q_p/Q_f).
- Using the nomograph in **Chart 26**, Q_f = 2.4 m^3/s and V_f = 2.5 m/s.
- Q_p/Q_f = 2.08/2.4 = 87%.
- Using the nomograph in **Chart 27** obtain the following factors:

  – 71.5% depth of flow, therefore y = 1.05 x 71.5% = 0.75 m

  – 77% waterway area, therefore A = 0.87 m^2 x 77% = 0.67 m^2

  – 112% outlet velocity, therefore V = 2.75 x 112% = 3.08 m/s.

This work can be tabulated as follows (see **Table 13.50**):

Variable	Pipe full	Applied factor	Pipe part
Q	2.40	-	2.08
V_o	2.75	1.12	3.08
y	1.05	0.715	0.75
A	0.867	0.77	0.67

Step 8 – Calculate outlet flow energy

Calculate the Froude Number (F_r) (**Equation (13.108)**):

$$F_r = \frac{V_{outlet}}{\sqrt{g(A/_B)}}$$

In order to calculate the Froude Number, calculate B (top width of flow) using **Equation (13.109)**:

$$B = 2\sqrt{y(D-y)}$$

Where,

y = depth flow = 0.715 x 1.05 = 0.75 m

$$B = 2\sqrt{0.75(1.05-0.75)}$$

B = 0.95 m

Therefore:

$$F_r = \frac{3.08}{\sqrt{9.81(0.67/_{0.95})}} = 1.17\ (supercritical\ flow)$$

Since F_r < 1.7 a rock pad will be suitable.

Step 9 – Design check

The outlet velocity = 3.08 m/s and is therefore greater than allowable stream velocity = 2.5 m/s. It is recommended that a suitable type of erosion control measure be determined.

Example 13.25 Culvert Design Procedure

Determine an appropriate culvert size given the following design data:

(a) Design discharge Q_{50} = 19.3 m³/s

(b) Allowable outlet velocity (with standard outlet protection). V_a = 1.8 m/s

(c) Flood level in natural channel F.L. = 31.8 m

(d) Invert level of channel at outlet IL_0 = 30.0 m

(e) Slope of culvert in metres per metre. S_0 = 0.01 m/m

(f) Allowable headwater depth in metres. AHW = 2.0 m

(g) Mean velocity = 0.55 m/s. Maximum velocity = 0.61 m/s.

From Manning's formula, maximum velocity is in the main channel and the mean velocity is over the total section including the overflow.

Solution

Step 1 - Determine the first trial culvert size:

(a) From the equation A = Q/V area of waterway: A = 19.3/1.8 m^2 = 10.72 m^2

(b) Select a culvert size where the soffit would be above or just below the tailwater to

utilise the full waterway opening, if practical.

Depth of Tailwater = Flood level - Invert

level of natural channel = 31.8 - 30.0 = 1.8m

Assuming a height of 1.5 m and selecting a standard size R.C. slab deck culvert:

Try 3/2400 x 1500 mm culvert (Area 10.80 m^2)

Step 2 - Find the headwater depth for the trial culvert

(a) Assume INLET CONTROL

Determine the discharge (Q) per cell i.e. Q = 19.3 / 3 = 6.43 m^3/s

Find the ratio of discharge to width Q/B =2.68 m^3/s per metre, then using the nomograph in **Chart 16** determine the HW/D ratio.

HW/D = 0.91

HW= 0.91 x 1.5= 1.37 m

This headwater is satisfactory as it is less than the allowable given in the design data, item (f).

(b) Assume OUTLET CONTROL

Since the tailwater level is above the culvert soffit at the outlet, the HW can be calculated from the equation:

$HW = H + h_o - LS_o$

$h_o = TW = 1.8$ m

$LS_o = 12 \times 0.01 = 0.12$

From the nomograph in **Chart 20** and using $k_e = 0.4$ from **Table 13.49**, for area of box = 2.4x1.5 = 3.60 m^2, H = 0.26 m

Therefore,

HW = 0.26 + 1.8 - 0.12 = 1.94 m

(c) As the HW for outlet control is higher than that for inlet control, outlet control is the governing factor; and being less than the allowable height of 2.0 m is acceptable.

Step 3

Try a culvert of another type or shape, if a comparison of alternative design costs is to be made and determine size and headwater by the above procedure.

Step 4

Compute the velocity through the 3/2400 x 1500 RCBC.

Since outlet control governs, and the tailwater is above the soffit of the culvert, the full waterway area is used.

Outlet velocity = Q/A_o = 19.3/10.8 = 1.79 m/s

Step 5

Make selection and record all relevant data on **Figure 13.54** or similar form.

Example 13.26 Culvert Outlet Protection

The following example demonstrates the procedure to determine the dimensions of a rock apron required to provide outlet protection against outlet velocities that are higher than acceptable.

For velocities in excess of 5 m/s, the use of energy dissipators should be considered and specialist advice should be acquired.

This example will demonstrate the design of an erosion control measure for a 1200 mm diameter RCPs (inlet control) that have been designed to convey a total discharge of 2.25 m^3/s. The maximum allowable stream velocity for the channel is 2.0 m/s. The depth and velocity at the outlet are 64% of the depth of full flow and 2.97 m/s (per cell) respectively.

Step 1 - First determine a suitable type of control measure by calculating the Froude Number

(F_r) (**Equation (13.108)**):

$$F_r = \frac{V}{\sqrt{g\left({}^{A}/_{B}\right)}}$$

In order to calculate the Froude Number, it is necessary to calculate area of flow at the outlet (A_o) and the top width of flow (B).

- $A_o = Q/V_o$
- $A_o = 2.25/2.97$
- $A_o = 0.76\ m^2$

Using **Equation (13.109)**:

$$B = 2\sqrt{y(D-y)}$$

Where,

y = 64% depth of full flow = 0.4 x 1.2 = 0.77 m:

$$B = 2\sqrt{0.77(1.2-0.77)} = 1.15m$$

$$F_r = \frac{2.97}{\sqrt{9.81\left({}^{0.76}/_{1.15}\right)}} = 1.16\ (supercritical\ flow)$$

Since F_r < 1.7, a rock pad will be suitable

Step 2 - Determine stone size and length of protection as shown below:

From **Table 13.48**, with an outlet velocity of 2.97m/s, adopt riprap Class I (D_{50}=335mm)

Calculate TW=(dc + D)/2, read d_c from **Chart 19**

TW= (0.82+ 1.2)/2=1.01m>0.5R_p, where R_p is the inside diameter of culvert (1.2m)

Therefore, use **Equation (13.111)** to determine L_a (length of protection)

$$L_a = \frac{5.44(Q - 0.142)}{{S_p}^{1.5}} + 3.05$$

$$L_a = \frac{5.44(2.25 - 0.142)}{1.2^{1.5}} + 3.05 = 11.8\text{m}$$

Step 3 - Determine rock apron dimensions based on a 5:1 expansion ratio. See **Figure 13.67**.

The outside edge of the normal concrete apron will extend 1.8 m beyond the outlet with a 30° wingwall flare.

The overall width between the wingwalls is therefore= 1.2 + 2(1.8 x tan 30°) = 3.3 m.

The width of the rock apron at its downstream extremity, at a 5:1 expansion ratio is therefore,

= 3.3 + 2 (11.8/5) = 8.0 m. See **Figure 13.67**.

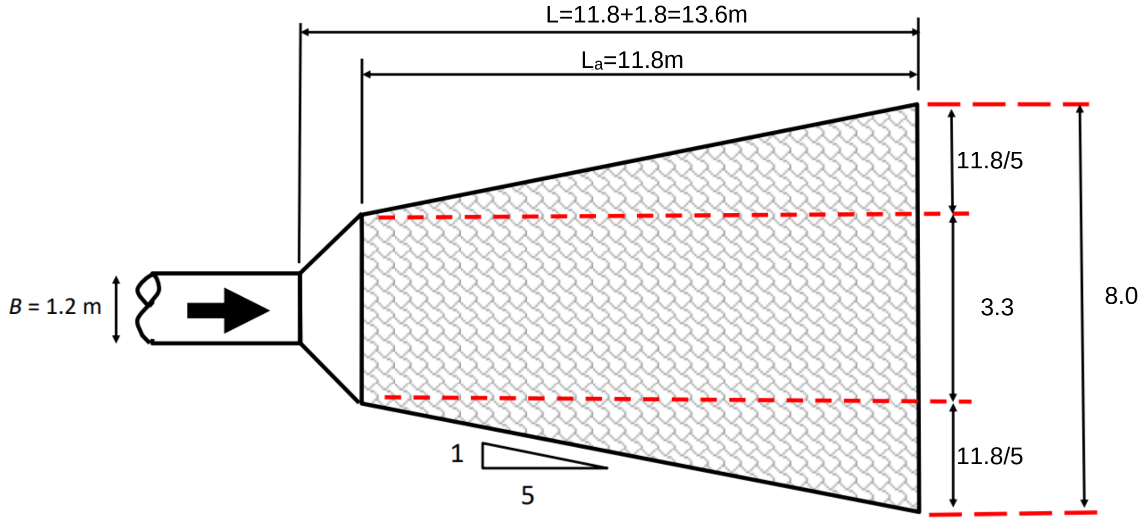


Figure 13.67 Solution to example for culvert outlet protection

13.7 HYDRAULIC DESIGN OF BRIDGES
13.7.1 INTRODUCTION

Bridges serve a variety of highway purposes including the elimination of conflicts with traffic and other modes of transportation, such as rail, marine, air, and pedestrian. Bridges enable watercourses to maintain the natural function of flow conveyance and sustain aquatic life. Bridges are also important and expensive highway-hydraulic structures and are vulnerable to failure from flood-related causes.

Structures measuring more than 6 m along the roadway centreline are conventionally classified as bridges. From a hydraulic perspective, a bridge is defined as:

- A structure built over a depression or obstacle for passageway.
- Part of a stream crossing system that includes the approach roadway across the floodplain and any openings.

Any structure designed hydraulically to operate in free surface flow at the design event is treated as a bridge in this Section, regardless of actual length.

To minimize the risk of failure, the hydraulic requirements of stream crossings must be recognized and considered in all phases of highway development, construction, and maintenance.

Features that are important to the hydraulic performance of a bridge include the approach fill alignment, skew, and profile; bridge location, skew, and length; span lengths; bent and pier location and design; and foundation and superstructure configuration and elevations. These features of a highway-stream crossing are usually the responsibility of location, design, and bridge engineers; however, the integrity and safety of the facility are often as dependent upon competent hydraulic design as on competent structural and geometric design.

In addition to the choices regarding hydrologic and hydraulic components of a bridge hydraulic analysis there are many other factors and requirements to consider.

One early consideration is the level of service the bridge is expected to provide. If the bridge is remote and carries a low volume of traffic, it can be designed with a lower hydraulic capacity resulting in a smaller and less expensive bridge. This means that the bridge and/or approach roadways will be overtopped more frequently, and the bridge owner can expect the bridge and approach roadways to require more frequent maintenance and repair. On the other hand, if the bridge is on an important route such that significant hardships or economic impacts would be encountered if the bridge were out of service, then it should be designed with a higher hydraulic capacity resulting in a larger and more expensive bridge and higher approach embankments. These bridges and/or approach roadways would be rarely overtopped and would need less frequent maintenance or repair. A smaller bridge may be less expensive from a capital (initial) cost perspective, but this does not necessarily always hold true from a life-cycle cost perspective. Most states or local jurisdictions have policies and criteria that govern the level of service expected from their roadways and bridges.

13.7.2 RIVERS AND RIVER CROSSINGS

13.7.2.1 RIVER MORPHOLOGY

Morphology is a study of forms, and geomorphology is a study of the development, configuration and distribution, or form, of the earth's surface. Fluvial or stream geomorphology is a study of the development and configuration of the earth's surface as formed by rivers. Many rivers can change pattern, dimension, and orientation as the result of one flood; others change at a much slower rate. All rivers change with time, and the rate and manner in which they will change can be recognized.

Planning and location engineers should be very conscious of river morphology and aware that methods are available for quantifying natural changes and changes that can occur as the result of stream encroachments and crossings.

River channels have inherent dynamic qualities by which changes continually occur in the stream position and shape. Changes may be slow or rapid, but all rivers are subjected to the forces that cause changes to occur. In alluvial rivers (i.e., rivers whose beds and banks are composed of materials deposited in water), it is the rule rather than the exception that banks will erode, sediments will be deposited, and islands and side channels will undergo changes with time.

13.7.2.2 TYPES OF RIVERS

Rivers can be divided into those with floodplains and those without. Floodplains are usually not the direct result of large flood flows, but rather the result of lateral movement of the river from one side of the plain to the other over geological time. By definition, the floodplain is low enough to be completely inundated by floods with fairly low return periods.

Rivers can be further classified as either braided, straight or meandering, whether they have floodplains or not. The character of each classification is shown in **Figure 13.68**.

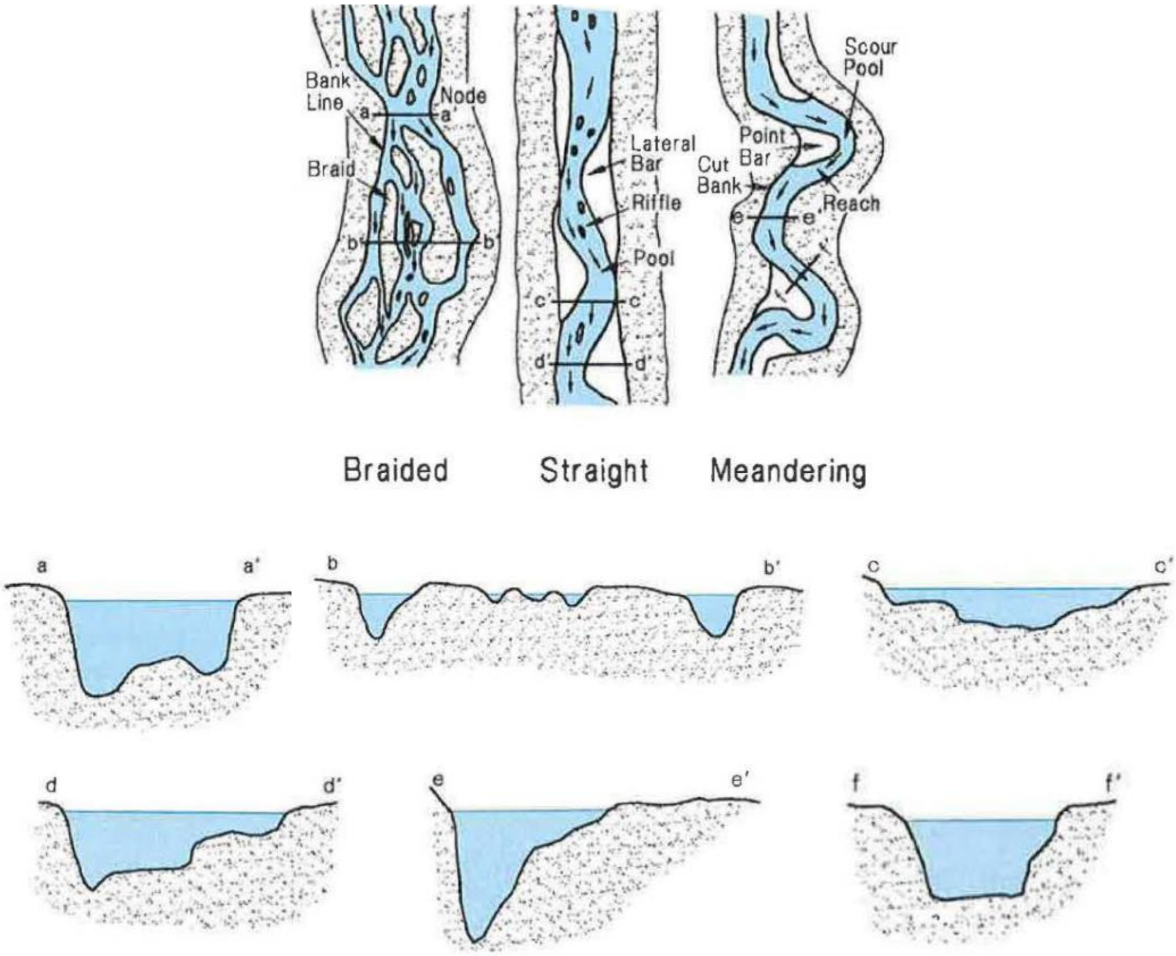


Figure 13.68 River channel patterns

Braided channels can, in some circumstances, be extremely unstable, with the channels shifting with each sharp change in discharge. This has resulted in many crossing failures.

As seen in **Figure 13.68**, even straight rivers are to some degree sinuous. The sinuosity is a measure of this meandering feature. The sinuosity is defined as the ratio of the river's thalweg to the length of the valley proper. The thalweg is the path of deepest flow. Rivers with sinuosity less than 1.5 are usually considered straight.

Meandering rivers are commonly associated with erodible floodplains, although very regular and highly developed meanders have occurred in rivers incised in solid rock.

13.7.2.3 DYNAMICS OF NATURAL RIVERS

Frequently, environmentalists and hydraulic engineers consider a river to be static, i.e. unchanging in shape, dimensions and pattern. However, an alluvial river continually changes its position and shape as a consequence of hydraulic forces acting on its bed and banks. These changes may be slow or rapid and may result from natural environmental changes or from changes by man's activities.

A river that may appear stable may be relatively unstable because of the slow but implacable shift of a channel through erosion and deposition at bends, the shift of a channel to form chutes and islands, and the cut-off of a bend to form oxbow lakes. These changes are dependent on flood events, bank stability, permanence of vegetation on banks and floodplain land use.

Experience has shown that modifications to perennial rivers can be quite extensive, however, the dynamic effects of rivers in arid and semi-arid regions, and especially ephemeral (flowing occasionally) stream channels can be even more extensive and dramatic. When an engineer modifies a river channel locally, this local change can frequently modify channel characteristics both up and down stream. The response of a river to man-induced changes often occurs in spite of attempts by engineers to keep the anticipated response under control.

All rivers are governed by the same basic forces which determine the response to man-induced changes. The design engineer should have an understanding of these forces which include:

i. geological factors, including soil conditions
ii. hydrological factors, including possible changes to runoff and the hydrological effects of changes in land use
iii. geometric characteristics of the stream, including the probable geometric alterations which may occur as a result of man's actions
iv. hydraulic characteristics such as depths, slopes and velocity and what changes may be expected in these characteristics in space and time.

13.7.2.3.1 RIVER HYDRAULICS

As just stated, rivers are dynamic and respond to changed environmental conditions. The extent of the change depends on the forces acting on the system. The mechanics of flow in rivers is a complex subject, the major complicating factors of which are:

i. the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a river system
ii. the continual evolution of river channel patterns, channel geometry, bars and forms of bed roughness with changing water and sediment discharge.

In order to understand the responses of a river to the actions of man and nature, a few hydraulic and geomorphic concepts are now presented.

As stated in **Section 13.7.2.2**, river forms are broadly classified as straight, meandering, braided or some combination of these classifications. However, any changes that are imposed on a river may change its form. The dependence of river form on the slope, which may be imposed independent of the other river characteristics, is illustrated schematically in **Figure 13.69**. By changing the slope, it is possible to change the river from a meandering one that is relatively tranquil and easy to control to a braided one that varies rapidly with time, has high velocities, is subdivided by sandbars and carries relatively large quantities of sediment. Such a change could be caused by the natural or artificial shortening of the length of the stream. Conversely, it is possible that a slight decrease in slope could change an unstable braided river into a meandering one. The significantly different channel dimensions, shapes and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel form can be anticipated.

An insight into the direction of change, the magnitude of change, and the time involved to reach a new equilibrium can be gained by studying the river in a natural condition; having knowledge of the sediment and water discharge; being able to predict the effects and magnitude of man's future activities; and applying to these a knowledge of geology, soils, hydrology and hydraulics of alluvial rivers.

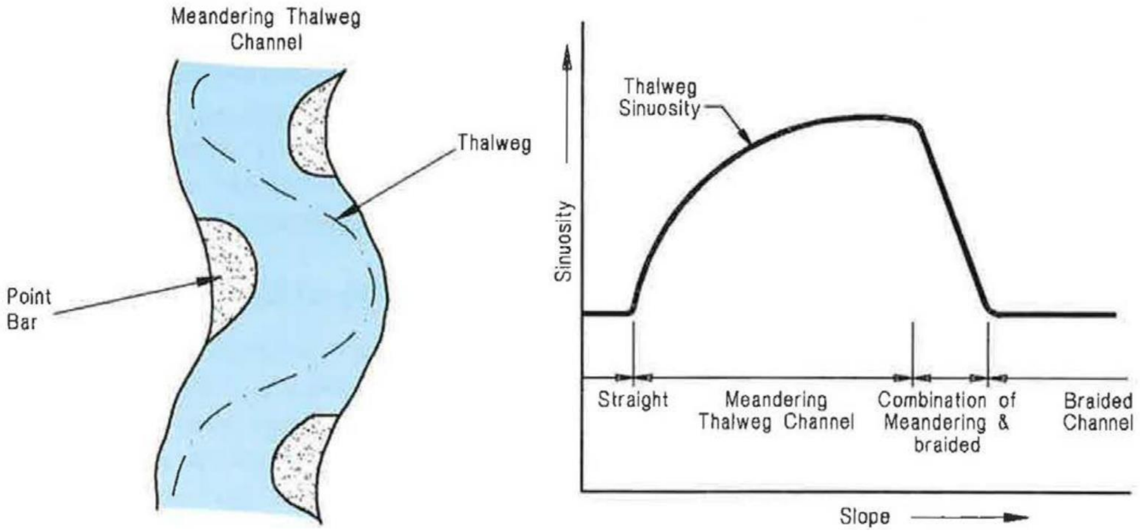


Figure 13.69 Sinuosity versus slope with constant discharge

13.7.2.4 TYPES OF ENCROACHMENT

Stream crossings generally impose some degree of encroachment of the river or floodplain. In some instances, particularly in mountainous regions or in river gorges, river crossings can be accomplished with absolutely no encroachment on the river. The bridge and approaches can be located far above and beyond any possible flood stage. More commonly, the economics of crossings may require substantial encroachment on the river and its floodplain. The encroachment can be in the form of earth fill embankments over the floodplain or into the main channel itself, reducing the required bridge length.

13.7.2.5 GEOMETRY OF BRIDGE CROSSINGS

The geometric properties of stream crossings are dependent on the conditions at the site. Abutments may be of the spill through type or vertical wall type. The approaches may be skewed or normal to the direction of flow, or one approach may be longer than the other, producing an eccentric crossing.

Abutments may be set back from the channel banks to provide room to pass the flood flow or simply to provide access under the bridge, or the abutments may extend up to the banks or even protrude over the banks, constricting the low flow channel. Piers, dual bridges for multi-lane highways, channel bed conditions, and guide banks add to the list of geometric classifications.

13.7.2.6 EFFECTS OF BRIDGE CONSTRUCTION ON RIVER SYSTEMS

Bridge construction can have significant general and local effects on the geomorphology and hydraulics of river systems. Hence, it is necessary to consider induced short-term and long-term responses of the river, the impact on environmental factors, the aesthetics of the river environment and short-term and long-term effects of erosion and sedimentation on the surrounding landscape and the river.

A. Immediate responses

Constrictions generally lead to general and local scour and the sediments removed through this action are usually dropped in the immediate reach downstream. In addition, the development of crossings and the constrictions of river sections may have a significant effect on the water level in the vicinity and upstream of the bridge.

As a consequence of the construction of bridges and their approaches, areas adjacent to the bridge site become highly susceptible to erosion. Rainfall causes surface runoff and the accompanying erosion can significantly increase the sediment yield to the river unless careful control is exercised. Fine sediments are easily transported and generally pollute the river downstream.

B. Delayed responses

Often it is necessary to employ training works in connection with bridges to favourably align flow with the bridge openings. When such training works are used, they can straighten the channel, shorten the flow line and increase the velocity within the channel. Any such changes made in the system that cause an increase in the gradient may cause an increase in velocities. The increase in velocity increases local and general scour with subsequent deposition downstream where the channel takes on its normal characteristics.

13.7.3 ENVIRONMENTAL CONSIDERATIONS

As the preservation of environmental quality has become a matter of national interest and priority, the decisions made on the location, design and construction of river crossings should, as far as possible, avoid or minimise the adverse effects on scenic, natural, historical, archaeological, recreational and social values and resources of the project area. To do this, the engineer must have sufficient knowledge to recognise potential problems.

Environmental impact must be included in the investigation of all bridge sites. To achieve this the following steps should be taken:

i. Several alternative bridge sites should be identified based on both environmental and technical considerations. An inventory should be made of the environmental resources at each site. The inventory should check the area for its uniqueness in supporting specific vegetation, wildlife and aquatic life, especially rare and endangered species. Where necessary relevant specialists should be consulted to provide advice and evaluate the response of the species to stress.
ii. If necessary, an interdisciplinary study should then be made of the impact, at each site, of construction on the environmental factors identified in the first step. The impact of alternative forms of construction should also be included in this study.
iii. Preliminary plans and cost estimates that provide the best compromise between functional cost and environmental impact should then be prepared for each site. These plans should include measures to rehabilitate the sites and ameliorate any expected harmful impacts. The final choice can then be made with an understanding of the impact of the crossing.

Specific considerations pertinent to site selection and bridge construction are described in more detail below.

A. Site Selection

While the site selection of a bridge is usually closely dictated by the location planning of the proposed road, the site must be selected with full knowledge of its environmental impact.

To minimise the environmental impact the site should be located where:

i. satisfactory geological and soil conditions exist
ii. it is away from the reaches of highly-unstable channels
iii. possible adverse effects on other existing bridges and hydraulic structures can be avoided
iv. it is possible to minimise the hazards from floods, landslides, cyclones, earthquake and subsidence

v. river banks are stable
vi. ecological impact is acceptable
vii. aesthetic considerations are favourable.

B. Construction Effects

Although construction may be of short duration as compared to the operating life of the project, some changes during construction could have long-term damaging effects. The impact of each of the construction activities on the environment should be assessed and measures taken to minimise such impact. Erosion control and other pollution control measures, the impact on water supplies, and restoration of the landscape after completion of construction must be considered.

13.7.4 FREE-SURFACE BRIDGE FLOW

Free-surface bridge flow refers to the range of flow conditions at a specific bridge in which the bridge low chord is not submerged. There are three types of flow that may be encountered in a bridge waterway design. These are labelled Types I, II and III as shown in **Figure 13.70**. The long-dashed lines shown on each profile represent normal water surface, or the stage the design flow would assume prior to placing a constriction in a channel. The solid lines represent the configuration of the water surface on the centre line of the channel in each case, after the bridge is in place. The short dashed lines represent critical depth, or critical stage in the main channel (y_{1c} and y_{4c}) and critical depth within the constriction, y_{2c}, for the design discharge in each case.

Since normal depth is shown essentially the same in all four profiles, the discharge and slope of the channel must increase in passing from Type I to Type IIA, to Type IIB, to Type III flow.

Type I flow – referring to **Figure 13.70-A**, it can be seen that normal water surface is everywhere above critical depth. This has been labelled Type I or subcritical flow, the type usually encountered in practice. The backwater expression for Type I flow is obtained by applying the conservation of energy principle between sections 1 and 4.

Type IIA flow – there are two variations of Type II flow, which will be described here as Types IIA and IIB.

For Type IIA flow (**Figure 13.70-B**), normal water surface in the unconstricted channel again remains above the critical depth throughout, but the water surface passes through critical depth in the constriction. Once the critical depth is penetrated, the water surface upstream from the constriction, and thus the backwater, becomes independent of conditions downstream (even though the water surface returns to normal at section 4). Thus, the backwater expression for Type I flow is not valid for Type II flow.

Type IIB flow – The water surface for Type IIB flow (**Figure 13.70-C**), starts out above both the normal water surface and the critical depth upstream and passes through the critical depth in the constriction. It then dips below the critical depth downstream from the constriction and then returns to normal. The return to normal depth can be rather abrupt as shown on **Figure 13.70-C**, taking place in the form of a poor hydraulic jump, since normal water surface in the stream is above the critical depth. A backwater expression applicable to both Types IIA and IIB flow has been developed by equating the total energy between section I and the point at which the water surface passes through critical stage in the constriction.

Type III flow – in type III flow (**Figure 13.70-D**), the normal water surface is everywhere below the critical depth and the flow throughout is supercritical. This is an unusual case requiring a steep gradient. However, such conditions do exist, particularly in mountainous regions. Theoretically, backwater should not occur for this type, since the flow throughout is supercritical. It is more than likely that an undulation of the water surface will occur in the vicinity of the constriction, however, as indicated on **Figure 13.70-D**.

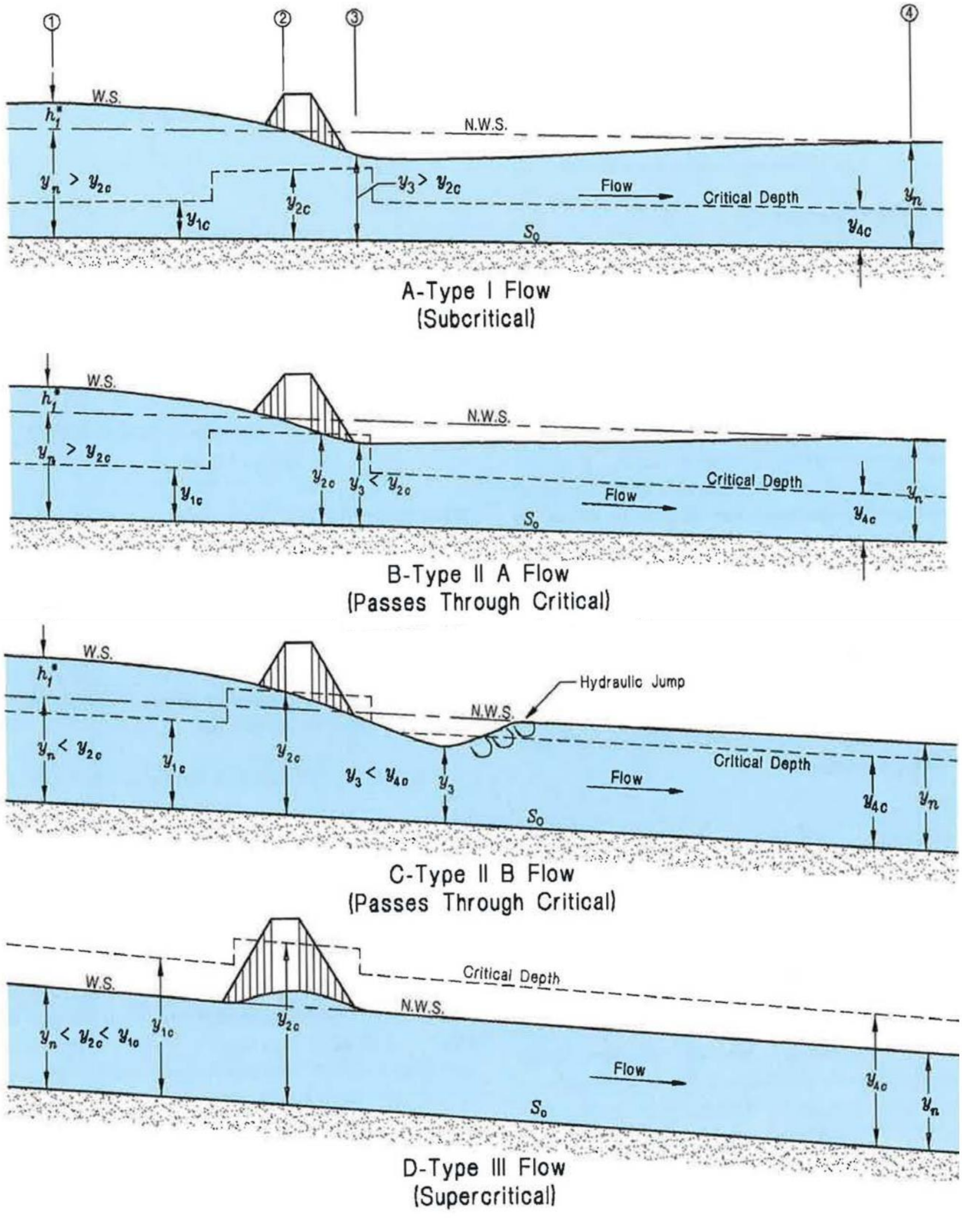


Figure 13.70 Types of flow

Open channel flow can be classified in many ways. Flow can be classified as either steady or unsteady. Flow can also be classified as uniform or non-uniform (varied). Non-uniform flow can be further classified as gradually varied or rapidly varied. The flow can also be subcritical or supercritical, and depending upon the turbulence of the flow field, the flow can be classified as laminar (low Re) or turbulent (high Re). As just discussed, there are four classifications needed to describe the type of flow in an open channel:

i. uniform or non-uniform
ii. steady or unsteady
iii. laminar or turbulent
iv. subcritical or supercritical.

One from each of these four types must exist. Because the classifying characteristics are independent, 16 different types of flow can occur. In most open channel flow problems related to the hydraulics of bridges, culverts and floodways, it is sufficient to study flow behaviour under steady flow conditions only. Although flow in natural channels is almost always unsteady, it is often analysed in a quasi-steady state, unless special circumstances prevail. This is due to the fact that, while flow may vary over time, it varies over a long enough time that it can be treated as steady flow.

13.7.5 BRIDGE DESIGN CONSIDERATIONS

Hydraulic factors that should be considered in the design of bridges include:

i. bridge opening and road grade
ii. bridge location selection
iii. scour and stream stability
iv. bridge design specifications and design criteria.

These factors are now briefly described.

13.7.5.1 BRIDGE OPENING AND ROAD GRADE DESIGN CONSIDERATIONS

In general, given a particular design discharge at a given crossing, the shorter a bridge the more backwater it will create. This same smaller bridge will also have higher velocities through the bridge opening and an increased potential for scour at the bridge foundation. A longer bridge at this same crossing will generate a smaller amount of backwater and will have lower velocities and potential for scour. Policy considerations and economics require an understanding of the impacts that the bridge could have on the flow of water in the floodplain and impacts it might have on adjacent properties.

The bridge waterway width is directly associated with the bridge length, from abutment to abutment. Hydraulic capacity should be a primary consideration in setting the bridge length. The bridge must provide enough capacity to:

- Avoid excessive backwater in order to prevent adverse floodplain impacts
- Prevent excessive velocity and shear stress within the bridge waterway

Freeboard refers to the vertical distance from the water surface upstream of the bridge to the low chord of the bridge. The freeboard requirement is associated with a particular design recurrence-interval event, which is usually the 50- or 100-year event. Rural, low-traffic routes often allow a lower recurrence interval for establishing hydraulic capacity and freeboard.

The road profile can have a significant effect on bridge crossing hydraulics. Even if a bridge is designed to provide freeboard above a 100-year flood, the approach roadways may be overtopped by that same flood. When the overtopping occurs over a long segment of roadway, the associated weir flow is an important component of the overall hydraulic capacity of the crossing. In such a case, raising the road profile will have the potential to increase backwater unless additional capacity is provided in the bridge waterway to compensate for the lost roadway overtopping flow capacity.

The design of the piers and abutments has an effect on the bridge hydraulic capacity. Although this effect is small compared to the bridge length and road profile, it can still be important. For example, a bridge that crosses a regulatory floodway must be shown to cause no increase in backwater over existing conditions. In such a case the energy losses that are affected by the number of piers and their geometry can be significant. Spill-through abutments, set well back from the tops of the main channel banks, are advisable when bridge hydraulic capacity must be optimized.

Frequently the bridge waterway design includes subtle changes to the channel cross section under the bridge and for a short distance upstream and downstream of the bridge. These changes are intended to enhance channel stability and, in some cases, to improve hydraulic efficiency. Channel stability can be enhanced, for instance, by grading the channel banks to side slopes of 2H:1V or flatter, and by providing channel bank revetment. Capacity can be improved by a moderate widening of the channel bottom in the immediate vicinity of the bridge, with appropriate width transitions upstream and downstream.

There are several potential bridge opening and road grade considerations that impact hydraulic capacity and upstream flood risk, especially when a road is upgraded and the bridge is replaced. These include bridge length, deck width, abutment configuration (spill through or vertical wall), number and size of piers, low chord elevation, freeboard, and road grade. If a crossing with a 25-year level of service is improved to a 50-year level of service, the road elevation may need to be increased. To avoid increased flood risk, the replacement bridge may need to be considerably longer and higher than the existing bridge. If there is inadequate freeboard, debris may collect along the deck and reduce flow conveyance.

13.7.5.2 BRIDGE LOCATION SELECTION AND ORIENTATION GUIDELINES

A. Bridge location selection

Some of the factors considered in the selection of bridge locations are:

i. the safety of the highway user,
ii. user costs,
iii. vertical and horizontal highway alignment,
iv. construction and maintenance costs,
v. foundation conditions,
vi. availability of funds,
vii. traffic needs,
viii. navigation requirements,
ix. development within the highway corridor,
x. social costs,
xi. natural resources,
xii. political considerations,
xiii. public opinion,
xiv. stream regime,
xv. environmental considerations, and
xvi. hazards from floods.

Situating the bridge at the proper location within the floodplain can greatly influence the performance and service life of the crossing. If possible, the crossing should be located based on the following considerations:

i. minimize skew,
ii. be located at the narrowest portion of the floodplain,
iii. be located on a stable reach of stream,

iv. minimize impacts of meander migration, and
v. have appropriately located auxiliary/relief openings (if needed).

Although the aforementioned factors, including non-technical ones, are used to determine the final location of a stream-crossing system, the hydraulics of the proposed location must have a high priority. Hydraulic considerations in selecting the location include flood plain width and roughness, flow distribution and direction, stream type (braided, straight, or meandering), stream regime (aggrading, degrading, or equilibrium), and stream controls. The hydraulics of a proposed location also affect environmental considerations such as aquatic life, wetlands, sedimentation, and stream stability. Finally, the hydraulics of a particular site determine whether or not certain national objectives such as wise use of flood plains, reduction of flooding losses, and preservative of wetlands can be met.

Where practicable, the alignment of the bridge should be chosen to avoid unstable sections of a watercourse channel, such as sharp or obviously mobile channel bends. When it is considered necessary to realign a waterway channel as part of a bridge design, the following issues and concerns should be investigated and appropriately addressed:

i. potential environmental impacts of sediment run-off from the construction of the new channel
ii. erosion potential of the downstream channel in response to the proposed realignment
iii. possible changes to existing bed conditions, including pool-riffle systems, within the channel
iv. the need for rock protection of the channel bed and banks given the potential to adversely affect the continuity and health of riparian vegetation and consequently the quality of the wildlife corridor
v. any reduction in the length of the main channel and a consequential increase in the hydraulic gradient and erosion potential
vi. the form, condition and location of the low-flow channel; where practical, all of these should be maintained.

The location of the low-flow channel can have a significant effect on channel stability and aquatic habitat values. It can also meander within the bed of the main channel and the form, condition and location of the low-flow channel can vary from flood event to flood event.

The selection of bridge waterway openings also takes into consideration the size and type of debris from upstream (**Plate 13.14**). Factors to be considered include:

i. hydrodynamic forces without debris
ii. forces due to debris mats
iii. forces due to log impact
iv. urban debris, e.g. shipping containers and vehicles.

Where large logs, trees and urban debris can be anticipated, consideration shall be given to increasing both the span length and the freeboard to permit passage of debris.

Piers and abutments shall be designed to minimize their effects on water flows and avoid the trapping of debris where this is considered likely, as well as remain stable after the effects of scour. If piers must be located within the channel, and if a pool is likely to form within the channel at the bridge location, then the foundation design must allow for future bed erosion.

Abutment slopes and the underlying material shall be designed for stability and shall be protected against erosion effects for the design flood velocities.

Plate 13.14 Debris accumulated upstream of a bridge

B. Bridge orientation guidelines

For selecting and orienting bridge locations, consider the following guidelines:

i. Bridges should be located and cantered on the main channel portion of the entire floodplain. This may mean an eccentricity bridge in the location with respect to the entire stream cross section, but it allows for better accommodation of the low flows of the stream
ii. Design the bridge waterway opening to provide a flow area sufficient to maintain the through-bridge velocity at no greater than the allowable through-bridge velocity under the circumstances of design discharge
iii. Orient headers and interior bents to conform to the streamlines at flood stage
iv. Accomplish this within reason, using standard skew values (15°, 30°, 45°, etc.) where feasible. Locate the toe of slope of the header away from deep channels, cuts, and high velocity areas
v. Locate and orient the bridge headers and piers to minimize the potential for excessive scour
vi. If the intrusion of either or both roadway headers into the stream floodplains is more than about 240m, consider including either relief openings or guide banks
vii. Incorporate existing vegetation in the overall bridge plan. Where practicable, leave trees and shrubs intact even within the right-of-way. Minimizing vegetation removal also tends to control turbulence of the flow into, through, and out of the bridge. On the other hand, you should consider safety and maintenance aspects of retaining vegetation within the right-of-way and near the travel lanes.

For some configurations, it is necessary to incorporate roadway approaches that accommodate overflow. Such overflow approaches allow floods larger than the design flow to overtop the roadway, thereby reducing the threat to the bridge structure itself. This action interrupts the function of the roadway, and the designer should consider the potential costs associated with such interruption and associated damage to the embankment.

13.7.5.2.1 AUXILIARY/RELIEF OPENINGS

The need for auxiliary waterway openings, or relief openings, arises on streams with wide floodplains. The purpose of the openings is to pass a portion of the flood flow that travels in the floodplain when the stream reaches a certain stage. The openings do not provide relief for the principal waterway opening as an emergency spillway of a dam does, but it has predictable capacity during flood events. However, the hydraulic engineer should be aware that the presence of overtopping or relief openings may not result in a significant reduction in flow through the principal bridge opening and may concentrate flow at undesirable locations.

The basic objectives in selecting the location of auxiliary openings include:

i. maintenance of flow distribution and flow patterns
ii. accommodation of relatively large flow concentrations on the floodplain
iii. avoidance of floodplain flow along the roadway embankment for long distances
iv. crossing of significant tributary channels
v. accommodation of eccentric stream crossings to provide for drainage (see **Figure 13.71**), and
vi. consideration of impacts associated with concentrations of flow at such locations.

The technological weakness in modelling auxiliary openings involves the use of some one-dimensional models to analyse two-dimensional flow. Two-dimensional models should provide a more accurate analysis of complex stream crossing systems.

The most complex factor in designing auxiliary openings is determining the division of flow between two or more structures. If incorrectly proportioned, one or more of the structures may be overtaxed during a flood event with possible damage to the structure and downstream property. The design of auxiliary openings should usually be conservatively large to guard against that possibility.

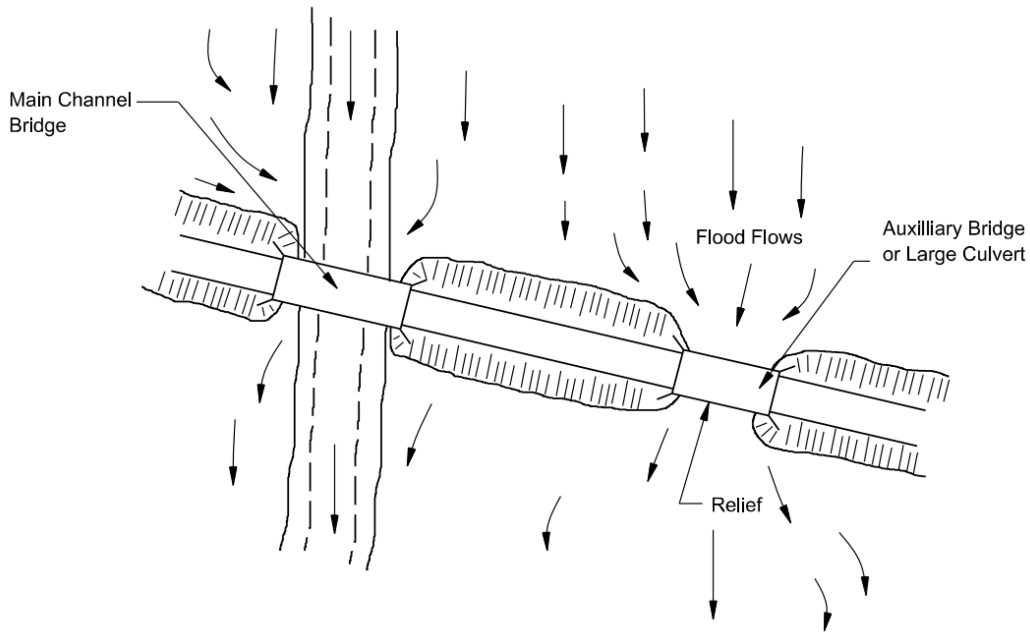


Figure 13.71 Eccentric Stream Crossings

13.7.5.3 SCOUR AND STREAM STABILITY CONSIDERATION AND GUIDANCE

Another critical component of the design and/or evaluation of a bridge opening is to design the bridge to be stable from scour at the piers, abutments, and across the contracted opening. From a hydraulic perspective, the magnitude of local scour at a pier is a function of depth and velocity of flow, alignment of the pier with flow, and pier type and location.

Depending on foundation costs and complexity it will be necessary to balance the number and size of piers, length and height, and anticipated total scour depth against increased costs of the superstructure associated with longer spans (girder type and allowable span) and foundation required to resist scour.

The magnitude of local scour at an abutment is a function of depth and velocity of flow, the skew of the embankment to the floodplain, as well as the amount of flow from the overbank that passes through the bridge opening. Highly-skewed bridges are not recommended as shown in **Plate 13.15** where a large skew angle to the flow direction has worsened the scour effects. It is also a function of where the abutment is located in relation to the main channel. It is recommended that an abutment not be located in or close to the main channel if at all possible.

The amount of contraction scour that occurs at a bridge crossing is a function of the degree that a bridge contracts floodplain flow. In general, bridges with higher degrees of contraction can be expected to have higher flow velocities and larger scour depths. If the depths of contraction scour are too large it may be necessary to increase the bridge length to reduce scour across the bridge opening.

Bridges should be designed to withstand scour from large floods (superflood) and from stream instabilities expected over the life of a bridge. Recommended procedures for evaluating and designing bridges to resist scour can be found in FHWA publications HEC-20 (FHWA 2012a) and HEC-18 (FHWA 2012b).

Plate 13.15 Scour at a highly skewed bridge

13.7.5.3.1 FACTORS THAT AFFECT STREAM STABILITY

Factors that affect stream stability, and potentially bridge and highway stability at stream crossings, can be classified as geomorphic factors and hydraulic factors.

A. Geomorphic Factors

These include:

i. stream size
ii. flow habit
iii. bed material
iv. valley setting
v. floodplains
vi. natural levees
vii. apparent incision
viii. channel boundaries
ix. tree cover on banks
x. sinuosity
xi. degree of braiding
xii. degree of anabranching
xiii. variability of width
xiv. development of bars

B. Hydraulic Factors

These include:

i. magnitude, frequency and duration of floods
ii. bed configuration
iii. resistance to flow
iv. water surface profiles
v. problems at bends
vi. problems at confluences
vii. backwater effects of alignment and location
viii. effects of highway profile, and
ix. bridge design

Rapid and unexpected changes may occur in streams in response to man's activities in the watershed (e.g., alteration of vegetative cover). Changes in perviousness can alter the hydrology of a stream, sediment yield and channel geometry. Channelization, stream channel straightening, stream levees and dikes, bridges and culverts, reservoirs, and changes in land use can have major effects on stream flow, sediment transport, and channel geometry and location. Knowing that man's activities can influence stream stability can help the designer anticipate some of the problems that can occur. Natural disturbances (e.g., floods, drought, earthquakes, landslides, volcanoes, forest fires) can also cause changes in sediment load and major changes in the stream channel. When natural disturbances do occur, it is likely that changes will also occur to the stream channel.

13.7.5.4 BRIDGE DESIGN SPECIFICATIONS AND DESIGN CRITERIA

The process by which a highway-stream crossing design is developed has changed significantly over the past few decades. Fortunately, hydraulic engineering technology is available to meet the challenge of changed construction economics and the needs created by increased traffic demands, the concerns for highway user safety, the environment and increasing flood losses because of floodplain occupancy.

Procedures for the design of a stream crossing should generally follow the sequence indicated below:

1. Hydrologic Analysis—The estimation of the stream discharge at the site for a range of flood exceedance probabilities. All existing flood-control measures, land use, and anticipated changes in the watershed should be considered in the hydrologic analysis.
2. Stream and Floodplain Analysis—The establishment of stream-flow characteristics at the crossing site for the range of discharges to be considered, including stages, flow distributions, velocities, stream morphology, sediment transport, and the influence of man's activities.
3. Identification of Criteria for Design—Certain criteria or standards, by which a design is judged to meet the objectives of a crossing design, override risk and economic considerations in some crossing designs. These criteria may include standards imposed because of legislative mandate, policy or the importance of the highway, such as national defence or for emergency vehicle access, and preclude politically, environmentally, or socially unacceptable solutions such as placing embankment in a significant marsh or wetland or leaving a community isolated during floods.
4. Analysis of Design Alternatives—The analysis of a highway-stream crossing involves an engineering, environmental, and economic evaluation of various design alternatives. The objective of the analysis is to achieve a design that will have the least expected cost to society considering expected losses and capital costs within the constraints imposed by criteria identified in No. 3 above. The analysis of stream-crossing design alternatives includes roadway alignment and profile, probable scour associated with each alternative, protective and preventive measures that are necessarily associated with each alternative, bridge lengths and locations on the floodplains, recommended orientation and location of piers and abutments, foundation depths and superstructure elevations, capital and risk costs associated with each alternative, the environmental and social impacts of each alternative, and the mitigation measures necessary to preserve or enhance the stream environment.

Figure 13.72 is a flowchart for a bridge drainage design process. Although the scope of the project and individual site characteristics make each design a unique one, this flowchart shall be used as a general guide.

1

Data Collection

A. Survey
- Topography
- Geology
- High-water marks
- History of debris accumulation and scour at the existing structure in the vicinity of the proposed crossing
- Review of hydraulic performance of existing structures
- Maps, aerial photographs
- Rainfall and stream gauge records if available
- Field reconnaissance visit

B. Other Relevant Information
- River basin studies
- Hydraulic performance of existing bridges

C. Influences on Hydraulic Performance of Site
- Other streams, reservoirs, water intakes
- Structures upstream or downstream
- Natural features of stream and flood plain
- Channel modifications upstream or downstream
- Flood plain encroachments
- Sediment types and bed forms

D. Environmental Impact
- Existing bed or bank instability
- Flood plain land use and flow distribution
- Environmentally sensitive areas (fisheries, wetlands, etc.)

E. Site-specific Design Criteria
- Preliminary flood risk assessment
- Application of GRDG drainage design criteria

2

Hydrologic Analysis

A. Catchment area morphology (characteristics)
- Drainage area (attach map)
- Catchment area and stream slope
- Channel geometry

B. Hydrologic computations
- Discharge for historical flood that complements the high-water marks used for calibration (if available).
- Discharges for specified frequencies (determined).

3

Hydraulic Analysis

A. Computer hydraulic model construction:
B Computer model calibration and verification (if data is available):
C Computer model sensitivity analysis (roughness, structural coefficients, future climate change and blockage scenarios)
D. Hydraulic performance for existing conditions
E. Hydraulic performance of proposed designs
F. Impact analysis
G. Mitigation Measures

4

Selection of Final Design

A. Flood Risk Assessment/Least-Cost Alternative (LTEC, or Least Total Expected Cost)
B. Measure of compliance with established hydraulic criteria
C. Consideration of environmental and social criteria
D. Design details such as riprap, scour abatement, and river training

5

Documentation

A. Complete project records (data, model), etc.
B. Complete correspondence and reports (survey, hydrological and hydraulic modelling reports)

Figure 13.72 Flowchart for a bridge drainage design process

13.7.5.4.1 GENERAL CRITERIA

In the design of a highway-stream crossing, standards or criteria should be defined by which the design can be measured for acceptability. Criteria commonly used for judging the acceptability of the hydraulic design of stream crossings include:

i. Backwater will not significantly increase flood damage to property upstream of the waterway crossing.
ii. Velocities through the bridge structure(s) will not either damage the highway facility or increase damages to adjacent property both upstream and downstream of the crossing.
iii. Maintain the existing pre-construction flow mechanism to the extent practicable.
iv. Pier spacing, orientation, and abutment are to be designed to minimize flow disruption and potential scour; spill-through type abutments using side slopes are preferred over deep abutments to minimize scour and backwater effect.
v. Select foundation design and/or scour countermeasures to avoid failure by scour.
vi. Allow freeboard at structure(s) designed to pass anticipated debris and minimise blockage of bridge openings.
vii. Acceptable risks of damage or viable measures to counter the unpredictability of alluvial streams.
viii. When two or more bridges are constructed in parallel over a channel, care should be taken to align the piers and to provide streamlined grading and protection for abutments. This abutment grading is to minimize expansion or contraction of flow between the two bridges.
ix. Commercial mining of sands and gravel in streams is common in Ghana because the material is clean and well graded, and the stream replenishes the supply. Borrow pits, either upstream or downstream of a highway-stream crossing, can cause or aggravate scour at the bridge. This fact should be considered when calculating bridge scour and should be estimated by sediment transport modelling using an industry standard software package.
x. Minimal disruption of ecosystems and values unique to the floodplain and stream.
xi. Provide a level of traffic service compatible with that commonly expected for the class of highway and compatible with projected traffic volumes.
xii. Design choices should support costs for construction, maintenance, and operation, including probable repair and reconstruction and potential liability; and
xiii. Adequate right-of-way shall be provided upstream and downstream of structure for maintenance operation.

In addition to the above criteria, the following general bridge design principles should be considered:

i. The final bridge design selection should consider the maximum backwater (**Figure 13.73**) allowed (0.5m) unless exceeding of the limit can be justified by special hydraulic conditions.
ii. The final design should not significantly alter the flow mechanism in both the main channel and the flood plain.
iii. The "crest-vertical curve profile" shall be considered as the preferred highway crossing profile when allowing for embankment overtopping (**Figure 13.74**).
iv. A specified clearance based on site specific conditions shall be established to allow for passage of debris if required.
v. Degradation or aggradation of the river as well as contraction and local scour shall be estimated as part of the final design preferably using a two- dimensional computer model and the design should either minimize scour or provide scour protection; and
vi. Foundation level shall be positioned below the total scour depth whenever practical.

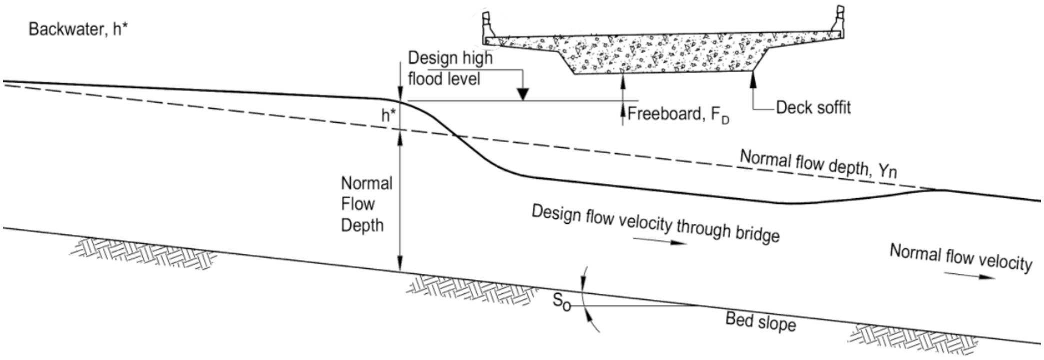


Figure 13.73 Illustration of some hydraulic definitions

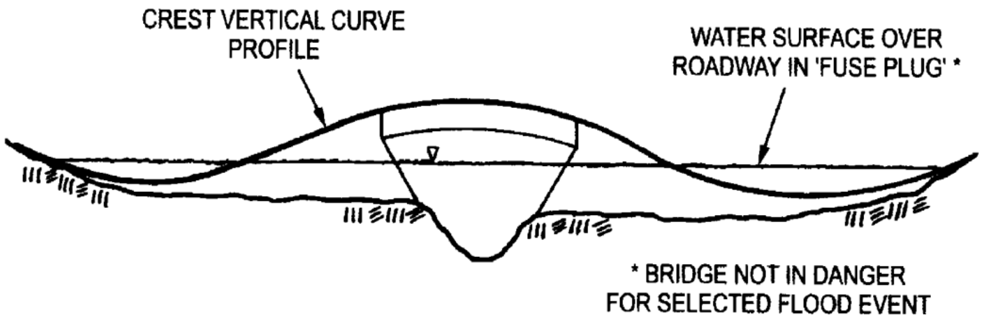


Figure 13.74 Crest Vertical Curve Profile

13.7.5.4.2 SPECIFIC CRITERIA

These criteria augment the general criteria. They provide specific, quantifiable values that relate to local site conditions. Evaluation of various alternatives according to these criteria can be accomplished by using the water surface profile computation.

A. Inundation

Inundation of the carriageway dictates the level of traffic services provided by the facility. The carriageway overtopping flood level identifies the limit of serviceability.

B. Risk Evaluation

The selection of hydraulic design criteria for determining the waterway opening, road grade, scour potential, riprap, and other features should consider the potential impacts to interruptions to traffic, adjacent property, the environment, and the infrastructure of the highway.

The evaluation of the consequence of risk associated with the probability of flooding attributed to a stream-crossing system is a tool by which site specific design criteria can be developed.

This evaluation considers capital cost, traffic service, environmental and property impacts, and hazards to human life.

The evaluation of risk is a two-stage process. The initial step, identified as risk assessment, is more qualitative than a risk analysis and serves to identify threshold values that must be met by the hydraulic design.

In many cases where the risks are low and/or threshold design values can be met, it is unnecessary to pursue a detailed economic analysis. In those cases where the risks are high and/or threshold values cannot be met, a Least Total Expected Cost (LTEC) analysis shall be considered.

The results of a least-cost analysis can be presented in a graph of total cost as a function of the overtopping discharge. The total cost consists of a combination of capital costs and flood damages (or risk costs). Risk costs decrease with increases in the overtopping discharge while capital costs simultaneously increase. The overtopping discharge for each alternative is determined from a hydraulic analysis of a specific combination of embankment height and bridge-opening length. The resulting least-cost alternative provides a trade-off comparison.

The alternatives considered in the least-cost analysis do not require the specification of a particular design flood. This information is part of the output of the least-cost analysis. In other words, the least-cost alternative has a specific risk of overtopping that is unknown before the least-cost alternative has been determined. Therefore, design flood frequencies are used only to establish the initial alternative.

Thereafter, specific flood-frequency criteria shall be considered only as constraints on the final design selection. Deviation from the least-cost alternative may be necessary to satisfy these constraints and the trade-off cost for doing so can be obtained from the least-cost analysis.

Risk based analysis does not recognize some of the intangible factors that influence a design. The minimum design that results from this type of analysis may be too low to satisfy the site condition.

C. Design Floods

Design floods for such purposes as the evaluation of backwater, clearance, and overtopping shall be established predicated on risk-based assessment of local site conditions. They should reflect consideration of traffic service, environmental impact, property damage, hazard to human life, and flood plain management criteria.

D. Backwater

Backwater and/or increases over existing condition should be decided based on site-specific conditions. It is important to eliminate or minimize backwater if possible, especially in urban areas. However, if there are constraints, up to 0.5 m increase in backwater level upstream of the bridge during the passage of the 100-year flood can be allowed, if practicable.

The expression for backwater is formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge, and a point downstream from the bridge at which normal stage has been re-established. The expression is reasonably valid if the channel in the vicinity of the bridge is essentially straight, the cross-sectional area of the stream is fairly uniform, the gradient of the bottom is approximately constant between the upstream and downstream section, the flow is free to contract and expand, there is no appreciable scour of the bed in the constriction and the flow is in the subcritical range.

E. Freeboard

Clearance/freeboard (**Figure 13.73**) should be determined based on site-specific condition. However, to permit the passage of debris, a minimum clearance (freeboard) of 1.5m should be provided between the computed approach water surface elevation and the low chord (or bottom of the slab) of the bridge. Where this is not practical, the clearance should be established by the Bridge Hydraulic Engineer based on the type of stream and level of protection desired. For example, consider the following cases:

i. Bridges on small streams that normally do not transport debris may be adequate with 0.5m of freeboard.
ii. Urban bridges with grade limitations may not provide any freeboard.
iii. Bridge replacement projects should at least match pre-existing low-chord elevations unless the results of a flood risk assessment indicate a different structure is the most beneficial option.
iv. Bridges that have relief due to grade variation.

Zero freeboard may be acceptable if the longitudinal gradient of the roadway provides for overtopping at Q_{100}. Where the provision of any freeboard is not practical, the designer should ensure that the waterway opening does not result in pressure flow at the $Q_{100\text{-year}}$ flood or ensure that the structure is designed accordingly.

For navigational channels, vertical and horizontal clearance should be provided based on normally expected flows during the navigation season.

F. Flow Distribution (Mechanism)

Any stream crossing that uses a combination of landfill and bridge within the floodplain disturbs flow distribution during floods. Preserve flow distribution to the extent practicable in order to:

i. Avoid disruption of the stream-side environment;
ii. Preserve local drainage patterns;
iii. Minimize damage to property by either excessive backwater or high local velocities;
iv. Avoid concentrating flow areas that were not subjected to concentrated flow prior to construction of the highway facility; and
v. Avoid diversions for long distances along the roadway embankment.

For many situations one-dimensional analysis techniques suffice for determining optimum bridge locations. When analysing complex sites, such as those at a bend, and skewed (**Figure 13.75**) crossings with one-dimensional models only, the designer need a great deal of intuition, experience, and engineering judgment to supplement the quantitative analysis. The development of two-dimensional techniques of analysis greatly enhances the capabilities of hydraulics designers to deal with these complex sites. On this situation, a HEC-RAS hydraulic modelling or something similar should be undertaken.

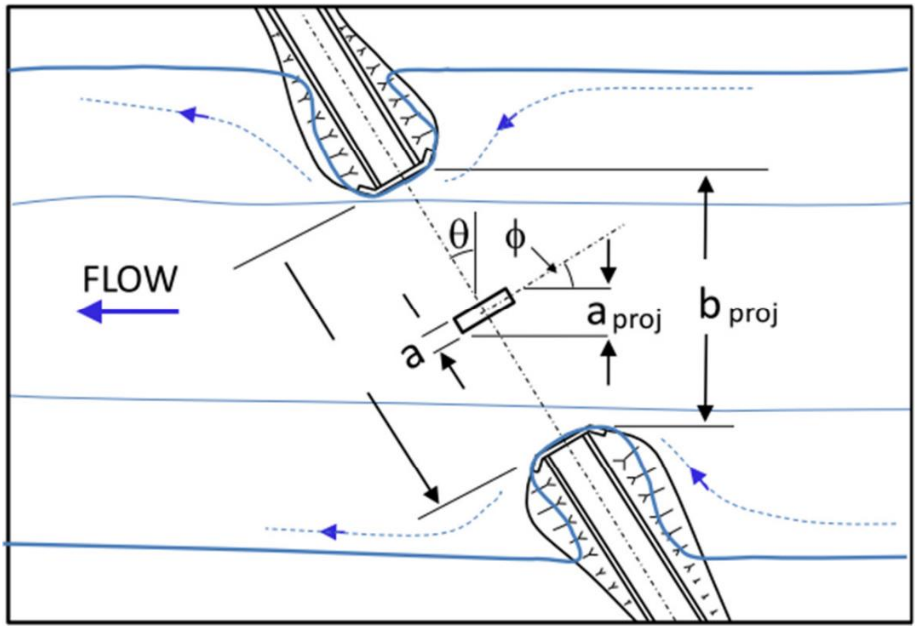


Figure 13.75 Illustration of Skew Bridge Crossing

13.7.6 HYDRAULIC ANALYSIS METHODS

There are a number of methods available for the hydraulic design of bridges. All hydraulic models, whether numerical or physical, rely on a set of assumptions and requirements in order to accurately simulate the flow condition at a structure. As no model will provide an exact representation of the complexity of the actual flow, it is important for engineers to understand these assumptions, as they form the limitations of that method.

Ignoring or violating these assumptions and limitations or failing to critically analyse the model will produce inaccurate results. This is unacceptable given the high cost and potential consequences of failure of bridges.

The available modelling approaches are listed below (Zevenbergen et al. 2012):

i. one-dimensional modelling: flow parameters change predominantly in one defined direction (x), along the channel
ii. two-dimensional modelling: flow parameters change in two directions (x and y) Horizontal velocity is computed as either components (V_x and V_y) or as a vector with constant magnitude and direction throughout the model.
iii. three-dimensional modelling [Computational Fluid Dynamics (CFD)]: flow parameters change in three directions (x, y, and z)
iv. physical modelling: geometrically scaled physical representations of bridges or bridge components.

Additionally, the decision should be made as to whether or not the flow condition should be modelled as steady or unsteady (i.e., changes with time). While almost all flow is unsteady to some extent, as the rate and depth of flow typically changes over long periods, the majority of bridge hydraulic analyses are conducted using steady flow.

There are several situations where unsteady flow conditions should be used, including:

i. run-off from precipitation, or when depth and velocity of flow in a river change rapidly with time
ii. unsteady or transient flows released from reservoirs during operations for flood control, hydropower generation, recreation, and wildlife management

iii. where floodplain storage and/or the loss of floodplain storage are significant
iv. tidal applications
v. dam break floods
vi. wind-generated storm surges or seiches
vii. landslide-generated waves
viii. earthquake-generated tsunami waves
ix. irrigation flows affected by gates, pumps, and diversions.

The appropriate analysis method for a structure should be selected primarily on the advantages and limitations of the method itself. However, the importance of the structures, project impacts, cost and schedule should also be considered. The most common approaches are one or two-dimensional modelling. While CFD and physical modelling can provide much higher levels of detail regarding flow patterns and hydrodynamic phenomena, their high cost and the lack of availability of computational and physical resources means they are typically used for localised situations, such as predicting local scour at piers, or modelling hydraulic forces on piers or the bridge superstructure. The decision to whether to use one or two-dimensional modelling should be based on the characteristics of the waterway being analysed. **Table 13.53** provides a summary of the appropriateness of one or two-dimensional modelling to a wide variety of waterway and structural situations. Generally, one-dimensional modelling is appropriate for situations where lateral velocities are small, such as in-channel flow, minor floodplain flow, or where bridge constriction is minimal.

Table 13.53 Bridge hydraulic modelling selection

Bridge hydraulic condition	Hydraulic analysis method	
	One-dimensional	**Two-dimensional**
Small streams	●	◗
In-channel flows	●	◗
Narrow to moderate-width floodplains	●	◗
Wide flood plains	◗	●
Minor floodplain constriction	●	◗
Highly variable floodplain roughness	◗	●
Highly sinuous channels	◗	●
Multiple embankment openings	◗/○	●
Unmatched multiple openings in series	◗/○	●
Low skew roadway alignment (< 20°)	●	◗
Moderately skewed roadway alignment (>20° and < 30°)	◗	●
Highly skewed roadway alignment (> 30°)	○	●
Detailed analysis of bends, confluences and angle of attack	○	●
Multiple channels	◗	●
Small tidal streams and rivers	●	◗
Large tidal waterways and wind-influenced conditions	○	●
Detailed flow distribution at bridges	◗	●
Significant roadway overtopping	◗	●
Upstream controls	○	●
Countermeasure design	◗	●

Key:

● well suited or primary use

◗ possible application or secondary use

○ unsuitable or rarely used

◗/○ possibly unsuitable depending on application

Source: Zevenbergen et al. (2012).

13.7.6.1 ONE-DIMENSIONAL BRIDGE HYDRAULIC ANALYSIS

One-dimensional analysis is a term covering a wide range of analysis methods with differing levels of details, from a single waterway section to a more detailed water surface profile involving multiple cross-sections and stream reaches. They typically incorporate the assumption of uniform flow, in which flow parameters change predominantly in the x-direction, along the centre line of the channel. An approximate method for the calculation of backwater is the FHWA's HDS 1 method (Zevenbergen et al. 2012).

The HDS 1 model for calculating backwater is as follows (**Equation (13.113)**):

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right) - \left(\frac{A_{n2}}{A_1} \right) \right] \frac{V_{n2}^2}{2g} \tag{13.113}$$

Where,

h_1^* = total backwater, m

K^* = total backwater coefficient

α_1, α_2 = kinetic energy distribution coefficients at cross-sections 1 and 2

A_{n2} = gross water area in constricted bridge waterway measured below normal stage at cross-section 2 (m^2)

V_{n2} = average velocity in constriction (total discharge divided by A_{n2}) (m/s)

A_1 = total flow area at cross-section 1, including addition caused by backwater (m^2)

A_4 = total flow area at cross-section 4, downstream of influence of bridge (m^2)

g = acceleration of gravity(m/s^2)

The locations of the cross-sections are shown in **Figure 13.76**.

The location of the cross-section is critical to ensuring accurate results. Any redistribution of flow caused by changing conveyance (area, depth or roughness) should be reasonably possible in the distance between cross-sections. Placing cross-sections too close together will create physically impossible results, while placing them too far apart will result in numerically unreliable answers due to the significant differences in energy slopes. Additionally, the cross-sections should be selected and orientated such that there is a consistent water surface for the simulated flow, otherwise the numerical solution to the model may differ significantly from reality.

13.7.6.1.1 LIMITATIONS

As noted in **Table 13.53**, there are several situations where one-dimensional analysis is inappropriate. This is due to the assumption of steady flow in one direction, along the centre line of the channel. The conditions of this flow are as follows:

i. a single water surface at each cross-section
ii. the flow is perpendicular to the cross-section along its entire length
iii. the energy slope for the cross-section applies to every point in the cross-section
iv. hydrostatic pressure exists throughout the cross-section
v. channel slope is small
vi. energy slope is the same as for the corresponding normal depth
vii. the channel is prismatic with constant alignment and shape
viii. roughness is constant through the reach.

As flow in natural channels is inherently three-dimensional and unsteady, taking these assumptions as absolute would preclude one-dimensional analysis in most cases. However, if the engineer is aware of these limitations and applies the model appropriately, then the accuracy of the results should not be overly compromised.

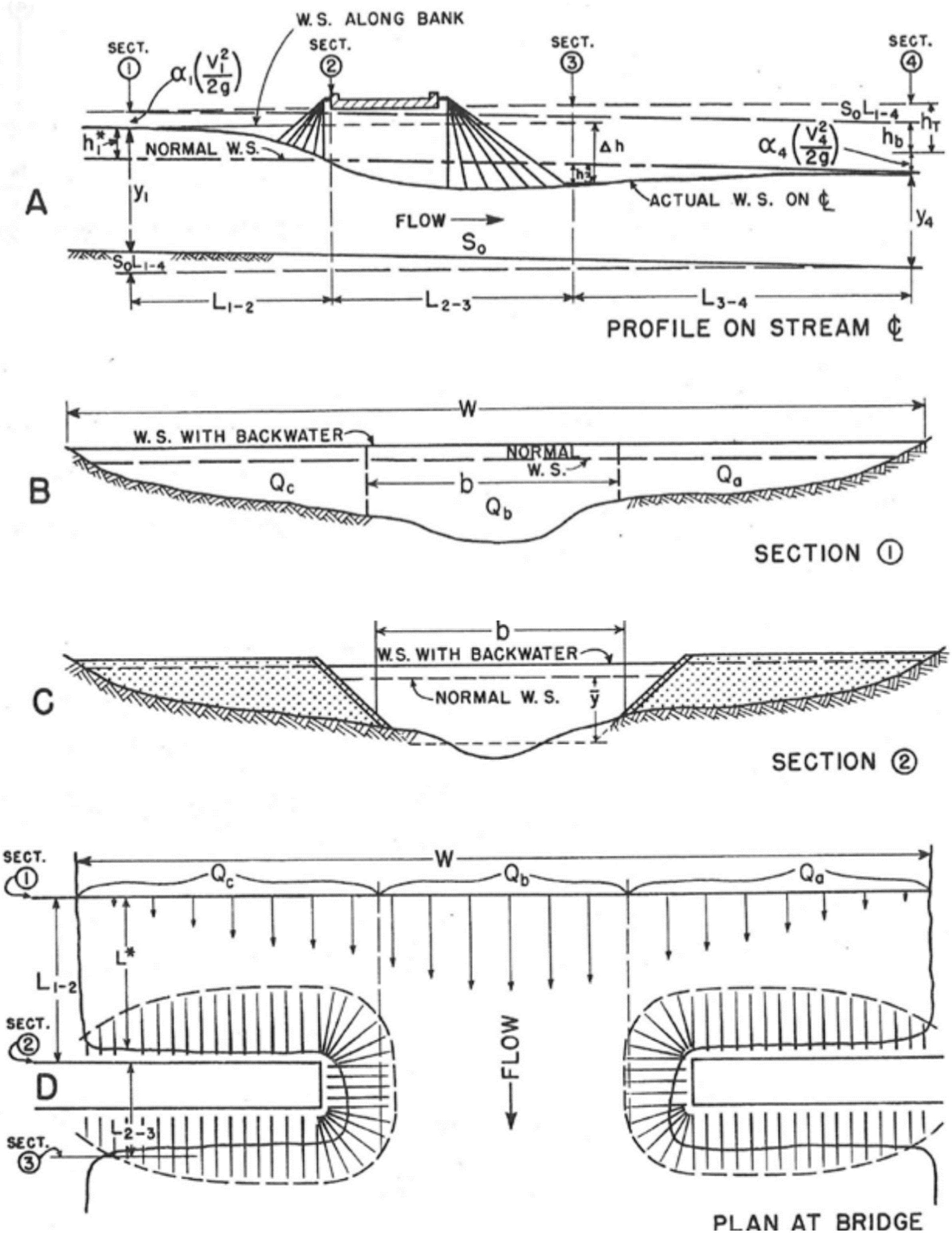


Figure 13.76 Sketch illustrating positions of cross-sections 1 through 4 in HDS 1 backwater method

13.7.6.1.2 SPECIAL CASES

There are several situations where different approaches to one-dimensional modelling may be required in order to accurately model special conditions that may exist at a structure. These may include:

i. skewed crossings
ii. crossings with parallel bridges
iii. split flow conditions
iv. crossings with multiple openings in the embankment
v. lateral weirs.

13.7.6.2 TWO-DIMENSIONAL BRIDGE HYDRAULIC ANALYSIS

Two-dimensional hydraulic analysis is a more rigorous form of analysis that relies on less assumptions than one-dimensional analysis and can more accurately model complex situations. While one-dimensional modelling typically relies on the solution of the energy equation, in two-dimensional modelling, the momentum and continuity equations are applied to a control volume, as illustrated in **Figure 13.77**. As opposed to one-dimensional modelling, velocities in both the x and y directions are considered; vertical velocities are considered negligible and hydrostatic pressure is assumed.

Velocities may be expressed as a vector quantity (with magnitude and direction) as separate x and y components. Additionally, the elevation of the bed (Z_b) and water (H) depth vary over the area. The force variables are pressure (P) at the horizontal surfaces, water weight (W), bed and water surface shear stress components (τ_b and τ_s).

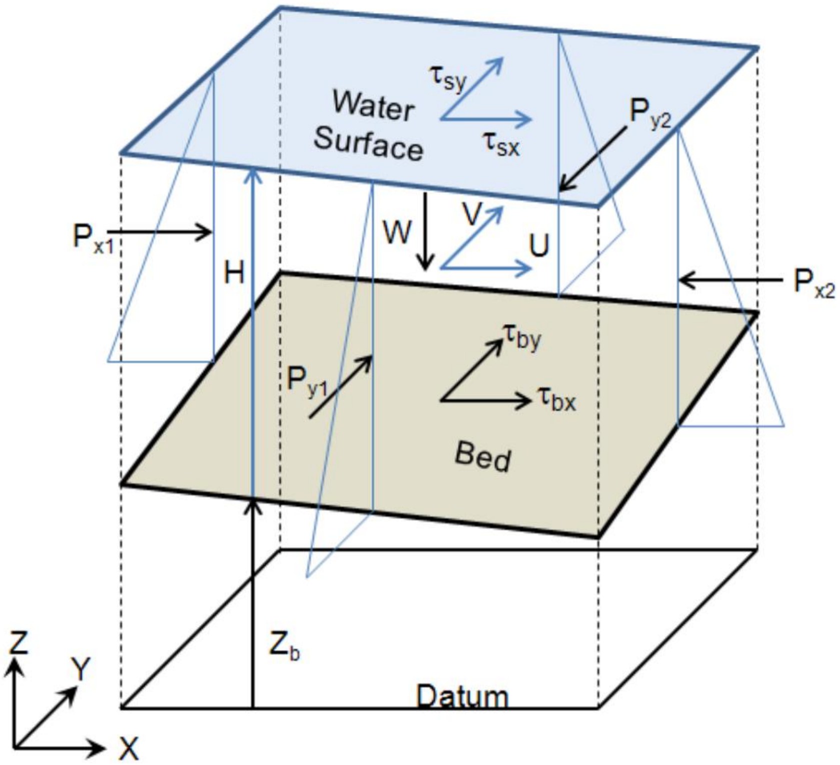


Figure 13.77 Control volume and two-dimensional hydraulic analysis variables

These equations are applied in two main model types, the finite element method and finite difference method. The finite element method uses an unstructured mesh or grid to solve the continuity and momentum equations through numerical integration techniques at each element.

The finite difference method is a numerical solution technique for differential equations. Both these methods have the advantage of being able to model flow conditions more accurately than one-dimensional analysis through techniques such as decreasing mesh/grid size in order to provide greater detail on changes in terrain (land use, roughness, topography, or bathymetry) or velocity, or by removing/deactivating elements/grid cells to represent road embankments or piers.

13.7.6.2.1 LIMITATIONS

While two-dimensional hydraulic analysis is able to account for many of the limitations faced by one-dimensional analysis, there are several situations where more three-dimensional or CFD analysis may be required to model the complete flow field. This is this case when analysis pier, abutment or debris obstruction losses, and road overtopping, due to the fact that vertical velocities are not included in two-dimensional models, and hydrostatic pressure is assumed. Additionally, while two-dimensional modelling can account for most processes associated with the pressure flow caused in submerged bridge decks, it is unable to account for flow separation at the leading edge of the deck.

13.7.6.2.2 SPECIAL CASES

As stated earlier, there are special cases of one-dimensional modelling that fall outside the typical model application. These include skewed crossings, parallel crossings, multiple openings and other less common applications. Most of these situations are not considered as special applications in two-dimensional models because the assumptions required by one-dimensional models are not required in two-dimensional models.

13.7.6.3 UNSTEADY FLOW ANALYSIS

Unsteady flow is defined as flow that changes with time. While in actuality, almost all flow can be considered to be unsteady to some extent, for many applications in bridge hydraulics the flow can be assumed to be steady for a short reach of the channel. There are several important differences between steady and unsteady flow analysis (Zevenbergen et al. 2012):

i. Steady flow analysis assumes flow can vary with respect to space, but not time ($\partial Q/\partial x \neq 0$, $\partial Q/\partial t = 0$); unsteady flow assumes it can vary in time and space.
ii. For small bed slopes (i.e. slopes less than 0.0004) or highly transient flows, such as tidal influences or dam breach flood waves, the peak stages do not necessarily coincide with the peak discharges, and the rating curves of stage versus discharge are not single-valued.
iii. The total flow downstream from a junction of two tributaries is not necessarily the combination of the two flows.
iv. Tributary flows entering a main stream channel may experience a flow reversal caused by flow in the main stem backing up into the tributary or vice versa, e.g. when a large tributary flood enters the main channel during a period of low flow.
v. If the inflow or stage at a boundary is changing rapidly, the acceleration terms in the momentum equation are important, and thus unsteady flow is a more robust solution.
vi. For full networks, where the flow divides and recombines, unsteady flow analysis should be used. This is due to the fact that the channel length, resistance and geometry will likely differ, causing the flow to travel at different speeds, affecting flow distribution.

Unsteady flow is typically analysed for one-dimensional and two-dimensional applications.

One-dimensional unsteady flow analysis is represented by the Saint-Venant equations outlined in Zevenbergen et al. (2012). These equations may be expressed in a three-dimensional space, with two axes corresponding to the distance along the channel (x) and time (t), along with the solution being sought such as discharge, depth, water surface elevation or velocity. This is illustrated in **Figure 13.78**, which also demonstrates features of unsteady analysis such as lag time and attenuation of flow.

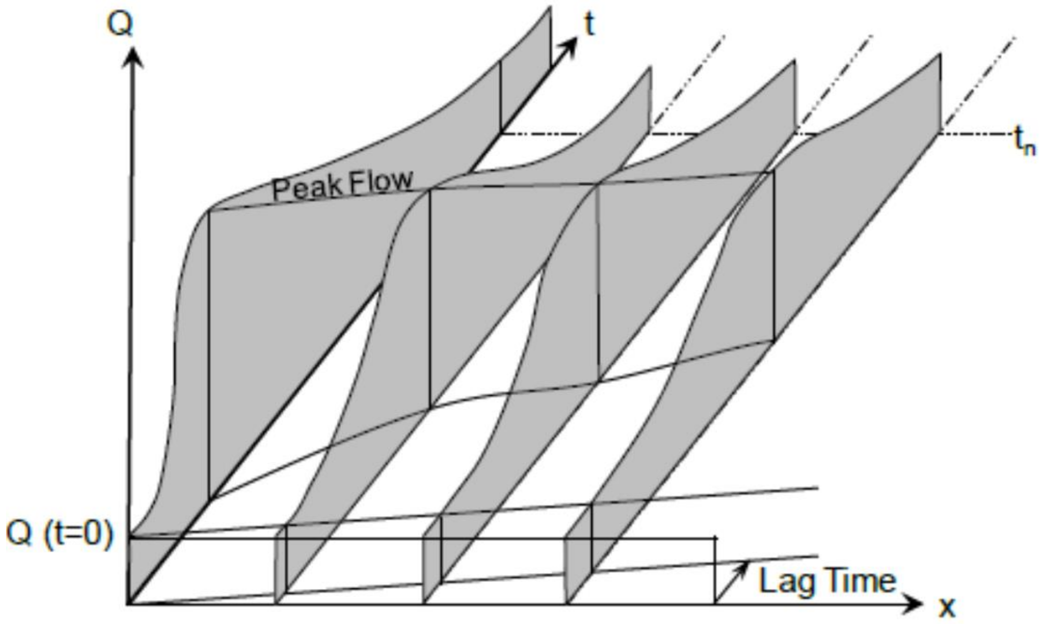


Figure 13.78 Unsteady flow analysis solution of discharge versus distance and time

13.7.6.3.1 LIMITATIONS

The model limits of an unsteady model must be carefully chosen to include all potential storage upstream and downstream of the location of interest. If a bridge hydraulic model including only the minimum number of cross-sections were used as the unsteady model geometry, the simulation would be inaccurate because storage and routing effects would be significantly underrepresented. Therefore, unsteady models almost always require much longer upstream and downstream limits than steady models.

13.7.6.3.2 SPECIAL CASES

As two-dimensional steady flow analysis is used in more complex situations than one-dimensional analysis, the same is true for unsteady flow analysis. This due the ability for the equations to include additional terms such as wind stress that may occur during hurricane storm surges. Other situations where two-dimensional unsteady flow analysis is appropriate include:

i. water levels and flow distributions around islands
ii. flow at bridges having one or more relief openings
iii. in extremely contracting and expanding reaches
iv. into and out of off-channel storage or flow situations such as overtopping of a levee flow at river junctions
v. circulation and transport in water bodies with wetlands
vi. water surface elevations and flow patterns in large rivers, reservoirs and estuaries.

13.7.7 HYDRAULIC DESIGN PROCEDURES

The following procedure is recommended for hydraulic design of bridges in determining a bridge waterway (length and deck level):

1. determine magnitude of flow at site for design recurrence interval
2. determine stage-discharge curve for the stream at the bridge site

3. determine the stage height at the bridge site for the design discharge from the stage-discharge curve (step 2)
4. select velocity of flow through bridge opening to limit scour or encourage scour as required
5. determine minimum length of bridge opening required to pass design discharge assuming water surface is at stage height
6. select a bridge deck level and trial length of bridge based upon minimum length of bridge opening and required length of spans
7. determine type of flow encountered
8. calculate backwater using the procedure relevant to the type of flow encountered
9. check the assumed deck level using the stage height and backwater for the design discharge based on the trial length of bridge.

If there is insufficient clearance beneath the bridge lift the bridge deck level and if necessary, recalculate the backwater. If there is insufficient freeboard to the top of the embankment, lift the embankment or reduce the backwater by using a larger bridge.

It is worth noting that checking for afflux is required where flooding upstream is very sensitive to development, especially in urban catchments. In flood-sensitive areas, zero or negligible afflux is required and as a result, a longer bridge matching the flood width may be required.

The selection of the type of superstructure could affect the level of service and freeboard for the crossing. The depth of superstructure may vary from 0.25 m to more than 1.8 m, not taking into account the cross fall and the deck slab.

13.7.8 SPECIAL CONSIDERATIONS

There are several other considerations that should be taken into consideration alongside typical bridge hydraulic analysis, which are discussed hereafter.

13.7.8.1 BACKWATER EFFECTS OF BRIDGE PIERS

Hydraulic drag at bridge piers is a force that must be resisted by the structure. For the stream, it is a resistance to flow that must be overcome by an increase in energy driving flow through the bridge waterway. This typically takes the form of backwater. While the total backwater upstream of the bridge is generally dominated by the severe constriction caused by road embankments and bridge abutments, piers can be a significant factor in bridge design (Zevenbergen et al. 2012). As bridge piers are required for most structures in order to maintain economical span length and superstructure depths, the hydraulic analysis should inform the structural design in order to minimise the impact of piers on the waterway.

13.7.8.2 COINCIDENT FLOWS AT CONFLUENCES

Where a bridge over a waterway is located near a confluence with another stream, the potential influence of the other waterway on the hydraulics at the bridge should be considered. If the structure is upstream of the confluence, it should be considered how the other waterway will affect the water surface profile through the bridge at various flood levels. If the bridge is located within or near the confluence zone, how the interaction with will affect the distribution and direction of flow through the confluence should also be considered (Zevenbergen et al. 2012). In order to consider the effects of the confluence, it is necessary to estimate the coincident flow probabilities of the waterways. That is, while it is unlikely that a 100 year flood level will occur in both simultaneously, the possible combinations of flows that have a 100 year recurrence level should be determined. Overestimating these probabilities will lead to a conservative design, while underestimating may have negative consequences for the structure.

13.7.9 COMPUTATION OF BACKWATER

This section presents a method for computing the backwater caused by bridge constrictions based on the work of Bradley (1978).

The expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge, section 1, and a point downstream from the bridge at which normal stage has been re-established, section 4 (**Figure 13.76**). The expression is reasonably valid if the channel in the vicinity is essentially straight, the cross-sectional area of the stream is fairly uniform, the gradient of the bed is approximately constant between sections 1 and 4, the flow is free to contract and expand, there is no appreciable erosion of the bed in the constriction due to scour, and the flow is in the subcritical range.

The expression for computation of backwater upstream from a bridge constriction is in **Equation (13.114)**:

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \tag{13.114}$$

Where,

h_1^* = total backwater, m

K^* = total backwater coefficient

α_1, α_2 = kinetic energy distribution coefficients at cross-sections 1 and 2 (**Figure 13.76**)

A_{n2} = gross water area in constriction measured below normal stage, m^2

V_{n2} = average velocity in constriction (or Q/A_{n2}), m/s

A_1 = total flow area at cross-section 1(**Figure 13.76)**, including that produced by the backwater, m^2

A_4 = water area at section 4 (**Figure 13.76**) where normal stage is re-established, m^2

g = acceleration of gravity, m/s^2

To compute backwater from **Equation (13.114)** it is first necessary to obtain the approximate value of h_1^* by using the first part of the expression (**Equation (13.115)**):

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} \tag{13.115}$$

The value of A_1 in the second part of the expression which depends on h_1^* can then be determined and the second term of the expression evaluated (**Equation (13.116)**):

$$\alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \tag{13.116}$$

This part of the expression represents the difference in kinetic energy between sections 4 and 1 (**Figure 13.76**), expressed in terms of the velocity head $\frac{V_{n2}^2}{2g}$. The equation may appear cumbersome, but it has been set up as shown to permit omission of the second part when the difference in kinetic energy between sections 4 and 1 (**Figure 13.76**) is small enough to be insignificant in the final result. To permit the design engineer to readily recognise cases in which the kinetic energy term may be ignored, the following guides are provided:

i. the bridge opening ratio (ratio of the bridge opening width to the total floodplain width), M, is greater than 0.7

ii. V_{n2} is less than 2 m/s, and

iii. $K^* \frac{V_{n2}^2}{2g}$ is less than 0.15 m.

The backwater obtained from the first term of **Equation (13.114)** can be considered sufficiently accurate, if values in the problem at hand meet all three conditions. Should one or more of the values not meet the above conditions, it is advisable to use **Equation (13.114)** in its entirety.

The following definitions are worth noting:

A. Conveyance - is a measure of the ability of a channel to transport flow. Using the Manning Equation (**Equation (13.84)**) for open channel flow, the discharge, q in a subsection of a channel can be determined. **Equation (13.84)** can be re-arranged as (**Equation (13.117)**):

$$\frac{Q}{S^{1/2}} = \frac{A}{n} R^{2/3} = k \tag{13.117}$$

Where,

k = conveyance of subsection

Conveyance can, therefore, be expressed either in terms of flow factors or strictly geometric factors. In bridge waterway computations, conveyance as a means of approximating the distribution of flow in the natural river channel upstream from a bridge. Total conveyance K_1 is the summation of the conveyances of the subsections.

B. Bridge opening ratio (M) - defines the degree of stream constriction involved. It is defined as the ratio of the flow which can pass unimpeded through the bridge constriction to the total flow of the river (**Equation (13.118)**):

$$M = \frac{Q_b}{Q_a + Q_b + Q_c} = \frac{Q_b}{Q} \tag{13.118}$$

or for the example shown on **Figure 13.79**, M=210/350=0.6

Because of the irregular cross-section common in natural streams and the variation in boundary roughness within any cross-section, the discharge is not uniform across a river but varies as might be indicated by the stream lines in **Figure 13.79**. The bridge opening ratio, M is most easily explained in terms of discharges, but it is usually determined from conveyance relationships. Since conveyance is proportional to discharge, assuming all subsections to have the same slope M can be expressed also as (**Equation (13.11)9**):

$$M = \frac{K_b}{K_a + K_b + K_c} = \frac{K_b}{K_1} \tag{13.119}$$

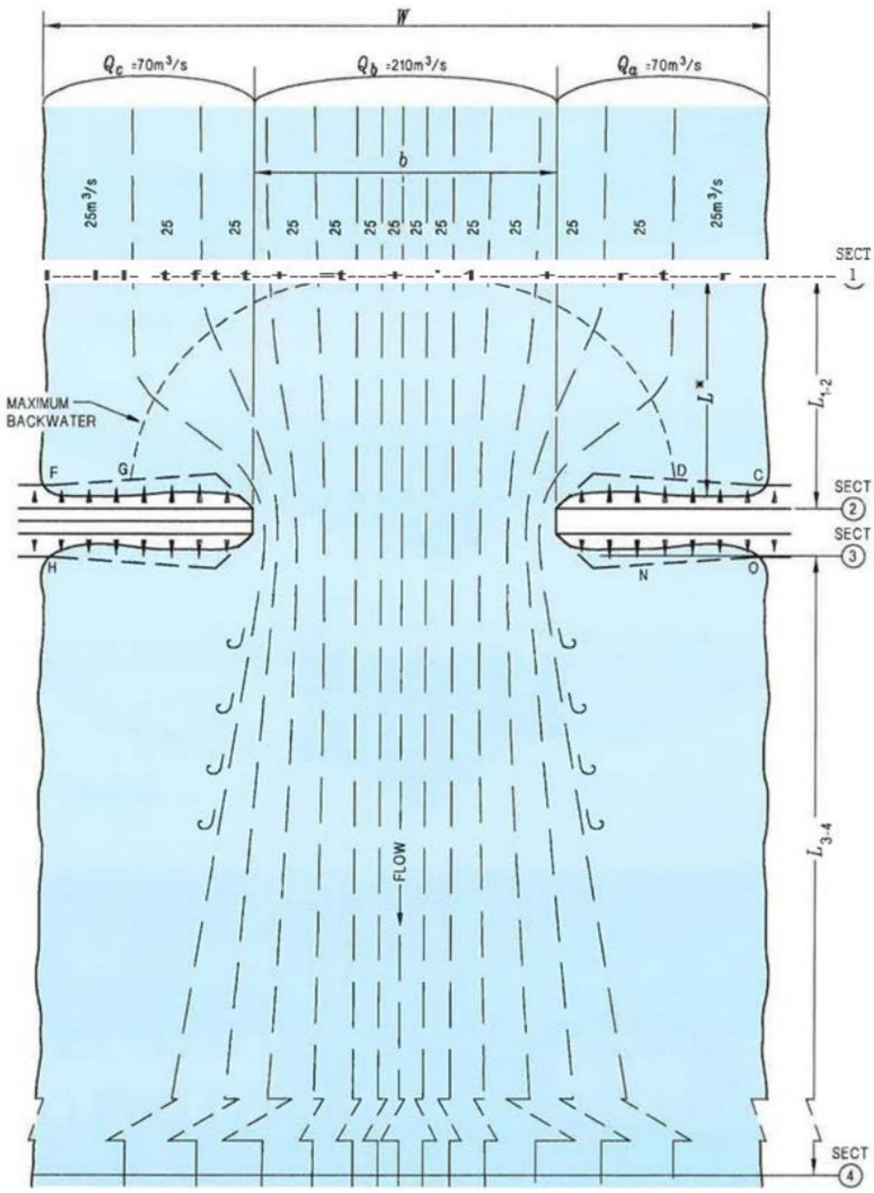


Figure 13.79 Flow lines for typical normal crossing

C. Kinetic energy coefficient - as the velocity distribution in a river varies from a maximum at the deeper portion of the channel to essentially zero along the banks, the average velocity head computed as $(Q/A_1)^2/2g$ for the stream at section 1 (**Figure 13.79**), does not give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head, above, by a kinetic energy coefficient, α_1, defined as (**Equation (13.120)**):

$$\alpha_1 = \frac{\sum(qv^2)}{QV_1^2} \tag{13.120}$$

Where,

v = average velocity (m/s) in a subsection
q = discharge (m³/s) in same subsection
Q = total discharge (m³/s) in river
V_1 = average velocity (m/s) in river at section 1 (**Figure 13.79**) or Q/A_1

A second coefficient α_2 is required to correct the velocity head for non-uniform velocity distribution under the bridge (**Equation (13.121)**):

$$\alpha_2 = \frac{\sum(qv^2)}{QV_2^2} \tag{13.121}$$

Where,

V_2 = average velocity in constriction, = Q/A_2

In this equation, v, q, and Q are defined as in **Equation (13.121)** but apply here to the constricted cross-section.

The value of α_1 can be computed, but α_2 is not readily assessed. The best that can be done in the case of the latter is to collect, tabulate and compare values of α_2 from existing bridges. **Figure 13.80** relating α_2 to α_1 and the contraction ratio, M, is included for estimating purposes only. The value of α_2 is usually less than α_1 for a given crossing, but this is not always the case. Actually, there should be no definite relationship between the two, but there is a trend. Local factors at the bridge such as asymmetry of flow, variation in cross-section and extent of vegetation in the bridge opening will influence the value of α_2. It is suggested that values adopted for α_2 should err on the high side.

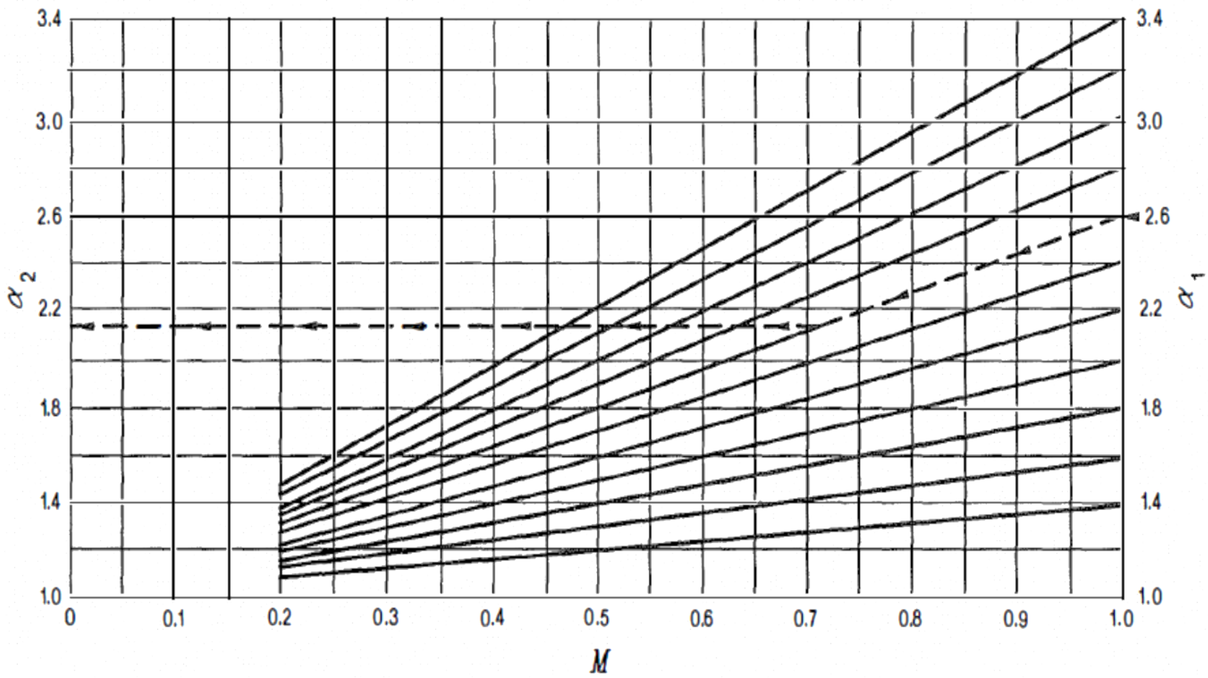


Figure 13.80 Aid for estimating α_2

13.7.9.1 BACKWATER COEFFICIENT

Two symbols are interchangeably used throughout this section, and both are backwater coefficients. The symbol K_b is the backwater coefficient for a bridge in which only the bridge opening ratio, M, is considered. This is known as a base coefficient and the curves on **Figure 13.81** are called base curves. The value of the overall backwater coefficient, K^*, is similarly dependent on the value of M, but also affected by:

i. number, size, shape and orientation of piers in the constriction
ii. eccentricity, or asymmetric location of bridge with respect to floodplains
iii. skew (bridge crosses floodplain at other than 90° angle).

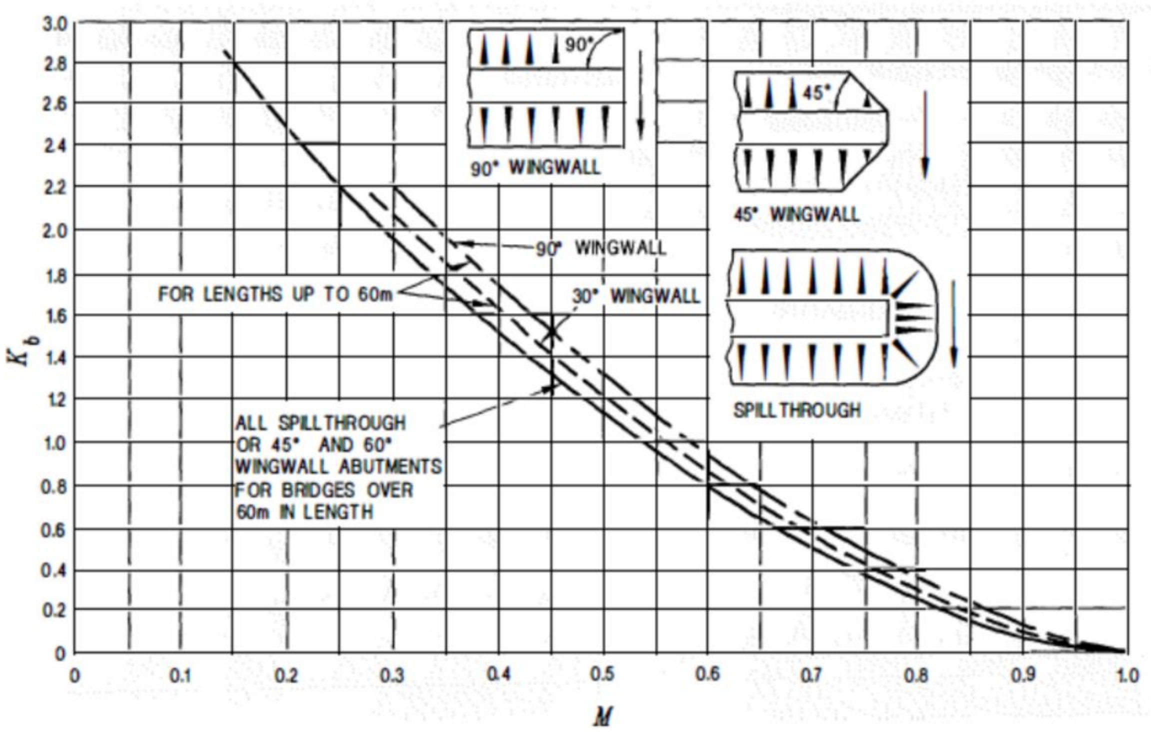


Figure 13.81 Backwater coefficient base curves (subcritical flow)

It will be demonstrated in the succeeding sections that the overall backwater coefficient K^* consists of a base curve coefficient K_b, to which are added incremental coefficients to account for the effect of piers, eccentricity and skew. The value of K^* is nevertheless primarily dependent on the degree of constriction of the flow at a bridge.

13.7.9.2 EFFECT OF M AND ABUTMENT SHAPE (BASE CURVES)

Figure 13.81 shows the base curve backwater coefficient, K_b, plotted with respect to M, for wingwall and spill-through abutments. Note how the coefficient, K_b, increases with channel wingwall constriction. The lower curve applies for 45° and 60° wingwall abutments and all spill-through types. Curves are also included for 30° wingwall abutments and for 90° vertical wall abutments for bridges up to about 60 m in length. For bridges over 60 m in length, regardless of abutment type, the lower curve is recommended because abutment geometry becomes less important to backwater as bridge length increases.

13.7.9.3 EFFECTS OF PIERS (NORMAL CROSSINGS)

Backwater caused by the introduction of piers in a bridge constriction is treated as an incremental backwater coefficient designated ΔK_p, which is added to the base curve coefficient. The value of the incremental backwater coefficient, ΔK_p, is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening, the type of piers (or piling in the case of pile bents), the value of the bridge opening ratio, M, and the skew of the piers to the direction of flood flow. The ratio of the water area occupied by piers A_p, to the gross water area of the constriction, A_{n2}, both based on the normal water surface, is assigned the letter J.

In computing the gross water area, A_{n2}, the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from, **Figure 13.82**.

The procedure is to enter chart A on **Figure 13.82** with the proper value J and read ΔK, and then obtain the correction factor, σ, from chart B for opening ratios other than unity. The incremental backwater coefficient is then shown in **Equation (13.122)**:

$$\Delta K_p = \sigma \Delta K \tag{13.122}$$

The incremental backwater coefficients for pile bents can, for all practical purposes, be considered independent of diameter, width, or spacing of piles, but should be increased if there are more than five piles in a bent. A bent with 10 piles should be given a value of K_p about 20 per cent higher than those shown for bents with five piles. If there is a possibility of debris collecting on the piers, it is advisable to use a larger value of J to compensate for the added obstruction.

For a normal crossing with piers, the total backwater coefficient becomes (**Equation (13.123)**):

$$k^* = k_p(\text{Figure 13.81}) + \Delta K_p \text{ (Figure 13.82)} \tag{13.123}$$

13.7.9.4 EFFECTS OF PIERS (SKEW CROSSINGS)

In the case of skew crossings, the effect of piers is treated as explained for normal crossings except for the computation of J, A_{n2} and M. The pier area for a skew crossing, A_p, is the sum of the individual pier areas normal to the general direction of flow, as illustrated by the sketch on **Figure 13.82**. Note how the width of pier, W_p, is measured when the pier in not parallel to the general direction of flow. The area of the constriction, A_{n2}, for skew crossings, is based on the projected length of bridge, b_s cos $\emptyset$ (**Figure 13.82**). Again, A_{n2} is a gross value and includes the area occupied by piers. The value of J is the pier area, A_p, divided by the projected gross area of the bridge constriction, both measured normal to the general direction of flow.

13.7.9.5 EFFECT OF ECCENTRICITY

Referring to the sketch on **Figure 13.83**, it can be seen that the symbols Q_a and Q_c at section 1 are used to represent the portion of the discharge obstructed by the approach embankments. If the cross-section is very asymmetrical, so that Q_a is less than 20 per cent of Q_c, or vice versa, the backwater coefficient will be somewhat larger than for comparable values of M shown on the base curves. The magnitude of the incremental backwater coefficient, ΔK_e accounting for the effect of eccentricity, is shown on **Figure 13.83**.

Eccentricity, e, is defined as 1 minus the ratio of the lesser to the greater discharge outside the projected length of the bridge, or as shown in **Equation (13.124)**:

$$e = \left(1 - \frac{Q_c}{Q_a}\right) with Q_c < Q_a \tag{13.124}$$

or **Equation (13.125)**:

$$e = \left(1 - \frac{Q_a}{Q_c}\right) with Q_c > Q_a \tag{13.125}$$

Reference to the sketch on **Figure 13.83** will aid in clarifying the terminology for instance, if Q_c/Q_a = 0.05, the eccentricity, e = (1 – 0.05) or 0.95 and the curve for 0.95 on **Figure 13.83** would be used for obtaining ΔK_e.

The largest influence on the backwater coefficient due to eccentricity will occur when a bridge is located adjacent to a bluff where a floodplain exists on only one side and the eccentricity is 1.0. The overall backwater coefficient for a very eccentric crossing with wingwall abutments and piers will be **Equation (13.126)**:

$$K^* = K_b(\text{Figure 13.81}) + \Delta K_p \text{ (Figure 13.82)} + \Delta K_e \text{ (Figure 13.83)} \tag{13.126}$$

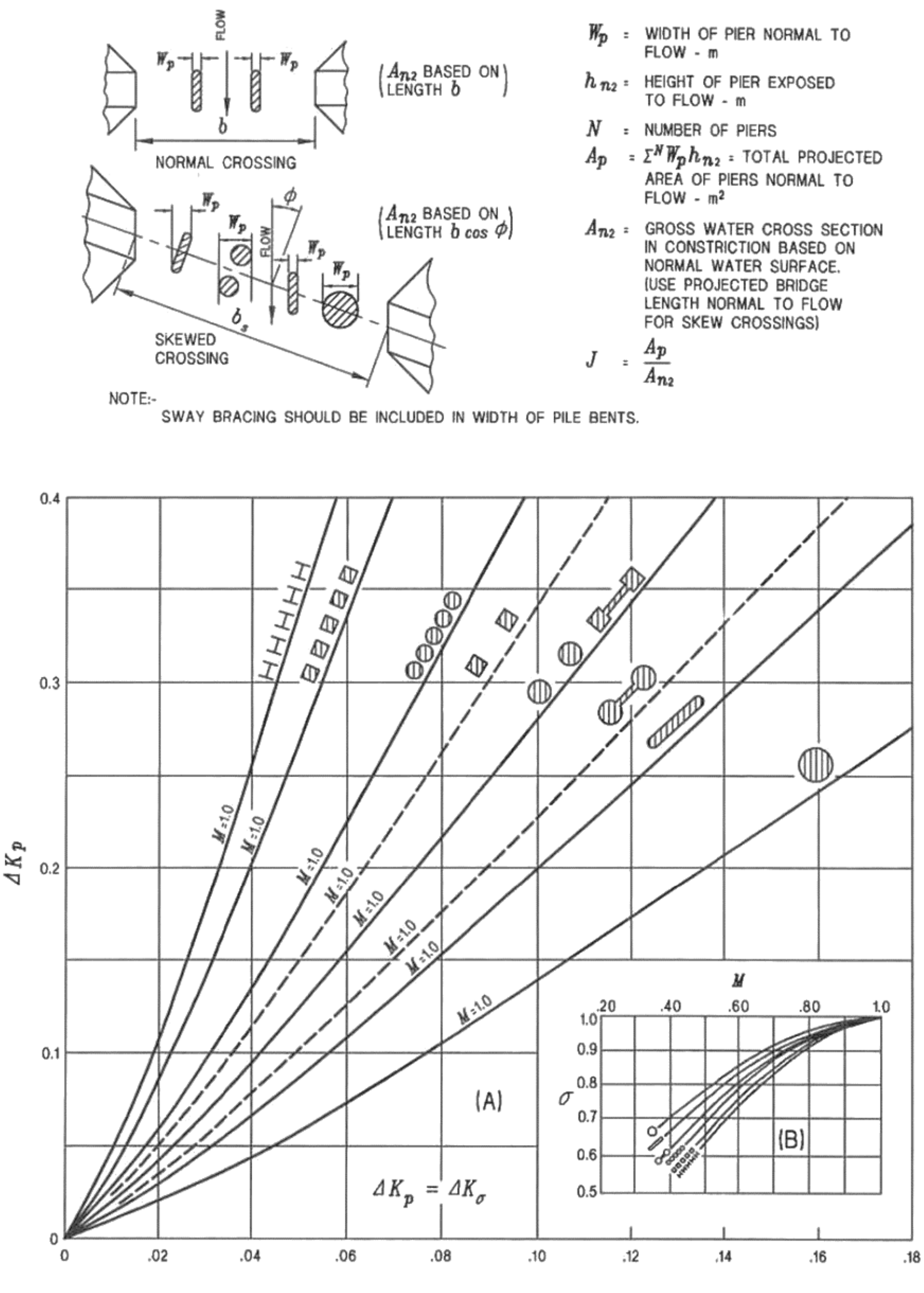


Figure 13.82 Incremental backwater coefficient for piers

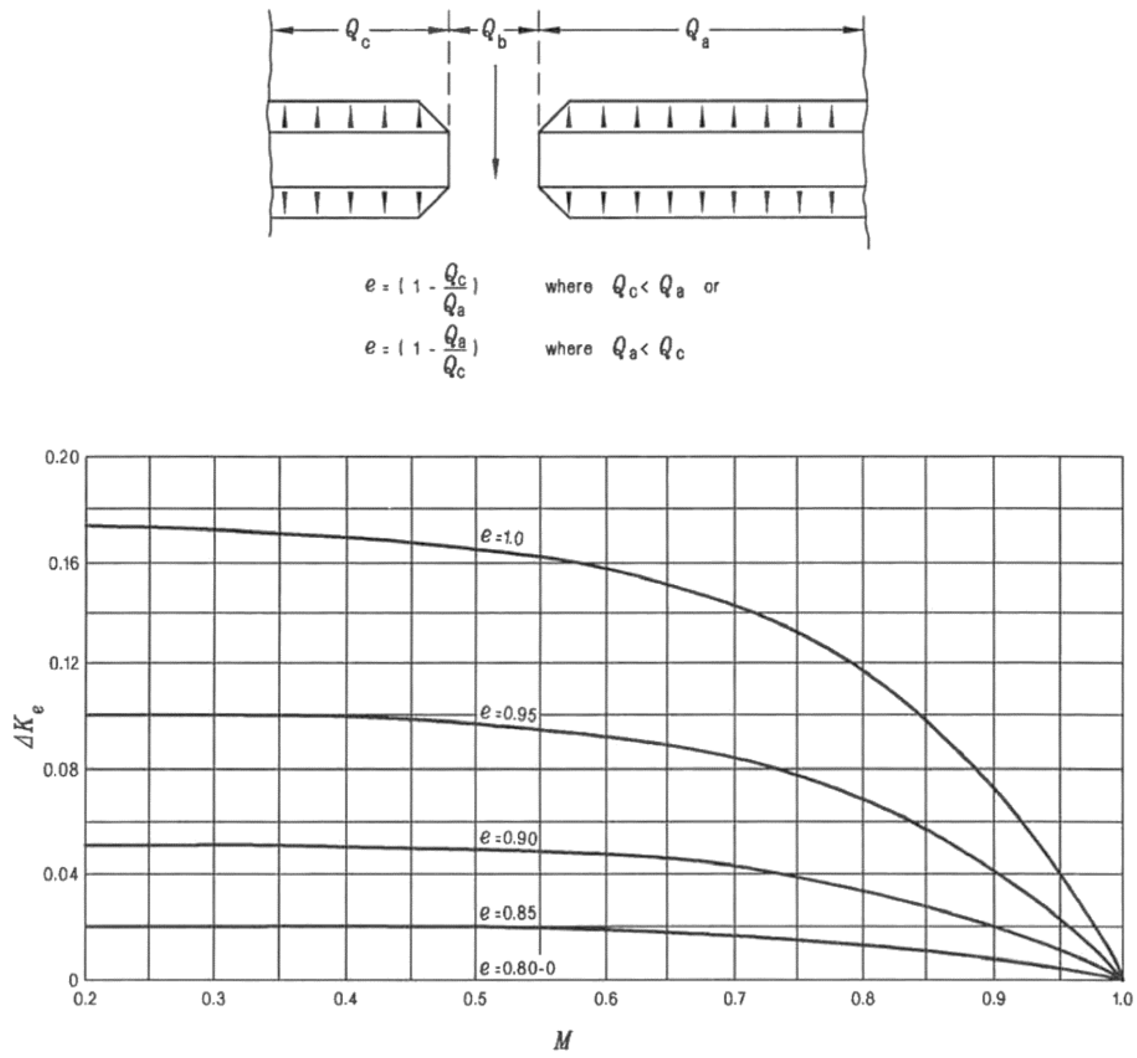


Figure 13.83 Incremental backwater coefficient for eccentricity

13.7.9.6 EFFECTS OF SKEW

The method of computation for skew crossings differs from that of normal crossings in that the bridge opening ratio, M, is computed on the projected length of bridge rather than on the full length. The length is obtained by projecting the bridge opening upstream parallel to the general direction of flood flow as illustrated on **Figure 13.84**. The general direction of flow is the direction of flood flow as it existed prior to the placement of embankments in the stream. The length of the constricted opening is $b \cos \emptyset$ and the area, A_{n2}, is based on this length. The velocity head, $V^2_{n2}/2g$, to be substituted in the expression for computing backwater based on the projected area A_{n2}.

Figure 13.85 shows the incremental backwater coefficient, ΔK_e, for the effect of skew, for wingwall and spill-through type abutments. The incremental coefficient varies with the opening ratio, M, the angle of skew of the bridge, $\emptyset$, with the general direction of flood flow, and the alignment of the abutment faces, as indicated by the sketch on **Figure 13.85**.

It should be noted that the incremental backwater coefficient, ΔK_e, can be negative as well as positive. The negative values result from the method of computation and do not indicate that the backwater will be reduced by employing a skew crossing. These incremental values are to be added algebraically to ΔK_b, obtained from the base curve. The total backwater coefficient for a skew crossing with abutment faces aligned with the flow and piers would be (**Equation**

(13.127)):

$$k^* = k_b(\text{Figure 13.81}) + \Delta K_p\ (\text{Figure 13.82}) + \Delta K_e\ (\text{Figure 13.85}) \qquad (13.127)$$

From the model tests carried out it was found that crossings with skew up to an angle of 20° produced no particularly objectionable results for any of the four abutment shapes investigated. As the angle increased above 20°, however, the flow pattern deteriorated; flow concentrations at abutments produced large eddies, reducing the efficiency of the waterway and increasing the potential for scour.

The above statement does not apply to cases where a bridge spans practically an entire valley and there is little constriction of the flow.

Figure 13.86 was prepared from the same model data as **Figure 13.85 (A)**. By entering **Figure 13.86** with the angle of skew and the projected value of M, the ratio $bs\ \cos\ \emptyset\ /b$ can be read from the ordinate. Knowing b and h_1^* for a comparable normal crossing, one can solve for b_s, the length of opening needed for a skewed bridge to produce the same amount of backwater for the design discharge.

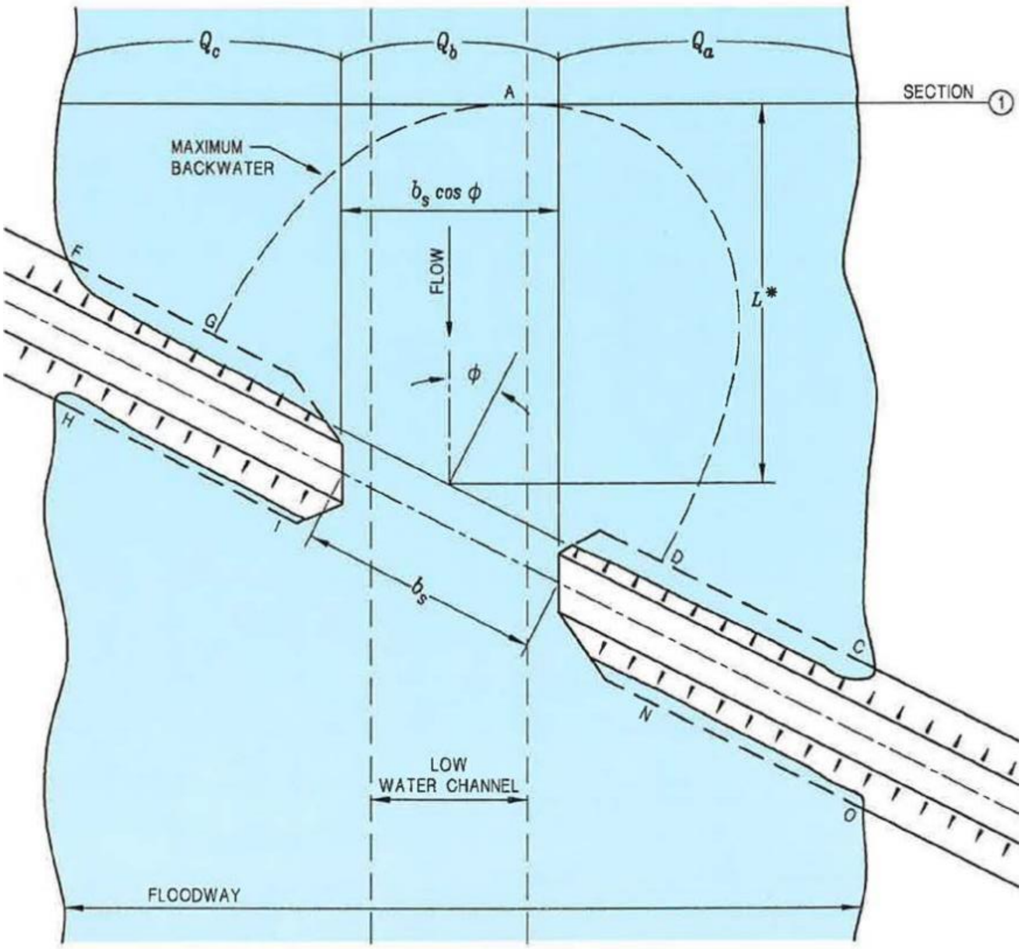


Figure 13.84 Skewed crossing

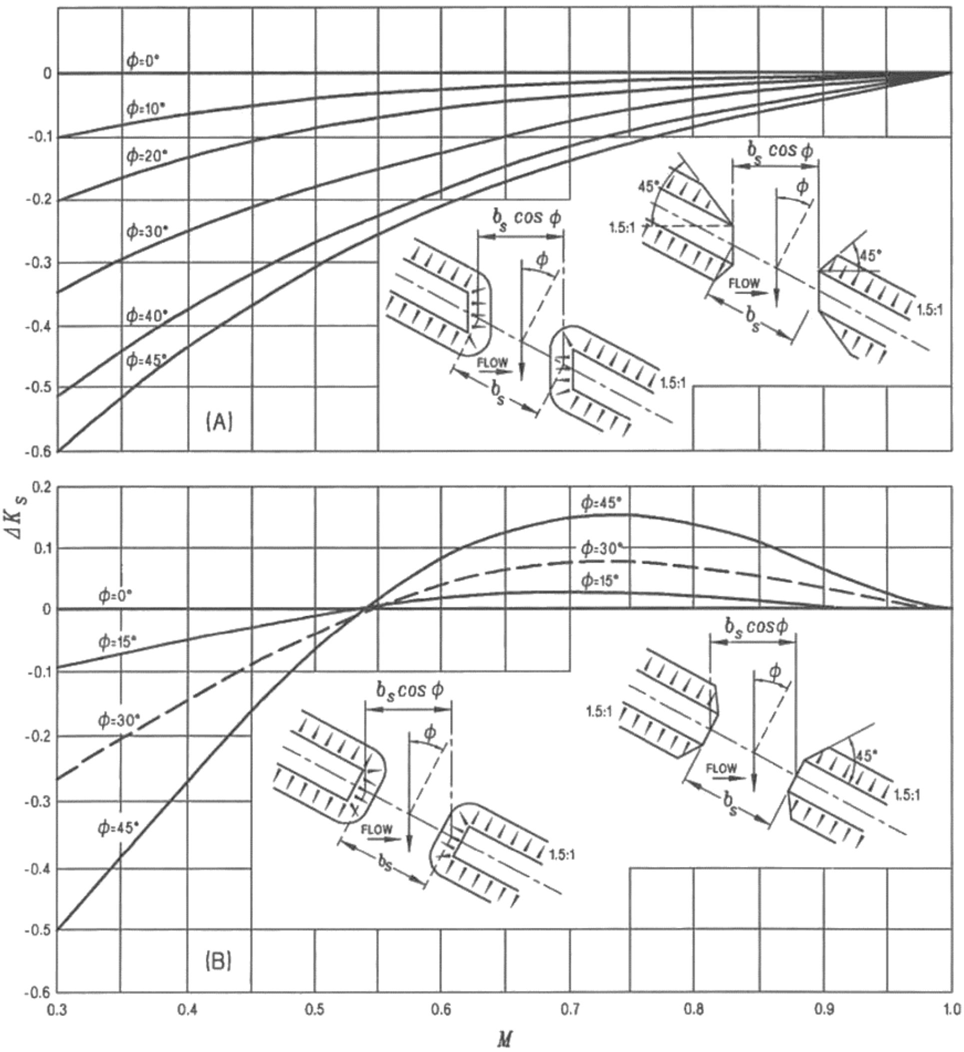


Figure 13.85 Incremental backwater coefficient for skew

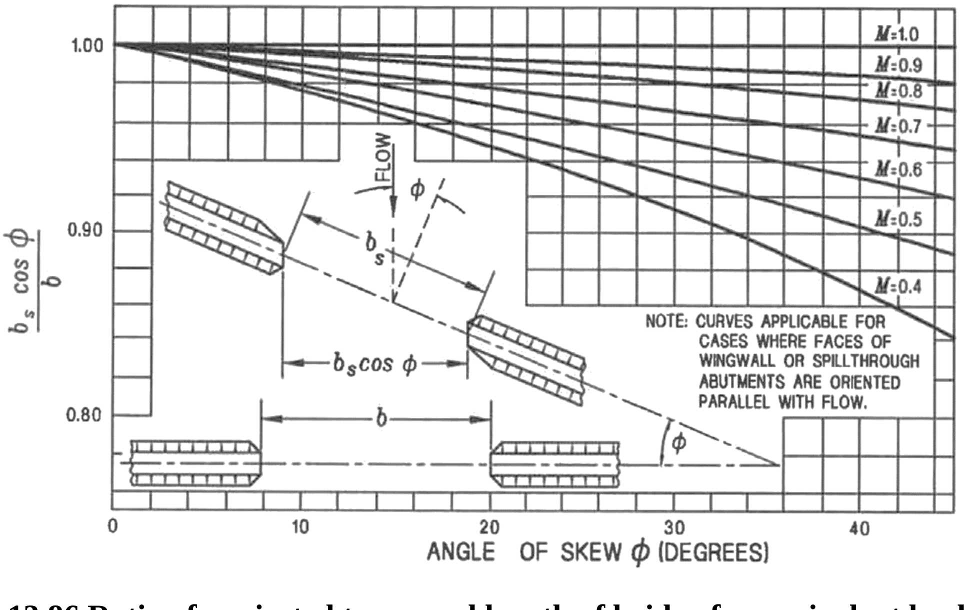


Figure 13.86 Ratio of projected to normal length of bridge for equivalent backwater (skewed crossing)

13.7.9.7 EFFECT OF DUAL BRIDGES

The backwater produced by dual bridges (such as two bridges, side by side, on a dual carriageway) is larger than that for a single bridge, yet less than the value which would result from considering the bridges separately.

When two bridges are in close proximity to one another, the flow pattern is elongated, but little different from that of a single bridge. As the bridges are spaced further apart, the embankment of the second bridge interferes with the expanding jet of the first, which must again contract and then re-expand downstream from the second bridge, producing additional turbulence and loss of energy. This is reflected in a larger backwater upstream of the first bridge.

To determine backwater for dual bridges which are essentially identical in design and parallel, and have an L_d/l, (see **Figure 13.87**) of not more than 11, it is first necessary to compute the backwater, h_1^* for a single bridge. The backwater for the dual combination, measured upstream from the first bridge is then (**Equation (13.128)**):

$$h_d^* = h_1^*\eta \tag{13.128}$$

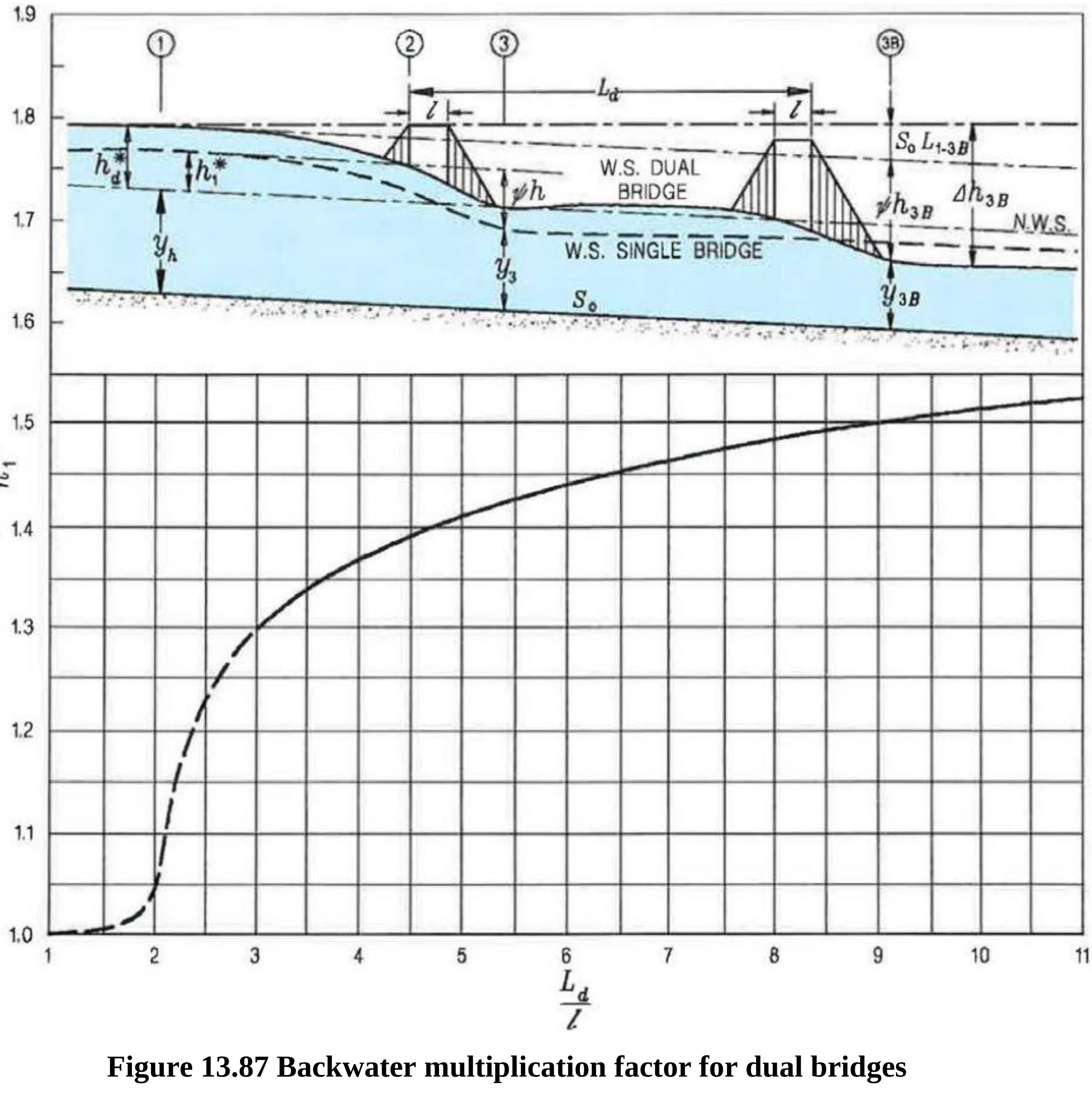


Figure 13.87 Backwater multiplication factor for dual bridges

13.7.10 DIFFERENCE IN WATER LEVEL ACROSS APPROACH EMBANKMENTS

The difference in water surface elevation between the upstream and downstream side of bridge approach embankments, Δh, has been interpreted erroneously as the backwater produced by a bridge. This is not the backwater as the sketch on **Figure 13.88** will attest. The water surface at section 3 (**Figure 13.76**), measured along the downstream side of the embankment, is lower than normal stage by the amount h_3^*.

The difference in level across embankments is always larger than the backwater, h_1^*, by the sum $h_3^* + S_0L_{1-3}$, where S_0 is the natural slope of the stream (**Figure 13.88**). The method of determining L_{1-3}, which is the distance from section 1 to section 3 (**Figure 13.76**) is given in the Section, "Location of Maximum Backwater". The differential level is significant in the determination of backwater at bridges in the field, since Δh is the most reliable head measurement that can be made.

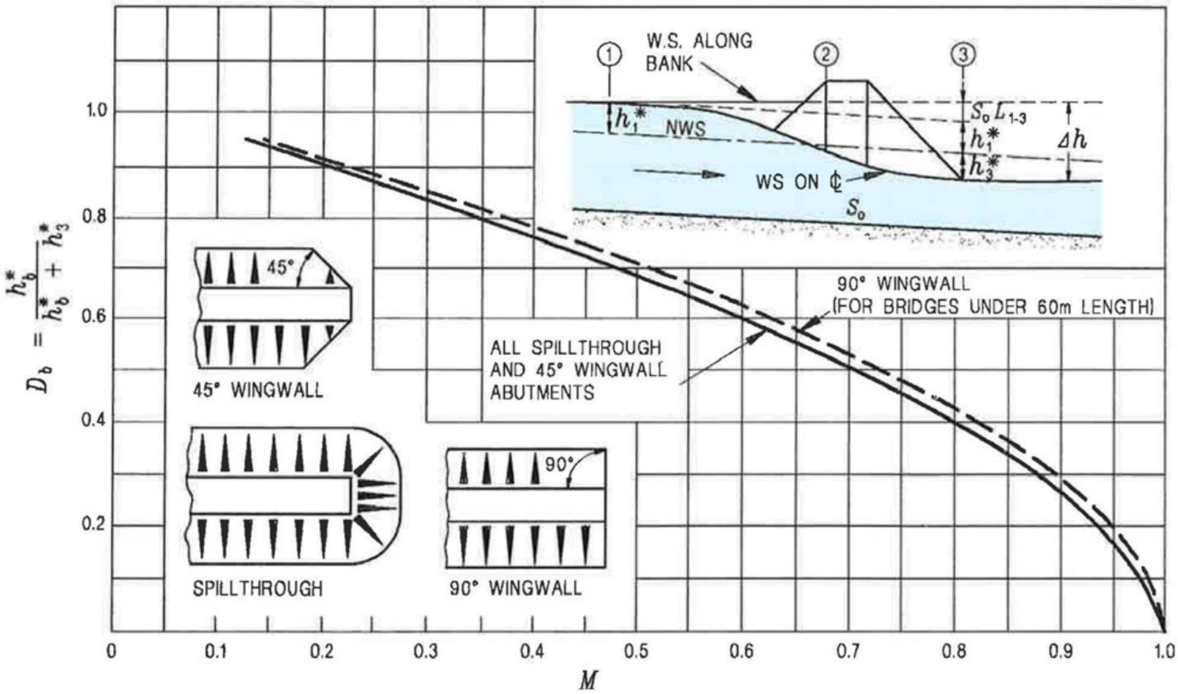


Figure 13.88 Differential water level ratio base curves

13.7.10.1 BASE CURVES

The base curves for determining downstream levels given on Figure 4.16 have been constructed entirely from model data.

To determine Δh it is first necessary to compute the backwater, h_b^*, for a normal crossing, without piers, eccentricity, or skew. The relevant curve for abutment type is then entered on **Figure 13.88** with the contraction ratio, M, to obtain the differential level ratio (**Equation (13.129)**):

$$D_b = \frac{h_b^*}{h_b^* + h_3^*} \tag{13.129}$$

or **Equation (13.130)**:

$$h_3^* = h_b^* \left[\frac{1}{D_b} - 1\right] \tag{13.130}$$

The elevation on the downstream side of the embankment is simply normal stage at section 3

(**Figure 13.76**), less h_3^*.

13.7.10.2 EFFECT OF PIERS

It was found during the model study that the introduction of piers into the bridge constrictions increased backwater, whilst the h_3^* showed no measurable change. Hence, no correction for piers is required when determining Δh.

13.7.10.3 EFFECT OF ECCENTRICITY

In the case of severely eccentric crossings, the difference in level across the embankment applies only to the side of the river having the greater floodplain discharge. In plotting the experimental differential level ratios with respect to M for eccentric crossings, it was found that the points fell directly on the base curve (**Figure 13.88**). The individual values of h; and h; for eccentric crossings are different from those for symmetrical crossings, but the ratio of one to the other, for any given value of M, remains unchanged. Thus, **Figure 13.88** can also be considered applicable to eccentric crossings if used correctly.

To obtain h_3^* for an eccentric crossing, with or without piers, enter the appropriate curve on **Figure 13.88** with the value of M and read D_b as before. In this case **Equation (13.131)**:

$$D_b = \frac{h_b^* + \Delta h_e^*}{h_b^* + \Delta h_e^* + h_3^*} \tag{13.131}$$

or **Equation (13.132)**:

$$h_3^* = (h_b^* + \Delta h_e^*)\left[\frac{1}{D_b} - 1\right] \tag{13.132}$$

Where,

$$\Delta h_e^* = \Delta K_e \alpha_2 \frac{V_{n2}^2}{2g} \tag{13.133}$$

13.7.10.4 DROP IN WATER SURFACE ACROSS EMBANKMENT (NORMAL CROSSING)

Having computed h_3* and knowing the total backwater h_1*, the difference in water surface elevation across the embankment is **Equation (13.134)**:

$$\Delta h = h_3^* + h_1^* + S_0 L_{1-3} \tag{13.134}$$

Where,

h_1* = total backwater (m), including the effect of piers and eccentricity.

S_0L_{1-3} = the normal fall in stream bed between sections 1 and 3 (**Figure 13.76**).

13.7.10.5 WATER SURFACE ON DOWNSTREAM SIDE OF EMBANKMENT (SKEWED CROSSING)

Individual values of h_1^* and h_3^* for skewed crossings again differ from those for symmetrical crossings, but the differential level ratio across the embankments at either end of the bridge can be considered the same as for normal crossings for any given value of M. The value of M, is of course, based on the projected length of bridge as explained in **Section 13.7.9** for the effect of skew. Thus, it is again possible to use **Figure 13.88** for skewed crossings. The differential level ratio, D_b, with or without piers, is obtained by entering the chart with the appropriate opening ratio, M. Then **Equation (13.135)**:

$$h_3^* = (h_b^* + \Delta h_s^*)\left[\frac{1}{D_b} - 1\right] \qquad (13.135)$$

13.7.11 LOCATION OF MAXIMUM BACKWATER

The maximum backwater occurs at point A upstream of the bridge centre line and at a distance L^* from the water line on the upstream side of the embankment (**Figure 13.89**). Where the floodplains on each side of the main channel are no wider than twice the bridge length and the hydraulic roughness is low then the water level in the areas ABCD and AEFG will be approximately the same as point A.

However, for wide and rough floodplains, there will be a flow gradient along the upstream side of the embankments. These flow gradients are likely to be more pronounced on the falling than rising stage of the flood.

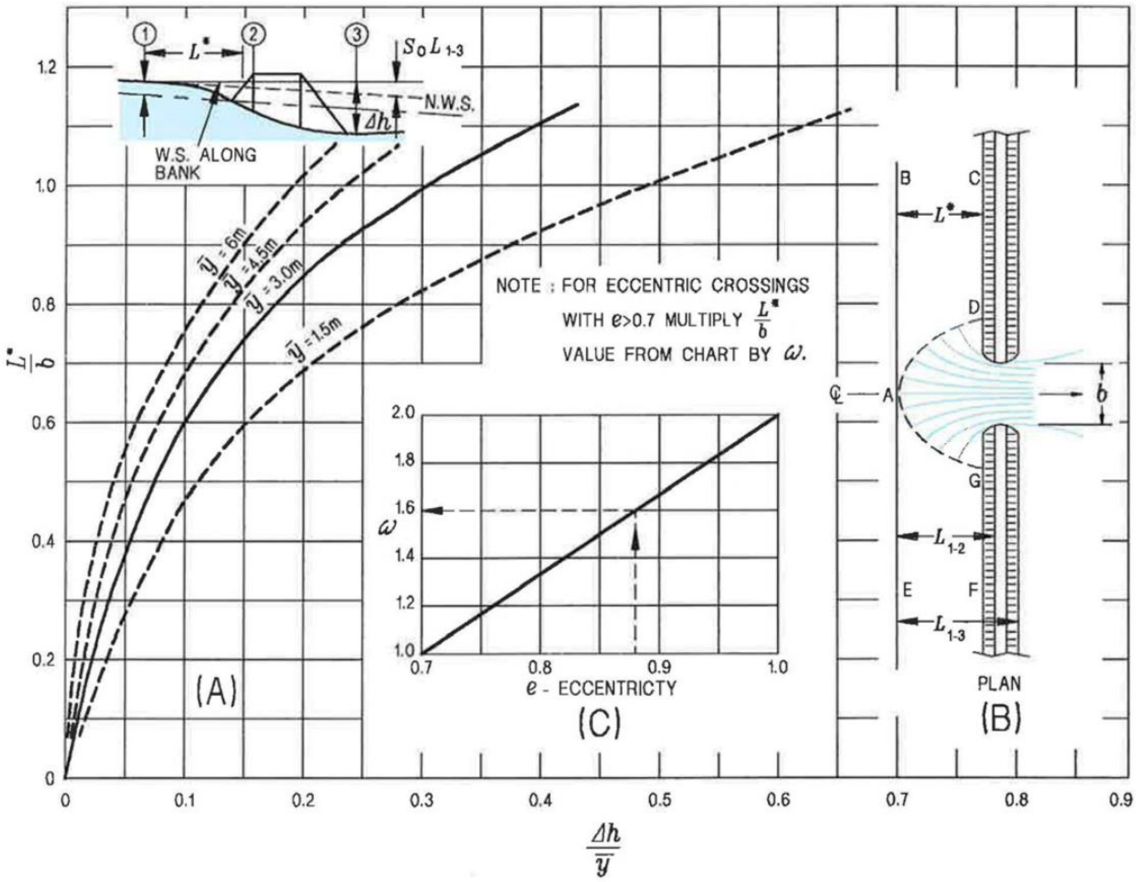


Figure 13.89 Distance to maximum backwater

13.7.11.1 NORMAL CROSSINGS

For normal crossings the distance to maximum backwater, L^*, may be obtained from **Figure 13.89** with:

- Δh = difference in water surface elevation (m) across embankment
- $\bar{y} = A_{n2}/b$
- A_{n2} = cross-sectional area (m^2) under the bridge below normal water surface
- b = width (m) of waterway.

A trial solution is required for determining the differential level across the embankments, Δh, but from the result of the backwater computation it is possible to make a fair estimate of Δh. To obtain the distance to maximum backwater for a normal constriction, use **Figure 13.89** with appropriate values of $\Delta h/y$ and obtain the corresponding value of L^*/b. Solving for L^* and adding to this the additional distance to section 3, which is known, gives the distance L_{1-3}. Then the computed difference in level across the embankment is (**Equation (13.136)**):

$$\Delta h = h_1^* + h_3^* + S_0 L_{1-3} \tag{13.136}$$

Should the computed value of Δh differ materially from the one chosen, the above procedure should be repeated until assumed and computed values agree.

13.7.11.2 ECCENTRIC CROSSINGS

Eccentric crossings with extreme asymmetry perform much like one half of a normal symmetrical crossing with a marked contraction of the jet on one side and very little contraction on the other.

For cases where the value of e is greater than 0.7, enter the abscissa on **Figure 13.89 (A)** with $\Delta h/\bar{y}$ and $\bar{y}$ and read off the corresponding value of L^*/b as usual. Next multiply this value by L^*/b by a correction factor, ω, which is obtained from **Figure 13.89 (C)**.

13.7.11.3 SKEWED CROSSINGS

In the case of skewed crossings, the water surface elevations along opposite banks of a stream are usually different than at point A. One may be higher and the other lower depending on the angle of skew, the configuration of the approach channel and other factors. To obtain the approximate distance to maximum backwater, L^*, for skewed crossings (**Figure 13.84**), the same procedure is recommended as for normal crossings except the ordinate of **Figure 13.89** is read as L^*/b_s, where is the full length of the skewed bridge.

13.7.12 EFFECT OF SCOUR ON BACKWATER

13.7.12.1 GENERAL

The estimation of backwater in the preceding Sections has been limited to the case where scour has not occurred. In actual practice where embankments have constricted the flow causing backwater and higher velocities through the bridge opening, scour will occur where the stream bed is composed of loose or soft material. If a flood persists for a sufficient period of time, equilibrium conditions will eventually result from the increase in waterway area, resultant reduction in backwater and velocity, and reduced capacity of the flow to cause further scour.

Figure 13.90 shows the effect of scour on bridge backwater.

In cases where the bridge foundations can be adequately protected and there is no adverse environmental impact, it may be permissible to encourage scour in the interest of utilising a shorter bridge. The same objective can be attained by enlarging the waterway area under a bridge by excavation during construction.

In such cases it is desirable to be able to determine the amount of backwater to be expected with increase in the waterway area.

13.7.12.2 BACKWATER DETERMINATION

A design curve derived from model experiments is included as **Figure 13.91**. The correction factor for scour, ($C = h^*_{1x}/h^*_1$), is plotted with respect to A_s/A_{n2}, where A_s is the additional area due to scour at the constriction and the other terms bearing the subscript, s designate values with scour and those not bearing this subscript represent the same values computed without scour.

Supposing the backwater at a given bridge was 0.3 m with no scour; it would be reduced to 0.16 m if scour increased the waterway area by 50 per cent, and it would be reduced to 0.09 m should the waterway area be doubled.

The same reduction applies equally well to the ratios h^*_{3x} $/h^*_3$ and $\psi\psi hss/\psi\psi h$ (see **Figure 13.90**) so one curve suffices for all three. Thus, to obtain backwater and related information for bridge sites where scour is to be encouraged or cannot be avoided, or where the waterway is to be enlarged during construction, it is first necessary to compute the backwater and other quantities desired according to the method outlined in **Section 13.7.9** for a rigid bed, using the original cross-section of the stream at the bridge site. These values are then multiplied by a common coefficient from **Figure 13.91** as follows (**Equation (13.137)**):

$$h_{1s} = Ch_1^* \tag{13.137}$$

$$h_{3s}^* = Ch_3^*$$

$$\psi h_s = C\psi h$$

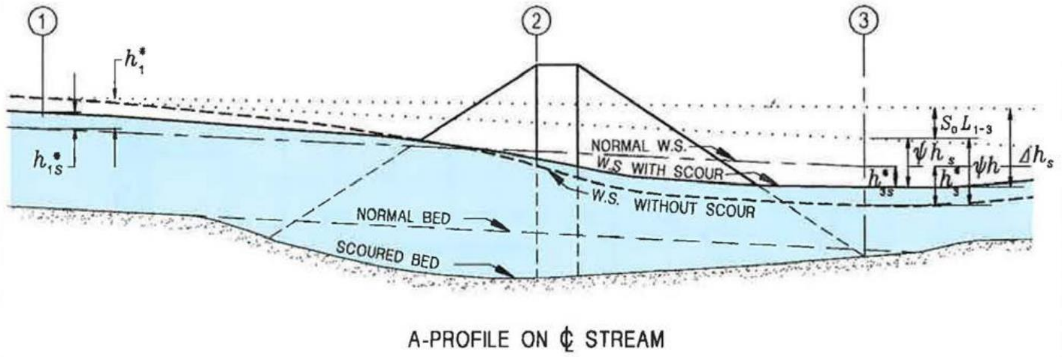


Figure 13.90 Effect of scour on bridge backwater

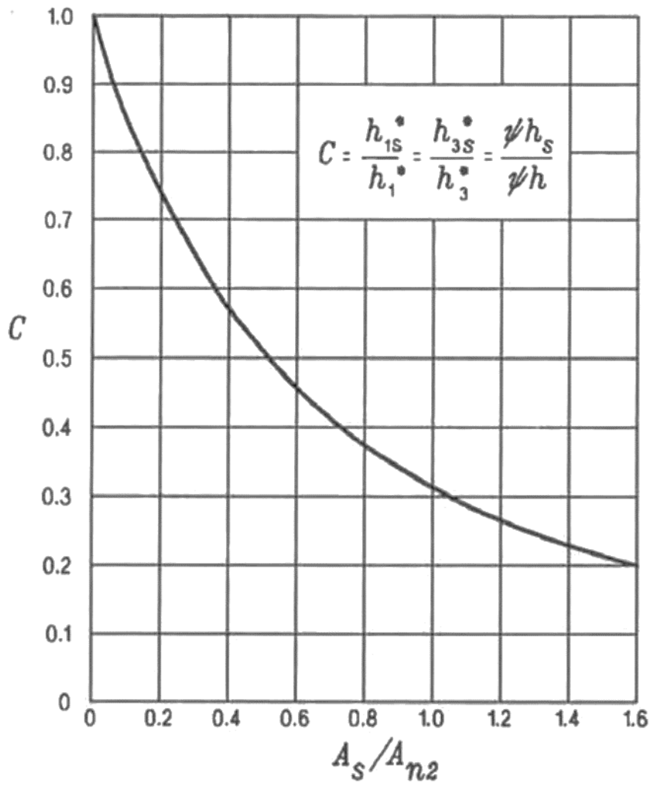


Figure 13.91 Correction factor for backwater with scour

13.7.12.3 ENLARGED WATERWAYS

The design engineer will probably be reluctant to depend on scour as a means of enlarging a waterway and thereby reducing backwater. If the waterway is enlarged by excavation, there is little to gain by excavating much beyond the limits (upstream and downstream) of the embankments, as the downstream channel acts as a control. If additional volume is removed upstream or downstream, the channel may simply refill by deposition. Any enlargement of the cross-section should be maintained to prevent reduction of area by the growth of vegetation.

Any proposal to reduce backwater by excavation to enlarge a bridge waterway area should be carefully studied. If there is reason to believe that the enlarged area cannot be maintained or that the stability of the stream will be disturbed, alternate solutions such as additional spans should be considered.

13.7.13 SUPERSTRUCTURE PARTIALLY INUNDATED

Cases arise in which it is desirable to compute the backwater upstream from a bridge or the discharge under a bridge when flow is in contact with the girders. Once flow contacts the upstream girder of a bridge, orifice flow is established so the discharge then varies as the square root of the effective head. The result is a rather rapid increase in discharge for a moderate increase in backwater. The greater discharge, of course, increases the likelihood of scour under the bridge.

Two cases are considered below; the first where only the upstream girder is in the water as indicated by the sketch on **Figure 13.92** and the second, where the bridge constriction is flowing full, all girders in the flow, as shown in **Figure 13.93**.

Where the normal water surface is higher than the soffits of the girders, backwater analyses should be carried out on the alternative assumptions that:

1. the upstream girder is in contact with the flow
2. all girders are in contact with the flow.

The higher of the two results should be adopted.

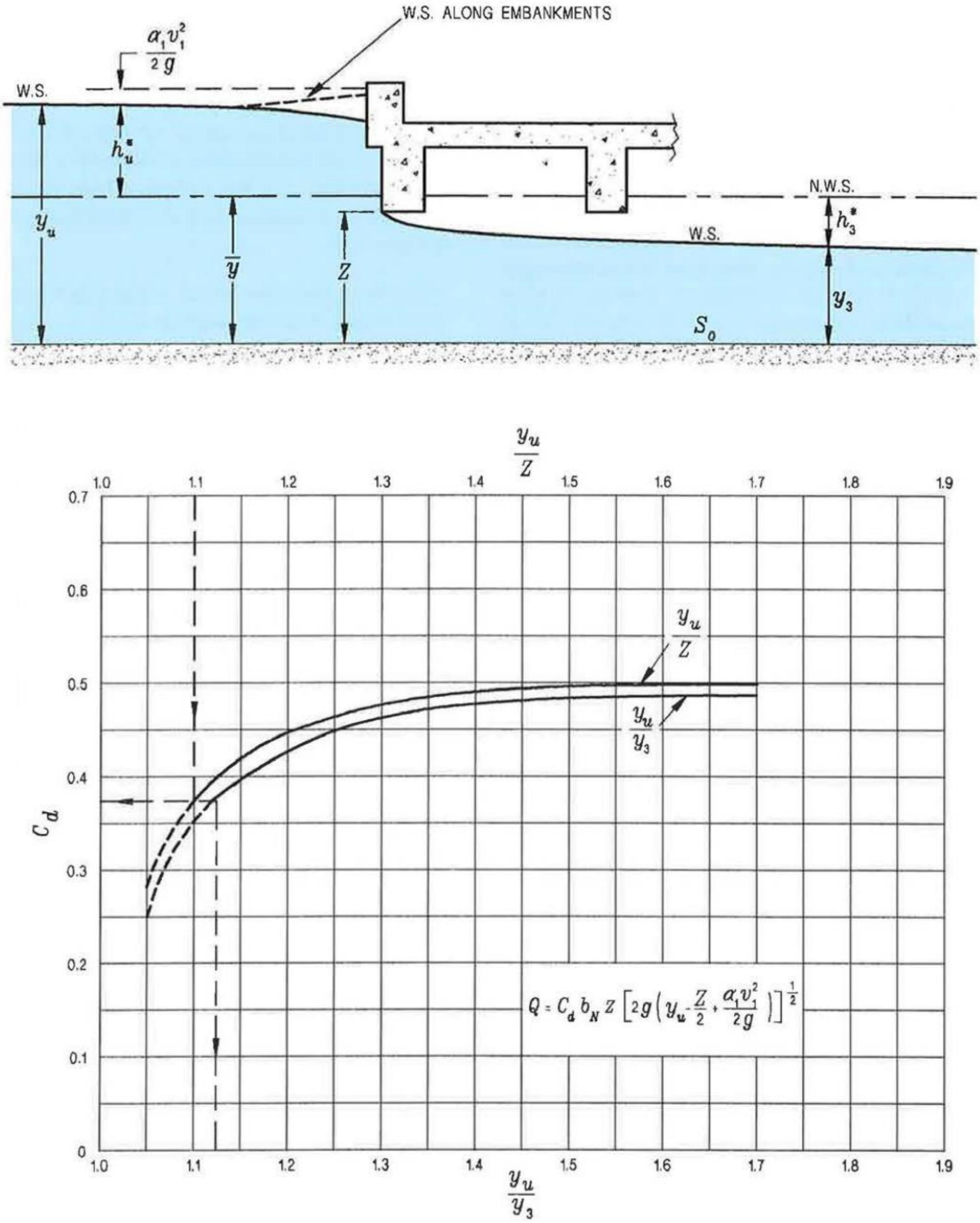
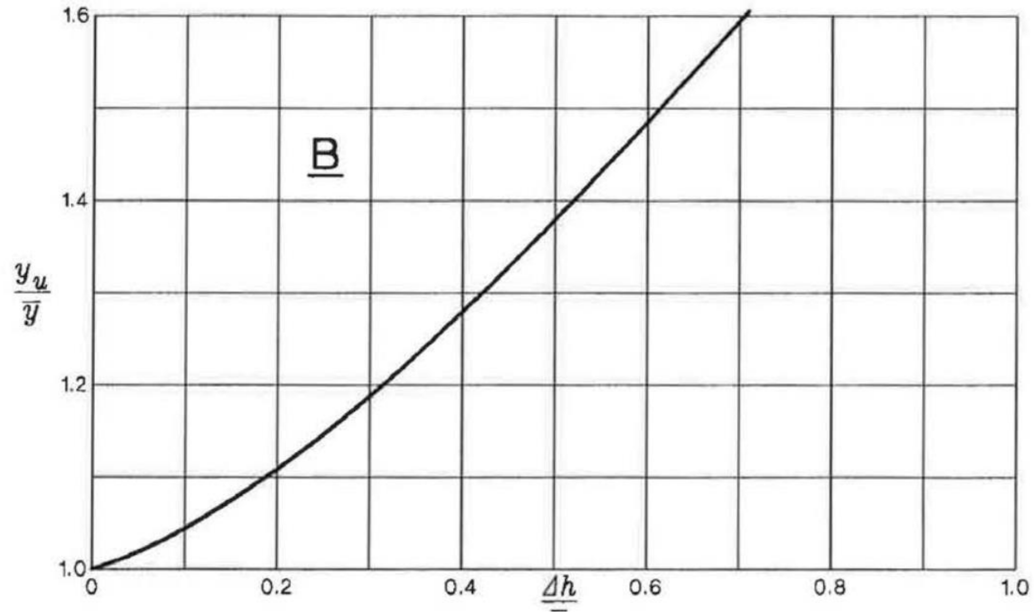


Figure 13.92 Discharge coefficients for upstream girder in flow (Case I)



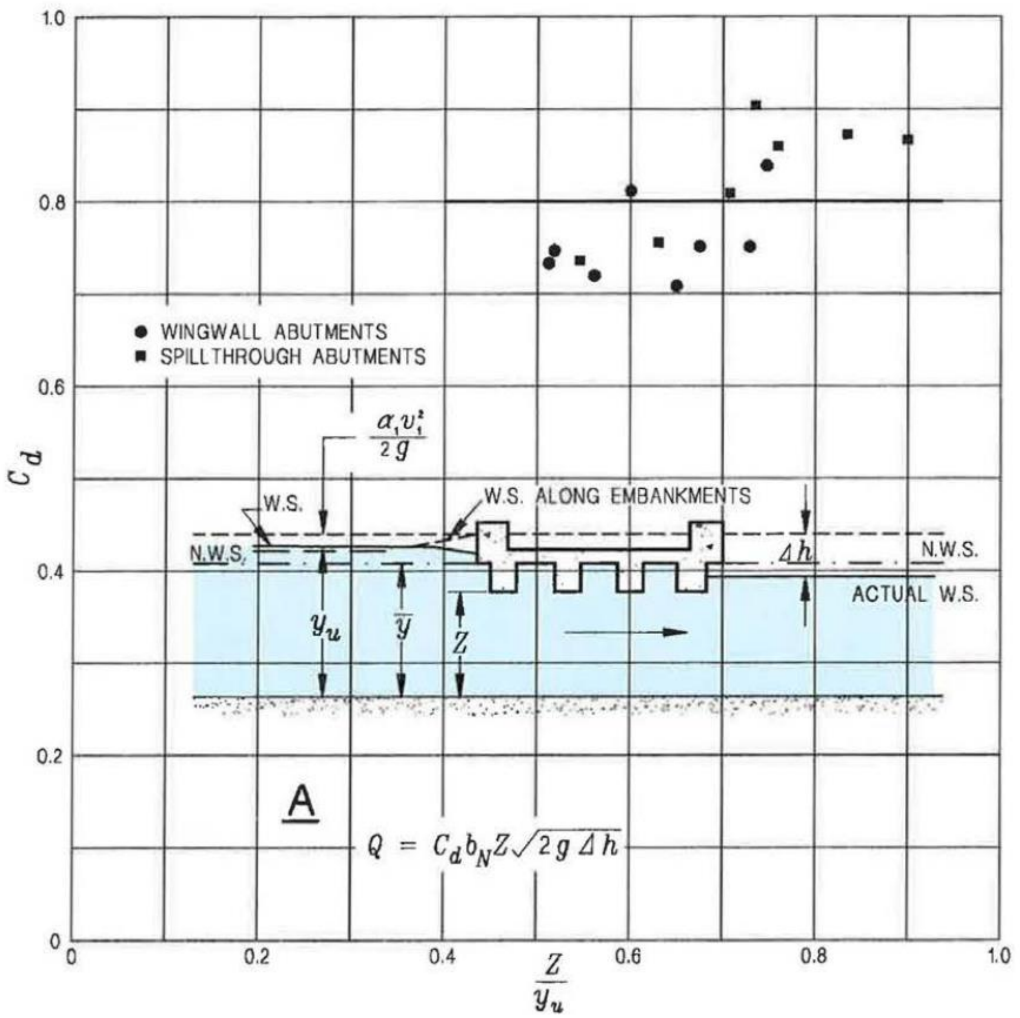


Figure 13.93 Discharge coefficients for all girders in flow (Case ll)

13.7.13.1 UPSTREAM GIRDER IN FLOW (CASE I)

The most logical and simple method of analysis is to treat this flow condition as a sluice gate problem.

Using a common expression for sluice gate flow (**Equation (13.138)**):

$$Q = C_d b_N Z \left[2g \left(y_u - \frac{Z}{2} + \frac{\alpha_1 V_1^2}{2g} \right) \right]^{1/2} \tag{13.138}$$

Where,

- Q = total discharge (m3/s)
- C_d = coefficient of discharge
- b_N = net width (m) of waterway (excluding piers)
- Z = vertical distance (m) under bridge from bottom of upstream girder to mean river bed level
- y_u = vertical distance (m) of upstream water surface to mean river bed at bridge

For Case I, the coefficient of discharge, C_d is plotted with respect to the parameter y_u/Z on **Figure 13.92**. The upper curve applies to the coefficient of discharge where only the upstream girder is in contact with the flow.

By substituting values in the above expression, it is possible to solve for either the water surface upstream or the discharge under the bridge, depending on the quantities known. It would appear that the coefficient curve (**Figure 13.92**) approaches zero as y_u/Z becomes unity. This is not the case since the limiting value of $y_u/Z$for which the expression applies is not much less than 1.1. There is a transition zone somewhere between y_u/Z = 1.0 and 1.1 where free surface flow changes to orifice flow or vice versa. The type of flow within this range is unpredictable. For y_u/Z = 1.0, the flow is dependent on the natural slope of the stream, while this factor is of little concern after orifice flow is established or y_u/Z = 1.1.

In computing a general river backwater curve across the bridge as shown on **Figure 13.92**, it is necessary to know the water surface elevation downstream as well as upstream from the bridge. The approximate depth of flow, y_3, can be obtained from **Figure 13.92** by entering the top scale with the proper value of y_u/Z and reading down to the upper curve, then over horizontally to the lower curve, and finally down to the lower scale as shown by the arrows. The lower scale gives the ratio of y_u/y_3.

13.7.13.2 ALL GIRDERS IN CONTACT WITH FLOW (CASE II)

Where the entire area under the bridge is occupied by the flow, the computation is handled in a different manner. To compute the water surface upstream from the bridge, the water surface on the downstream side and the discharge must be known. Or if the discharge is desired, the drop in water surface across the roadway embankment Δh, and the net area under the bridge is required. The experimental points on **Figure 13.93** (A), which are for both wingwall and spill through abutments, show the coefficient of is charge to be essentially constant at 0.80 for the range of conditions tested. The equation recommended for the average two to four lane concrete girder bridge for Case II is (**Equation (13.13)**):

$$Q = 0.8 b_N Z (2g\Delta h)^{\frac{1}{2}} \tag{13.139}$$

Where the symbols are defined as in the expression for Case I. Here the net width of waterway (excluding width of piers) is used again. It is preferable to measure Δh across embankments rather than at the bridge proper. The partially inundated bridge behaves in a similar way to a submerged box culvert but on a larger scale. Submergence, of course, can increase the likelihood of scour under a bridge.

For working out general backwater curves for a river, it is desirable to know the drop in water level across existing bridges as well as the actual water surface elevation either upstream or downstream from the bridge.

Once Δh is computed from the above expression the depth of flow upstream, y_u can be obtained from chart B, **Figure 13.93** where $\bar{y}$ is the depth from normal stage to mean river bed at the bridge in metres.

13.7.14 FLOW PASSES THROUGH CRITICAL DEPTH (TYPE II)

13.7.14.1 GENERAL

The computation of backwater for bridges on streams with fairly steep gradients, by the method outlined for Type I flow (refer **Figure 13.70**), may result in unrealistic values. When this occurs, it probably indicates that the flow encountered is Type II, and the backwater analysis for subcritical or Type I flow does not apply.

The water surface for Type IIA flow passes through critical stage under the bridge but returns to normal or sub-critical flow some distance downstream. In the case of Type IIB flow, the water surface passes through critical stage under the bridge and then dips below critical stage downstream. The sole source of data for Type II flow is from model studies, which cover a limited range of contraction ratios.

13.7.14.2 BACKWATER COEFFICIENTS

The expression for the backwater coefficient for Type II flow is (**Equation (13.140)**):

$$C_b = \frac{h_1^* + \bar{y} - y_{2c}}{\alpha_2 V_{2c}^2/2g} + \frac{\alpha_1}{\alpha_2}\left[\frac{V_1}{V_{2c}}\right]^2 - 1 \qquad (13.140)$$

Where,

- $\bar{y}$ = normal depth (m) in constriction = A_{n2}/b
- y_{2c} = critical depth (m) in constriction = A_{2c}/b
- V_{2c} = critical velocity (m/s) in constriction = Q/A_{2c}
- A_{2c} = area (m^2) in constriction below critical depth
- α_1 = kinetic energy coefficient
- α_2 = velocity head coefficient for the constriction

The backwater coefficient has been assigned the symbol C_b to distinguish it from the coefficient for subcritical flow.

The coefficient curve of **Figure 13.94** accounts for the contraction ratio only, which is the major factor involved.

The effect of piers, eccentricity, and skew have not been evaluated because of the tentative nature of the curve. The incremental coefficients on **Figure 13.85**, **Figure 13.86** and **Figure 13.88** for piers, eccentricity and skew, are not applicable to type II flow problems.

The expression for backwater for Type II flow, with no allowance for piers, eccentricity and skew, is then (**Equation (13.141)**):

$$h_1^* = \alpha_2 \frac{V_{2c}^2}{2g}\,(C_b + 1) - \alpha_1 \frac{V_1^2}{2g} + y_{2c} - \bar{y} \qquad (13.141)$$

13.7.14.3 RECOGNITION OF FLOW TYPE

The prime difficulty here is to determine which type of flow will occur at a proposed bridge site in the field prior to starting backwater calculations. No definite answers can be given since most problems encountered of this nature will be borderline cases. It is suggested that the Type I approach is tried first and if the result appears unrealistic, repeat the backwater calculation using the Type II approach. It is more than likely that the difference in the two results will be great enough to readily spot the erratic one. If the computed backwater for the Type II flow is smaller than that computed for Type I flow, the flow will definitely be Type II.

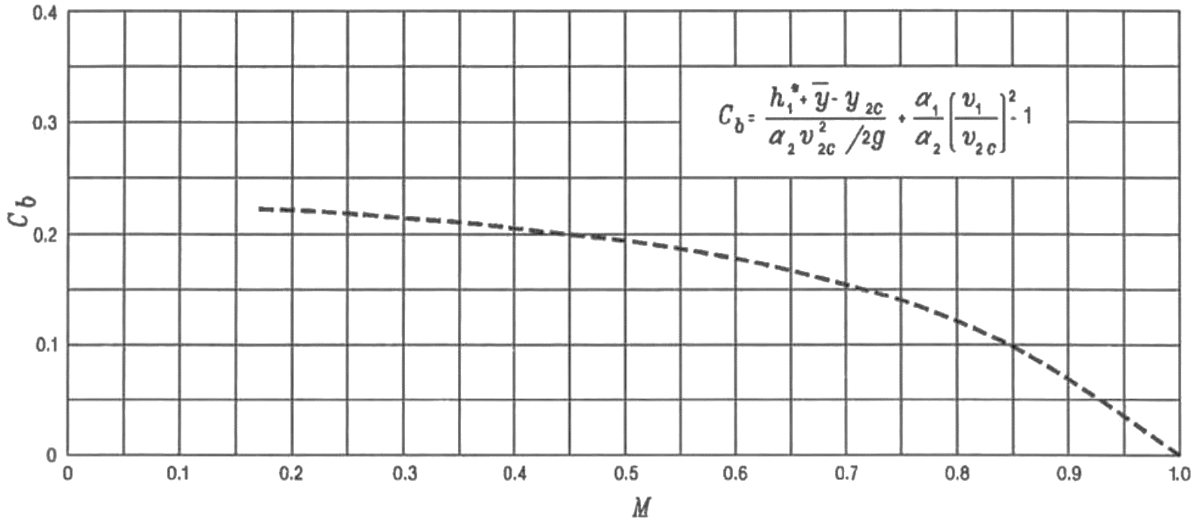


Figure 13.94 Tentative backwater coefficient curve for Type II flow

13.7.15 BRIDGE DECK DRAINAGE DESIGN

The drainage of bridge decks is an important component of bridge performance. The surface water needs to be removed from the bridge deck effectively to minimise the safety hazard of hydroplaning (water on the pavement surface) and prevent corrosion of the structure. The collected run-off needs to be discharged appropriately to meet the environmental requirements, prevent erosion of the surrounding ground and possibly undermining the foundations, and the width of flow on a bridge deck should not exceed that specified for its road approaches.

Every effort should be made to ensure that longitudinal sags are not located on bridges.

Proper bridge deck drainage provides many other benefits, including:

i. Long-term maintenance of the bridge is enhanced.
ii. The structural integrity of the bridge is preserved.
iii. Aesthetics are enhanced (e.g., the avoidance of staining substructure and superstructure members).
iv. Erosion on bridge end slopes is reduced.

13.7.15.1 DRAINAGE OF CARRIAGEWAY

Transverse and longitudinal drainage of the carriageway should be undertaken by providing a suitable

cross-fall and camber or gradient, respectively. Water flowing downgrade on bridge approaches should not be permitted to run onto the bridge unless permitted otherwise by the road agency. To reduce costs, short bridges should be detailed without formal superstructure drainage wherever possible, with the run-off from the bridge discharged into outfall drains at the end of the structure, as specified by the agency.

Longer bridges require drainage facilities; otherwise flow widths may exceed the allowable limits (which is typically 1.2 m if the shoulder width is greater than 1.2 m). Inlet structures, such as flush grates connect to the under-deck pipe work, which discharges away from the structure, waterway, or other thoroughfare beneath the structure. Drainage inlets should be of rigid, ultraviolet and corrosion-resistant material, not less than 100 mm in their least dimension, and should be provided with provision for cleanouts.

Deck drainage should be detailed to prevent the discharge of drainage water against any portion of the structure and to prevent erosion adjacent to the point of impact of the discharge from the outlet of the downpipe. The overhanging portions of a concrete deck should be provided with a drip bead or notch, which should be continuous where possible.

Drainage from bridges should not discharge directly into waterways, onto traffic lanes, railway corridors or any other thoroughfare below. As such, the use of scuppers for bridge deck drainage should be precluded.

13.7.15.2 DETAILING FOR DRAINAGE

Design details should ensure that water drains from all parts of the structure and should prevent the retention of dirt, leaves or other foreign matter. Where drainage pipes are provided in the closed cells of bridges, the pipes should be of durable material. Where pipes carrying liquids are located inside closed cells, drainage should be provided in case of leaking or bursting of the pipes.

If the bridge is located in a sag road section, there should be a maximum allowable ponding depth near the parapet. Drainage relief should be provided at the parapet by installing 100 mm diameter pipe at the parapet. This is to prevent excessive ponding of water on the bridge deck, hence increase the dead load to the bridge, and to limit the ponding depth to the shoulder of the bridge.

13.7.15.3 DRAINAGE OF BALLAST RAILWAY BRIDGES

Consideration should be given to the effective drainage of ballast-topped railway bridges, and waterproofing should be provided where necessary.

13.7.16 BRIDGE SCOUR

13.7.16.1 INTRODUCTION

The most common cause of bridge failures is from floods eroding bed material from around bridge foundations. Scour is the engineering term for the erosion of soil, alluvium or other materials surrounding bridge foundations (piers and abutments) by flowing water. Safe bridge design must account for scour conditions that may occur over the life of the bridge.

Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are more scour resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand-bed streams. Under constant flow conditions, scour will reach maximum depth in sand- and gravel-bed material in hours; cohesive bed material in days; glacial till, sandstones, and shale in months; limestone in years, and dense granite in centuries. Under flow conditions typical of actual bridge crossings, several floods may be needed to attain maximum scour. Determining the magnitude of scour is a complicated process due to its occasionally cyclic nature.

Scour is greatest during flood events when flow velocity and depth is highest, but the event-related scour is in addition to the long-term stream instability components of channel shifting, aggradation, and degradation.

Each of the scour components discussed in this Section should be considered during bridge design. It is important for bridge engineers to recognize that these scour and stream instability components be considered over the life of the bridge.

The FHWA HEC-18 and HEC-20 manuals are the primary source of guidance and procedures for incorporating scour and stream instability into safe bridge design.

The following factors related to scour and stream instability should be considered in bridge design:

i. Evaluation of bridge design alternatives shall consider stream instability, backwater, flow distribution, stream velocities, scour potential, flood hazards, tidal dynamics (where appropriate) and consistency with established criteria.
ii. Studies shall be carried out to evaluate the stability of the waterway and to assess the impact of construction on the waterway.
iii. (Consider) whether the stream reach is degrading, aggrading, or in equilibrium.
iv. (Consider) the effect of natural geomorphic stream pattern changes on the proposed structure.
v. For unstable streams or flow conditions, special studies shall be carried out to assess the probable future changes to the plan form and profile of the stream and to determine countermeasures to be incorporated in the design, or at a future time, for the safety of the bridge and approach roadways.
vi. For the design flood for scour, the streambed material in the scour prism above the total scour line shall be assumed to have been removed for design conditions.
vii. Locate abutments back from the channel banks where significant problems with debris build-up, scour, or channel stability are anticipated.
viii. Design piers on floodplains as river piers. Locate their foundations at the appropriate depth if there is a likelihood that the stream channel will shift during the life of the structure or that channel cut-offs are likely to occur.

13.7.16.2 TYPES OF SCOUR

Total scour at a bridge crossing may comprise one or more of the following four inter-related components:

i. **Aggradation and degradation** – long-term stream bed elevation changes due to natural or man-induced causes within the reach of the river on which the bridge is located. Aggradation involves the deposition of material eroded from other sections of a stream reach, whereas degradation involves the lowering or scouring of the bed of a stream over relatively long reaches due to a deficit in sediment supply from upstream.
ii. **Scour due to river morphology** – occurs naturally in the stream; it is a function of flow conditions and associated channel characteristics. It includes general bed movement and scour at channel contractions and bends.
In addition, naturally-occurring lateral migration of a stream may also occur. This can erode abutments, the approach roadway or change the total scour by changing the flow angle of attack.
iii. **Contraction scour** – the scour resulting from the contraction of flow by bridge approach embankments encroaching onto the floodplain and/or into the main channel (**Plate 13.16**). This scour causes a lowering of the streambed across the stream or waterway bed at the bridge.
Scour due to a naturally-occurring channel contraction is similar to scour resulting from contraction of flow by bridge approach embankments and is treated as contraction scour. Contraction scour is different from long-term degradation in that contraction scour occurs in the vicinity of the constriction may be cyclic and/or related to the passing of

a flood.

iv. **Local scour** – involves removal of material from around piers, abutments, guide banks and embankments (**Plate 13.17**). It is caused by an acceleration of flow and resulting vortices induced by the flow obstructions.

The design scour depth at a site is assessed by combining the effects of these scour components.

Plate 13.16 Excessive scour due to significant constriction of waterway

Plate 13.17 Local scour at a bridge pier due to significant constriction of waterway

13.7.16.3 FACTORS AFFECTING SCOUR

Factors which can affect the extent of scour at a bridge include:

i. slope and alignment of the natural stream
ii. bed material of stream and floodplains
iii. vegetation in the stream, and floodplains
iv. changes or potential changes in the prevailing conditions in the stream or the catchment, whether man-made or natural
v. depth, velocity and alignment of flow through the constriction
vi. alignment and layout of the bridge and training works
vii. accumulation of debris (**Plate 13.14**)
viii. size, shape, orientation and arrangement of piers, footings and piles
ix. amount of bed material in transport.

13.7.16.4 CLEAR-WATER AND LIVE-BED SCOUR

There are two conditions under which contraction and local scour may occur: clear-water scour and live-bed scour. Clear-water scour occurs when there is no movement of the bed material of the stream upstream of the crossing, but the acceleration of the flow and the vortices created by piers or abutments causes the material in the crossing to move. Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach into the bridge cross-section. With live-bed contraction scour the area of the contracted section increases until, in the limit, the transport of sediment out of the contracted section equals the sediment transported in. Live-bed scour is cyclic in nature, i.e. the scour hole that develops during the rising stage of a flood refills during the falling stage.

Typical clear-water scour situations include:

i. coarse bed material streams
ii. flat gradient streams during low flow
iii. armoured stream beds
iv. vegetated channels.

During a flood event, bridges over streams with coarse bed material are often subjected to clear-water scour at low discharges, live-bed scour at the higher discharges and then clear-water scour on the falling stages.

Clear-water scour reaches its maximum over a longer period of time than live-bed scour (see **Figure 13.95** for a comparison of live-bed and clear-water scour, as a function of time, at a pier in a sand-bed stream). This is because clear-water scour occurs mainly in coarse bed material streams. In fact, clear-water scour may not reach a maximum until after several floods. Maximum clear-water scour is about 10% greater than the equilibrium live-bed scour.

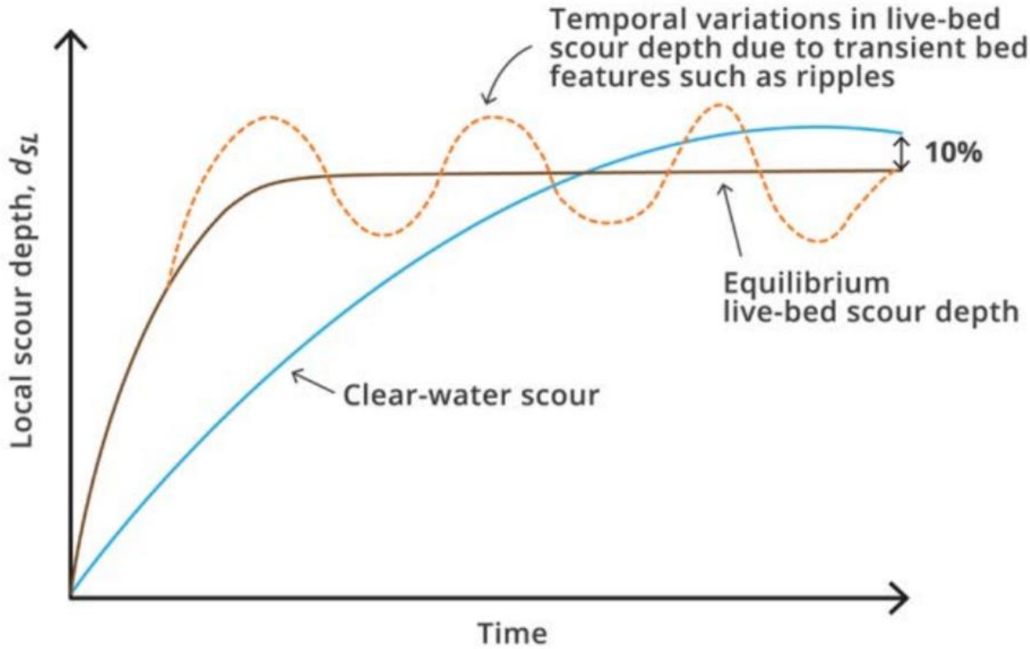


Figure 13.95 Illustrative pier scour depth in a sand-bed stream as a function of time

13.7.16.5 AGGRADATION AND DEGRADATION

To determine what long-term bed elevation changes will occur in the life of a structure, the design engineer should carry out an assessment using the principles of river mechanics. Some of the factors that affect long-term stream bed elevation changes are:

i. dams or reservoirs (upstream or downstream of a bridge)
ii. changes in watershed land use (urbanisation, deforestation, etc.)
iii. channelisation
iv. cut-offs of meander bends (natural or man-made)
v. changes in the downstream channel control
vi. sand or gravel mining from the stream bed
vii. diversion of water into or out of the stream
viii. natural lowering of the total system.

13.7.16.6 SCOUR DUE TO RIVER MORPHOLOGY

With the rise in stage accompanying flood passage through an alluvial river reach, there is an increase in velocity and shear stress on the bed. As a result, the channel bed tends to scour during high flow. Because sediment is being contributed from upstream, as the shear decreases with the fall of stage the sediment tends to be deposited on the bed. This occurs with both perennial and ephemeral alluvial streams.

A greater depth of scour will occur at channel contractions, because of the decrease in waterway area and resultant increase in velocity. This scour may occur over the full width of the stream bed. At bends, the non-uniform velocity distribution may cause scour of the bed and bank at the outside of the bend and deposition on the inside of the bend. High velocities at the outside of the bend or downstream of the bend can substantially contribute to local scour at abutments and piers.

13.7.16.7 CONTRACTION SCOUR

Contraction scour occurs when the flow area is constricted by a bridge (**Figure 13.96**). The decrease in flow area increases the average velocity and bed shear stress through the constriction. This results in an increase in erosive forces and more bed material being removed from the constricted reach than is transported into it.

The resultant scour lowers the natural bed elevation in the constriction. As the bed elevation is lowered, the flow area increases, and the velocity and shear stress decrease until relative equilibrium is reached. At this point, the quantity of bed material transported into the reach is equal to that removed from the reach.

Contraction scour is similar to the scour that would occur naturally in a stream contraction and is typically cyclic. That is, the bed scours during the rising stage of a runoff event, and fills on the falling stage.

The constriction of flow due to a bridge can be caused by a decrease in flow area of the stream channel by the abutments projecting into the channel or by the approaches to a bridge cutting off the floodplain flow.

Clear-water scour is more likely to occur at the bridge section, because the floodplain flow normally does not transport significant concentrations of bed material sediments. This clear water picks up additional sediment from the bed upon reaching the bridge opening.

When bridge superstructures obstruct flow or become completely submerged, pressure flow occurs. Under these circumstances contraction scour and local scour can be expected to increase substantially. The depth of scour will depend on the velocity of the approach flow and the clearance between the superstructure and the stream bed. For the same approach velocity, constriction and local scour can be expected to increase with decreasing clearance between the superstructure and the stream bed.

A guide bank at an abutment decreases the risk from scour at the abutment by realignment of the stream lines of the floodplain flow, so that they enter the bridge opening clear of the bridge abutment. However, scour will occur at the upstream end of the guide bank and this may extend downstream under the bridge.

Another method of decreasing abutment scour is to install relief bridges. They decrease the scour problem by decreasing the quantity of clear-water returning to the main channel.

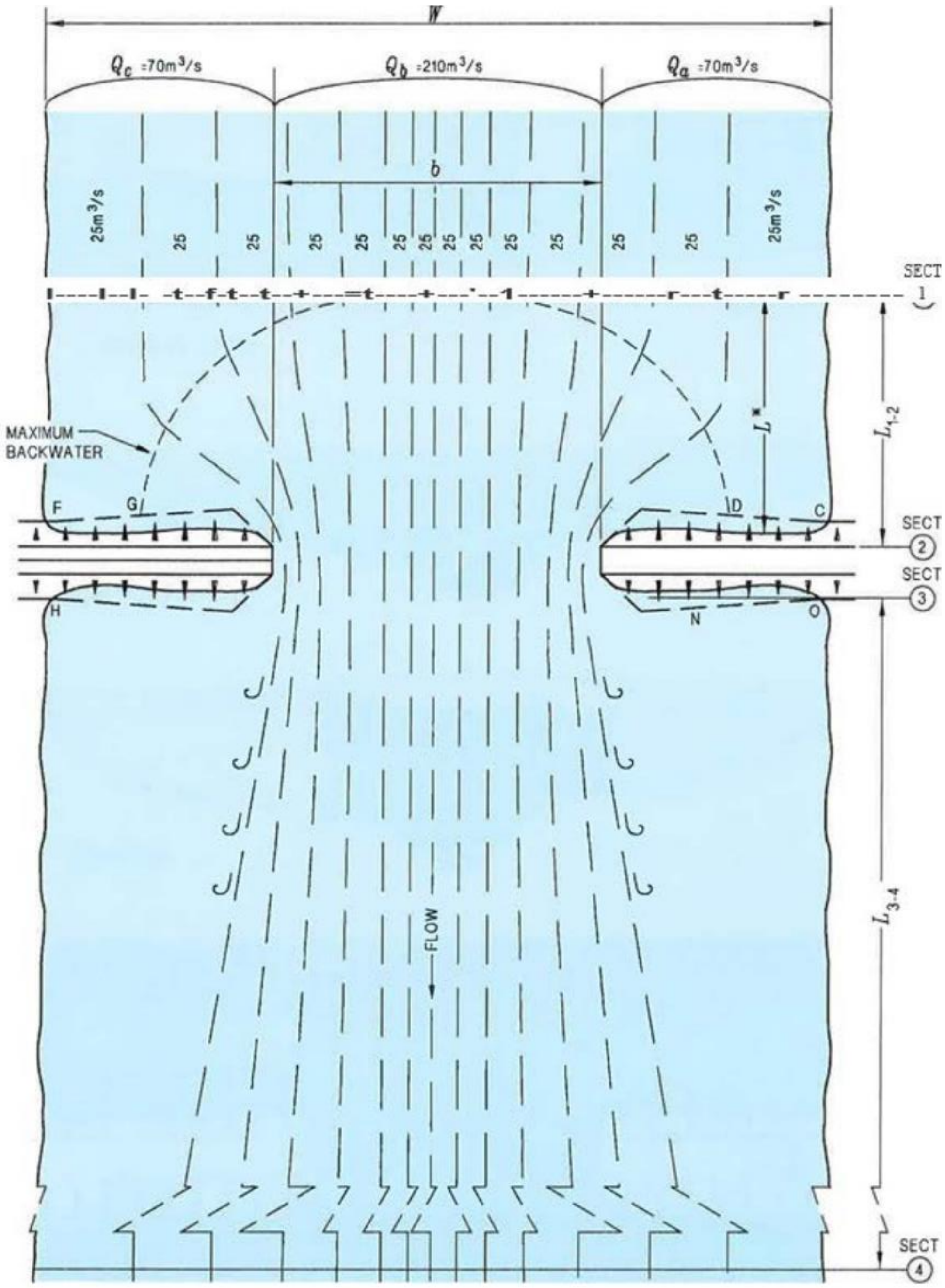


Figure 13.96 Flow lines for typical normal crossing

Contraction scour can be caused by different bridge site conditions. There are four conditions as illustrated on **Figure 13.97** and now described (Arneson et al. 2012).

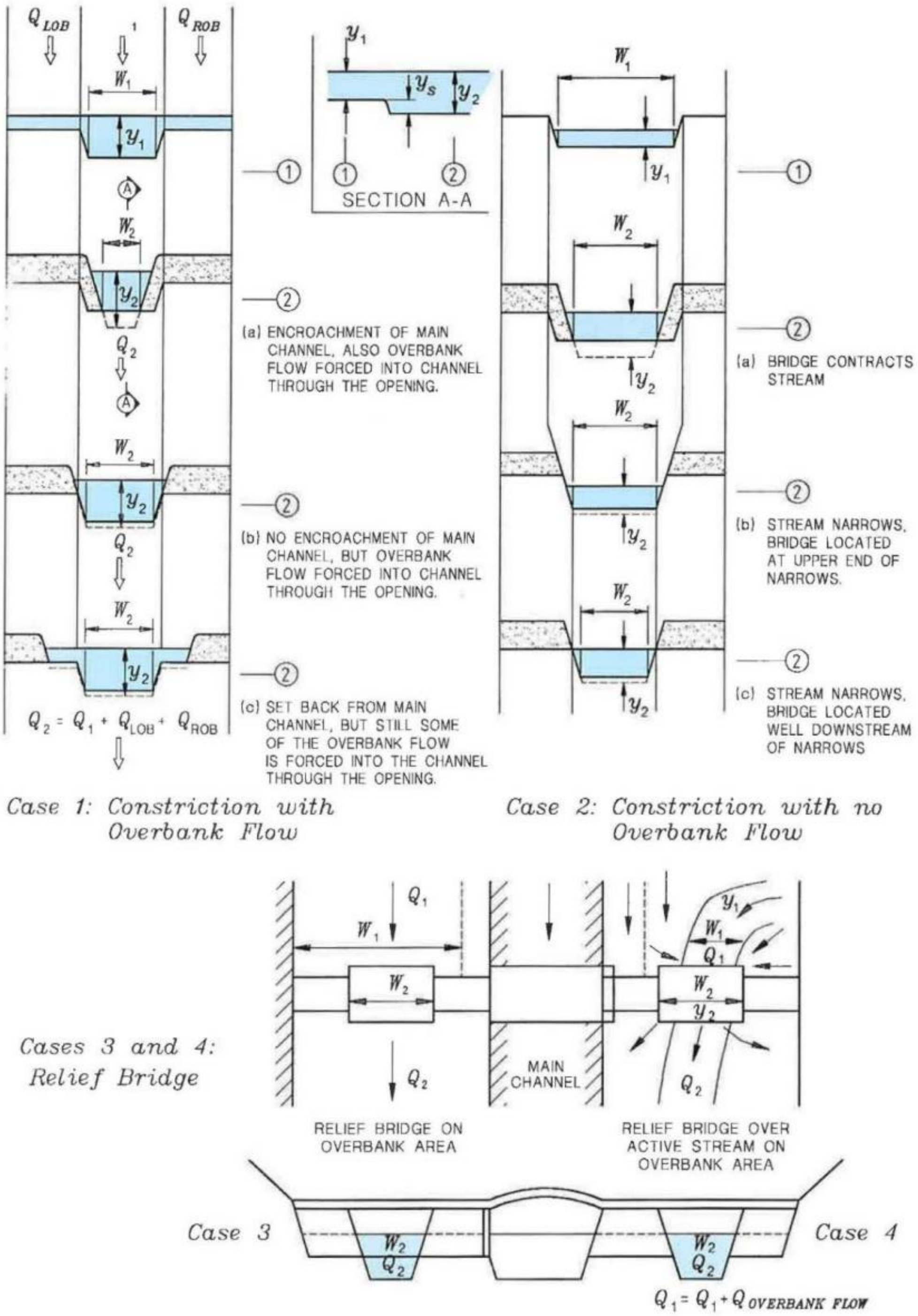


Figure 13.97 The four main cases of contraction scour

Case 1 – Overbank flow is forced back to the main channel by the bridge approaches. There are three different situations in which this can occur:

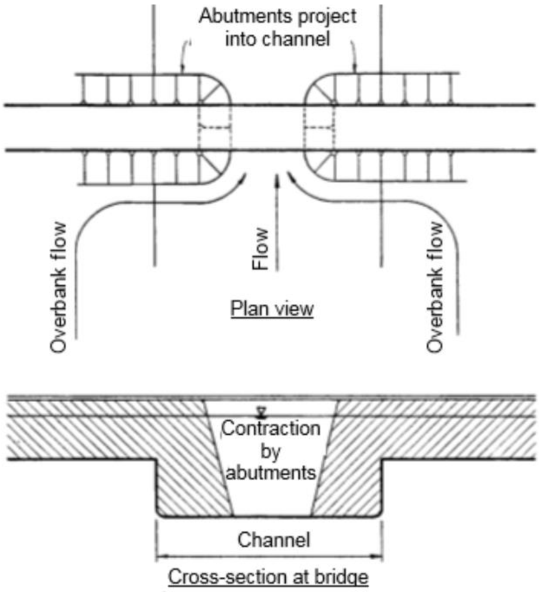
a. The river channel width becomes narrower, either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river (**Figure 13.98-A**).
b. No constriction of the main channel is involved, but the overbank flow area is completely obstructed by the embankment (**Figure 13.98-B**).
c. Abutments set back from the stream channel (**Figure 13.98-C**).

Case 2 – Flow is confined to the main channel with no overbank flow. The normal river channel width becomes narrower due to the bridge itself or the bridge site being located at a narrower reach of the river (**Figure 13.99-A** and **Figure 13.99-B**).

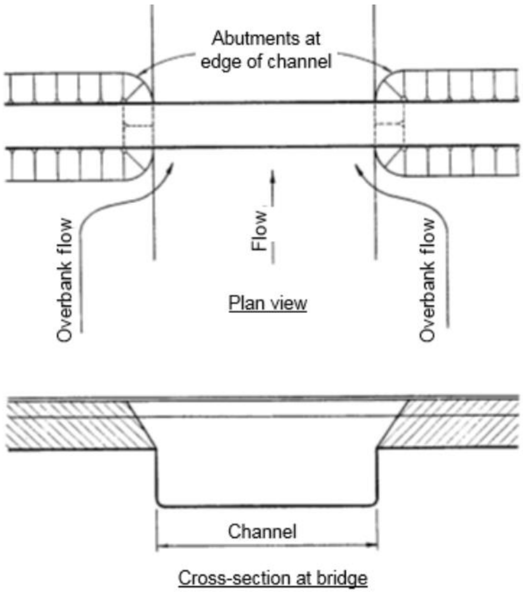
Case 3 – A relief bridge in the overbank area with little or no bed material transport in the overbank area; i.e. clear-water scour (**Figure 13.100**).

Case 4 – A relief bridge over a secondary stream in the overbank area (similar to Case 1) (**Figure 13.101**).

A. Case 1a: abutments project into



B. Case 1b: abutments at edge of channel



C. Case 1c: abutments set back from channel

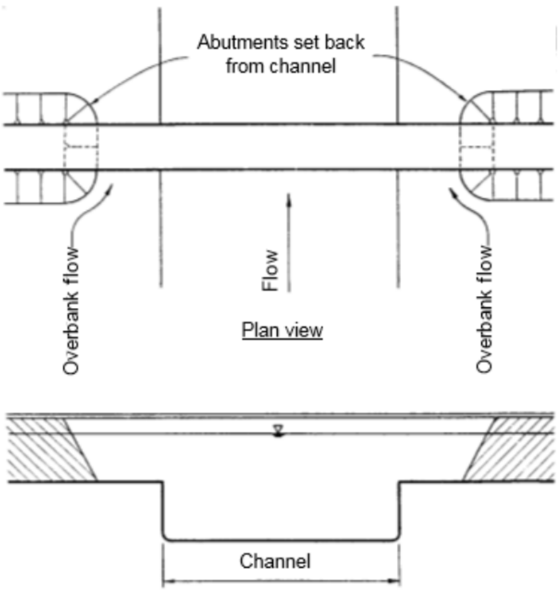


Figure 13.98 Case 1

A. Case 2a: river narrows | **B. Case 2b: bridge abutments and/or piers constrict flow**

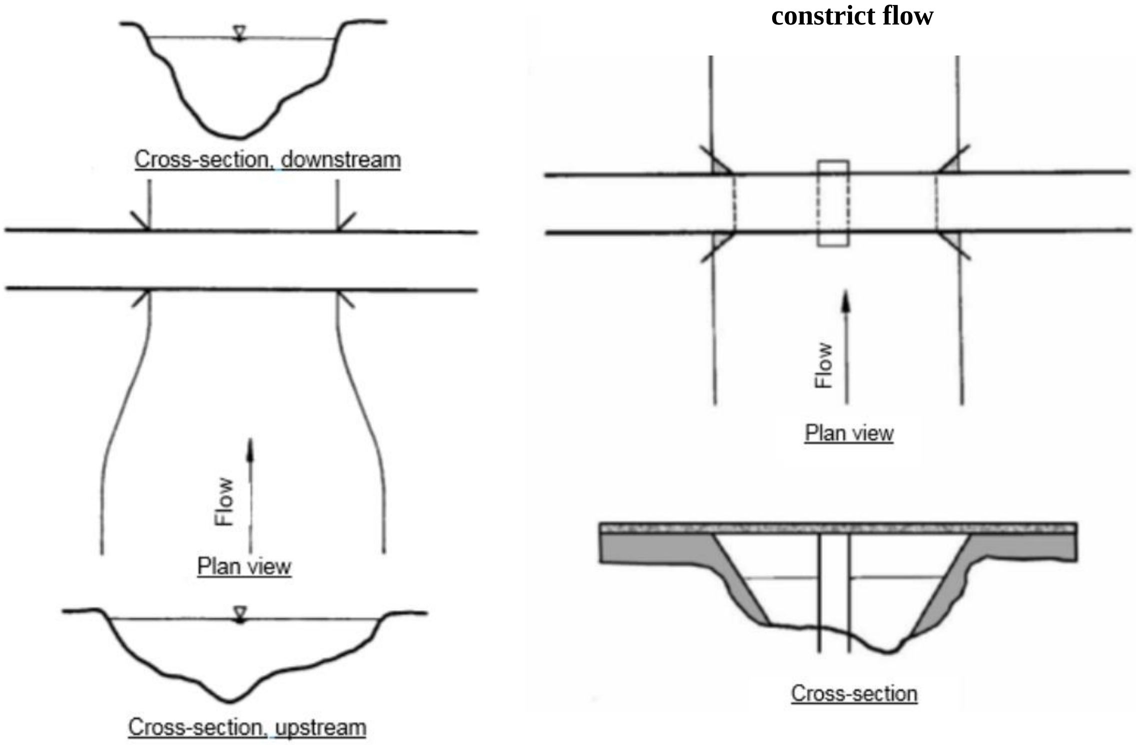


Figure 13.99 Case 2

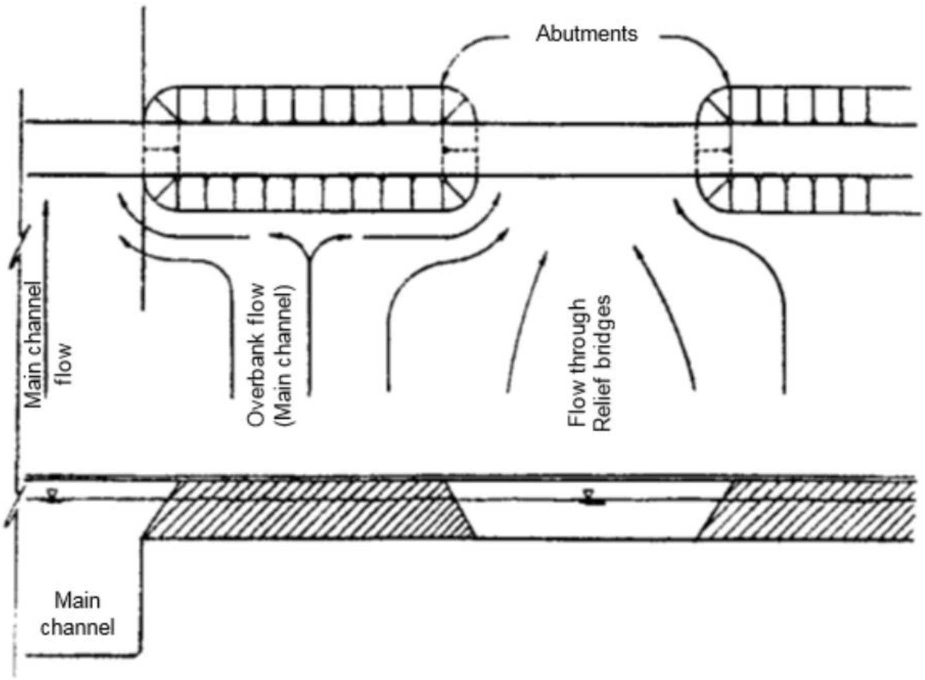


Figure 13.100 Case 3: relief bridge over floodplain

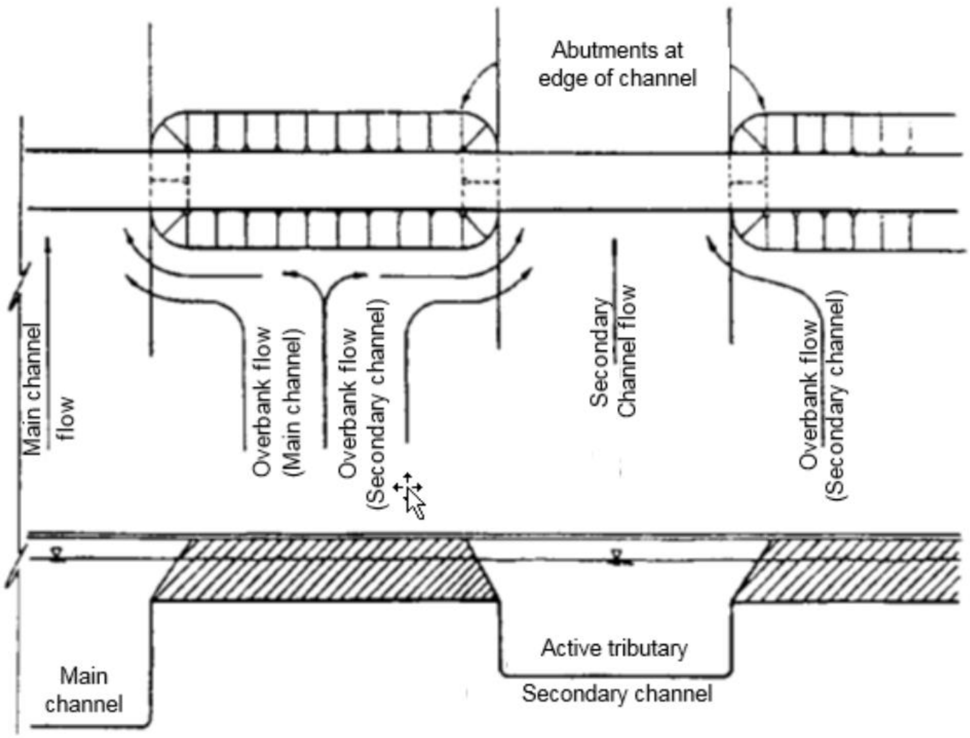


Figure 13.101 Case 4: relief bridge over secondary stream

Notes:

1. Cases 1, 2, and 4 may either be live-bed or clear-water scour depending on whether there is bed material transport from the upstream reach into the bridge reach during flood flows.
2. For case 1c, the depth of contraction scour depends on factors such as
   - how far back from the bank line the abutment is set
   - the condition of the overbank (is it easily eroded, are there trees on the bank, is it a high bank, etc.)
   - whether the stream is narrower or wider at the bridge than at the upstream section
   - the magnitude of the overbank flow that is returned to the bridge opening
   - the distribution of the flow in the bridge section, and other factors.

   In this case, the main channel under the bridge may be live-bed scour; whereas, the set-back overbank area may be clear-water scour.
3. Case 3 may be clear-water scour even though the floodplain bed material is composed of sediments with a critical velocity that is less than the flow velocity in the overbank area. The reasons for this are
   - there may be vegetation growing part of the year
   - if the bed material is fine sediments, the bed material discharge may go into suspension (wash load) at the bridge and not influence contraction scour.
4. Case 4 is similar to Case 3, but there is sediment transport into the relief bridge opening (live-bed scour).

   This case can occur when a relief bridge is over a secondary channel on the floodplain.

13.7.16.8 LOCAL SCOUR

The basic mechanism causing local scour at a pier (**Plate 13.18**) or abutment (**Plate 13.19**) is the formation of vortices at their base, resulting from acceleration of the flow around the nose of the pier or embankment.

The vortex removes bed material from the base of the obstruction and a scour hole develops.

As the depth of scour increases, the strength of the vortices is reduced, thereby reducing the transport rate from the base region, and eventually equilibrium is re-established and scouring ceases.

Plate 13.18 Example of local scour at piers

Plate 13.19 Example of local scour at an abutment

In addition to a horseshoe vortex around the base of a pier, there is a vertical vortex downstream of the pier called the wake vortex. Both vortices remove material from the pier base region. However, the intensity of these forces diminishes rapidly as the distance downstream of the pier increases. Therefore, immediately downstream of a long pier there is often deposition of material.

Figure 13.102 and **Figure 13.103** show the usual form of scour at an abutment and abutment and adjacent pier, respectively, and **Figure 13.104** shows the usual form of local scour holes at piers.

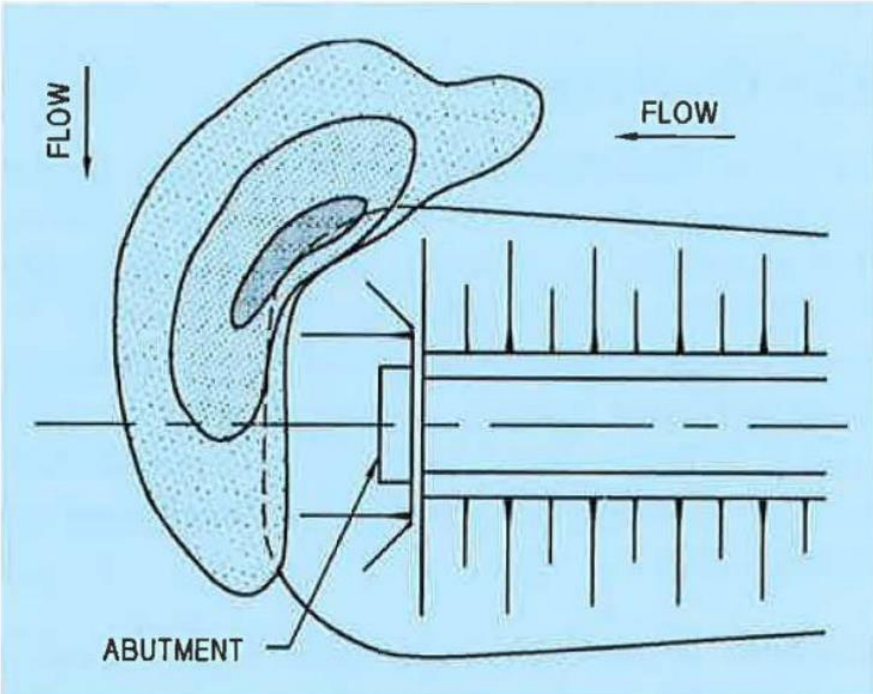


Figure 13.102 Scour at an abutment

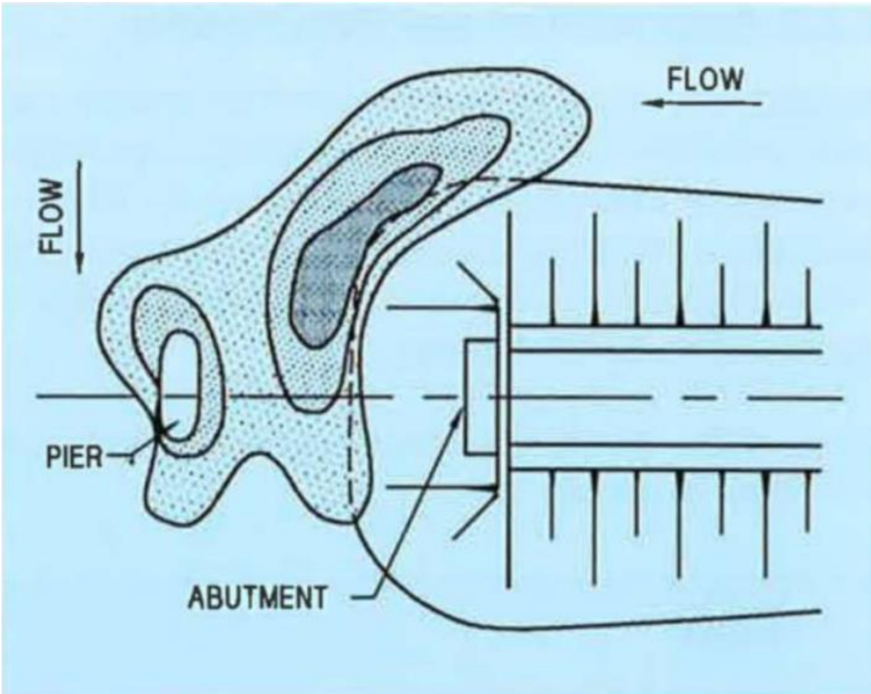


Figure 13.103 Scour at an abutment and adjacent pier

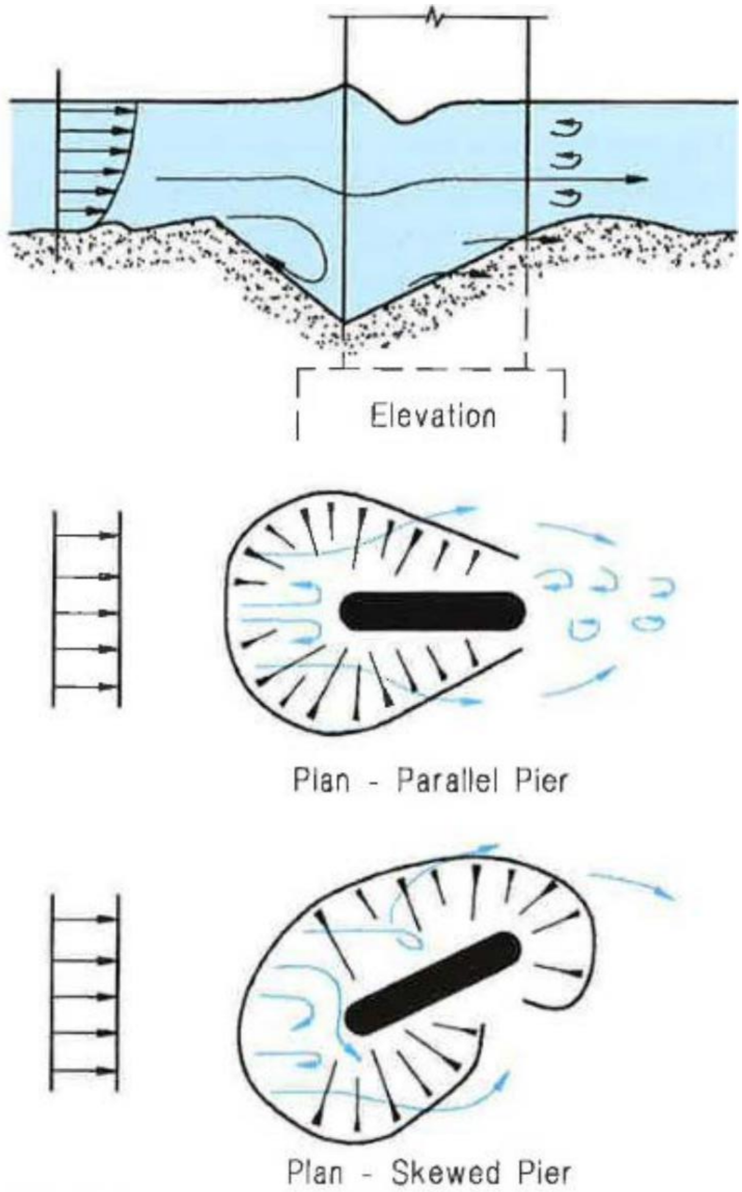


Figure 13.104 Usual form of local scour holes at piers

13.7.16.9 BRIDGE SCOUR DESIGN AND EVALUATION

Damage to bridge approaches from rare flood events can usually be repaired quickly. However, a bridge that has sustained major damage or failed from scour can create safety hazards to motorists, as well as large social impacts and economic losses over a long period of time. A greater assurance that scour will not endanger the foundations of a bridge is required, therefore, than is warranted for the design of its approaches.

The hydraulic capacity of a bridge varies widely, as it is highly dependent on both the design of the structure and the waterway itself. However, design of scour protection should consider the flood event that produces the highest velocity and greatest bed shear. This will generally be the flood event that overtops the bridge and is usually during the rising limb of the event. This suggests that the greatest velocity does not always correspond with the peak flood level (TMR 2013).

The aim of bridge design should identify the flood event that produces the highest velocities and worst case. The scour design event should be considered as the design flood event that produces an overtopping event plus an additional 300 mm in water surface. This additional overtopping amount and increased blockage factors can be tailored to the site characteristics for factors such as debris loading. Specifying a particular design event will ignore the subtle differences between designs and is being deliberately avoided.

Furthermore, an extreme flood event (2000 year ARI or 0.05% AEP) may not be the design flood that produces the greatest turbulence or highest velocity in a reach. The structural design of the bridge still requires the design to be tested against the extreme flood event.

The floods that should be used for scour estimation are as follows:

i. for the evaluation of bridge foundations – the 2000 year ARI (0.05% AEP) flood or the overtopping flood, if it produces more severe scour conditions
ii. for the design of protection works to the fill around bridge abutments and to bridge approaches – the total waterway design flood

Waterway investigations of bridge sites should address both the sizing of the bridge waterway and the designing of the foundations to resist scour. The scope and depth of the investigation should be commensurate with the importance of the road and the consequences of failure.

The size of a bridge will have an impact on scour. The waterway area is determined by the length and height of the deck, as well as the channel geometry. The waterway area of the bridge and flow conditions of the channel will determine the velocities through the bridge, with high velocities resulting in scour.

As flood levels are increased, the flow begins to be built up behind the bridge deck. Up to the point of overtopping, the flow through the bridge becomes pressure flow as flow is driven through the bridge opening by the additional hydraulic head (thickness of the superstructure). This often also includes debris trapped on guardrail which will increase hydraulic head by the additional blockage. Therefore, in terms of scour reduction it is preferable to increase the waterway area of the bridge by extending the bridge rather than increasing the deck level and associated bridge approach embankments. As there will be less constriction (horizontal and vertical) in the channel, the flow velocities through longer bridges will be lower, minimising pressure flow and scour of the bed material. However, increasing bridge length is typically more costly than increasing the height.

Abutment scour usually occurs within several zones of sediment and soil, leading to different rates of erosion. The bed of the main channel is more erodible than the floodplain, because the bed is formed of loose sediment while the floodplain is formed from more cohesive soil often protected by a cover of vegetation. Abutments are essentially short, erodible (in the direction of flow) contractions. As the flow width narrows, the stream velocity increases as does the associated turbulence. Higher flow velocities and large-scale turbulence around an abutment may erode the abutment.

Field observations indicate that, two prime scour regions develop as follows:

i. where the channel or overbank bed is least resistant to hydraulic erosion; this could be the main bed if flow velocities are sufficiently large
ii. where the flow velocities and turbulence are greatest; this usually is near the abutment.

Optimum bridge design should consider the approach levels where high flood immunity is not feasible.

Building the bridge approach embankment across the floodplain at, or above, the level of the bridge deck will exacerbate contraction scour. In some situations, typically when crossing floodplains on inland rivers, it may be preferable to deliberately reduce the level of the approach embankment to below the deck soffit. **Figure 13.105** illustrates the difference between a sag and crest vertical curves in a bridge design. A crest vertical curve (Approach B) will lower the approach to below the bridge deck. As water levels rise, the bridge approach acts as a weir and flow overtops the bridge approach, protecting the bridge. In Approach A, severe vertical contraction scour results from flow being dammed behind the road embankment and being forced through, and over, the bridge. Shallow flow over a long bridge approach will generally exceed the flow through the bridge's waterway opening. This lowers velocity and pressure flow and reduces the hydraulic forces acting at the bridge and scour of the bed. Allowing the bridge approaches to be overtopped will reduce scour and time of submergence, allowing the flood to travel unhindered downriver. This is particularly appropriate in locations where bridge flood immunity is very low or where afflux is a concern due to nearby houses.

Velocities through bridge approaches should be kept below 2.5 m/s or lower. This requires finding a careful balance between time of submergence and bridge scour concerns.

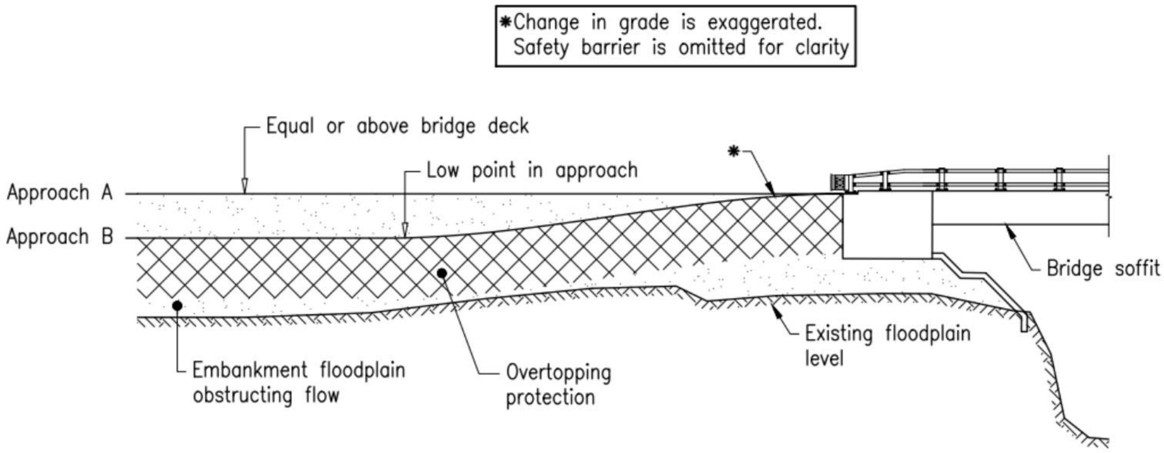


Figure 13.105 Crest vertical curve (Approach B) will minimise vertical contraction scour (Approach A blocks more floodplain flow)

13.7.16.9.1 NEW BRIDGES

The following general design procedure for scour is recommended for determining bridge type, size and location of substructures:

1. Determine relevant flood event(s). If there is an overtopping event that causes greater hydraulic stresses to the bridge than the hydraulic design event then that flood should be used for computing scour and designing the foundations.
2. Develop hydraulic parameters necessary to estimate scour for the flood flows in Step 1 by applying a one- or two-dimensional hydraulic model. The full range of hydraulic conditions that could impact the flow conditions at and near the bridge being designed.
3. Estimate total scour for the hydraulic conditions identified from Steps 1 and 2 above. The resulting scour computed from the selected flood event should be considered in the design of a foundation.
4. Plot the total scour depths obtained in Step 3 on a cross-section of the stream channel and floodplain at the bridge site.
5. Evaluate the results obtained in Steps 3 and 4 to determine if they are reasonable. This should be based on the judgment of a multi-disciplinary team comprised of hydraulic, geotechnical, and structural engineers. There are many factors that could affect the magnitude of the overall scour estimate, including but not limited to storm duration, erodibility of channel materials, flow conditions or debris.

6. Evaluate the proposed bridge size, configuration, and foundation elements on the basis of the scour analysis performed in Steps 3 through 5. Modify the design as necessary taking into account various measures to minimise scour such as increasing bridge length, adjusting the location of the bridge, changing the configurations of substructure elements and providing guide banks.
7. Perform the bridge foundation analysis on the basis that all streambed material in the scour prism above the total scour line (Step 4) has been removed and is not available for bearing or lateral support.

13.7.16.9.2 EXISTING BRIDGES

Scour countermeasures can be designed and installed for scour critical bridges. Normally the critical flood or, otherwise, the 50 or 100 ARI (2% or 1% AEP) floods are to be used for scour protection design.

13.7.16.9.3 DESIGN PROCEDURES FOR ABUTMENT PROTECTION

The common approach is to design the abutment protection to accommodate the waterway design flood (or serviceability limit states - SLS) without damage, and assume the abutment is fully scoured under the super-flood event.

Abutments should be protected against scour by the use of one of several approaches in order to assure that they or the fill material placed around them does not fail. These approaches include:

i. The use of a designed scour countermeasure (such as rock riprap and/or guide banks) to keep scour from developing at the base of the abutment or adjacent embankments. This method is a reasonable and cost-effective approach for determining abutment foundation depth, but relies on a properly designed and inspected scour countermeasure.
ii. Assuming that all embankment fill material has washed away and that the abutment essentially behaves as a pier. This method is advantageous in that the failed embankment can be more easily repaired than a failed abutment, but provides a disadvantage due to the adverse flow conditions in the floodplain and channel near the abutment. Therefore, if scour estimated by treating the abutment as a pier could lead to deep foundation depths.
iii. Using procedures specifically developed for estimating abutment scour. As these are empirical methods, the hydraulic variables must be accurately and realistically determined.

The following procedure can be used for the design of the protection measure:

1. Select either the total waterway design flood or a lesser flood, if it is expected to produce the most severe scour conditions. When a bridge is designed to be overtopped with a flood with an ARI less than the total waterway flood, it is quite likely that this flood will produce the worst conditions.
2. Develop water surface profiles for the flood flows in Step 1.
3. Estimate the contraction scour and local pier scour for the worse condition from Step 1.
4. Plot the total scour depths obtained in Step 3 on a cross-section of the stream at the bridge site. The cross-section should include the soil profiles obtained from the geotechnical investigation at the site.
5. Considering the limitations in current scour estimation procedures, are the scour estimates reasonable? Based on engineering judgement the adopted scour depth(s) may differ from the calculated values.
6. Determine the rock riprap protection required at abutments and guide banks for the depth of contraction scour adopted in Step 5. If the local pier scour impacts on the

abutment, then provide sufficient rock protection to protect both the abutment and pier.

The following should be considered in determining the form and extent of protection to be provided:

i. the degree of uncertainty in the scour prediction method
ii. the potential for and consequences of failure
iii. the added cost of making the bridge abutments and approaches less vulnerable to scour
iv. the overall flood flow pattern at the bridge site.

13.7.16.9.4 FOUNDATION DESIGN TO RESIST SCOUR

Foundations in the floodplain should generally be placed at the same level as those in the stream channel, to allow for possible lateral shifting of the stream. Abutment foundations should be placed below the elevation of the thalweg in the bridge opening.

Spread footings on soil – the top of the footing should be placed below the design scour line. When there is any risk of scour undermining spread footings, deep foundations in the form of piles should be used.

Spread footings on rock highly resistant to scour – the bottom of the footing should be placed directly on the cleaned rock surface. Small embedments should be avoided since blasting to achieve keying frequently damages the rock structure and makes it more susceptible to scour. If lateral restraint is required, it should be provided with steel dowels drilled and grouted into the rock.

Spread footings on erodible rock – careful assessment is required of rock that is potentially erodible. The decision on whether rock will be susceptible to scour should be based on an analysis of intact rock cores, including rock quality designations and local geology, as well as hydraulic data and anticipated structure life.

Excavation into erodible rock should be made with care. All loose rock should be removed from the excavation and any overbreak beneath the footing should be made up with lean concrete. The footing should be poured against the sides of the excavation for the full depth of the footing. The excavation above the top of the footing should be backfilled with rock riprap sized to withstand flood flow velocities.

Piled foundations with pile caps – the top of the pile cap should be placed at a depth equal to the contraction scour depth. This will minimise obstruction to flood flows and resulting local scour.

Factors that will assist in minimising scour are:

i. the alignment of the piers with the direction of flood flows
ii. the use of round piers, especially where there are complex flow patterns during flood events
iii. the use of piers streamlined to decrease turbulence and minimise the potential for the build-up of debris
iv. the use of spill-through abutments, rather than vertical wall abutments, which produce twice as much scour.

13.7.16.9.5 EVALUATION OF FOUNDATION DESIGN FOR ULS SCOUR

The following procedure can be used:

1. Determine whether the bridge will or will not be overtopped with a flood with an ARI less than 2000 years. If it will be overtopped, use the overtopping flood to evaluate the foundation design. If not, then the 2000 year ARI (0.05% AEP) flood should be used to

evaluate the foundation design.
2. Develop water surface profiles for the flood flows in Step 1.
3. Using the procedures in Section 5.4, estimate the total depths of scour for the worse condition from Step 1 above.
4. Plot the total scour depths obtained in Step 3 on a cross-section of the stream at the bridge site. The cross-section should include the soil profiles obtained from the geotechnical investigation at the site.
5. Considering the limitations in current scour prediction procedures, are the answers obtained in Steps 3 and 4 reasonable? Based on engineering judgement the adopted scour depth(s) may differ from the calculated values.
6. Evaluate the proposed substructure type, size and location and modify them, if necessary. The overall flood flow pattern at the bridge site should be visualised to assist in identifying those elements of the bridge at risk from scour.
7. Substructure design should be carried out on the basis that all stream bed material above the total scour line (Step 4) has been removed and is not available for bearing or lateral support.

13.7.16.9.6 SCOUR RELATED TO CONSTRUCTION

Removal of vegetation and development of borrow areas upstream of a bridge and its approaches will often result in changes in flow patterns, which may affect the depth and extent of scour along embankments, abutments and piers.

Clearing of vegetation and removal of borrow material should be controlled in the vicinity of bridges and their approach embankments.

13.7.16.10 METHODS OF ESTIMATING SCOUR

There are many methods for estimating scour at bridges. They have all been developed from a limited range of laboratory data, with very little verification in the field. Therefore, all theoretical estimates of scour should be treated with caution and engineering judgement used in applying them.

13.7.16.10.1 DESIGN APPROACH

The steps involved in estimating scour at bridges are as follows:

1. Obtain the fixed-bed channel hydraulics for determining scour analysis variables, including the magnitude of the discharges for the design floods, possible future factors that will produce a combination of high discharge and/or low tailwater control and the water-surface profiles for the discharges.
2. Determine long-term profile degradation or aggradation, taking into account historic records, observational data, or using other empirical methods.
3. Compute the magnitude of the natural contraction scour.
4. Determine the magnitude of local scour at piers.
5. Determine the foundation elevation for abutments.
6. Plot and evaluate the total scour depths, including estimated long-term bed elevation change, contraction scour, and local scour at the piers and abutments. These data are plotted on the cross-section of the stream channel or other general floodplain at the bridge crossing.
7. Re-evaluate the bridge design.
8. Determine the protection required at the abutments.

The design approach is based on the assumption that the scour components develop independently. Thus, the potential local scour is added to the contraction scour without considering the effects of the contraction scour on the channel and bridge hydraulics. However, if the contraction scour is significant, the channel and/or bridge hydraulics should be adjusted for the effects of the contraction scour before estimating the local scour.

13.7.16.10.2 LIVE-BED CONTRACTION SCOUR

The modified version of Laursen's 1960 equation (Arneson et al. 2012) for live-bed scour at a long contraction can be used to estimate the depth of scour in a contracted section [**Equation (13.142)** and **Equation (13.143)**]. These equations assume that bed material is being transported from the upstream section:

$$\frac{y_2}{y_1} = \left[\frac{Q_2}{Q_1}\right]^{0.86} \left[\frac{W_1}{W_2}\right]^{k1} \qquad (13.142)$$

$$y_s = y_2 - y_0 \qquad (13.143)$$

Where,

y_1 = average depth in the upstream main channel, m
y_2 = average depth in the contracted section, m
y_s = average scour depth, m
y_0 = existing depth in the contracted section before scour (m)
Q_1 = flow in the upstream channel transporting sediment (m^3/s)
Q_2 = flow in the contracted channel (m^3/s)
W_1 = bottom width of the upstream main channel that is transporting bed material (m)
W_2 = bottom width of main channel in contracted section less pier width(s) (m)
k_1 = exponent determined based on the mode of bed material transport (**Table 13.54**)

Table 13.54 Estimation of exponent k_1

V*/T	k_1	mode of bed material transport
< 0.50	0.59	mostly contact bed material discharge
0.50 to 2.0	0.64	some suspended bed material discharge
> 2.0	0.69	mostly suspended bed material discharge
Where, $V^* = (\vartheta_0/\Delta)^{1/2} = (g y_1 S_1)^{1/2}$, shear velocity in the upstream section, m/s T = fall velocity of bed material based on the D50 (m/s) (**Figure 13.106**) g = acceleration of gravity (9.81 m/s^2) S_1 = slope of energy grade line of main channel, m/m ϑ_0 = shear stress on the bed, N/m^2 Δ = density of water (1000 kg/m^3)		

Notes:

- Q_2 may be the total flow going through the bridge opening as in cases 1a and 1b, but is not the total flow for case 1c. For case 1c contraction scour must be computed separately for the main channel and the left and/or right overbank areas.
- Q_1 is the flow in the main channel upstream of the bridge, not including overbank flows.
- The Manning 'n' ratio is eliminated in the Laursen live-bed equation to obtain **Equation (13.143)**. This was

done for the following reasons. The ratio can be significant for a condition of dune bed in the upstream channel and a corresponding plane bed, washed out dunes or anti-dunes in the contracted channel. However, Laursen's equation does not correctly account for the increase in transport that will occur as the result of the bed planning out (which decreases resistance to flow, increases the velocity and the transport of bed material at the bridge). In this cause, Laursen's equation will indicate a decrease in scour; in reality, there would be an increase in scour depth. In addition, at flood flows, a plane bedform will usually exist upstream and through the bridge waterway, and the values of Manning 'n' will be equal. Consequently, the 'n' value ratio is not recommended or presented in **Equation (13.143)**.

- W_1 and W_2 are not always easy to define. It can be acceptable in some cases to use the top width of the main channel to define these widths. It is important to be consistent so that W_1 and W_2 refer to either bottom widths or top widths.
- The average width of the bridge opening (W_2) is normally taken as the bottom width, with the width of the piers subtracted.
- Laursen's equation will overestimate the depth of scour if the bridge is located upstream of a natural contraction or if the contraction is caused by the bridge abutments and piers. However, it is currently the best available equation.
- The y_0 depth may be approximated by y_1 in sand channel streams where the contraction scour hole is filled in on the falling stage. The depth may be approximated by Sketches or surveys through the bridge can help in determining the existing bed elevation.
- Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armouring the bed. Scour depths should be calculated for live-bed scour conditions using the clear-water scour equation in addition to the live-bed equation where coarse sediments are present, and that the smaller calculated scour depth be used.

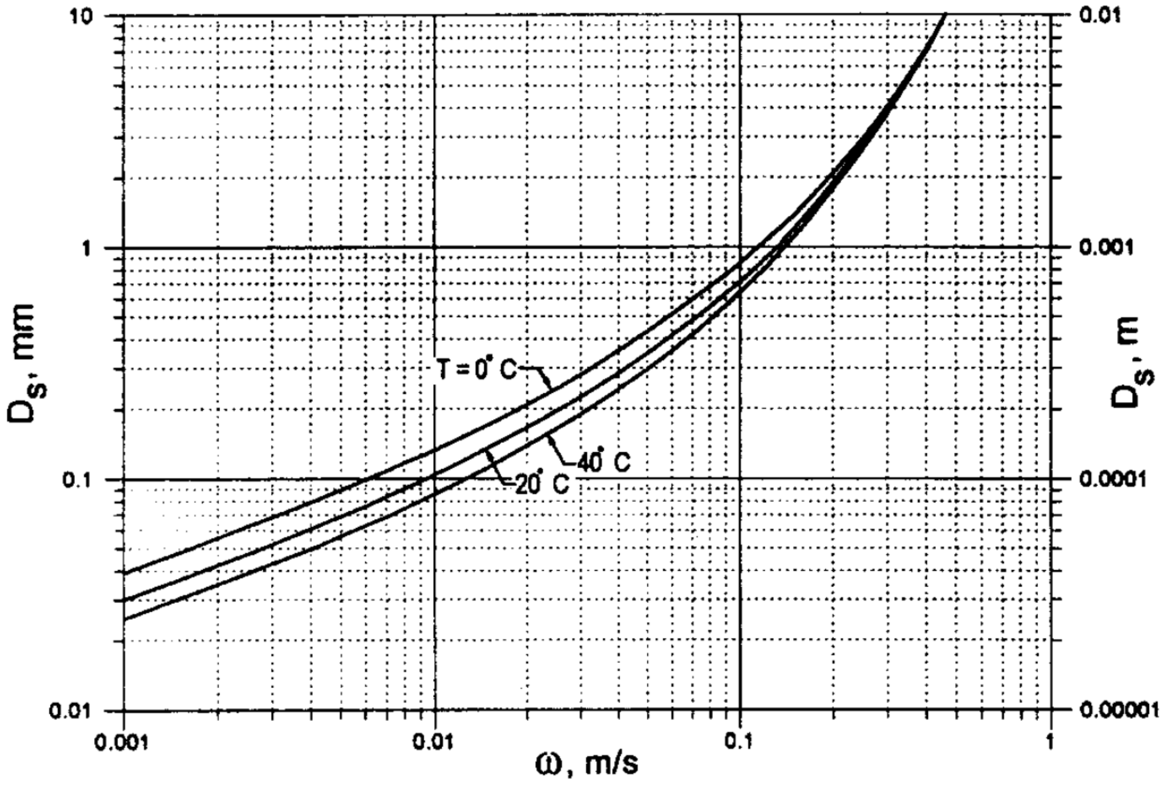


Figure 13.106 Fall velocity of sand-sized particles

Figure 13.107 and **Figure 13.108** defines the terms used for case 1a, and case 1c, respectively.

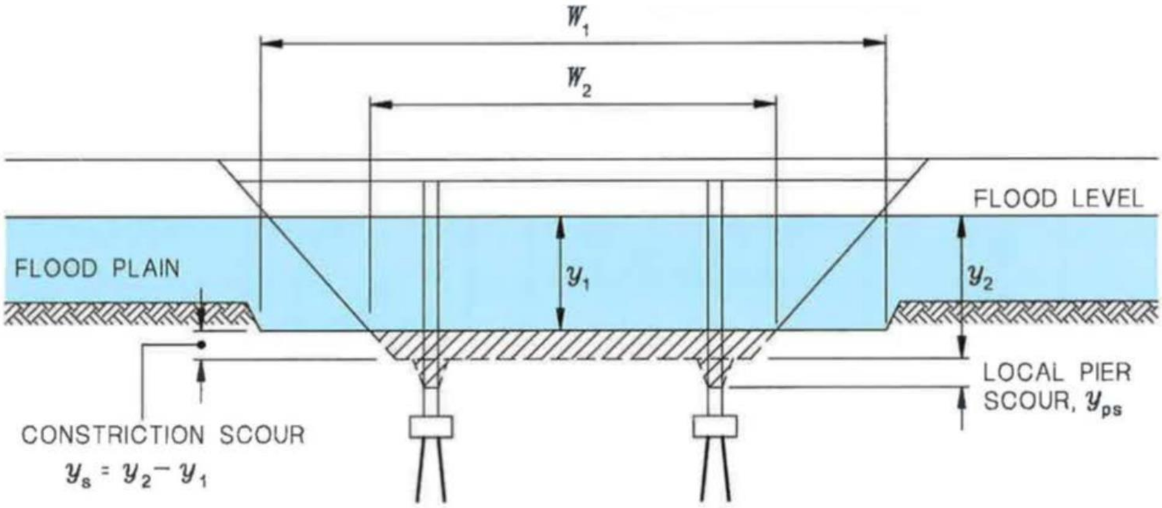


Figure 13.107 Definition sketch for scour depths for case 1a

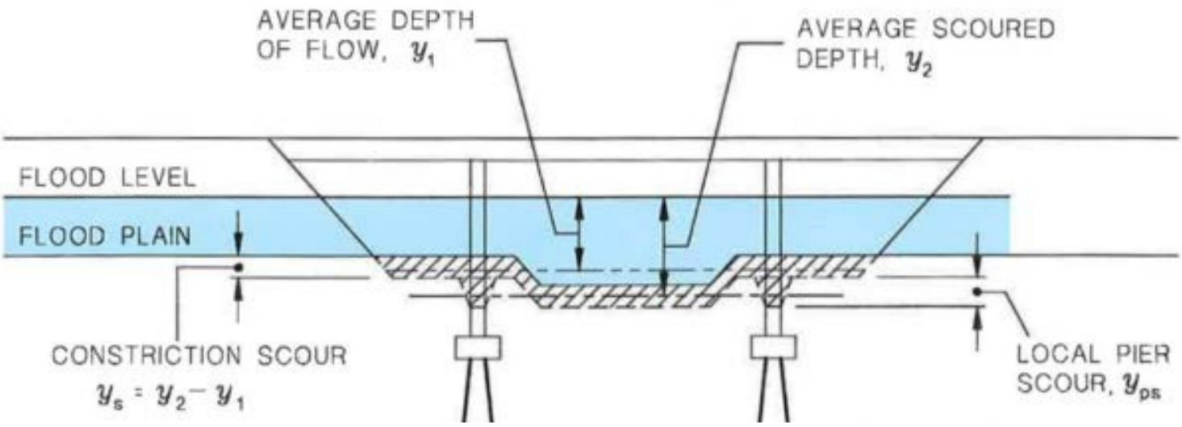


Figure 13.108 Definition sketch for scour depths for case 1c

13.7.16.10.3 CLEAR-WATER CONTRACTION SCOUR

The recommended clear-water contraction scour equation is based on a development suggested by Laursen (Arneson et al. 2012). The equation is (**Equation (13.144)**):

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2}\right]^{3/7} \tag{13.144}$$

Where,

- y_2 = average equilibrium depth in the contracted section after contraction scour (m)
- Q = discharge through the bridge or on the set-back overbank area at the bridge associated with the width W (m^3/s)
- D_m = diameter of the smallest non-transportable particle in the bed material (1.25 D_{50}) in the contracted section (m)
- D_{50} = median diameter of bed material (m)
- W = bottom width of the contracted section less pier widths (m)
- W_2 = bottom width of main channel in contracted section less pier width(s) (m)
- K_u = 0.0077

The average contraction scour depth, y_s, can be determined using **Equation (13.143)**.

Equation (13.144) is a rearranged version of **Equation (13.142)**. D_{50} equal to 0.2 mm is a reasonable lower limit that can be applied to this equation. Using a size smaller than 0.2 mm will over-estimate clear-water contraction scour.

The scoured section can be considered slightly armoured due to the fact that D_{50} is not the largest particle in the bed material. Therefore, the D_m is assumed to be 1.25 D_{50}. The depth of scour for stratified bed material can be determined by using the clear-water scour equation sequentially with successive D_m of the bed material layers.

13.7.16.10.4 CONTRACTION SCOUR WITH BACKWATER

The live-bed contraction scour equation is derived assuming a uniform reach upstream and a long contraction into a uniform reach downstream of the bridge. The equation computes a depth after the contraction where the sediment transport in the downstream reach is equal in and out. The clear-water contraction scour equations are derived assuming that the depth at the bridge increases until the shear-stress and velocity are decreased so that there is no longer any sediment transport. With the clear-water equations it is assumed that the flow is uniform. A level water surface is used in both equations to calculate contraction scour depth (**Equation (13.143)**). A more consistent computation would be to write an energy balance before and after the scour. For live-bed the energy balance would be between the approach section and the contracted section. Whereas, for clear-water scour it would be the energy at the same section before and after the contraction scour.

Backwater can decrease the velocity, shear stress and the sediment transport in the upstream section in extreme cases, increasing the scour at the contracted section. The backwater can, by storing sediment in the upstream section, change live-bed scour to clear-water scour.

13.7.16.10.5 CONTRACTION SCOUR IN COHESIVE MATERIALS

The live-bed and clear-water contraction scour equations presented so far are developed for cohesionless sediments and provide estimates of scour for a hydraulic conditions sufficient to produce ultimate scour. For silts and clays, the critical shear stress increases due to cohesion (Arneson et al. 2012). The only reliable way of determining critical shear for silt and clay particles is to perform materials testing (refer also to Briaud et al. 2011). Briaud et al. (2011) outlines an equation to compute ultimate scour for cohesive materials, based on laboratory data (**Equation (13.145)**). This computes the centreline scour downstream of the bridge entrance (scour in the vicinity of the entrance is 35% greater) and assumes that upstream flow depth is equal to the flow depth at the constriction (**Equation (13.146)**):

$$y_{s-ult} = 0.94 y_1 (\frac{1.83 V_2}{\sqrt{g y_1}} - \frac{K_u \sqrt{\frac{\tau_c}{\rho_w}}}{g n y_1^{\frac{1}{3}} 0} \tag{13.145}$$

$$\tau = \gamma (\frac{V_2 n}{K_u})^2 y_0^{-1/3} \tag{13.146}$$

Where,

- $y_{s\text{-ult}}$ = Ultimate scour depth, m
- y_1 = upstream average flow depth (m)
- V_2 = average flow velocity in the contracted section (m/s)
- τ_c = critical shear stress (N/m^2)

ρ_w = density of water, (kg/m^3)
n = Manning n
K_u = 1.0
γ = specific weight of water (N/m^3)
y_0 = existing depth in the contracted section before scour (m)

Including cohesion will typically reduce ultimate scour in comparison to fine-sand in the clear-water contraction scour equation.

Ultimate scour may not be reached in the lifetime of a bridge for cohesive materials if there is not sufficient flooding. Further information is required to estimate scour over the life of a bridge, including the erosion rate versus excess shear curve, as well as flow magnitudes and durations. Initial shear stress can be calculated using **Equation (13.146)**. If the shear stress does not exceed the critical value for the material then no scour will occur during that flow period. If it is exceeded, then the ultimate scour can be calculated using **Figure 13.109** from Briaud et al. (2011) which illustrates the generalised relationships between critical shear and erosion.

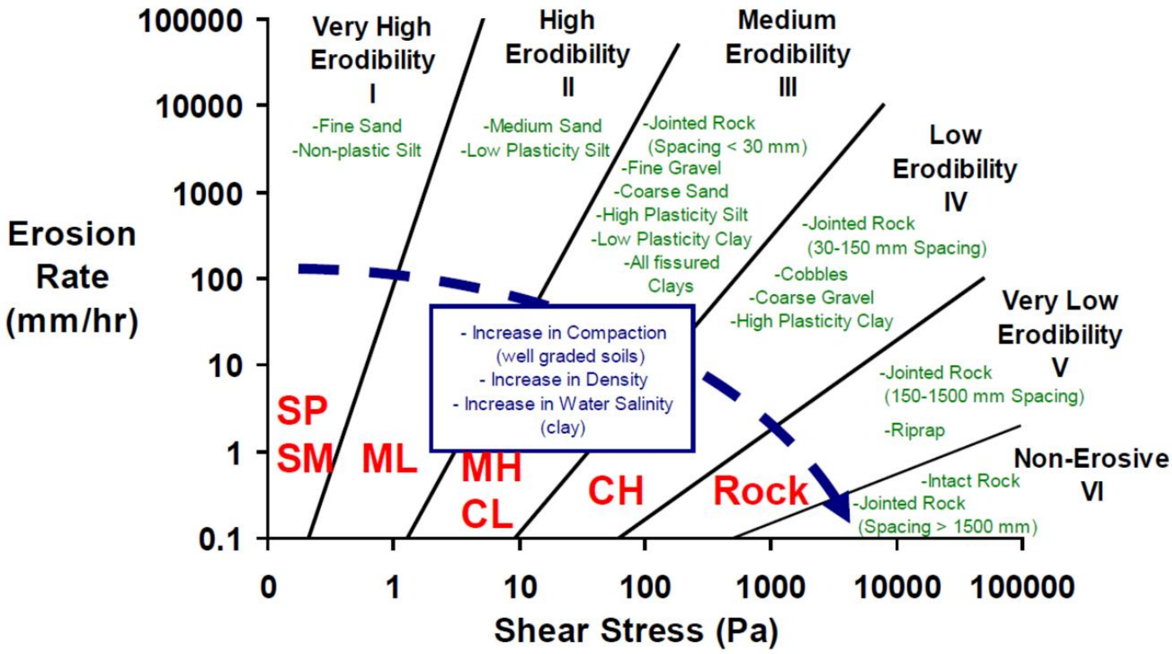


Figure 13.109 Generalised relationships for scour in cohesive materials

Time rate of scour is also an important consideration for contraction scour in cohesive soils. The actual scour that occurs during the first flood event during the life of the bridge depends on the initial scour rate, ultimate scour for the flow and its duration (see **Equation (13.147)**):

$$y_s(t) = \frac{t}{\dfrac{1}{\dot{z}_i} + \dfrac{t}{y_{s-ult}}} \quad (13.147)$$

Where,

$\dot{z}_i$ = initial rate of scour (m/hr)

y_s = average scour depth, m

t = duration of flow (h)

For subsequent flood events, scour will only occur when the ultimate scour of the event exceeds previous scour. This will always occur when the shear exceeds previously occurring shear, but may also occur when it does not. During the life of a bridge, scour in cohesive material is cumulative and can increase even during smaller events that occur after large flood events. **Equation (13.147)** can be used to compute scour for subsequent events, provided that the time is adjusted using **Equation (13.148)** and **Equation (13.149)**:

$$t = t_{event} + t_e \tag{13.148}$$

$$t_e = \frac{y_{s-ult} y_{s-prior}}{\dot{z}_i \left(y_{s-ult} - y_{s-prior}\right)} \tag{13.149}$$

Where,

t_e = equivalent time scour event would have required to reach prior scour (h)

$\dot{z}_i$ = initial rate of scour (m/hr)

$y_{s\text{-ult}}$ = Ultimate scour depth, m

$y_{s\text{-prior}}$ = cumulative scour that has been reached in prior flood events (m)

These steps must be completed for all scour events over the life of a bridge. This is due to the fact that the scour a bridge will experience during one flood depends not only on the magnitude and duration of that flood, but also the amount of scour that has occurred in previous floods. Therefore, the sequence of floods will also affect the total scour observed over the life of the bridge. The greatest cumulative scour will occur when floods increase in magnitude over the life of the bridge, and vice versa for the smallest cumulative scour.

Even where cumulative scour is not calculated, ultimate scour should be computed for the design flood. The hydraulic engineer should work closely with a geotechnical engineer to fully account for scour in cohesive soils.

13.7.16.10.6 CONTRACTION SCOUR IN ERODIBLE ROCK

Contraction scour can also occur in erodible rock. Some rock types are vulnerable to weathering and abrasion. In addition to hydraulic forces, channels in rock materials may degrade due to wetting and drying, abrasion, and chemical reactions. Some rock, such as weakly-cemented sandstone and other friable rock, may be as erodible as sand while other rock may be extremely erosion resistant. The concepts from the previous section can also be applied to erodible rock; however, it is not only necessary to determine the critical shear and erosion rate information, but also account for other potential factors. In order to fully account for scour in rock, the hydraulic engineer must work closely with a geotechnical engineer and geologist.

13.7.16.10.7 MEAN VELOCITY METHOD

This is an alternative method which uses the concept of cross-sectional velocity as a criterion of contraction scour. It can be used as a check on the contraction scour estimates obtained from **Equation (13.143)** for live-bed and clear-water scour for Cases 1, 2 and 4. The method is as follows:

i. Calculate mean velocity of flow in the unrestricted main channel for the discharge at which contraction scour will just commence.
ii. Determine average contraction scour level that will make the mean velocity through the bridge opening equal to the estimated mean velocity calculated in (i) above.

It is assumed that the discharge at which contraction scour will just commence is that of the total waterway design flood. This flood has been chosen as experience has shown that it appears to give realistic estimates of contraction scour. The total waterway design flood can be either the 50 or 100 year ARI (0.02 or 0.01 AEP) flood depending upon the road design class.

It is not anticipated that estimates of contraction scour will vary greatly with the use of either flood, as velocities in the main channel of a stream should be of similar magnitude for both floods.

13.7.16.10.8 SCOUR AT ABUTMENTS

Where the abutments and roadway embankments obstruct river flow, scour will occur at bridge abutments.

Several causes of abutment failures observed during post-flood field inspections of bridge sites include:

i. overtopping of abutments or approach embankments
ii. lateral channel migration or stream widening processes
iii. contraction scour
iv. local scour at one or both abutments.

Abutment damage is often caused by a combination of these factors. The abutments most vulnerable to damage are generally those located at or near the channel banks. Large scour holes with depths as much as four times that of the approach floodplain flow depth have been observed where abutments are set back from channel banks, especially on wide floodplains.

Abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel at the abutment. The discharge returned to the main channel at the abutment is not simply a function of the abutment and roadway length. Abutment scour depth depends on abutment shape, discharge in the main channel at the abutment, discharge intercepted by the abutment and returned to the main channel at the abutment, sediment characteristics, cross-sectional shape of the main channel at the abutment (especially the depth of flow in the main channel and depth of the overbank flow at the abutment), and alignment. In addition, there may be tree-lined or vegetated banks, low velocities, and shallow depths upstream of the abutment.

Various methods have been used in estimating abutment scour in determining the potential depth of scour to aid in the design of the foundation and placement of rock riprap and/or guide banks. Some examples include Froehlich's live-bed scour equation, the HIRE equation in FHWA's HDS 6 (Arneson et al. (2012) and the approach developed under NCHRP Project 24-20 (Ettema, Nakato, & Muste 2010). Froehlich's live-bed scour equation is reproduced below for illustration. Refer to Arneson et al. (2012) for full details.

Froehlich's live-bed scour equation (**Equation (13.150)**) was established based on the analyses of 170 live-bed scour measurements in laboratory flumes by regression analysis:

$$\frac{y_s}{y_a} = 2.27K_1K_2\left(\frac{L'}{y_a}\right)^{0.43}Fr^{0.61} + 1 \qquad (13.150)$$

Where,

K_1 = coefficient for abutment shape

K_2 = coefficient for angle of embankment to flow:
$(\theta/90)^{0.13}$
$\theta < 90^o$ if embankment points downstream
$\theta > 90^o$ if embankment points downstream

L' = length of active flow obstructed by the embankment (m)

A_e = flow area of the approach cross-section obstructed by the embankment (m^2)

Fr = Froude Number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$

V_e = velocity at approach cross-section obstructed by the embankment Q_e/A_e (m/s)

Q_e = flow obstructed by the abutment and approach embankment (m^3/s)

y_a = average depth of flow on the floodplain (A_e/L) (m)

L = length of embankment projected normal to the flow (m)

y_s = scour depth (m)

13.7.16.10.9 LOCAL SCOUR AT PIERS

Local scour at piers is a function of bed material characteristics, bed configuration, flow characteristics, fluid properties, and the geometry of the pier and footing. The bed material characteristics are either granular or non-granular, cohesive or non-cohesive, erodible or non-erodible rock.

In order to determine local pier scour, flow characteristics such as velocity, depth upstream of the pier, angle of attack and whether or not the flow is free surface or pressure need to be determined. The viscosity of the flow is also required (surface tension can be ignored).

The key pier geometry characteristics are its type, dimensions, and shape. Types of piers can include single column, multiple columns, or rectangular; with or without friction or tip bearing piles; with or without a footing or pile cap; footing or pile cap in the bed, on the surface of the bed, in the flow or under the deck out of the flow. Important dimensions are the diameter for circular piers or columns, spacing for multiple columns, and width and length for solid piers. Shapes can include round, square or sharp nose, circular cylinder, group of cylinders, or rectangular (**Figure 13.110**). In addition, piers may be simple or complex. A simple pier is generally considered to be a single shaft, column or multiple columns exposed to the flow, whereas, a complex pier may have the pier, footing or pile cap, and piles exposed to the flow.

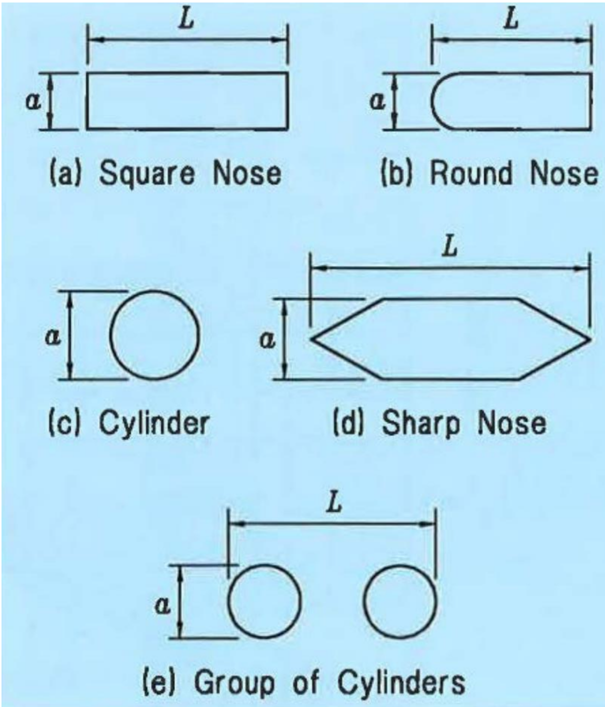


Figure 13.110 Common pier shapes

The HEC-18 pier scour equations (based on the Colorado State University (CSU) equation) are recommended for both live-bed and clear-water pier scour (**Equation (13.151)** and **Equation (13.152)**). Refer to **Figure 13.111** for the definition sketch of pier scour.

$$\frac{y_s}{y_a} = 2.0K_1K_2K_3\left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} \tag{13.151}$$

$$\frac{y_s}{y_a} = 2.0K_1K_2K_3\left(\frac{y_1}{a}\right)^{0.35} Fr_1^{0.43} \tag{13.152}$$

as a rule of thumb, the maximum scour depth for round nose piers aligned with the flow is:

$y_s \leq 2.4$ times the pier width (a) for $Fr \leq 0.8$

$y_s \leq 3.0$ times the pier width (a) for $Fr > 0.8$

Where,

y_s = scour depth (m)
y_1 = flow depth directly upstream of pier
K_1 = correction factor for pier nose shape
K_2 = correction factor for angle of attack of flow
K_3 = correction factor for bed condition
a = pier width (m)
L = length of pier (m)
Fr_1 = Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
V_1 = mean velocity of flow directly upstream of the pier (m/s)
g = acceleration of gravity, 9.81 (m/s^2)

This equation can be applied to wide pier applications, more complex (3-element) substructure configurations, multiple columns skewed to the flow, estimating scour from debris on piers, and scour in tidal waterways.

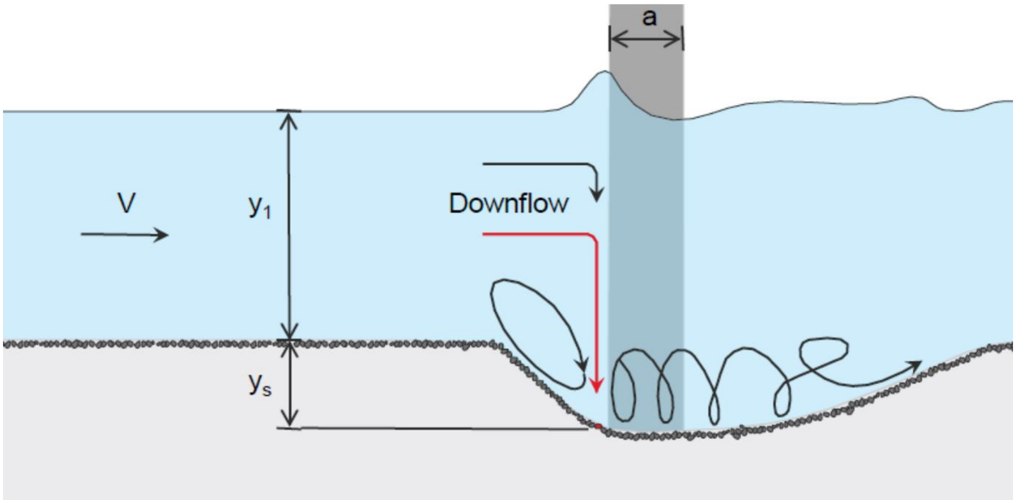


Figure 13.111 Definition sketch for pier scour

Note: The correction factor K_1 should be determined using **Table 13.55** for flow angle of attack up to 5 degrees. For greater angles, K_2 (**Table 13.56**) dominates and K_1 should be considered as 1.0. If L/a is larger than 12, use the values of L/a = 12 as a maximum.

Table 13.55 Correction factor, K_1 for pier nose shape

Shape of nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Sharp nose	0.9
(e) Group of cylinders	1.0

Table 13.56 Correction factor, K_2 for angle of attack of flow

Angle (^{0})	L/a = 4	L/a = 8	L/a = 12
0	1	1	1
15	1.5	2	2.5
30	2	2.5	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5

Angle = skew angle of flow

L = length (m) of pier

a = width (m) of pier

Top width of scour hole – for practical purposes the top width of a scour hole in cohesionless bed material from one side of a pier or footing can be taken as 2.8 y_s.

Footings and pile caps – where the top of the footing (or pile cap) is at or below the stream bed (after taking into account contraction scour and long-term degradation) it is recommended that the pier width be used for the value of 'a' in the pier scour equation.

Where the footing or pile cap extends above the stream bed, a second computation should be made using the width of the footing (or pile cap) for the value of a and the depth and average velocity in the flow zone obstructed by the footing for the y_1 and V_1 respectively in the scour equation. The larger of the two scour computations should be used. The average velocity of flow at the exposed footing (V_f) should be determined using **Equation (13.153)** (Jones 1989):

$$\frac{V_f}{V_1} = \frac{\ln(10.93\frac{y_f}{k_s} + 1)}{\ln(10.93\frac{y_1}{k_s} + 1)} \qquad (13.153)$$

Where,

V_f = average velocity in the flow zone below the top of the footing (m/s)

V_1 = mean velocity of approach flow upstream of the pier (m/s)

y_f = distance from the bed to the top of the footing

k_s = the grain roughness of the bed = D_{84} (m) of the bed material

y_1 = depth of flow upstream of the pier (m)

Exposed pile groups – pile groups that project above the stream bed (as a result of contraction scour or long-term degradation) can be analysed conservatively by representing them as a single pier width equal to the projected area of the piles, ignoring the clear space between them. Good judgement needs to be used in accounting for debris which may collect on the piles and cause the pile group to act as a much larger obstruction to flow.

If a pile group is exposed to the flow as the result of local scour then it is unnecessary to consider the piles in calculating pier scour.

The correction factor K_1 in **Equation (13.141)** for the multiple piles would be 1.0 regardless of the layout of the piles. If the pile group is a square, then K_1 would be 1.0. However, if the pile group is a rectangle, then the dimensions used would be based on the assumption that they are a single pier and the appropriate L/a value used for determining K_2.

Multiple columns – for multiple columns skewed to the flow, the scour depth depends on the spacing between the columns. The correction factor K_2 for angle of attack would be smaller than for a solid pier. The pier width, a in **Equation (13.141)** would be the total projected width of all the columns in a single bent, normal to the flow angle of attack. The correction factor K_1, for the multiple columns would be 1.0 regardless of column shape. If debris is likely to pile up against the multiple columns, it would be logical to consider the multiple columns as a solid elongated pier. In this instance the appropriate L/a value and flow angle of attack would then be used to determine K_2 from **Table 13.56**.

Pier scour in cohesive material – Pier scour in cohesive materials generally progresses more slowly and is more dependent on soil properties than for non-cohesive sediments. These properties include critical velocity, critical shear stress, and the erosion rate for hydraulic conditions that exceed the critical value.

It should be noted that the HEC-RAS software can be used to compute scour estimates for contraction and local scour at piers and abutments (but only for cohesionless soils).

13.7.16.10.10 PRESSURE FLOW SCOUR

Scour in pressurised flow conditions can be estimated as follows. Refer to **Figure 13.112** for the definition and geometric parameters.

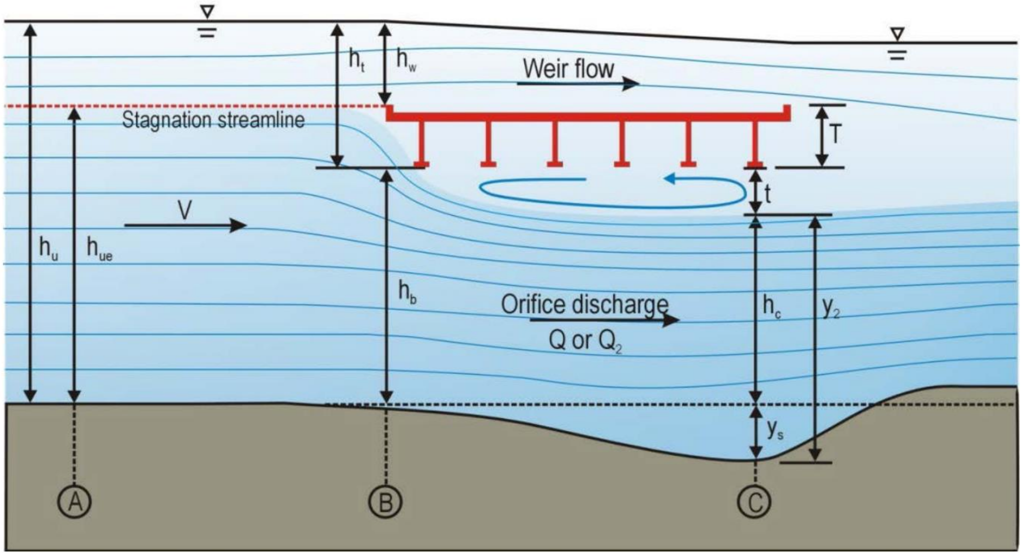


Figure 13.112 Pressure flow scour at a fully submerged bridge site

The bridge 'superstructure' mentioned in this section refers to a continuous cross-section of the structural and non-structural elements that span the waterway and that can produce significant blockage when it is partially or fully inundated. Discharge under the superstructure can be conservatively assumed to be all approach flow below the top of the superstructure at height h_b+T, where h_b is the vertical size of the bridge opening prior to scour and T is the height of the obstruction including girders, deck, and parapet. For floods that do not create overtopping, all discharge upstream goes into the bridge opening.

The depth at the location of maximum scour is comprised of three components:

- h_c – the vertically contracted flow height from the streamline bounding the separation zone under the superstructure at the maximum scour depth
- y_s – the scour depth
- t – the maximum thickness of the flow separation zone (this zone does not convey any net mass from the upstream opening of the bridge to the downstream exit).

The pressure scour depth y_s is determined by using the horizontal contraction scour equations to calculate the height, $y_s + h_c$, required to convey flow through the bridge opening at the critical velocity. This height is equivalent to y_2 (the average depth in the contracted section) in the clear-water contraction scour (**Equation (13.144)**) and the live-bed contraction scour (**Equation (13.142)**). Combining this relation with the definitions of t and h_b (**Equation (13.154)**):

$$y_s = y_2 + t - h_b \tag{13.154}$$

It should be noted that h_b in pressure flow scour is analogous to y_0 (existing depth in the contracted section before scour) in contraction scour (see **Equation (13.142)** and **Equation (13.143)**). These equations show that the scour depth of pressure flow can be significantly greater than that of non-pressure flow because depth available to convey flow through the opening under the bridge is reduced by the flow separation thickness, t. This thickness can be calculated using **Equation (13.155)**:

$$\frac{t}{h_b} = 0.5\left(\frac{h_b h_t}{h_u^2}\right)^{0.2}\left(1 - \frac{h_w}{h_t}\right)^{-0.1} \quad (13.155)$$

Where,

h_b = vertical size of the bridge opening prior to scour (m)

h_u = upstream channel flow depth (m)

h_t = distance from the water surface to the lower face of the bridge girders, equals $h_u - h_b$ (m)

h_w = weir flow height, $h_w = h_t - T$ for $h_t > T$, $h_w = 0$ for $h_t \leq T$

13.7.16.11 SCOUR COUNTERMEASURES

Scour countermeasures are those works incorporated into an existing stream crossing to monitor, control, inhibit or minimise stream stability problems and bridge scour to make a bridge less vulnerable to damage or failure from scour. In many cases, the best countermeasure is appropriate design that avoids causing stream instability. The alternatives available for protecting an existing bridge from scour are listed below, roughly in order of increasing cost:

i. providing rock protection at piers and abutments
ii. constructing or lengthening guide banks
iii. constructing channel improvements
iv. strengthening bridge foundations
v. constructing sills or drop structures
vi. constructing relief bridges or lengthening existing bridges.

A guide for providing rock protection at abutments and piers follows. All bridges over streams subject to scour should be regularly inspected to ensure that they are not at risk of damage or failure as a result of scour. Where a bridge is found to be vulnerable to scour it should be closely monitored and consideration given to closing it until scour countermeasures are installed. In some instances, it may be appropriate to install interim or temporary measures to protect the bridge and public until suitable long-term countermeasures can be put in place.

Over the last several decades, a wide variety of countermeasure structures, armouring materials and monitoring devices have been used to mitigate scour and stream stability problems at existing bridges.

While standard countermeasures such as riprap will be familiar to most bridge inspectors and engineers, it is unlikely they are knowledgeable of the full spectrum of countermeasures available and in use.

13.7.16.11.1 COUNTERMEASURE GROUPS AND CHARACTERISTICS

A wide variety of countermeasures have been used to control channel instability and scour at bridge foundations. The countermeasure matrix, presented in **Table 13.57**, is organized to highlight the various groups of countermeasures and to identify their individual characteristics.

Countermeasures have been organized into groups based on their functionality with respect to scour and stream instability. The four main groups of countermeasures are: hydraulic countermeasures, structural countermeasures, biotechnical countermeasures, and

monitoring. The following outline identifies the countermeasure groups in the matrix:

Group 1. Hydraulic Countermeasures

i. Group 1.A: River training structures
- Transverse structures

- Longitudinal structures
- Areal structures

ii. Group 1.B: Armouring countermeasures
- Revetments and bed armour
- Rigid
- Flexible/articulating
- Local scour armouring

Group 2. Structural Countermeasures

i. Foundation strengthening
ii. Pier geometry modification

Group 3. Biotechnical Countermeasures

Group 4. Monitoring

i. Fixed instrumentation
ii. Portable instrumentation
iii. Visual monitoring

13.7.16.11.2 GROUP 1. HYDRAULIC COUNTERMEASURES

Hydraulic countermeasures are those which are primarily designed either to modify the flow or resist erosive forces caused by the flow. Hydraulic countermeasures are organized into two groups:

i. river training structures and
ii. armouring countermeasures.

The performance of hydraulic countermeasures is dependent on design considerations such as edge treatment and filter requirements.

i. **Group 1.A River Training Structures**.
River training structures are those which modify the flow. River training structures are distinctive in that they alter hydraulics to mitigate undesirable erosional and/or depositional conditions at a particular location or in a river reach. River training structures can be constructed of various material types and are not distinguished by their construction material, but rather, by their orientation to flow. River training structures are described as transverse, longitudinal or areal depending on their orientation to the stream flow.
    - **Transverse river training structures** are countermeasures which project into the flow field at an angle or perpendicular to the direction of flow.
    - **Longitudinal river training structures** are countermeasures which are oriented parallel to the flow field or along a bankline.
    - **Areal river training structures** are countermeasures which cannot be described as transverse or longitudinal when acting as a system. This group also includes countermeasure "treatments" which have areal characteristics such as channelization, flow relief, and sediment detention.

Table 13.57 Stream instability and bridge scour countermeasures matrix

Countermeasure Group	Countermeasure Characteristics														
	FUNCTIONAL APPLICATIONS						SUITABLE RIVER ENVIRONMENT								MAINTENANCE
	Local Scour		Contraction Scour	Stream Instability		Overtopping Flow	River Type	Stream Size	Bend Radius	Velocity	Bed Material	Debris Load	Bank Slope	Floodplain	Estimated Allocation of Resources
	Abutments	Piers	Floodplain and Channel	Vertical	Lateral	Approach Embankments	B = braided M = meandering S = straight	W = wide M = moderate S = small	L = long M = moderate S = short	F = fast M = moderate S = slow	C = coarse bed S = sand bed F = fine bed	H = high M = moderate L = low	V = very steep S = steep M = mild	W = wide M = moderate N = narrow/none	H = high M = moderate L = low
GROUP 1. HYDRAULIC COUNTERMEASURES															
GROUP 1.A. RIVER TRAINING STRUCTURES															
TRANSVERSE STRUCTURES															
Impermeable spurs (jetties, groins, wing dams)	◗	◗	○	○	●	○	B, M	W, M	L, M	✓	✓	✓	✓	✓	M - L
Permeable spurs (fences, netting)	◗	◗	○	○	●	○	B, M	W, M	L, M	M, S	S, F	L	✓	✓	H - M
Transverse dikes	○	○	○	○	●	○	B, M	W, M	✓	✓	✓	✓	✓	✓	M - L
Bendway weirs/Stream barbs	◗	◗	○	○	●	○	M	✓	M, S	✓	✓	✓	✓	✓	L
Hardpoints	○	○	○	○	●	○	✓	✓	✓	✓	✓	✓	✓	✓	L
Drop structures (check dams, grade control)	◗	◗	◗	●	○	○	✓	✓	✓	✓	✓	✓	✓	✓	M
Embankment Spurs	◗	○	◗	○	○	○	✓	✓	✓	✓	✓	✓	✓	W	L
LONGITUDINAL STRUCTURES															
Longitudinal dikes (crib/rock toe/embankments)	◗	○	○	○	●	◗	✓	✓	L, M	✓	✓	M, L	✓	✓	M - L
Retards	◗	○	○	○	●	○	✓	✓	L, M	✓	S, F	L	✓	✓	H - M
Bulkheads	●	○	○	○	●	○	✓	✓	✓	✓	✓	✓	V, S	✓	M
Guide banks	●	◗	◗	○	◗	◗	✓	W, M	✓	✓	✓	✓	✓	W, M	M - L
AREAL STRUCTURES/TREATMENTS															
Jacks/tetrahedron jetty fields	○	○	○	○	●	○	B, M	W, M	L	M, S	S, F	M, L	✓	W, M	M
Vanes	○	◗	○	○	●	○	B, M	W, M	L, M	M, S	S, F	L	✓	✓	H - M
Channelization	◗	◗	○	○	●	○	B, M	✓	✓	✓	✓	✓	✓	✓	M
Flow relief (overflow, relief bridge)	◗	◗	●	○	○	●	✓	✓	✓	✓	✓	✓	✓	W	M
Sediment detention basin	○	○	○	●	○	○	✓	✓	✓	✓	C, S	✓	✓	✓	H - M

● well suited/primary use

◗ possible application/secondary use

○ unsuitable/rarely used

N/A not applicable

✓ suitable for the full range of the characteristic

Table 13.57 Stream instability and bridge scour countermeasures matrix (continued)

Countermeasure Group	Countermeasure Characteristics														
	FUNCTIONAL APPLICATIONS						SUITABLE RIVER ENVIRONMENT								MAINTENANCE
	Local Scour		Contraction Scour	Stream Instability		Overtopping Flow	River Type	Stream Size	Bend Radius	Velocity	Bed Material	Debris Load	Bank Slope	Floodplain	Estimated Allocation of Resources
	Abutments	Piers	Floodplain and Channel	Vertical	Lateral	Approach Embankments	B = braided M = meandering S = straight	W = wide M = moderate S = small	L = long M = moderate S = short	F = fast M = moderate S = slow	C = coarse bed S = sand bed F = fine bed	H = high M = moderate L = low	V = very steep S = steep M = mild	W = wide M = moderate N = narrow/none	H = high M = moderate L = low
GROUP 1.B. ARMORING COUNTERMEASURES															
REVETMENTS AND BED ARMOR															
Rigid															
Soil cement	●	●	◗	◗	●	●	✓	✓	✓	✓	S, F	✓	✓	✓	L
Roller compacted concrete	●	◗	●	●	●	●	✓	✓	✓	✓	S, F	✓	✓	✓	L
Concrete pavement	◗	○	●	◗	●	◗	✓	✓	✓	✓	✓	✓	S, M	✓	M
Rigid grout filled mattress/concrete fabric mat	◗	○	◗	◗	●	◗	✓	✓	✓	✓	✓	✓	S, M	✓	M
Fully grouted riprap	○	○	○	○	◗	○	✓	✓	✓	✓	✓	✓	S, M	✓	M
Flexible/articulating															
Riprap	●	●	◗	◗	●	◗	✓	✓	✓	✓	✓	✓	S, M	✓	M
Self launching riprap (windrow)	○	○	○	○	◗	○	✓	✓	✓	✓	C,S	✓	V, S	✓	H - M
Riprap fill-trench	◗	○	○	○	●	○	✓	✓	✓	✓	✓	✓	✓	✓	M
Gabions/gabion mattress	●	●	◗	◗	●	◗	✓	✓	✓	✓	S, F	M, L	✓	✓	M
Wire enclosed riprap mattress (rail bank/sausage)	●	○	○	○	●	○	✓	✓	✓	M, S	S, F	M, L	S, M	✓	M
Articulated blocks (interlocking and/or cable tied)	●	●	●	◗	●	●	✓	✓	✓	✓	✓	✓	S, M	✓	M - L
Concrete/grout mattress (fabric-formed)	●	◗	●	◗	●	◗	✓	✓	✓	M, S	✓	✓	S, M	✓	M - L
Partially grouted riprap	●	●	◗	◗	●	○	✓	✓	✓	✓	✓	✓	S, M	✓	L
LOCAL SCOUR ARMORING															
Riprap (fill/apron)	●	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, M	✓	H - M
Fully grouted riprap	◗	○	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, M	✓	H - M
Concrete armor units (Toskanes, tetrapods, etc.)	◗	◗	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	M	✓	M - L
Grout filled bags/sand cement bags	●	◗	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	M, L	M	✓	H - M
Gabions/gabion mattress	●	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	S, F	M, L	S, M	✓	M
Articulated blocks (interlocking and/or cable tied)	●	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, M	✓	M - L
Sheet pile/cofferdam	◗	◗	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	M - L
Partially grouted riprap	●	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, M	✓	L

● well suited/primary use

◗ possible application/secondary use

○ unsuitable/rarely used

N/A not applicable

✓ suitable for the full range of the characteristic

Table 13.57 Stream instability and bridge scour countermeasures matrix (continued)

Countermeasure Group	Countermeasure Characteristics														
	FUNCTIONAL APPLICATIONS						SUITABLE RIVER ENVIRONMENT								MAINTENANCE
	Local Scour		Contraction Scour	Stream Instability		Overtopping Flow	River Type	Stream Size	Bend Radius	Velocity	Bed Material	Debris	Bank Slope	Floodplain	Estimated Allocation of Resources
	Abutments	Piers	Floodplain and Channel	Vertical	Lateral	Approach Embankments	B = braided M = meandering S = straight	W = wide M = moderate S = small	L = long M = moderate S = short	F = fast M = moderate S = slow	C = coarse bed S = sand bed F = fine bed	H = high M = moderate L = low	V = very steep S = steep M = mild	W = wide M = moderate N = narrow/none	H = high M = moderate L = low
GROUP 2. STRUCTURAL COUNTERMEASURES															
FOUNDATION STRENGTHENING															
Crutch bents/Underpinning	○	●	●	●	◗	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L
Cross bracing	○	●	●	●	○	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L
Continuous spans	○	●	●	●	○	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L
Pumped concrete/grout under footing	●	●	◗	◗	◗	N/A	✓	✓	✓	✓	✓	✓	✓	✓	M
Lower foundation	●	●	●	●	●	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L
PIER GEOMETRY MODIFICATION															
Extended footings	N/A	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L
Pier shape modifications	N/A	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	M
Debris deflectors	N/A	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	H - M
Sacrificial piles/dolphins	N/A	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	H - M
GROUP 3. BIOTECHNICAL COUNTERMEASURES															
Vegetated geosynthetic products	○	○	○	○	●	◗	M, S	M, S	✓	M, S	✓	M, L	M, S	✓	H - M
Fascines/woody mats	○	○	○	○	●	○	✓	M, S	✓	M, S	✓	L	M, S	✓	H - M
Vegetated riprap	○	○	○	○	●	◗	✓	✓	✓	✓	✓	✓	M, S	✓	M - L
Root wads	○	○	○	○	●	○	✓	M, S	✓	M, S	✓	L	M	✓	H - M
Live staking	○	○	○	○	●	○	✓	M, S	✓	M, S	✓	M, L	M, S	✓	H - M
GROUP 4. MONITORING															
FIXED INSTRUMENTATION															
Sonar scour monitor	◗	●	●	●	◗	○	✓	✓	✓	✓	✓	L	✓	✓	M
Magnetic sliding collar	●	●	●	●	◗	○	✓	✓	✓	✓	S, F	✓	✓	✓	M
Float out device	●	●	●	●	●	●	✓	✓	✓	✓	S, F	✓	✓	✓	L
Sounding rods	◗	●	●	●	◗	○	✓	✓	✓	M, S	C	M, L	✓	✓	H
PORTABLE INSTRUMENTATION															
Physical probes	●	●	●	●	●	○	✓	✓	✓	M, S	✓	M, L	✓	✓	L
Sonar probes	●	●	●	●	●	○	✓	✓	✓	M, S	✓	L	✓	✓	L
VISUAL MONITORING															
Periodic Inspection	●	●	●	●	●	●	✓	✓	✓	✓	✓	M, L	✓	✓	H
Flood watch	●	●	●	●	●	●	✓	✓	✓	✓	✓	M, L	✓	✓	H

● well suited/primary use

◗ possible application/secondary use

○ unsuitable/rarely used

N/A not applicable

✓ suitable for the full range of the characteristic

ii. **Group 1.B Armouring Countermeasures.**
Armouring countermeasures are distinctive because they resist the erosive forces caused by a hydraulic condition. Armouring countermeasures do not necessarily alter the hydraulics of a reach but act as a resistant layer to hydraulic shear stresses providing protection to the more erodible materials underneath. Armouring countermeasures generally do not vary by function but vary more in material type. Armouring countermeasures are classified by two functional groups: revetments and bed armouring or local scour armouring.

- **Revetments and bed armouring** are used to protect the channel bank and/or bed from erosive/hydraulic forces. They are usually applied in a blanket type fashion for areal coverage. Revetments and bed armouring can be classified as either rigid or flexible/articulating. Rigid revetments and bed armouring are typically impermeable and do not have the ability to conform to changes in the supporting surface. These are subject to failure due to undermining. Flexible/ articulating revetments and bed armouring can conform to changes in the supporting surface and adjust to settlement; however, these countermeasures can fail by removal and displacement of the armour material.
- **Local scour armouring** is used specifically to protect individual substructure elements of a bridge from local scour. Generally, the same material used for revetments and bed armouring is used for local scour armouring, but these countermeasures are designed and placed to resist local vortices created by obstructions to the flow.

13.7.16.11.3 GROUP 2. STRUCTURAL COUNTERMEASURES

Structural countermeasures involve modification of the bridge structure (foundation) to prevent failure from scour. Typically, the substructure is modified to increase bridge stability after scour has occurred or when a bridge is assessed as scour critical. These modifications are classified as either foundation strengthening or pier geometry modifications.

- **Foundation strengthening** includes additions to the original structure which will reinforce and/or extend the foundations of the bridge. These countermeasures are designed to prevent failure when the channel bed is lowered to an expected scour elevation, or to restore structural integrity after scour has occurred. Design and construction of bridges with continuous spans provide redundancy against catastrophic failure due to substructure displacement as a result of scour. Retrofitting a simple span bridge with continuous spans could also serve as a countermeasure after scour has occurred or when a bridge is assessed as scour critical.
- **Pier geometry modifications** are used to either reduce local scour at bridge piers or to transfer scour to another location. These modifications are used primarily to minimize local scour.

13.7.16.11.4 GROUP 3. BIOTECHNICAL COUNTERMEASURES

Vegetation has been used increasingly over the past few decades to control streambank erosion or as a bank stabilizer. It has been used primarily in stream restoration and rehabilitation projects and can be applied independently or in combination with structural countermeasures. There are several terms that describe vegetative streambank stabilization and countermeasures. The use of 'soft' revetments (consisting solely of living plant materials or plant products) is often referred to as bioengineering. The techniques that combine the use of vegetation with structural (hard) elements include biotechnical engineering and biotechnical slope protection. Where riprap constitutes the "hard" component of biotechnical slope protection, the term vegetated riprap is also used.

The matrix considers representative categories for biotechnically engineered counter-measures (which incorporate rock) including:

- **Vegetated geosynthetic products**
- **Fascines/woody mats**
- **Vegetated riprap**
- **Root wads**
- **Live staking**

Biotechnical engineering can be a useful and cost-effective tool in controlling bank or channel erosion, while increasing the aesthetics and habitat diversity of the site. However, where failure of the countermeasure could lead to failure of a bridge or highway structure, the only acceptable solution in the immediate vicinity of a structure is a traditional, "hard" engineering approach.

13.7.16.11.5 GROUP 4. MONITORING

Monitoring describes activities used to facilitate early identification of potential scour problems. Monitoring could also serve as a continuous survey of the scour progress around the bridge foundations. While monitoring does not fix the scour problem at a scour critical bridge, it allows for action to be taken before the safety of the public is threatened by the potential failure of the bridge. Monitoring can be accomplished with instrumentation or visual inspection. A well-designed monitoring program can be a very cost-effective countermeasure. Two types of instrumentation are used to monitor bridge scour: fixed instruments and portable instruments.

- **Fixed instrumentation** describes monitoring devices which are attached to the bridge structure to detect scour at a particular location. Typically, fixed monitors are located at piers and abutments. The number and location of piers to be instrumented should be defined, as it may be impractical to place a fixed instrument at every pier and abutment on a bridge. Instruments such as sonar monitors can be used to provide a timeline of scour, whereas instruments such as magnetic sliding collars can only be used to monitor the maximum scour depth. Data from fixed instruments can be downloaded manually at the site or it can be telemetered to another location.
- **Portable instrumentation** describes monitoring devices that can be manually carried and used along a bridge and transported from one bridge to another. Portable instruments are more cost effective in monitoring an entire bridge than fixed instruments; however, they do not offer a continuous watch over the structure. The allowable level of risk will affect the frequency of data collection using portable instruments.
- **Visual inspection** describes standard monitoring practices of inspecting the bridge on a regular interval and increasing monitoring efforts during high flow events (flood watch). Typically, bridges are inspected on a biennial schedule where channel bed elevations at each pier location are taken. The channel bed elevations should be compared with historical cross sections to identify changes due to scour. Channel elevations should also be taken during and after high flow events. If measurements cannot be safely collected during a high flow event, the bridge owner should determine if the bridge is at risk and if closure is necessary. Underwater inspections of the foundations could be used as part of the visual inspection after a flood.

13.7.16.12 DESIGN OF COUNTERMEASURES

13.7.16.12.1 FILTER LAYER

Many scour countermeasures consist of a filter layer (geotextile or granular) overlain by a heavy-duty armour (usually rock riprap) and possibly a form of containment (basket or cables) holding it together. Correct design of the filter layer is essential and often overlooked. Filters limit the loss of fines, while providing a free-flowing interface. To achieve this, the permeability of the geotextile should be ten times that of the underlying soil. If the filter is too broad the rock riprap will roll off the filter and compromise the countermeasure.

Various considerations for filter design are required, including base soil properties, particle size distribution, plasticity, porosity and hydraulic conductivity of the soil and filter materials.

There are two main filter materials, including granular and geotextile.

13.7.16.12.2 ROCK RIPRAP AT BRIDGE PIERS

Properly designed riprap used for erosion protection has an advantage over rigid structures due to its flexibility when under attack by river currents, as it can remain functional even if some individual stones are lost and can be repaired relatively easily. Properly constructed riprap can provide long-term protection if it is inspected and maintained on a periodic basis as well as after flood events. This section considers the application of riprap as a pier scour countermeasure.

Design of a pier scour countermeasure system using riprap requires knowledge of the:

i. river bed and foundation material
ii. flow conditions including velocity, depth and orientation
iii. riprap characteristics of size, density, durability, and availability
iv. pier size, shape, and skew with respect to flow direction
v. type of interface material between the riprap and underlying foundation.

The system deployed will typically include a filter layer, either a geotextile fabric or a filter of sand and/or gravel, specifically selected for compatibility with the subsoil. The filter allows infiltration and ex-filtration to occur while providing particle retention. Typical pier riprap configurations are shown in **Figure 13.113**.

It is worth noting that, for bridge piers, rock riprap is not a permanent countermeasure for scour at piers at existing bridges and should not be used to protect piers at new bridges. The class of rock protection required to protect a pier from scour is determined from the velocity obtained by multiplying the velocity of flow approaching the pier by a coefficient, K_p for pier shape.

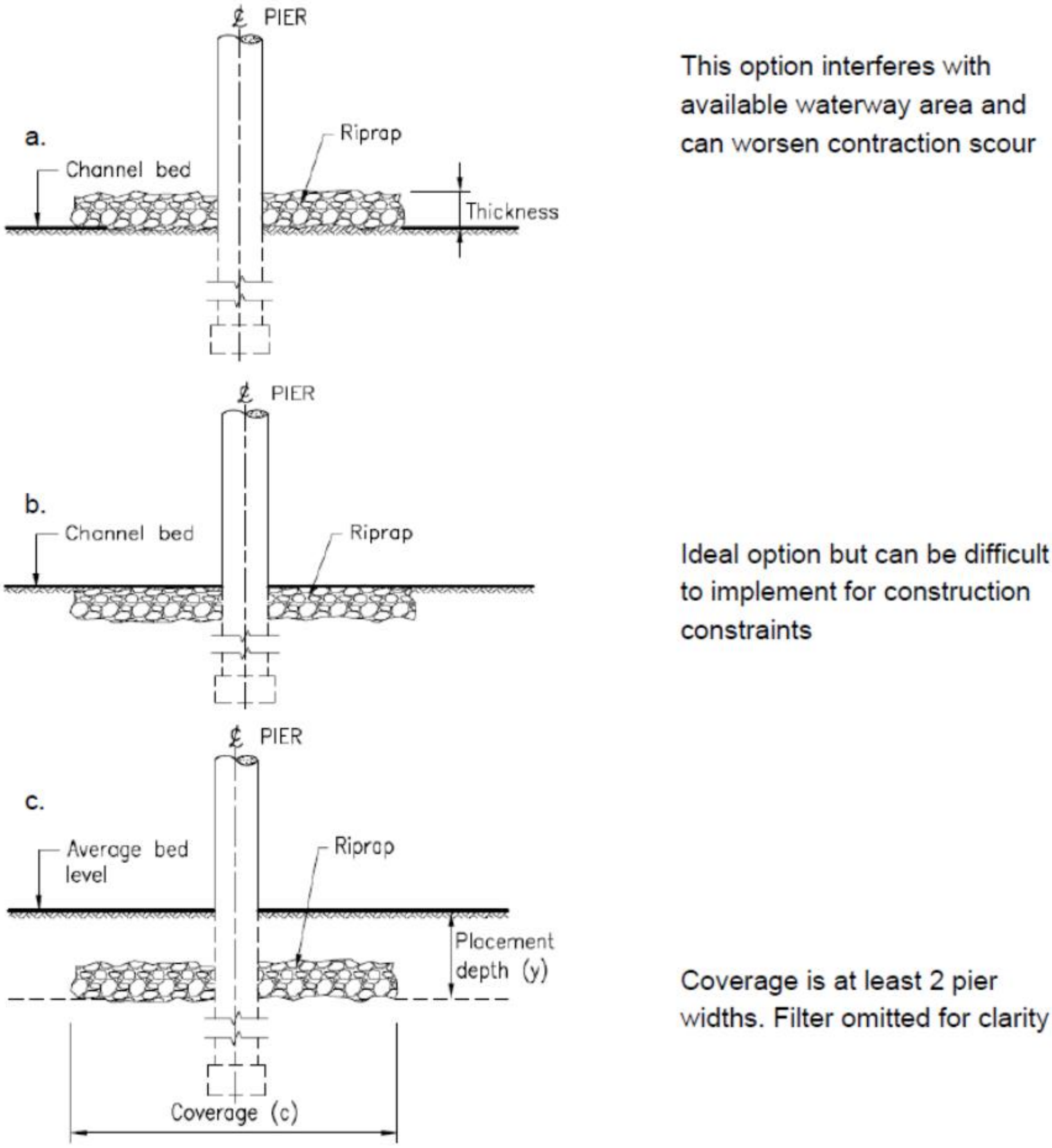


Figure 13.113 Typical pier riprap configurations (filter omitted for clarity)

The average velocity of flow, V, approaching the pier is estimated by taking the average velocity in the bridge opening and multiplying it by a coefficient that ranges from 0.9 for a pier near the bank in a straight uniform reach of the stream to 1.7 for a pier in the main current of flow around a bend. For piers located on the floodplain the velocity on the floodplain should be used. Where a pier is located adjacent to an abutment and the bridge approach is cutting off significant flow, then some allowance should be made for the resulting increase in velocity caused by the cut-off flow entering the bridge opening.

Coefficient for pier shape, K_p can be taken as 1.5 for round-nose pier, and 1.7 for rectangular pier.

Rock protection should be provided as follows:

i. The class and thickness of rock is determined from Table 5.11 for the velocity given by $V \times K_p$.
ii. The riprap mat should extend horizontally at least twice the pier width, measured from the pier face.

iii. The top of the riprap mat should be placed at the same elevation as the stream bed.
iv. In some conditions a filter cloth or gravel filter will be required under the riprap mat. If a well-graded riprap is used a filter may not be needed.

Bridge pier riprap design is primarily based on research conducted under laboratory conditions with little field verification. Flow turbulence and velocities around a pier are of sufficient magnitude that large rocks move over time. The removal of riprap at piers by turbulent high velocity flow has led to the loss of bridges. This is typically the result of a cumulative effect of a sequence of high flows rather than during one storm. Therefore, rock riprap pier scour protection should be monitored and inspected to ensure stability after each high-flow event.

Properly designed rock riprap will afford protection against progressive erosion for abutments and piers where scour is expected. This type of protection has generally been found to be the most practical and economic solution for the protection of spill-through abutments and guide banks. Alternatively, rock filled wire mattresses or hydraulically filled concrete mattresses may be used.

13.7.16.12.3 SIZING ROCK RIPRAP AT BRIDGE PIERS

The required size of stone for riprap at bridge piers is determined by the rearranged Isbash equation (**Equation (13.156)**), as recommended by Lagasse et al. (2009):

$$d_{50} = \frac{0.692(V_{des})^2}{(S_g - 1)2g} \tag{13.156}$$

Where,

d_{50} = particle size for which 50% is finer by weight, (m)
V_{des} = design velocity for local conditions at the pier, (m/s)
S_g = specific gravity of riprap (usually taken as 2.65)
g = acceleration due to gravity, (9.81 m/s^2)

It is important that the velocity used is representative of conditions in the immediate vicinity of the bridge pier including the constriction caused by the bridge. If the cross-section or channel average velocity, V_{avg} is used, then it must be multiplied by factors that are a function of the shape of the pier and its location in the channel (**Equation (13.157)**):

$$V_{des} = K_1 K_2 V_{avg} \tag{13.157}$$

Where,

K_1 = shape factor, equals to 1.5 for round-nose piers or 1.7 for square-faced piers
K_2 = velocity adjustment factor for location in the channel (ranges from 0.9 for a pier near the bank in a straight reach, to 1.7 for a pier located in the main current of flow around a sharp bend)
V_{avg} = channel average velocity at the bridge, m/s

If a velocity distribution is available from the flow distribution output of a 1D model or directly from a 2D model, then only the pier shape coefficient (K_1) should be used. In this case, the maximum velocity in the active channel Vmax is often used since the channel could shift and the highest velocity could impact any pier (**Equation (13.158)**):

$$V_{des} = K_1 V_{max} \tag{13.158}$$

Where,

Vmax = maximum velocity in the active channel, m/s

Once a design size is established, a standard gradation class can be selected, if design criteria and economic considerations permit. Using standard sizes the appropriate gradation can be achieved by selecting the next larger size class, thereby creating a slightly over-designed riprap installation, but economically a less expensive one.

13.7.16.12.4 ROCK RIPRAP AT BRIDGE ABUTMENTS

The class of rock protection required to protect an abutment (without a guide bank) is determined from the velocity given by multiplying the average velocity, V, in the bridge opening by a factor of 1.33, to allow for the turbulently mixing flow action at bridge abutments.

Rock protection should be provided as follows:

i. The class and thickness of rock required to protect the embankment slope and the toe of the embankment should be determined from **Table 13.58** for the velocity given by 1.33*V.
ii. The apron at the toe of the abutment slope should extend along the entire length of the abutment toe, around the curved portions of the abutment to the point of tangency with the plane of the embankment slopes.
iii. The top of the riprap mat should be placed at the same elevation as the stream bed.
iv. In some conditions a permeable geotextile fabric or gravel filter will be required under the riprap mat. If a well-graded riprap is used a filter may not be needed.

Additional requirements for rock riprap are as follows.

Grading of rock – the grading of rock riprap affects its resistance to erosion. The stone should be reasonably well graded throughout the riprap layer thickness. The grading of the various standard classes of rock protection should be in accordance with **Table 13.59**.

The riprap should be well graded from the smallest to the maximum specified. Stones smaller than the specified 10% size should not be present in an amount exceeding 20% by weight of each riprap load.

Quality of rock – the riprap should be hard, dense and durable, as well as being resistant to weathering, free from overburden, spoil, shale and organic matter. Rock that is laminated, fractured, porous, or otherwise physically weak is unacceptable to be used as scour protection.

Stone shape is another important factor in the selection of an appropriate riprap material. Riprap constructed with angular material generally has the best performance. Round material can be used as riprap provided it is not placed on slopes greater than 3:1. Flat slab-like stones can be easily dislodged by flow and should be avoided. The breadth or thickness of a single stone should be not less than one-third its length as an approximate guide for good stone shape.

Method of placement of rock protection – the thickness of the rock protection has been determined assuming the following method of placement.

A footing trench should be excavated, along the toe of the slope as shown on **Figure 13.114**. Rock should be placed so as to provide a minimum of voids. The larger rocks should be placed in the foundation course and on the outside surface of the slope protection. The rock may be placed by dumping and may be spread in layers by bulldozers or other similar equipment.

The embankment and rock protection should be raised in progressive horizontal layers in order to obtain good results where filter fabrics are not used. The larger rocks are placed at the face by bulldozer, and a graded sand/gravel filter material pushed tightly in behind the rock protection where required. The general level of the embankment is then raised to the next level. Local surface irregularities of the slope protection should not vary from the planned slopes by more than 300 mm measured at right angles to the slope.

Filter material – Filter material is material placed between the embankment fill and the rock slope protection to that is designed to prevent fine embankment material from being washed out through the voids of the riprap. The filter may be a permeable geotextile fabric membrane or a graded sand/gravel filter.

Geotextile filter fabrics have generally replaced sand/gravel filters in the construction of roadworks, but they do have a use in the construction of rock protected spill-through abutments as described above. The manufacturer's instructions should be followed when designing geotextile fabric filters. For the design of sand/gravel filters reference should be made to Hydraulic Engineering Circular No 11 (Federal Highway Administration 1989).

When rock slope protection consists of quarry-run rock dumped into place, most of the finer material will naturally settle against the embankment face and the coarser stones will work to the outside, avoiding the need for a filter. But where the face stones are nearly uniform in size and embankment material is vulnerable to scour, a filter will be necessary.

Embankment material should never be carried out over the rock slope protection so that the rock becomes a part of the fill. With this type of construction fill material will filter down through the voids of the large stones and the portion of fill above the rock will be lost.

Table 13.58 Design of rock slope protection

Velocity (m/s)	Class of rock protection, W_c (tonne)	Section thickness, T (m)
< 2	None	-
2.0–2.6	Facing	0.5
2.6–2.9	Light	0.75
2.9–3.9	¼	1
3.9–4.5	½	1.25
4.5–5.1	1.0	1.6
5.1–5.7	2.0	2
5.7–6.4	4.0	2.5
> 6.4	Special	–

Table 13.59 Standard classes of rock slope protection

Rock class	Rock size (1) (m)	Rock mass (kg)	Minimum percentage of rock larger than
Facing	0.4	100	0
	0.3	35	50
	0.15	2.5	90
Light	0.55	250	0
	0.4	100	50
	0.2	10	90
¼ tonne	0.75	500	0
	0.55	250	50
	0.3	35	90
½ tonne	0.9	1000	0
	0.7	450	50
	0.4	100	90
1 tonne	1.15	2000	0
	0.9	1000	50
	0.55	250	90
2 tonne	1.45	4000	0
	1.15	2000	50
	0.75	500	90
4 tonne	1.8	8000	0
	1.45	4000	50
	0.9	1000	90

1 Assuming a specific gravity of 2.65 and spherical shape.

13.7.16.12.5 SIZING ROCK RIPRAP AT ABUTMENTS

It is recommended that the following equations be used to determine the size of rock riprap for protecting abutments from scour for spill-through and vertical wall abutments (Lagasse et al. 2009). Depending on the Froude Numbers ($V/(gy)^{1/2}$), the median stone diameter can be calculated as follows:

For Froude Numbers ≤ 0.80 (**Equation (13.159)**):

$$\frac{d_{50}}{y} = \frac{K}{(S_g - 1)}\left[\frac{V^2}{gy}\right] \tag{13.159}$$

Where,

d_{50} = median stone diameter, (m)

V = characteristic average velocity in the contracted section, (m/s)

S_g = specific gravity of riprap (usually taken as 2.65)
g = acceleration due to gravity, (9.81 m/s^2)
y = depth of flow in the contracted bridge opening (m)
K = velocity multiplier to account for the apparent local acceleration of flow at the point of rock riprap failure, equals 0.89 for a spill-through abutment; 1.02 for a vertical wall abutment

For Froude Numbers > 0.80 (**Equation (13.160)**):

$$\frac{d_{50}}{y} = \frac{K}{(S_g - 1)}\left[\frac{V^2}{gy}\right]^{0.14} \tag{13.160}$$

Where,

K = 0.61 for a spill-through abutment; 0.69 for a vertical wall abutment **Figure 13.40**

13.7.16.12.6 STEEL-WIRE GABION AND MATTRESSES

Gabion and mattresses are containers constructed of steel-wire mesh and filled with rocks. The length of a gabion mattress is greater than the width, and the width is greater than the thickness, refer **Figure 13.41** for typical dimensions. Diaphragms are inserted width wise into the mattress to create compartments.

Galvanized or polyvinyl chloride coated wire is used to resist corrosion, and either welded or twisted into a lattice. Angular rock is preferred to fill the containers due to the higher degree of natural interlocking of the stone fill, however rounded cobbles can also be used. During installation, individual mattresses are connected together by lacing wire or other connectors to form a continuous structure.

Using wire mesh allows the gabions to deform and adapt to changes in the subgrade while maintaining stability. Less excavation of the bed is required and smaller stones can be used when compared to riprap, leading to a more economical solution. The smaller stones used in gabion mattresses are those that are smaller than those that would individually be too small to withstand the hydraulic forces of a stream.

The following types of gabions are commonly used as armouring countermeasures:

i. **Gabion sacks**: are used where it is not possible to dewater the construction area. In the absence of cofferdams, gabion sacks are placed directly in water. The size of a gabion sack can range between 500 mm to 900 mm.
ii. **Gabion boxes or baskets**: are larger in size than sacks and more suitable for higher velocities. The minimum dimension of a gabion box ranges between 600 mm to 1.2 m.
iii. **Rock filled mattresses**: are the most commonly used form of gabion. They are thinner than sacks or boxes and have less weight per unit area. Minimum thickness varies between 200 mm to 450 mm. The mattress is manufactured in greater lengths and tied together. For higher scour depths, two mattresses can be placed on top of each other.
iv. **Wire enclosed riprap**: differs from mattresses in that it is larger in size and is a continuous framework rather than individual interconnected boxes or baskets. They can be used for slope protection at riverbanks or as guide banks. Riprap sizes that are used are less uniform when compared to other three types discussed above.

It should be noted that gabions and mattresses have durability concerns due to the durability of the steel wire mesh. The maximum life for gabion is 50 years as claimed by the manufacturers.

13.7.16.12.7 GROUT-FILLED MATTRESSES

Grout-filled mattresses (mats) are comprised of a double layer of strong synthetic (woven nylon or polyester) fabric, sewn into a series of pillow-shaped compartments that are connected by internal ducts. An example is shown in **Plate 13.20**. The compartments are filled with a concrete grout that flows from compartment to compartment via the ducts. Mats are typically sewn together or otherwise connected prior to filling.

Plate 13.20 A possible flexible collar arrangement at a pile to seal joint with a mattress

The grout forms a mat made up of a grid of interconnected blocks when set. The mats are reinforced by cables laced through the mat before the concrete is pumped into the form. This is typically called an articulating block mat (ABM). As flexibility and permeability are important functions for stream instability and bridge scour countermeasures, systems that incorporate filter points or weep holes (allowing for pressure relief across the mat) combined with small-diameter ducts (to allow breakage and articulation between the blocks) are preferred.

Grout-filled mat systems can range from very smooth, uniform surface conditions that approach cast-in-place concrete in terms of surface roughness, to extremely irregular surfaces exhibiting the roughness of moderate size rock riprap. Comprehensive technical information on mat types and configurations are available from manufacturers due to the specialised nature of this type of revetments. Mats are typically available in standard nominal thicknesses of 100, 150 and 200 mm and are occasionally produced up to 300 mm thick.

There has been limited field use of these systems as a scour countermeasure for bridge piers. They have typically been used for shoreline protection, protective covers for underwater pipelines, bridge abutment spill slopes, and channel armouring where the mat is placed across the entire channel width and keyed into bridge abutments or stream banks. This type of scour protection, however, is not recommended for large river where the river bank is very high.

The benefits of grout-filled mats are that the fabric installation can be completed quickly, without the need for dewatering. Because of the flexibility of the fabric prior to filling, laying out the forms and pumping those with concrete grout can be performed in areas where room for construction equipment is limited.

13.7.16.12.8 GUIDE BANKS

Guide banks are an effective means of decreasing the risk from scour at bridge abutments. This is achieved by moving the contraction of the streamlines and the generated vortices away from the abutment to the upstream end of the guide bank. Guide banks also protect approach embankments by reducing the flow along the face of the embankment, minimising scour.

Three principal considerations are involved in proportioning a guide bank:

i. geometry
ii. height
iii. length.

Geometry – Karaki (1960) found that a guide bank in the form of a quarter ellipse, with ratio of major (length) to minor (offset) axes of 2.5:1 performed as well or better than any other shape tested. The equation for this ellipse (see **Figure 13.114**) is (**Equation (13.161)**):

$$\frac{X^2}{L_s^2} + \frac{Y^2}{(0.4L_s)^2} = 1 \tag{13.161}$$

Where,

X = length of major axis, m

Y = length of minor axis, m

L_s = length of guide bank, m

Height – is based on the anticipated high water level. The guide bank should have sufficient height and freeboard to avoid overtopping and be protected from wave action.

Length – is estimated using the method recommended in Bradley (1978) in which the length of guide bank, L_s, is determined from the discharge ratio Q_f/Q_{30}, relating the flow over the left or right floodplain to a specific portion of the flow under the bridge, a representative velocity adjacent to the abutment of the bridge, and the length of the guide bank needed. L_s is determined from **Figure 13.115**.

Definition of the symbols used are:

- Q = total stream discharge (m^3/s)
- Q_f = lateral or floodplain flow (one side) measured at the upstream reach of the structure (m^3/s)
- Q_{30} = Q/b x 30 = discharge (m^3/s) in 30 m of stream adjacent to abutment, measured at section 1 in **Figure 13.79**
- b = length (m) of bridge opening
- A_{n2} = water area (m2) under bridge referred to normal stage
- $V_{n2} = Q/A_{n2}$ = average velocity (m/s) through bridge opening
- Q_f/Q_{30} = guide bank discharge ratio
- L_s = top length (m) of guide bank (measured as shown on **Figure 13.115**).

The length of the guide bank should be increased with an increase in floodplain discharge (as observed in **Figure 13.115**), with an increase in velocity under the bridge, or both. In order read the chart, the ordinate with the proper value of Q_f/Q_{30} should be entered, moving horizontally the appropriate V_{n2} curve, then downward from the abscissa to obtain the required bank length. If the length read is less than 10 m, a guide bank is generally not required. A guide bank not less than 30 m be constructed for chart lengths between 10 m and 30 m. This length is required to direct curvilinear flow around the end of the guide bank to merge with the main channel flow and establish a straight course down the river before reaching the bridge abutment.

This type of flow can have several times the scour capacity than that of parallel flow, depending on factors such as the radius of curvature, velocity, depth and others. Holding all factors constant, the depth of scour will increase with a corresponding decrease in radius of curvature. Therefore, the deepest scour produced by a guide bank will occur near the nose where the radius of curvature is least.

Rock protection should be provided for the guide bank and should be extended out from the toe of the bank on the bed such that it will fall into the scour hole as it forms, preventing undermining of the guide bank.

Figure 13.114 shows this and further details for the construction of guide banks.

The selection of Class of rock protection required for impinging and parallel flow is based on the following assumption:

The velocity ratios are, V_p: V_m: V_i = 2:3:4

Where,

V_p = velocity of parallel flow along tangent bank

V_m = mean velocity through bridge opening

V_i = velocity of impinging flow against curved bank.

The classes of the rock riprap required can be obtained from **Table 13.59**. The rock protection for parallel and impinging flow should be distributed along the guide bank as shown on **Figure 13.114**. The level to which the toe of the rock is to be carried will be dependent upon the anticipated depth of scour. The grading of the various Class of rock should be in accordance with **Table 13.59**. Where necessary a filter should be placed between the embankment fill and the rock slope protection.

Example – given that the face slope of the bridge abutment is 1.5:1, the specific gravity of the rock is approximately 2.65 and the mean velocity of flow through the bridge for the design flow is 3.5 m/s:

V_p = 2/3 x 3.5 = 2.33 m/s

V_i = 4/3 x 3.5 = 4.67 m/s.

From **Table 13.59** the rock protection required for parallel flow is Facing Class with a thickness of 0.5 m and for impinging flow is 1 tonne Class with a thickness of 1.6 m.

13.7.16.12.9 SPURS

Spurs, retards or groynes are structures or embankments projecting into a stream from the bank at some angle to deflect flowing water away from critical zones, to prevent erosion of the bank, and establish a more desirable channel alignment or width. By deflecting the current from the bank, a spur or a series of spurs may protect the stream bank more effectively and at less cost than rock protecting the bank. Also, by moving the location of any scour away from the bank, failure of the rock protection on the spur can often be repaired before damage is done to structures along and across the river.

Spurs are also used to protect road embankments that form the approaches to a bridge crossing. Often these embankments cut off the overbank flood flows causing these flows to run parallel to the embankment en route to the bridge opening. Spurs constructed perpendicular to the embankment keep the potentially erosive current away to the direction of flow, as shown on **Figure 13.116**. Therefore, spurs of equal length need not be spaced closer than three times their projected length. For a group of four or more, the spacing may be up to four times their projected length.

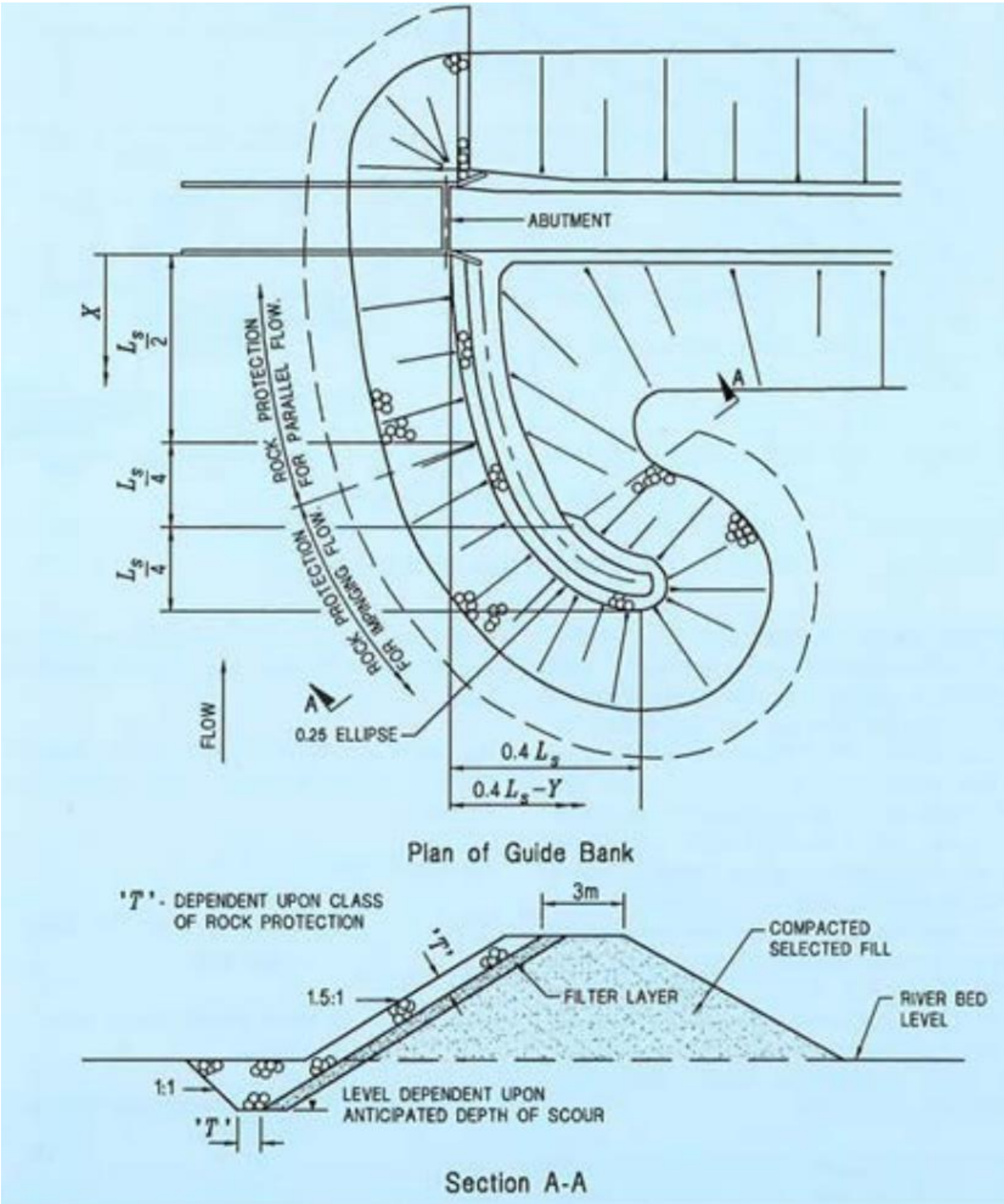


Figure 13.114 Guide bank details

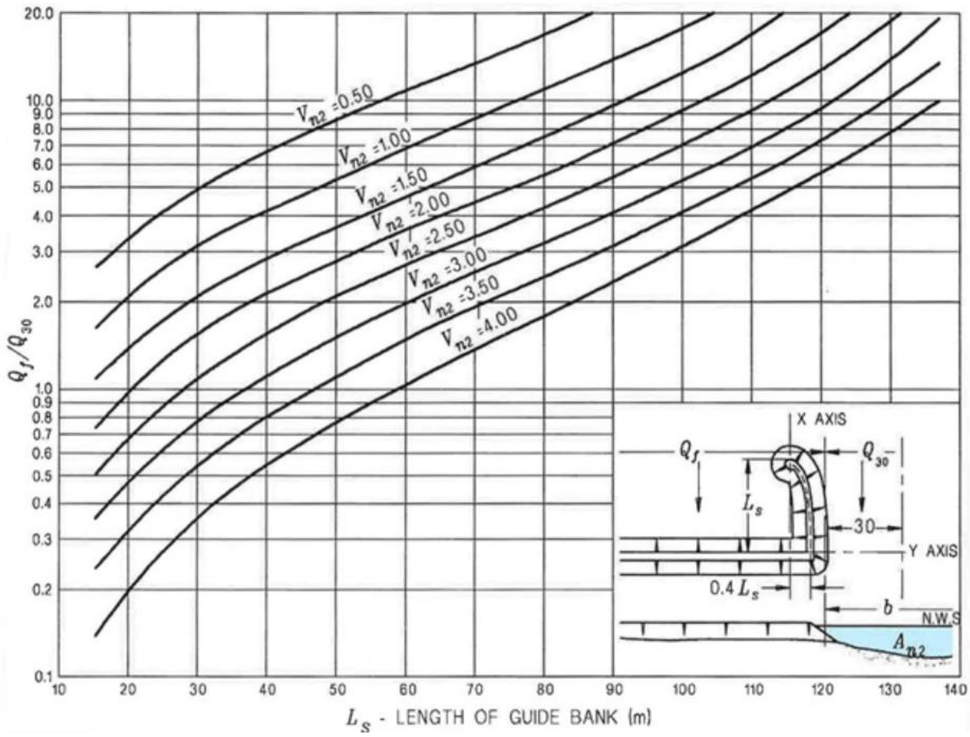


Figure 13.115 Chart for determining length of guide banks

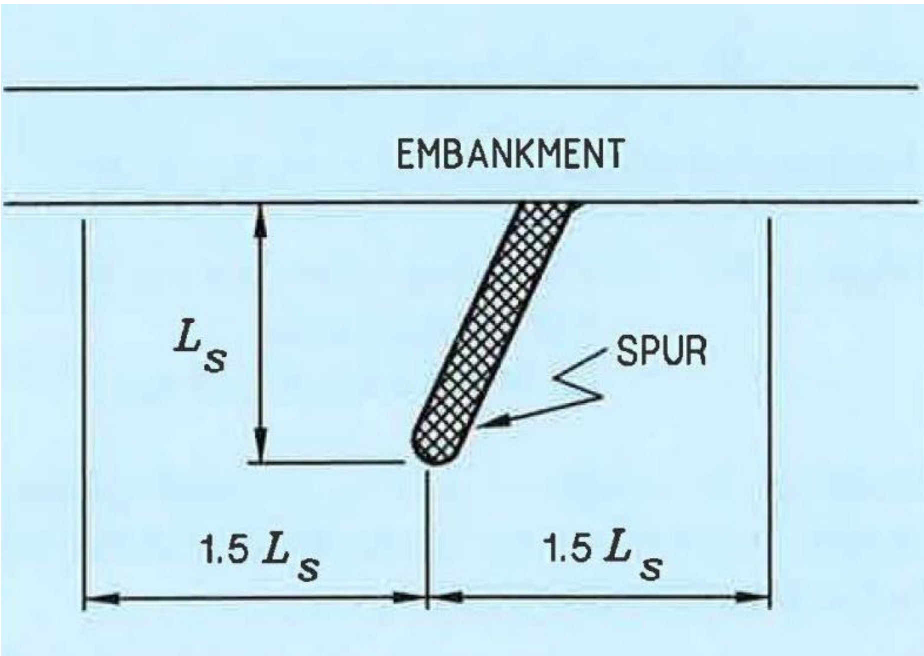


Figure 13.116 Approximate length of embankment protected by spurs

13.8 LOW-LEVEL RIVER CROSSINGS

A Low-Level River Crossing (LLRC) is a submersible road structure, designed in such a way as to experience no or limited damage when overtopped. This type of structure is appropriate when the inundation of a road for short periods is acceptable for the road user. LLRC are also referred to as **floodway crossing**.

LLRCs are either **drifts** or **causeways** which are defined as follows:

i. **Drift** - A drift is defined as a specially prepared surface for vehicles to drive over when crossing a river. A drift does not contain any openings underneath the surface for allowing passing water through. The surface layer may consist of gravel, concrete, grouted stone or commercial products such as Armorflex (concrete blocks held together longitudinally with polyester, galvanised steel or stainless steel cables) and Hyson Cells. These are mats comprising square, hollow geocells - fabricated from thin plastic film – and filled in-situ with grout to form a layer of interlocking concrete blocks. Drifts are also referred to in the literature as **fords**.

ii. **Causeway** - A vented causeway (referred to as a causeway) in essence also consists of a suitable surface layer over which vehicles may drive but contains openings underneath allowing water to pass through the structure. These openings may be of circular or rectangular shape and could be formed by means of pre-cast pipes or portal culverts, corrugated iron void formers, short span decks (less than 2 m), etc. Vented causeways are also referred to in the literature as **vented fords**.

The design of LLRC involves similar hydrological analysis and hydraulic designs to culverts and bridges. The capacity of a structure is determined as the sum of the discharge that could be accommodated over the structure within acceptable depth (see **Section 13.6.14.7**), and the discharge to be accommodated underneath the structure (see **Section 13.6.13**). The HY-8 software can be used to determine the discharge over and underneath the crossing structure.

The geometric dimensions of a drift depend on the volume of water expected to cross the road and the geometry of the intended crossing place. Typical arrangements for drifts and vented fords are shown in **Figure 13.117** and **Figure 13.118** respectively.

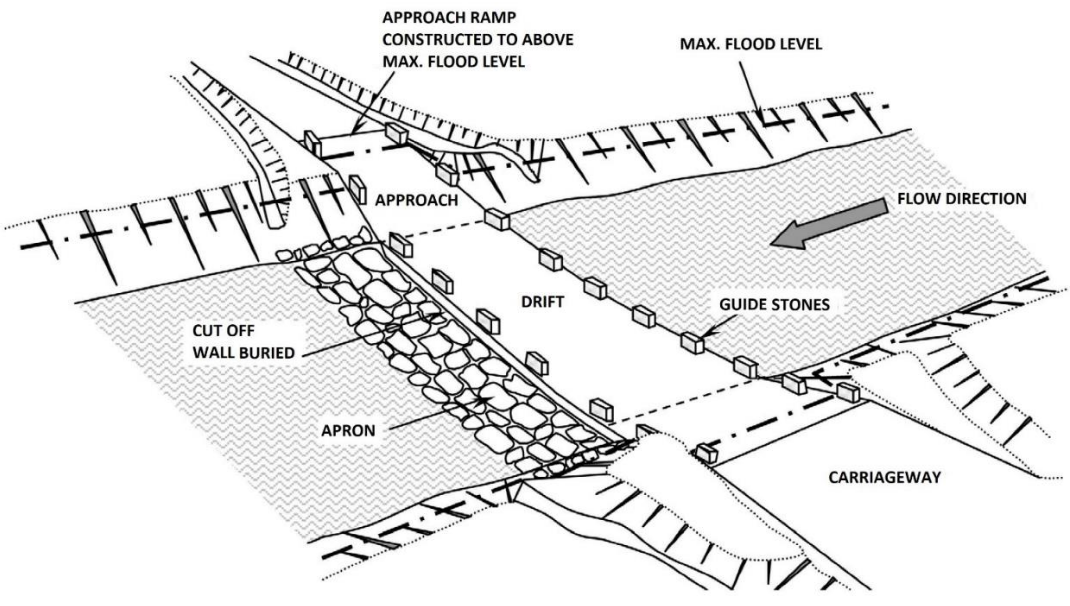


Figure 13.117 Layout of a drift

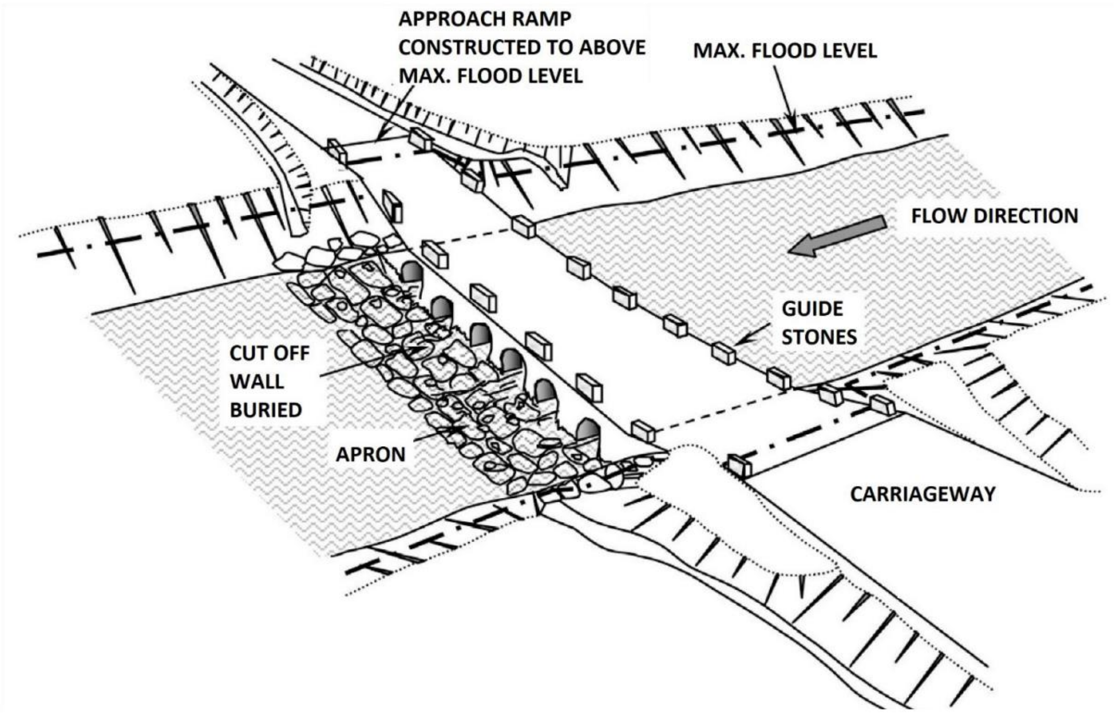


Figure 13.118 Layout of a vented ford

13.8.1 DRIFTS

The basic considerations for use of drifts are:

i. Stream drifts are structures which provide a firm place to cross a river or stream. Relief drifts transfer water across a road without erosion of the road surface. Water flows permanently or intermittently over a drift, so in times of flow vehicles are required to drive through the water, or in the case of very high flows, to wait until it is safe to cross.
ii. Drifts are particularly useful in areas that are normally dry with occasional heavy rain causing short periods of flood water flow.
iii. Drifts provide a cost-effective method for crossing wide rivers which are dry for the majority of the year or have very slow or low permanent flows.
iv. Drifts are particularly suited to areas where material is difficult to excavate, thus making culverts difficult to construct.
v. Drifts are suited to use in flat areas where culverts cannot be buried because of a lack of gradient.
vi. Drift efficiency is directly related to the drift invert slope and cross-sectional area.
vii. Drifts have low maintenance requirements.
viii. The drift approaches must extend above the maximum design flood level flow in order to prevent erosion of the road material.
ix. If necessary, guide stones or posts should be provided on the downstream side of the drift and be visible above the water when it is safe for vehicles to cross the drift.
x. Buried cut off walls are required upstream and downstream of the drift to prevent undercutting by water flow or seepage.
xi. Approach ramps should be surfaced with a non-erodible material and provided to the drift in the bottom of the watercourse with a maximum gradient of 10%. The designed gradient will take account of the anticipated mix of traffic but should be at least 5%.
xii. Drifts should not be located near or at a bend in the river.
xiii. Some form of protection is usually required downstream of a drift to prevent erosion.
xiv. Appropriate pavement design is needed to resist the adverse effects of submergence and

high flow velocity.

xv. Drifts force vehicles to slow down resulting in longer journey times.

xvi. Drifts are likely to cause less erosion from their discharge than culverts.

In general, drifts should be used where the following conditions apply:

i. The difference in elevation between the invert of the side drain and/or natural watercourse and the roadway shoulder break point is less than 300 mm. Where the water level is estimated to exceed 200 mm, the approaches must be lengthened to accommodate the high water level.

ii. The subgrade material is rocky and difficult to excavate.

iii. There is evidence that the natural soils of the side drain and/or watercourse comprise mainly silt and could lead to the rapid blocking of a culvert.

iv. Where discharge, if concentrated, may lead to erosion of agricultural land.

v. Where the cost of a culvert of similar capacity is significantly higher than the cost of a drift.

13.8.2 VENTED FORDS AND CAUSEWAY

The key features of vented fords and causeways are:

i. These structures are designed to allow the normal dry weather flow of the river to pass through pipes below the road. Occasional larger floods will also flow over the road, which may make the road impassable for short periods of time.

ii. Vented causeways are the same concept as vented fords but are longer with more openings. They are used to cross wider watercourse beds.

iii. The level of the road on the vented ford should be high enough to prevent overtopping except at times of peak flows. There should be sufficient pipes to accommodate standard flows. The location of pipes will depend on the flow characteristics of the river.

iv. Vented fords should be built across the whole width of the water-course.

v. A vented ford requires approach ramps, which must be surfaced with a non-erodible material and extend above the maximum flood level.

vi. As with drifts, the approach ramps should have a gradient of between 5% and 10%, depending on the mix of vehicles using the road.

vii. Watercourse bank protection is required to prevent erosion around the structure.

viii. The upstream and downstream faces of a vented ford require buried cut-off walls (preferably down to rock) to prevent water undercutting or seeping under the structure.

ix. An apron downstream of the pipes and area of overtopping is required to prevent scour by the water that flows out of the culvert pipes or over the structure.

x. The watercourse downstream from the structure must be protected from erosion. In flood conditions there could be considerable turbulence immediately downstream of the structure.

xi. The road surface longitudinal alignment of the vented ford should be a slight sag curve to ensure that, at the start and end of overtopping, water flows across the centre of the structure.

xii. Construction materials for vented fords can be, riprap, gabions, and reinforced concrete.

xiii. There should be guide stones or posts on each side of the structure to mark the edge of the carriageway and indicate when the water is too deep for vehicles to cross safely. Guide stones or posts should be painted with reflective paints to make it visible both day and night for road users and should be 0.6 m high.

xiv. At locations where there is a high risk of floating timber other debris causing damage to or blockage of a vented ford, then consideration should be given to including upstream guide slides that help direct such debris safely up and over the main structure.

xv. Appropriate pavement design is needed to resist the adverse effects of submergence and high flow velocity.

13.8.3 THE APPLICATION OF LLRCS

13.8.3.1 BASIC CHARACTERISTICS OF LLRCS

Two distinctive characteristics of LLRCs are that the:

i. structure will be inundated from time to time and hence not available for use. It is necessary for the users of these structures to be aware of this limitation, and to accept and respect it; and
ii. that the cost of construction is generally considerably lower than that of a conventional bridge.

LLRCs are appropriate under the following circumstances:

i. when the inundation of the structure, and the associated disruption in traffic flow, is acceptable for short periods of time;
ii. where alternative routes that can be used during flooding are available;
iii. where traffic volumes are low, typically on the tertiary road network;
iv. where high-level bridges are not economically justified; and
v. where funding available for construction is limited.

The use of LLRCs might be influenced by:

i. their non-availability for use from time to time;
ii. the risk associated for road users who might use them during periods of inundation, and be washed away; and
iii. the required maintenance after floods to repair and remove debris.

13.8.3.2 ROAD NETWORK ASPECTS TO BE REVIEWED WHEN LLRCS ARE CONSIDERED

The road network aspects to be considered when LLRCs are evaluated, are:

i. If a particular community or land use has one access road only and the access road crosses a river without a structure, the decision whether to construct a LLRC or a high-level bridge depends on the acceptability of short periods of inaccessibility, the construction cost and the economic justification of the options. Under low traffic volume conditions, say less than 500 vehicles per day, the return on investment of a LLRC is generally better than that of a high-level bridge. The reason is the considerably lower construction cost, while disruption in traffic is generally of a limited nature.
ii. If an alternative access route to a community or land use is available during periods of flooding, the impact of using the alternative route in terms of travel cost and travel time should be assessed in relation to the possible savings in construction cost associated with a LLRC.

With larger rivers, attention should be paid to the total road network in the area, the number and locations of river-crossing structures, as well as the levels of these structures in terms of design return periods. Rather than designing all river-crossing structures for the same return period, variation in the return periods used for design could be considered. In this way the number of accessible structures during flooding will be reduced, whilst alternatives will remain available. In contrast with the first option, a situation may occur where all the structures under consideration are overtopped at the same time. Generally, one would prefer structures on the primary road network not to be inundated during floods, while inundation of structures on the tertiary network is acceptable.

13.8.3.3 SIGHT DISTANCE

It is important that adequate approach sight distance be provided to allow drivers time to recognise water over the road and to stop on a wet road. It is important that drivers can see the presence of water on the road and the sight distance should be checked to ensure that stopping distance is achieved to the height of the water surface at a depth of 150mm. For short floodway crossings, the driver should be able to see a length of water equivalent to at least 1sec of travel at the design speed. This may determine the minimum length permissible.

In addition, a driver must be able to see any washout in the floodway crossing after the water has subsided and be able to stop before reaching it. An object height of zero should be used in determining the appropriate stopping sight distance to account for this.

Headlight sight distance should also be achieved to an object height of zero to ensure that the driver is alerted to the presence of water on the road, and to detect washouts.

13.8.4 CULVERT DIMENSIONS AND DESIGN CONSIDERATIONS FOR CULVERTS UNDER THE LLRC

The proposed drainage structures at each site will need to be designed to account for the hydraulic jump of water passing over the crossing or risk undermining from the downstream side.

i. The culverts should be aligned parallel with the waterway channel, not perpendicular to the road.
ii. Multiple banks of culverts are recommended at each site to:
   - Maintain the transport of sediment.
   - Account for the likely frequent (during high flows) reconfiguration of the channel bed and movement of the low flow channel.
   - Reduce the likelihood of blockage.
iii. Generally, box culverts should be constructed in lieu of pipe culverts to better support the transportation of debris and sediment through the structure. This requires them to be placed at a depth and with provisions to ensure scour does not undermine them.
iv. Appropriate bed and bank stabilisation work should be incorporated into the design to minimise damage and/or failure during high flow events associated with overtopping and outflanking. Rock riprap is generally considered the most effective method of stabilisation. Wire mattresses and gabions could be considered as suitable alternatives depending on site circumstances.
v. Riprap involves the controlled placement of quarried, angular rock against a stream bed and/or bank to prevent further erosion of the bank in the medium term. Formally designed rock protection involves undertaking hydraulic calculations to determine an appropriate rock size to ensure that the rock will not be washed away. The rock is also graded and placed to a design thickness to ensure that it forms an interlocking mass.
vi. Where high energy hydraulic conditions are expected, rock size is likely to be very large and potentially not feasible for some events. In this case the maximum feasible rock size should be used, and maintenance will be required.
vii. General design requirements for rock riprap work are summarised as follows:
   - Rock used for the works shall be hard, durable, well graded, and angular in shape, free from cracks, overburden, shale and organic matter.
   - Each case will have to be designed for the specific circumstance applying to that site and the
   - appropriate size of rock can be determined. Commonly, the D_{50} of the rock will need to be 350 - 450mm. (By definition 50% of the mass shall consist of stones with an equivalent spherical diameter equal to or larger than this dimension).

- Rock protection shall have a minimum thickness of twice the median diameter (D_{50}) of the rock material.
- The finished rock protection shall be fully keyed into the bed and banks of the river and not left proud of the riverbed and banks.
- The rock should be placed on the bank rather than tipped down the batter (batters not steeper than 1 on 3).
- Some settlement of the rock may require the placement of additional material on the slope.

viii. Where floodway crossing length permits, floodway crossings should have at least two sets (banks) of culverts at opposite ends of the floodway crossing to allow for the natural changes in the bed alignment as it meanders over time (see **Figure 13.119**).

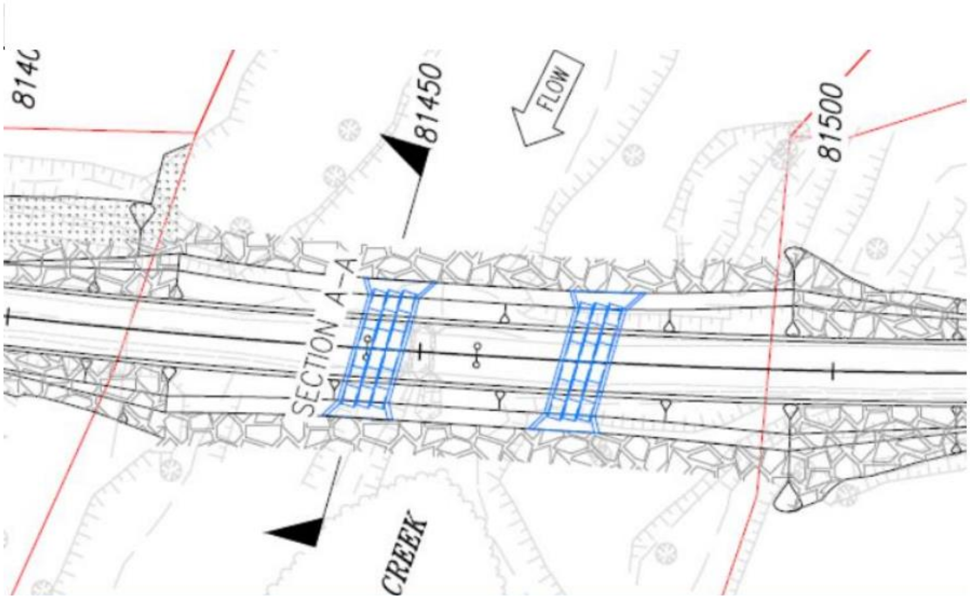


Figure 13.119 Floodway crossings with two sets of culverts both in line with the flow

13.8.5 FLOOD DAMAGE

Designers must be aware of the potential types of failures that can result from floods and adopt the most appropriate protection for the situation under consideration (refer to **Section 13.8.7**). These examples of extreme flood damage show that failures can be caused by either or both upstream and downstream protection failures (See **Plate 13.21**).

Plate 13.21 Flood damage examples

The LLRC apron on the downstream side of the LLRC sends supercritical flows away from the structure into the downstream channel where a hydraulic jump develops as the flows return to a sub-critical condition. The turbulence in the hydraulic jump will scour the natural bed and develop a scour hole (See **Plate 13.22**).

To counter the actions of scouring under the apron, the Type 1 – Floodway Crossing Protection (**refer to Section 13.8.7.1**) is designed with a vertical shear key wall (upstand or cut-off wall) of at least 0.5m. The length of the apron should be sufficient to avoid this type of scouring.

Type 2 protection (**refer to Section 13.8.7.2**) relies on riprap to counter the erosion and this must be sufficiently long to avoid the effects of this scouring. The purpose of the riprap is to move the scour hole location away from the LLRC structure and prevent undermining over time. The rock riprap requirements for the culvert outlet aprons include 6.0m of riprap in the downstream direction. A scour hole that develops beyond 6.0m from the concrete apron appears to be acceptable. Riprap can be thickened at the end to reduce the chance of rock movement, but over a long time and following large events some rock will require “topping up” with more similar rock. This is likely at the edges of the rock mass.

Plate 13.22 Scour beyond downstream apron

13.8.6 LOW-LEVEL RIVER CROSSING PROFILES

Selection of the type of LLRC and associated protection treatment against scour is governed by:

i. whether flow across the crossing is free or submerged, and
ii. under free flow conditions, whether plunging or surface flow occurs downstream from the crossing.

The tailwater height when the flood is at the point of overtopping the road usually controls the degree of protection required for a floodway crossing. Therefore, the cost of providing adequate bridge and/or culvert waterways to raise the tailwater high enough to require minimum protection becomes a prime consideration as well as the cost of the protection itself.

Overtopping flows of long duration and at frequent intervals may cause pavement failures and softening of the embankment, thus aggravating any tendency to scour. Even with a high tailwater when the flood is at the point of overtopping the road, the Times of Submergence may indicate more elaborate protection than natural grass.

The density of the natural vegetation, fertility of the soils and climatic variation are also important considerations on the ability of vegetation to effectively protect the floodway crossing embankment when determining protection requirements. The stability of the stream bed must also be considered, with some sites requiring deep cut off walls to minimise the effect of scoring on even a well-designed crossing. Mobile stream beds complicate floodway crossing design sometimes requiring a low- height bridge rather than culverts to maintain trafficability as the stream bed rises and falls depending on the flows in the stream.

For low standard floodway crossings in flat country, the bed heights surveyed downstream should extend sufficiently to allow design checks to be made regarding the possibility of ponding at the site due to unforeseen natural deposits, farm dams and access roads downstream.

13.8.6.1 TYPES OF PROFILES

There are essentially five types of LLRC (floodway crossing) that cater for a range of calculated volume and velocity situations:

i. Profile 1 – Full protection.
ii. Profile 2 – Significant protection.
iii. Profile 3 – Downstream protection only.
iv. Profile 4 – Minimal protection.
v. Profile 5 – Bed level floodway crossing – no culverts.

These floodway crossing types have performed satisfactorily with their associated limitations noted. The crossing type to be adopted will be determined after the assessment of the following site characteristics:

i. hydrology and hydraulic requirements,
ii. geotechnical considerations,
iii. environmental considerations, and
iv. site characteristics.

Experience has shown that most failures of floodway crossings with downstream batter protection commences by scouring at the downstream aprons and/or the downstream edge of the road formation, and therefore a reduction in standard in these areas should not be considered in order to reduce costs.

All floodway crossings design should consider the following:

i. Although not clearly defined, it appears that protection of the upstream batters may only be required in floodway crossings of low flood immunity in major streams. As a precaution where only downstream protection is adopted, protection for a distance of about 6.0 m on each side of major culverts on the upstream side may be placed to offset possible scour due to turbulence from the mixing of longitudinal and direct flows at the culvert inlets.
ii. Provision of culverts to ensure the tailwater is usually not more than 600 mm below the downstream edge of the road formation when overtopping first occurs (actual range 300–700 mm). It is important to note that a high afflux may not be acceptable in some areas and a high afflux may require non-standard additional outlet protection at culverts to control erosion. This does not apply in the case of Profile 5 – Bed level floodway crossings.
iii. Full protection of the top surface of the road formation, as for floodway crossings with grassed batters.
iv. Protection of at least the downstream batter.
v. Provision of adequate downstream aprons. For height of road embankment (H) equal to or more than 2.0 m, the downstream apron should extend at least 1.5H metres away from the toe of the embankment. For H less than 2.0 m, the downstream apron should extend at least H away from the toe of the embankment unless otherwise specified.
vi. Hydrostatic pressure on any downstream protection using a concrete slab (refer to Section Introduction)

A brief description of successful types of floodway crossing protection follows, and sectional details of these types are shown in Sections Types of Profiles. These types are not in any order of preference and operational and cost comparisons should be made where more than one suitable protection solution exists.

However, the availably of local materials and the stability of the stream bed and site will have a large bearing on the type of protection selected.

Floodway Crossing protection types are addressed separately in Section Floodway Crossing Protection Examples.

A. Profile 1 – Floodway crossing (full protection, both upstream and downstream)

This type of floodway crossing requires full protection on both upstream and downstream batters where the calculated volume and velocity are extremely high having the potential to cause profoundly serious damage to the floodway crossing structure should the protection underperform. Up and downstream protection is essential in this scenario (**Figure 13.120**).

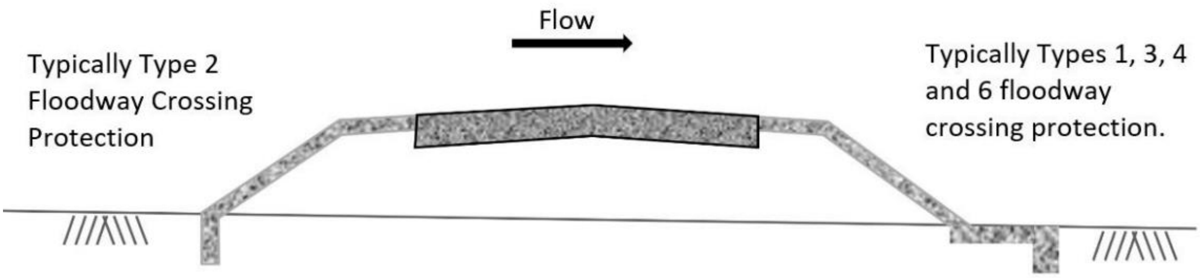


Figure 13.120 Full protection, both upstream and downstream

B. Profile 2 – Floodway crossing (significant protection, upstream shoulder and full downstream)

This type of floodway crossing has full downstream protection and only shoulder protection on the upstream side (**Figure 13.121**). This type has been extensively used and performed satisfactorily in remote areas where the natural ground has a high resistance to erosion and the tail water is quite high. If the natural surface does not have a high resistance to erosion, an appropriate apron will be required. This type of floodway crossing should not be used in lieu of Type 1 Floodway Crossing because of cost constraints.

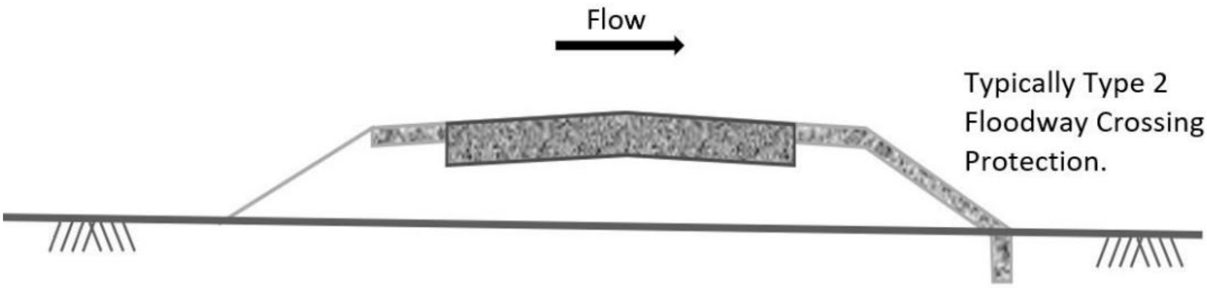


Figure 13.121 Significant protection, upstream shoulder and full downstream

C. Profile 3 – Floodway crossing (downstream protection)

This type of floodway crossing has full downstream protection with no upstream protection (**Figure 13.122**). It has been successfully used in situations where the floodwaters are of a lower volume and low velocity. If the natural surface does not have a high resistance to erosion, an appropriate apron will be required.

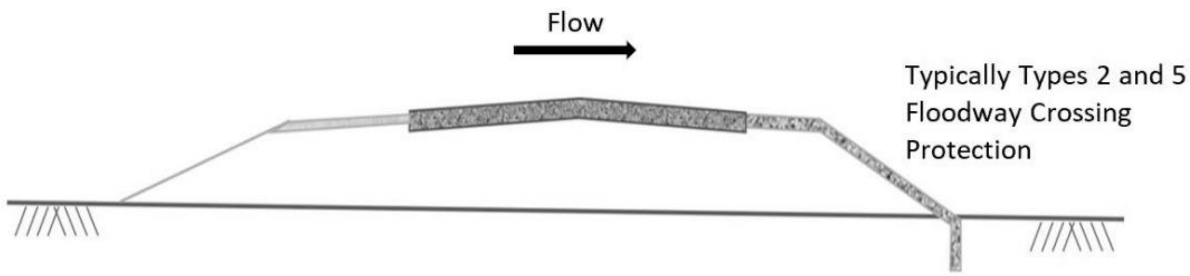


Figure 13.122 Downstream protection

D. Profile 4 – Floodway crossing (minimal protection)

This is not a typical type of floodway crossing but is useful when the water flowing towards the roads is of a sheet flow nature with little defined water channels (**Figure 13.123**). The low-profile design of the floodway crossing allows the overland water to flow over the road. Erosion of the downstream shoulder will generally occur unless treated to counter the erosion forces. Anti-ponding culverts should be placed at regular intervals along the road (these will require small levees to divert water to the culverts) thus reducing ponding on the upstream side of the formation.

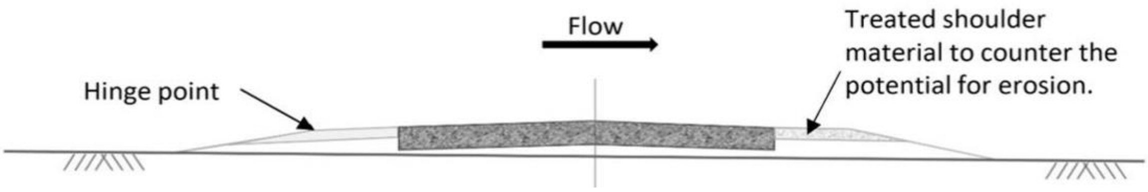


Figure 13.123 Minimal protection

E. Profile 5 – Bed level floodway crossings

Bed level floodway crossings are typically used in situations where flow is intermittent and usually for short periods (**Figure 13.124**). If using such a crossing results in excessive times of closure, this solution would not be acceptable.

A 1% one-way crossfall in the direction of flow is used to ensure that the crossing is free draining thus preventing ponding of water on the upstream side, which may result in the saturation of pavements and subgrade. The 1% crossfall is only applied when the floodway crossing is not on a curve (the preferred situation). If a crossing on a curve cannot be avoided, the curve superelevation (typically 3%), regardless of the flow direction, is retained. These floodway crossings should have concrete margins on the upstream and downstream ends to allow a chip seal to encapsulate and protect the pavement.

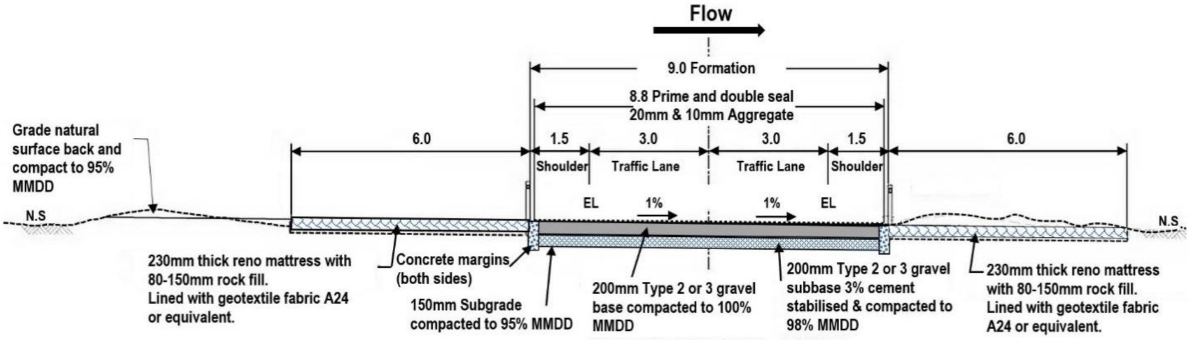


Figure 13.124 Bed level crossing

13.8.7 LOW-LEVEL RIVER CROSSING PROTECTION EXAMPLES

It is important to identify the types of potential damage that the floodway crossing under consideration is likely to sustain before deciding on the required protection details (refer to **Section 13.8.5**). This includes a complete assessment of the characteristics of the materials being used for the embankment and what protection they may need to remain stable and competent. For example, dispersive clays and silts should be avoided if possible, and if they must be used, then adequate protection will be essential.

The type of protection adopted can be reinforced concrete, gabions, dumped riprap, stone pitching, cement stabilised material, bituminous seal (refer diagrams shown for other types) or grass. The type of protection to be adopted will be based on the design requirements and the availability of materials. In some remote areas the availability of materials can be extremely limited, and cost will be an important factor in this assessment.

The structural design of floodway crossing protection must be undertaken for each situation. The concrete slab should be provided with sufficient drainage under the slab to prevent uplift forces damaging it. This may require weep holes to be provided in conjunction with an appropriate drainage layer under the slab, the details depending on the permeability of the underlying material and the drainage paths available from it.

Where weepholes are required, they would normally be about 90 mm diameter at 1.8 m (maximum) centres with 300 x 300 x 150mm no-fines concrete blocks behind the weepholes. The weepholes should be placed about 300 mm above the apron height or just above long-standing water height if higher. The more porous types of protection such as rock on filter cloths or layers and some cement stabilised gravels, depending on the grading, do not require weepholes.

13.8.7.1 TYPE 1 FLOODWAY CROSSING PROTECTION – RC MARGIN, BATTER AND APRON

This type of reinforced concrete floodway crossing protection has been constructed in many areas around the world. The reinforcement selected should not only satisfy strength requirements, but also prevent temperature and shrinkage cracks. Upstream and downstream protection are necessary for Type 1 Floodway Crossings (**Figure 13.125**).

With a suitable width of downstream apron and weepholes, the Type 1 floodway crossing protection is recommended as suitable for all crossings where other than grass protection is required, cost permitting.

In addition, the Type 2 floodway crossing protection is suitable for upstream protection in Profile 1 Floodway Crossings.

The slope (V on H) adopted should be assessed as a roadside hazard and treated accordingly. Depending on the outcome of that analysis, the use of flatter batters may be acceptable, or safety barriers may be required.

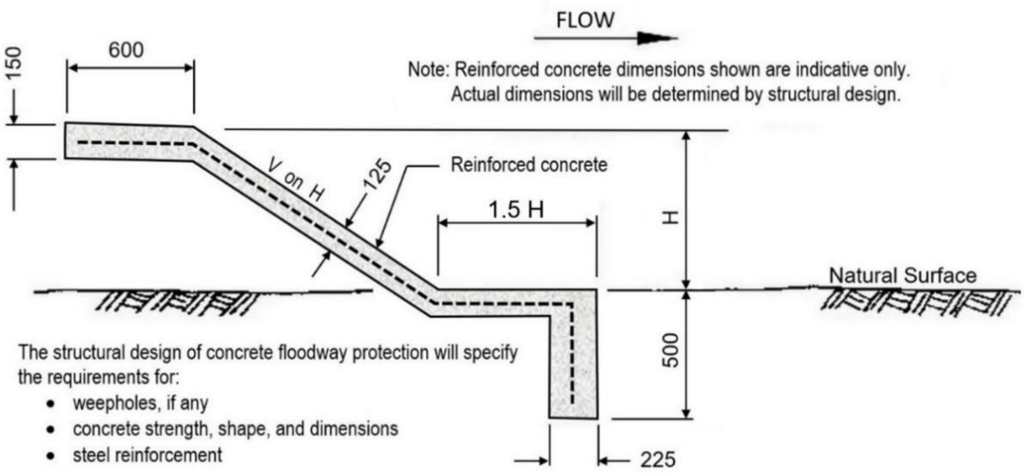


Figure 13.125 Type 1 floodway crossing protection – RC margin, batter and apron

13.8.7.2 TYPE 2 FLOODWAY CROSSING PROTECTION – RC MARGIN AND BATTER WITH NO APRON

This type of protection is like Type 1 without the downstream apron (**Figure 13.126**). It has been used successfully where the natural ground is not prone to scouring. In some situations, gabions have been added at the toe of the batter where scouring could be an issue due to local situations.

This type of protection is also appropriate for upstream protection for Profile 1 Floodway Crossings. See **Plate 13.23** damage starting from the upstream side of the floodway crossing approach embankment. The effect of the safety barrier may be a least partially responsible for losing the seal/pavement on what otherwise may have been a reasonable design.

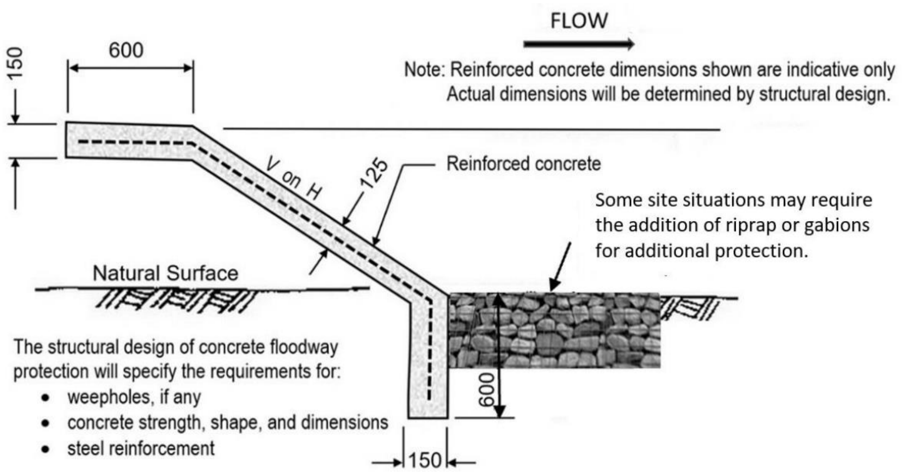


Figure 13.126 Type 2 floodway crossing protection – RC margin and batter with no apron

Plate 13.23 Example of floodway crossing damage starting from the upstream side of the floodway crossing

13.8.7.3 TYPE 3 FLOODWAY CROSSING PROTECTION – RC MARGIN AND BATTER WITH DISHED APRON

This is an example of a reinforced concrete floodway crossing (**Figure 13.127**) where the tailwater depth is uncertain but probably quite low (perhaps 700 mm or more below the downstream edge of the formation when the flood begins to overtop the road). This type performs well but the additional cost (as compared to Type 1) must be justified. Requires specialist design. Commentary is same as Section Type 1 Floodway Crossing Protection – RC Margin, Batter and Apron.

The slope (V on H) adopted should be assessed as a roadside hazard and treated accordingly. Depending on the outcome of that analysis, the use of flatter batters may be acceptable, or safety barriers may be required.

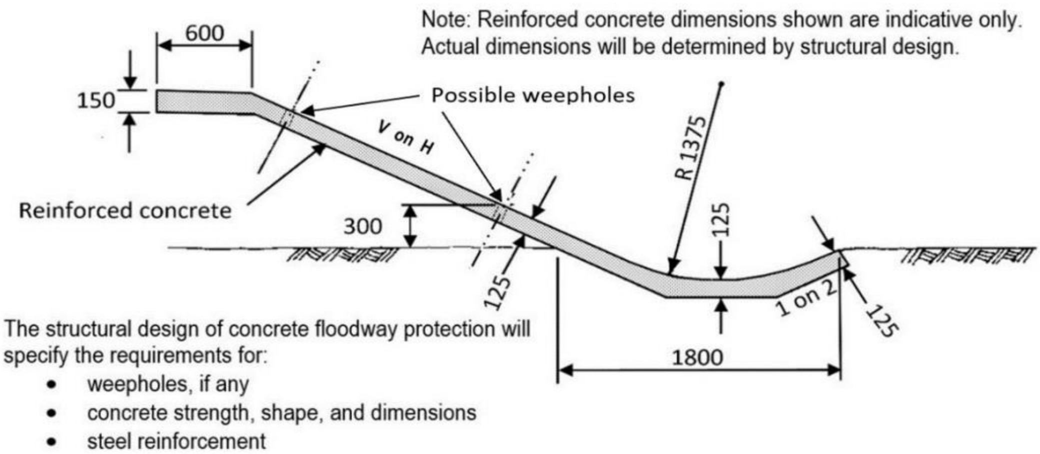


Figure 13.127 Type 3 floodway crossing protection – RC margin and batter with dished apron

13.8.7.4 TYPE 4 FLOODWAY CROSSING PROTECTION – STONE MATTRESSES AND GABIONS

The increased use of stone mattresses and gabions has given confidence in this type of construction. Rock fill with size 70–100 mm and median diameter of 85 mm is considered adequate for most uses. Type 4 floodway crossing protection is illustrated in **Figure 13.128**.

One of the important features of this type of treatment is that localised forces can be generated at the sharp change in slope between the road pavement and batter. A rounded shoulder (1.5 m radius) with a 1:3 slope should be adopted. Significant forces can act on upstands near the shoulder; therefore, any mattress or dumped riprap protection should not be allowed to project above the margin slab. Also applies to Type 6.

Mattresses must be pinned/anchored. Consider a cut off wall.

Cut off walls may not be necessary as mattresses usually achieve their optimum position with a little scour by dropping into a scour proof position.

The slope (V on H) adopted should be assessed as a roadside hazard and treated accordingly. Depending on the outcome of that analysis, the use of flatter batters may be acceptable, or safety barriers may be required.

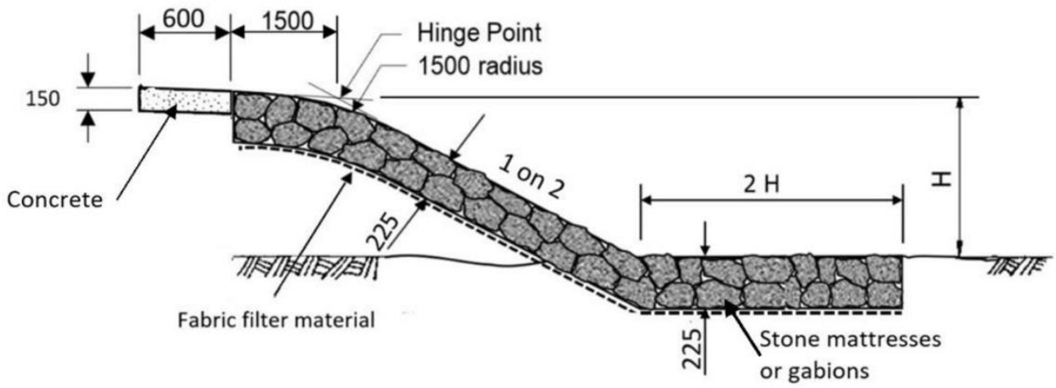


Figure 13.128 Type 4 floodway crossing protection – stone mattresses and gabions

13.8.7.5 TYPE 5 FLOODWAY CROSSING PROTECTION – BITUMINOUS SEAL

This type of protection incorporating a bituminous seal (**Figure 13.129**)is probably the lowest cost of the types shown, but its use is limited.

It should only be used where:

i. fill height is not higher than 900 mm,
ii. tailwater at overtopping is not more than 300 mm below the crown of the road, and
iii. time of submergence is small (hours).

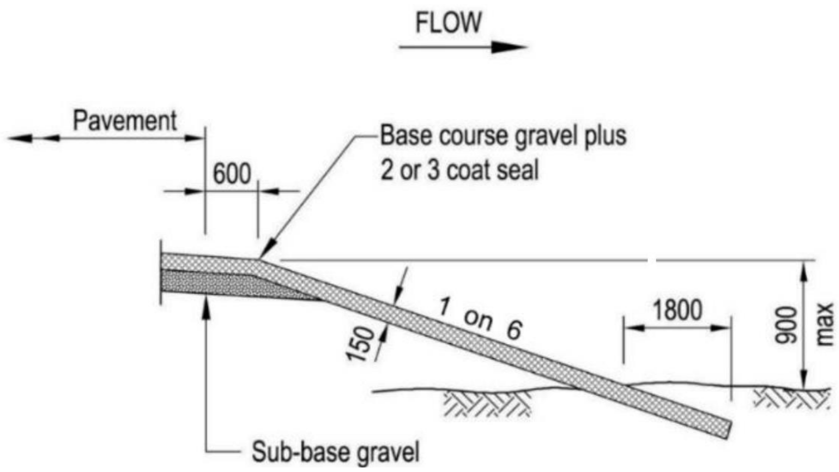


Figure 13.129 Type 5 floodway crossing protection – bituminous seal

13.8.7.6 TYPE 6 FLOODWAY CROSSING PROTECTION – DUMPED RIPRAP

Dumped riprap protection detail is shown in **Figure 13.130**.

The important features that should be considered in the design of this type of protection are:

i. Localised uplift forces can be generated at the sharp change in slope between the road pavement and batter. A rounded shoulder (1.5 m radius) with a 1:3 slope should be adopted, as this configuration does not display negative pressures and the net force on the bed is always downwards.
ii. Significant forces can act on upstands near the shoulder. Therefore, dumped riprap protection should not be allowed to project above the concrete slab or wall.
iii. Ingress of water at the upstream edge of the concrete wall may result in significant forces acting on the wall and material downstream of the wall. Leakage at the edge should be prevented and adhesion between the concrete and stabilised base course maximised.
iv. As the riprap consists of rock with grading requirements, it may have limited application in some jurisdictions where supplies of such rock are scarce in areas where floodway crossings are constructed.

Dumped riprap is also used for protection of open channels and for culvert outlet protection. The details shown in this section have been developed specifically for floodway crossings.

The slope (V on H) adopted should be assessed as a roadside hazard and treated accordingly. Depending on the outcome of that analysis, the use of flatter batters may be acceptable, or safety barriers may be required.

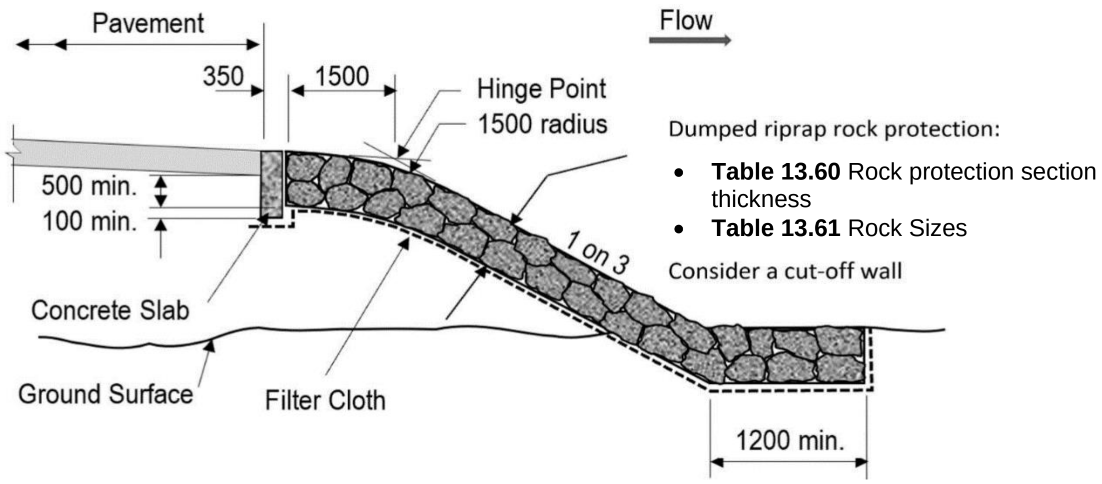


Figure 13.130 Type 6 floodway crossing protection – dumped riprap rock protection

Table 13.60 Rock protection section thickness

Velocity (m/s)	Class of rock protection (tonne)	Section thickness (m)
< 2	None	–
2.0–2.6	Facing	0.5
2.6–2.9	Light	0.75
2.9–3.9	¼	1.00
3.9–4.5	½	1.25
4.5–5.1	1.0	1.60
5.1–5.7	2.0	2.00
5.7–6.4	4.0	2.50
> 6.4	Special	–

Table 13.61 Rock Sizes

Rock class	Rock size (m)*	Approximate Rock Mass (kg)	Minimum percentage of rock larger than Rock Size in Second Column (%)
Facing	0.4	100	0
	0.3	35	50
	0.15	2.5	90
Light	0.55	250	0
	0.4	100	50
	0.2	10	90
¼ tonne	0.75	500	0
	0.55	250	50
	0.30	35	90
½ tonne	0.90	1000	0
	0.70	450	50
	0.40	100	90
1 tonne	1.15	2000	0
	0.90	1000	50
	0.55	500	90
	1.45	4000	0
	1.15	2000	50
	0.75	500	90
150 Rock Pitching	0.15 x 0.15 x 0.15		100
400 Rock Pitching	0.40 x 0.40 x 0.20		60

*Assuming a specific gravity of 2.65 and spherical shape for Facing, Light, Quarter Tonne, Half Tonne, One Tonne and Two Tonne.

13.8.7.7 GRASS BATTERS

Grass protection on floodway crossing batters is defined as turf or seeded grass and is used in all profile types except where full protection applies.

Grass protection of floodway crossing batters should not be considered in areas with low rainfall and subject to drought conditions.

There are some ‘engineered or selectively bred’ grasses that have very tight root systems which can resist erosion much better that most native and agricultural grasses. They are climate and soil type specific and are more effective at resisting erosion than most grasses.

Because the physical properties of grass such as species, stiffness, cover density and rooting pattern vary with soil type and climate, only general guidelines based on constructed floodway crossings are possible.

Floodway crossings with grassed batters should have the following features:

i. Bitumen seal or asphalt pavements with concrete or other rigid margins/shoulders (stone pitching, cement stabilised gravel, etc.) containing the bitumen in place.
ii. Alternatively, concrete blocks/nib walls along the top edges of the formation with a bitumen seal or asphalt pavement between them may be constructed. These containing blocks may be as simple as 10% by volume cement stabilised gravel strips 600 mm wide at the top by 180 mm deep. Concrete pavements instead of bituminous types will, of course, cover the full width of the formation.
iii. Culverts under the floodway crossing section to raise the tailwater to not more than 300 mm below the downstream edge of the road formation when overtopping first occurs.
iv. Overtopping occurs for a period of less than 12 hours in a 2% AEP flood. However, the type of material in the embankment and its saturated strength may require reduction of this allowable time of submergence.
Conversely there are some low floodway crossings which withstand submergence for much longer.

For this type of protection, it is desirable to have good grass cover when the overtopping flood occurs. This in turn requires an ability to maintain grass cover during the dry season and drought.

13.9 SUBSURFACE DRAINAGE

13.9.1 INTRODUCTION

Water in the structural layers of a pavement is the chief cause of road failures. This water causes the mechanical properties of the material to weaken, because excessively high pore pressures develop under traffic conditions. This weakening of mechanical properties and the washing out of underlying foundation materials is generally known as “pumping”.

The purpose of subsurface drainage is to control the moisture content of the pavement and the underlying material in order to assist in maintaining pavement strength and serviceability throughout the design life.

13.9.2 SOURCES OF MOISTURE

The main mechanisms by which moisture can enter a road subgrade and/or pavement are shown diagrammatically in **Figure 13.131** and include:

i. longitudinal seepage from higher ground, particularly in cuttings and in sag vertical curves
ii. rise and fall of water table level under a road
iii. rainfall infiltration through the road surfacing
iv. capillary moisture from the verges
v. capillary water from a water table
vi. vapour movements from a water table
vii. lateral movement of moisture from pavement materials comprising the road shoulder
viii. water flowing or standing in table drains, in catch drains, in median areas, within raised traffic
ix. islands or adjacent to the road (not illustrated)
x. leakage of water supply and drainage lines (not illustrated)
xi. passage of water through construction joints in pavements, and back and front of kerb and
xii. channel, between old and new pavements and behind bridge abutments (not illustrated).

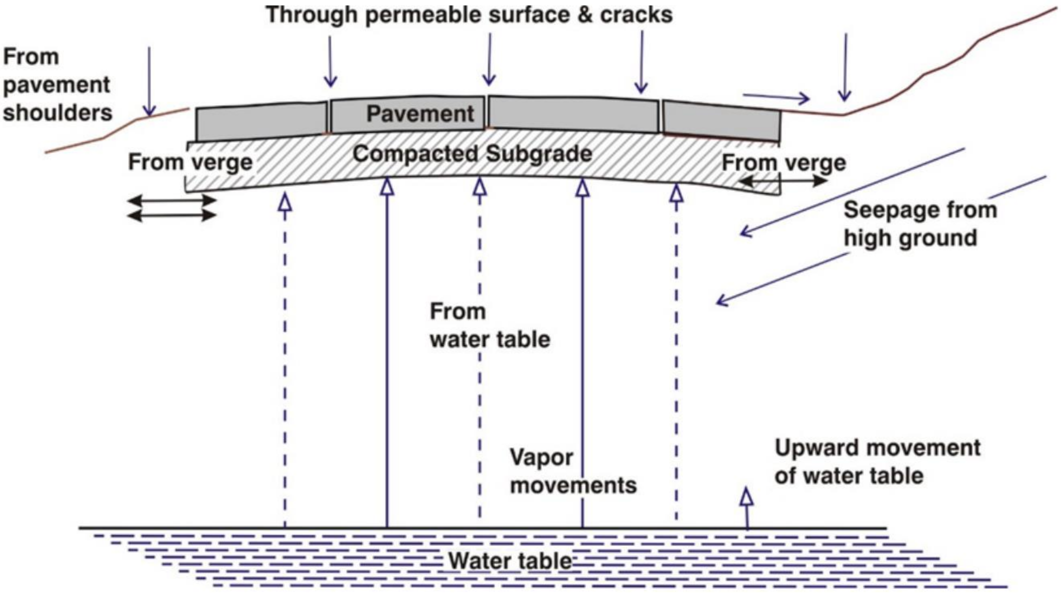


Figure 13.131 Sources of moisture (Adapted from ARRB (1987))

The design and installation of subsurface or subsoil drains beneath/adjacent to road pavements is essential where groundwater or seepage is known or considered to be present.

It is important to note that the construction of an underground stormwater drainage system with associated granular pipe bedding can result in the interception of seepage and the concentration of this intercepted water at drainage structures. The installation of subsoil drains should be considered in conjunction with the drainage pipes to allow seepage water to be collected and discharged into the drainage system.

Subsurface or subsoil drains are provided in order to avoid the following types of premature failures:

i. loss of subgrade strength and shape due to an increase in moisture content in moisture susceptible materials
ii. overload of the subgrade due to hydrostatic transmission of live load through a saturated pavement
iii. layer separation and potholing.

It is important to note that, in some flood plains and low-lying areas, a permanent, high-level water table may exist. Subsoil drains may be ineffective in such areas, particularly where it is difficult to provide an outlet. In some cases, such drains could act in reverse and provide a means of access for water to the pavement.

In these circumstances, the most effective measure which can be taken to control subgrade moisture conditions is to raise the subgrade above the surface of the ground. A height of 1.2 m above the water table is suggested (Earley 1979). This is usually impossible in urban street construction, in which case the pavement design should take into consideration the soaked conditions. In some situations, a cement or bituminous stabilised sub-base and/or base may be used.

13.9.3 CONTROL OF ROAD MOISTURE

The three basic techniques for controlling moisture are:

i. **Layer protection**: For example, seal coats, plastic sheeting and other impermeable barriers placed at various levels in the pavement structure. The durability of this type of moisture control is suspect, and pavement failure is likely if the barrier lets some moisture into it.

ii. **Rendering subgrade insensitive**: Lime or cement stabilisation reduces the moisture sensitivity of pavement layers. The advantage is that the load capacity of the stabilised material does not significantly decrease with increasing moisture content. The disadvantages are additional expense and a significant reduction in permeability (provided that cracks do not develop).

iii. **Subsurface drainage**: A correctly designed and maintained subsurface drainage system is considered the most effective way of ensuring a stable moisture condition.

This section is concerned only with the subsurface drainage.

13.9.4 TYPES OF SUBSURFACE DRAINS

Subsurface drainage systems are generally installed in a road either to remove water from the subgrade and pavement materials or to intercept water before it reaches the road structure.

Subsurface drains can be divided into two broad categories:

i. Pavement drains and
ii. Formation (cut-off or interceptor) drains.

Subsurface drainpipes may be surrounded by a single stage filter, or by two-stage filters. Filter materials can consist of aggregates (ranging in size from sand to cobble size), geotextiles or combinations of aggregates and geotextiles.

Filter materials whether of natural or man-made (geotextiles) need to be carefully designed for the following:

- permeability – to allow for free water movement
- piping – to limit the migration of small soil particles
- uniformity – to ensure an appropriate match to the in situ materials
- robustness – to allow for in service changes (e.g., partial blinding due to particle movement)
- constructability.

Examples are shown in **Figure 13.132**. The level of filtering will be determined by the prevailing soil types and any environmental requirements on the discharge. In some cases, a second-stage filtering may be required, and this can take the form of a geotextile wrap either around the pipe or around all the filter material.

A more recent form is the geocomposite drain, sometimes known as a fin drain, or the geocomposite edge drain. These are prefabricated with a polymer core wrapped in a geotextile. They can be installed in much narrower trenches than traditional pipe-based drains (see Pavement Drain Type 4 in **Figure 13.132**).

Where the polymer has the shape of an egg carton it may be easily crushed or buckled. Only the variety made of high-density polyethylene is acceptable for roadwork. A narrow trench is advantageous when placing the drains along existing pavements that previously had no subsurface drainage, or along the joint of a pavement widening. The disadvantages are that they:

i. have less hydraulic capacity than a subsoil drainage pipe and may not be suitable where there is visible seepage
ii. are difficult to clean if they become clogged
iii. require careful selection of the filter fabric to correspond to the existing permeability conditions.

Flexible pavements are particularly susceptible to absorption of moisture from failure of the waterproof surfacing, which can allow vehicle tyres to pump water into the pavement under high pressure. This further emphasises the importance of preventing ponding of water on the surface.

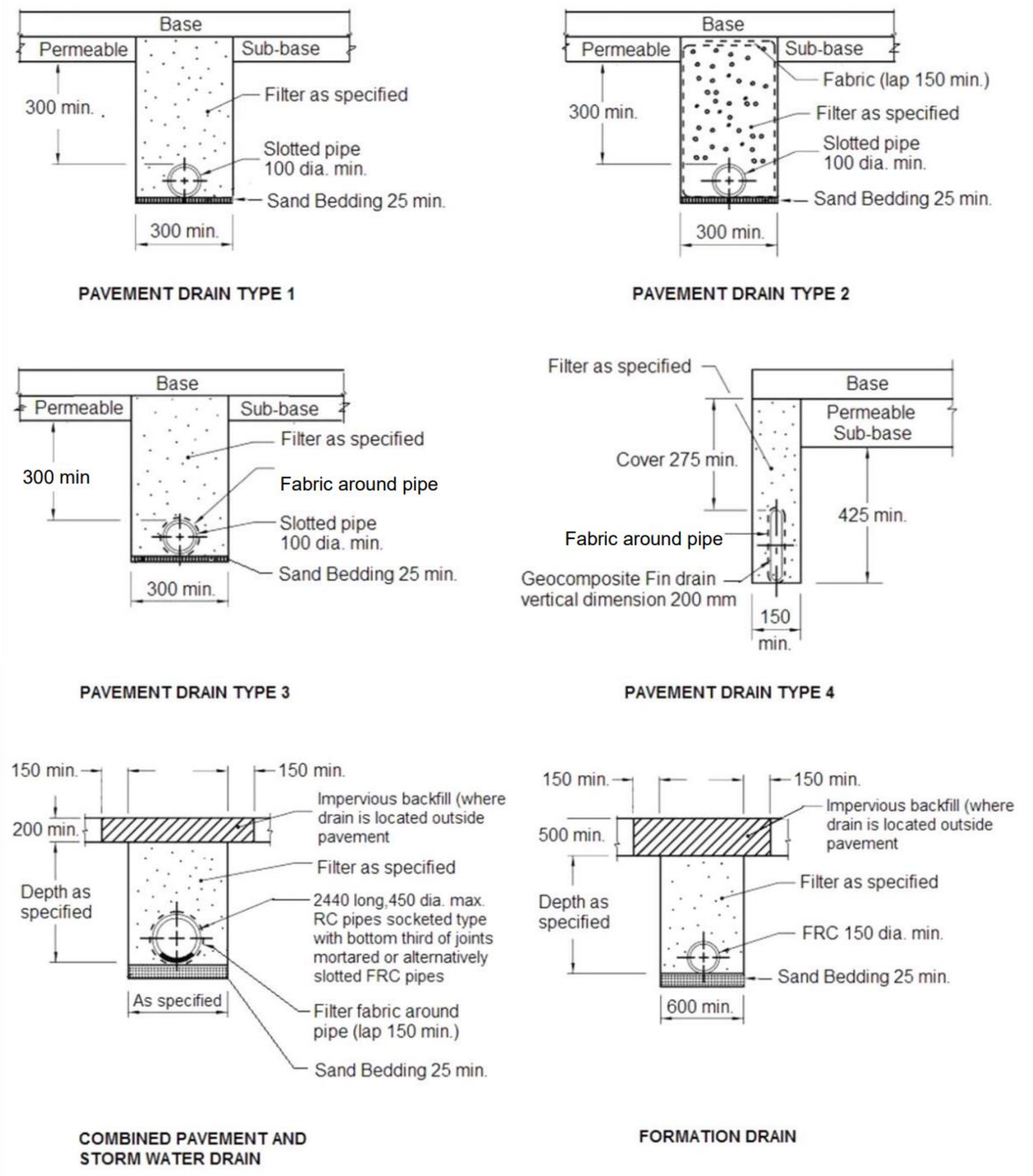


Figure 13.132 Subsurface drain types

13.9.5 PLANNING OF SUBSURFACE DRAINAGE

In the planning of an underground drainage system, the first step is to undertake a groundwater investigation at the site. There are two basic groundwater conditions for which drainage systems are essential:

i. **Groundwater with a hydraulic gradient smaller than the slope of the ground.** Typical warning signs are wet patches and visible outflows on the side of a cut. formation drains should be installed in such cases.

ii. **A groundwater table close to the surface**: Signs of this are collapsing wet spots in flat areas. This condition arises where water infiltrates from high areas or through a leaking surface layer. Pavement drainage should be installed in this case.

Observation should be carried out on:

i. Geology – fractured, fissured or jointed rock, impermeable dykes or alternating layers of permeable and impermeable material
ii. Vegetation – variations in colour and vigorous growth or hydrophilic vegetation
iii. Topography – shape of the land, depression, valley lines, catchments, etc.; and
iv. Road surface failures – pumping, rutting, tension cracks.

A design method should be chosen to calculate the capacity of the drainage system (Note: storm water may not be discharged into a subsurface system).

In the case of groundwater intercepted from cuts, it is generally not practical to carry out a sophisticated calculation, probably because too wide a variety of materials is found in a cut, and seasonal changes have a strong influence on the groundwater discharge. In practice, the capacity of an intercept drain may be determined by in-situ flow measurement (during the wet season after the channel has been constructed) or, if excessive quantities of groundwater have not been observed, or there is no groundwater in the dry season, nominal drainage may be provided.

During the planning and design of a drainage system the following points should be borne in mind:

i. A thorough investigation of the sub-surface drainage requirements is essential. Such an investigation should be performed during the rainy season, because the dry season may present a completely false picture of groundwater conditions.
ii. A sub-surface drainage system should ensure that the road structure is without free water for about 1.0 m below the road surface most of the time.
iii. It is, therefore, advisable to provide sub-surface drainage in all cuts and underneath some low embankments.
iv. cut-off drains should be provided at the end of a cut.
v. Water should be able to escape from an embankment that drains towards a solid bridge abutment.
vi. The capacity of the drainage network should be adequate.
vii. The drainage network should require minimum maintenance.
viii. The drainage network should not become blocked.
ix. Repairs and alterations are extremely expensive.
x. The drainage network should be economical.

13.9.6 LOCATIONS OF SUBSURFACE DRAINS

Where moisture ingress is unlikely, truck traffic is light and similar pavement designs in the vicinity have already performed satisfactorily without subsurface drains, they may be omitted. It is difficult to describe all circumstances that warrant the installation of subsurface drains but, where soils are not free draining (i.e. Clays, silts, loam) or where there is a likelihood of water ponding near the pavement, subsurface drains should be considered.

While provision of subsurface drains without design may appear excessive, it is prudent to provide drains extensively on roads where soils are not free draining. Omission of subsurface drains on some roads has caused premature pavement failure and considerable expense in installing them afterward. The following guidelines are offered where seepage is not obvious.

13.9.6.1 LONGITUDINAL SUBSURFACE DRAINS

Longitudinal drains are normally located parallel to the roadway centreline, with both horizontal and vertical alignments. This type of drainage usually includes a trench of substantial depth, a collector pipe, and a protective filter of some kind, or it may be less elaborate. Examples of types of longitudinal drains commonly used in control of seepage and groundwater are shown in **Figure 13.133**and **Figure 13.134**.

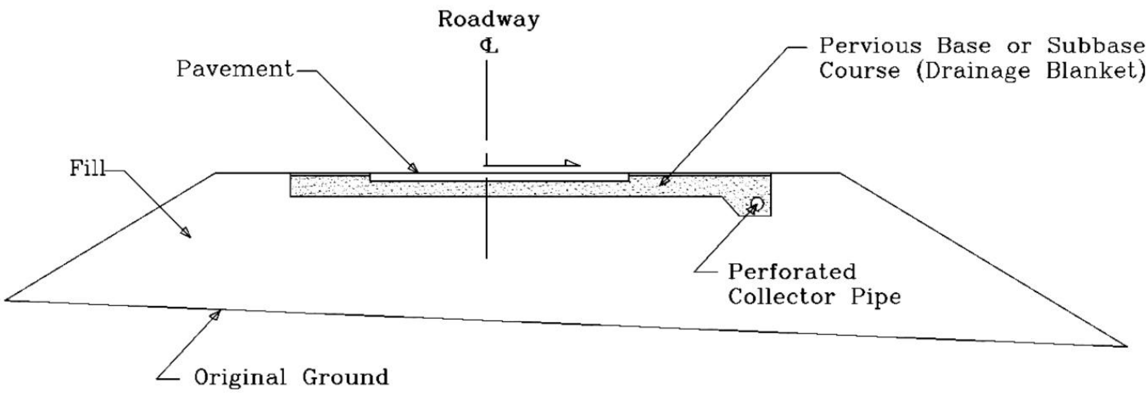


Figure 13.133 Longitudinal collector drain used to remove water seeping into pavement structural section

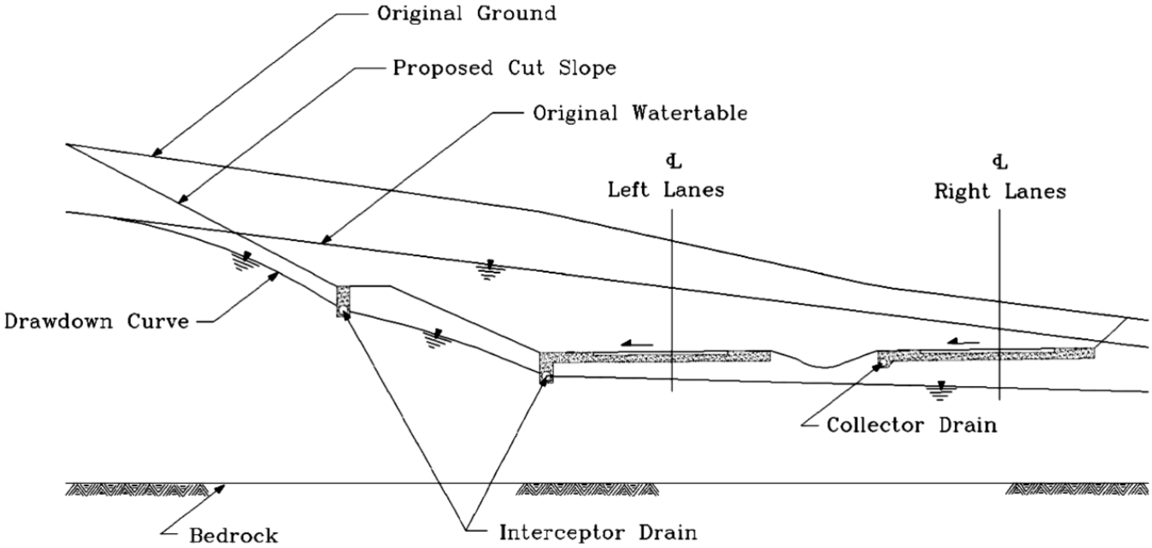


Figure 13.134 Multiple, multipurpose longitudinal drain installation

Sometimes, a multiple drain installation is needed for control of water under certain situations. **Figure 13.134** shows an example where a multiple longitudinal drain in a section of an expressway/motorway cut in a wet hillside. In order to intercept the flow and draw down the water table below the left cut slope, it was necessary to use two lines of relatively deep longitudinal drains.

In addition to intercepting water flowing from the hill slope, the interceptor drain beneath the left shoulder of the left lanes drains any water that may enter the base or subbase of the left lanes from infiltration or frost action. The shallow collector drain along the left edge of the right lanes performs this same function.

In many cases it is not possible to compact the subgrade material to desired specification. These materials are then removed, and other more suitable material is transported in to replace it. The resulting backfilled subcuts are then susceptible to the 'bathtub effect', meaning that water will accumulate in the volume of replaced materials. It is important to provide drainage for these subcuts. The drainage of subcuts can be accomplished with longitudinal drains if the subcuts are continuous along the pavement, or the drains might be placed on a transverse angle to the pavement if the subcut's volumes are localized. The design of drains for longitudinal subcuts follows the same procedures used for longitudinal edge drains. For more localized subcut situations the design of the drains can follow the procedures used for transverse drains.

Subsurface drains should be placed along:

i. the low sides of pavements
ii. both sides of the pavement near any cut-to-fill line
iii. both sides of a kerbed pavement
iv. both sides of the pavement where the crossfall is flatter than 0.02 m/m in a superelevation development
v. the high side of pavement where seepage is evident, or where water may enter from batters, full-width pavement, service trenches or abutting properties
vi. joins between an existing pavement and a pavement widening where pavement depths or permeabilities could create a moisture trap
vii. both sides of pavements for expressways/motorways and major/minor arterial roads.

Medians

Subsurface drains should be considered along the:

i. low side of a dished median where the median drain invert level is less than 0.2 m below subgrade level of the adjacent pavement
ii. low side of a kerbed median where the cross slope is 0.10 m/m or more
iii. sides of a median greater than 2 m wide
iv. sides of a median with a fixed watering system
v. centre of flat grassed medians without fixed watering systems and less than 6 m wide.

13.9.6.2 TRANSVERSE SUBSURFACE DRAINS

Transverse drains are a class of subsurface drains that run laterally beneath the roadway. The common placement of these drains is at right angles to the roadway centreline, although they may be skewed in some cases, creating what is often referred to as the "herringbone" pattern (Moulton, 1980). This type of drainage system is often used at pavement joints to drain infiltration and groundwater which may be in the bases and subbases. Transverse drains may be used in conjunction with a horizontal drainage blanket and longitudinal collector drain system, which provides an effective means for rapid removal of water from the pavement section.

Transverse drains may involve a trench, collector pipe, and protective filter, as shown in **Figure 13.135**, or they can consist of simple "french drains" (shallow trenches filled with open graded aggregate), although this is not generally recommended. The degree of sophistication employed in the designs of this type of drainage system depends on the source of the subsurface water and the function of the drain. This type of drain is especially effective when used in situations where the general direction of the groundwater flow tends to be parallel to the roadway (common when the roadway is cut more or less perpendicular to the existing contours). This application is illustrated in **Figure 13.136**.

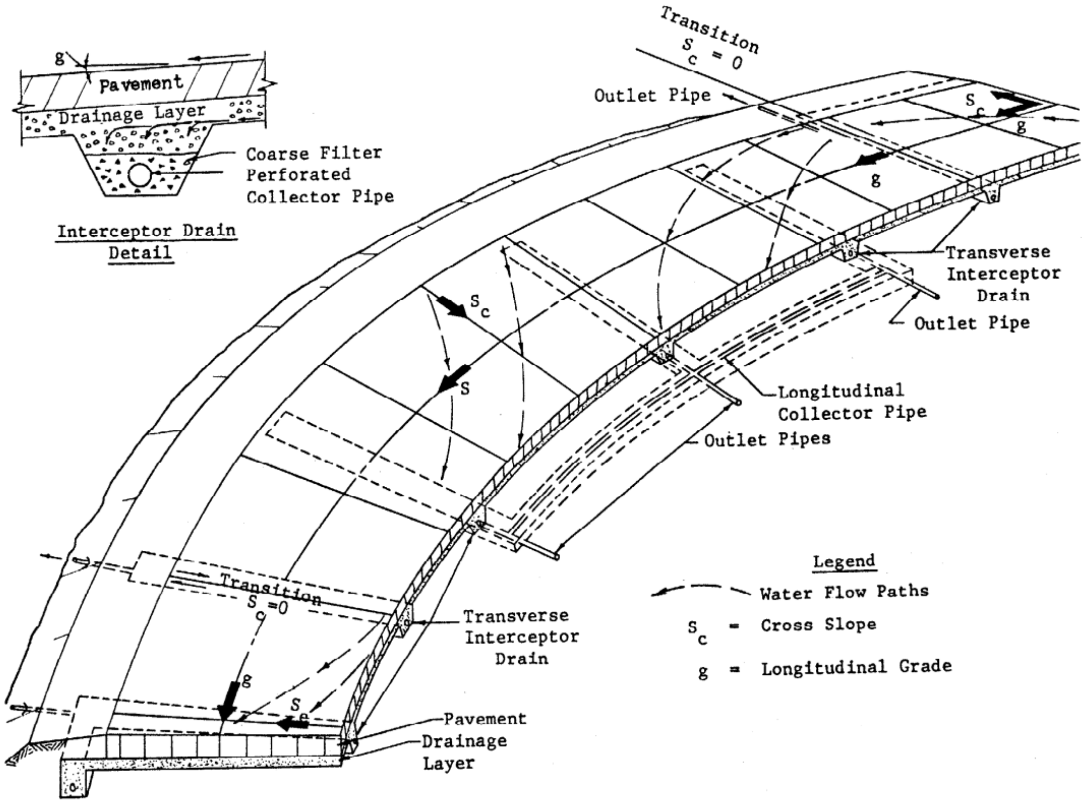


Figure 13.135 Transverse drains on super-elevated curve (Moulton, 1980)

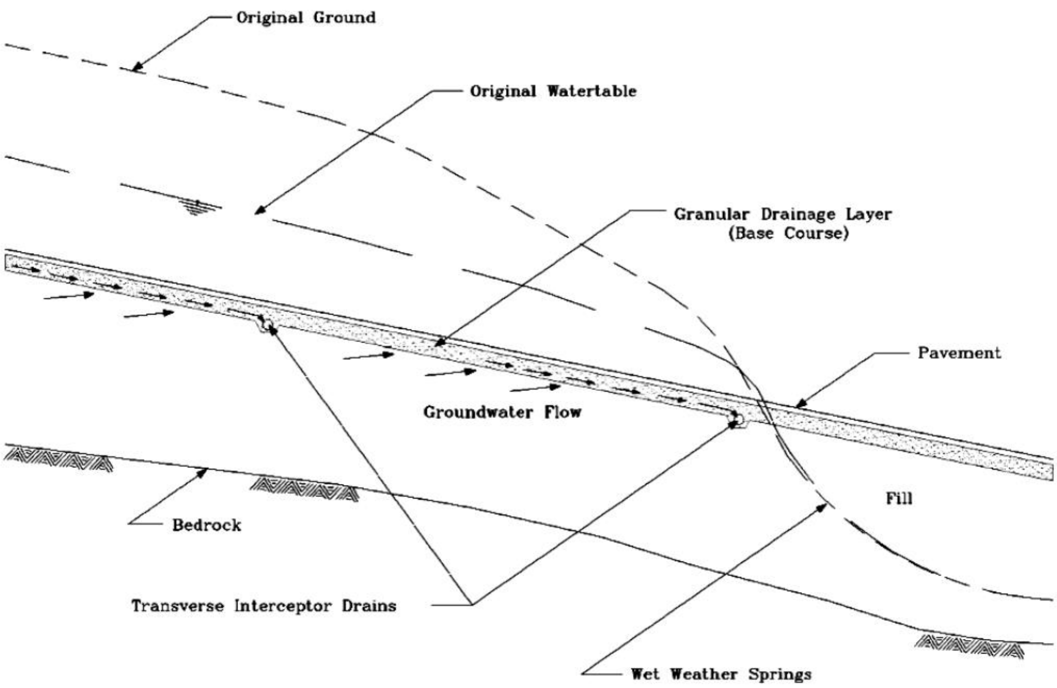


Figure 13.136 Transverse interceptor drain installation in roadway cut with alignment perpendicular to existing contours

Transverse subsurface drains (**Figure 13.137**) should be considered:

i. approximately 5 m upstream of cut-to-fill lines
ii. along changes of pavement depth or permeability
iii. at both ends of bridge approach slabs, that is
   – immediately behind the bridge abutment to the full depth of the abutment
   – in the subgrade at the interface of the road pavement and the approach slab
iv. at superelevation changes, to limit the length of the longest drainage path within the pavement to about 50 m.

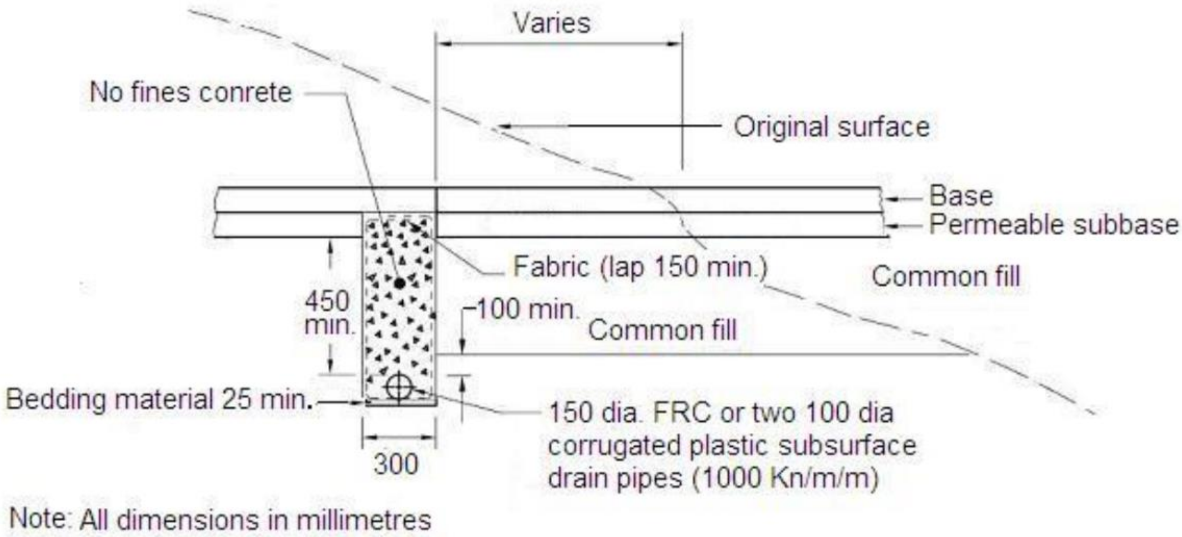


Figure 13.137 Transverse pavement drain

13.9.6.3 CUT-OFF (FORMATION) DRAINS

Subsurface cut-off drains should be considered:

i. along both sides of cuts where the road is known to be below the water table, or where seepage is encountered during construction, or where seepage is expected in wet weather
ii. transversely at any seepage areas, and further downgrade if required. The transverse drains may be laid in a herringbone pattern if necessary to achieve the minimum grade.

In many instances hillslopes along the side of roadways can have ground water seeping from higher ground, which leads to instability of the hillslope in many instances. This ground water is also a source of water for the pavement foundation. An illustration of a field situation near a pavement section with hillslope seepage is shown on **Figure 13.138**. The soil profile has a bottom boundary layer which is considered to be effectively impervious. The ground water flow toward the highway shows that the water table intersects with the hillslope surface near the road ditch, and ground water is seeping through the slope into the ditch. In addition, ground water is flowing beneath the road and entering into the subgrade and base course material.

Placing a cut-off (formation, interceptor) drain upgradient from the ditch, or beneath the ditch itself, can help to control the hillslope seepage and decrease or even eliminate the flow beneath the roadway, thus removing the source of water from entering into the pavement foundation. An illustration of the situation with a cut-off drain is shown in **Figure 13.139**. There it is seen that the water table is drawn down by the cut-off drain to the level of the drain. The water table down gradient of the cut-off drain may rise up above the level of the drain due to seepage flowing under the drain.

The design of a cut-off drain requires an estimate of the hydraulic conductivity of the hillslope soil, k, the thickness of the saturated zone for the ground water, which is shown as height H in **Figure 13.138**, the slope of the bottom boundary of the soil profile, S, and the height of the drain above the impermeable boundary, Ho. If we want to prevent ground water from entering into the subgrade and base course material, then the interceptor drain needs to be placed at an elevation below the elevation of those foundation layers, as shown in **Figure 13.139**.

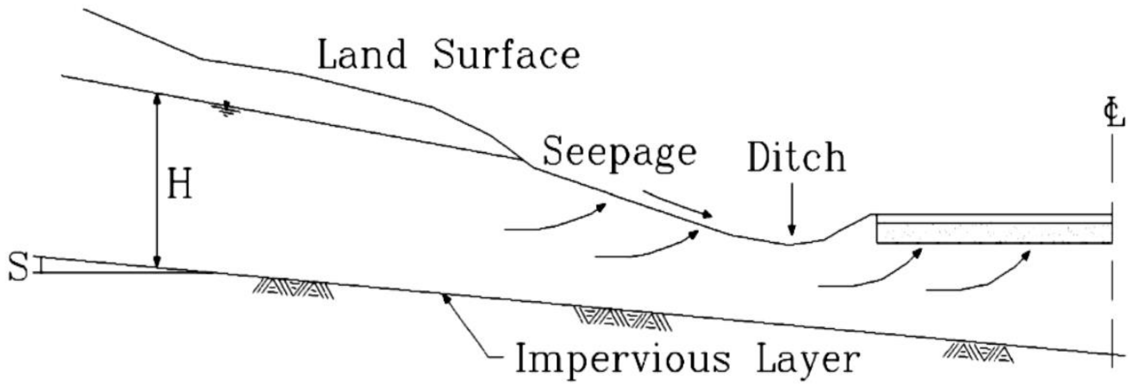


Figure 13.138 Illustration of ground water flow along a sloping impervious layer toward a roadway.

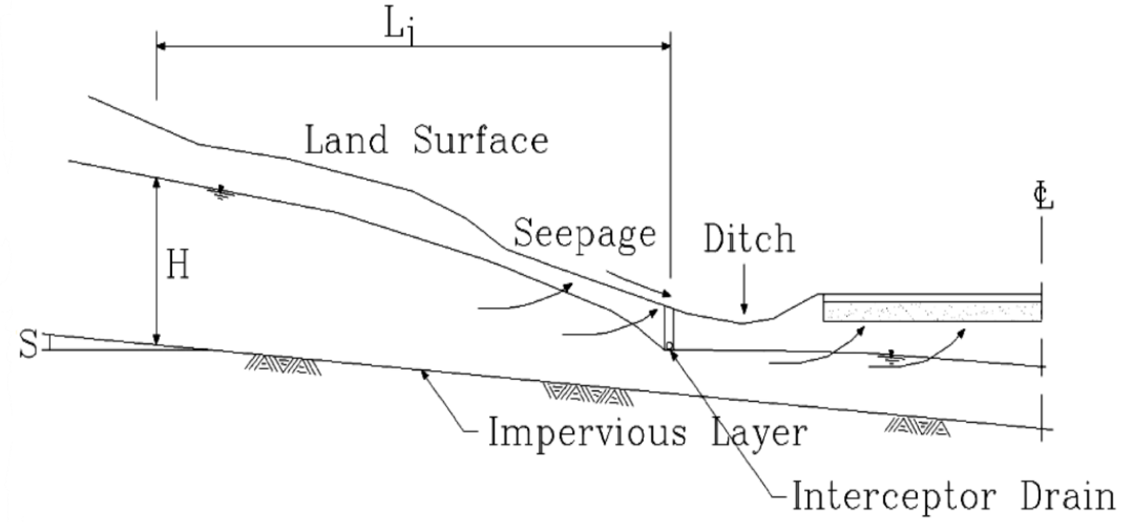


Figure 13.139 Illustration of the effect of an interceptor drain on the drawdown of the ground water table.

13.9.6.4 COMBINED STORMWATER AND GROUNDWATER DRAINS

Where the road is below the water table for most of the year, and flows are expected to exceed the capacity of the standard size subsurface drain, it can prove economical to lay slotted pipes in deep trenches to cater for both groundwater and stormwater flows.

13.9.6.5 LOCATIONS OF SUBSURFACE DRAINS ON RURAL ROADS

Longitudinal pavement drains are seldom placed along lightly trafficked rural roads, except where the subgrade consists of granitic sands mixed with rock. Factors to be taken into account are the cost of installing subsurface drains as a proportion of total construction cost, and of subsequent maintenance; as comprehensive tests are not carried out on many rural roads, provision of subsurface drainage is a matter for judgement by the designer and the construction manager.

Mitre drains may be used as an alternative to piped subsurface drains on low cost rural roads, but on rural arterials, slotted pipes should be used, so that maintenance can be carried out.

13.9.6.6 ACCESS TO SUBSURFACE DRAINS

Inlets and outlets for subsurface drains should be located clear of the traffic lanes. Where the inlet must be located in the shoulder, a pit with a trafficable steel cover should be used. The inlet should not be located in a position where it would be possible for stormwater to enter the subsurface drainage system.

In urban areas, subsurface drains usually start and end in drainage pits. In rural areas, the flushout riser inlet (**Figure 13.140**) is less expensive than a pit for intermediate access and outlets are as shown in **Figure 13.141**.

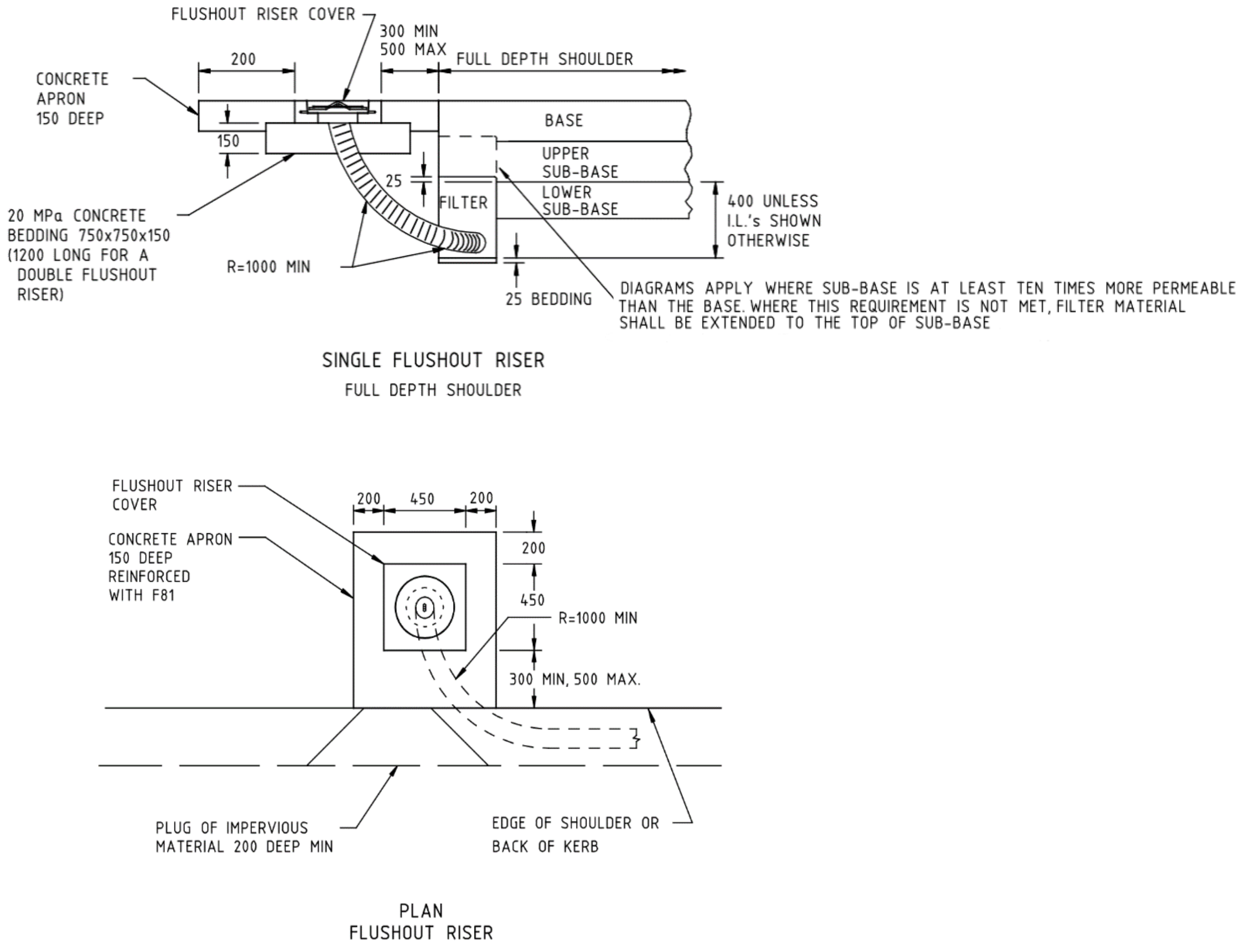


Figure 13.140 Typical flushout riser (all dimensions in mm)

Pits for subsurface drainage should be spaced not further than 150 m apart for ease of inspection and cleaning of the pipes. Maximum distance between a flushout riser and an outlet should generally not exceed 120m to facilitate inspection and flushing. In cuttings where groundwater is not present, the distance to the outlet of a pavement drain may be up to 360m, but intermediate pits should generally be placed at a maximum spacing of 120m. Where groundwater occurs in a cutting, the seepage should be conveyed from the subsurface drain into an impervious collector pipe to minimise water penetration of pavement remote from the problem area. The minimum size of pits is shown in **Table 13.62**. Subsurface drain outlets should be paved to prevent erosion.

Table 13.62 Minimum pit width

Pit depth (m)	Minimum pit width or diameter (m)
Less than 1.5	0.6 or 0.75 x 0.75 in traffic lane
More than 1.5	1.05

It is most important that there are no more than three intermediate subsurface drainage pits provided before an outlet is provided from the system (i.e., into stormwater drainage pits, batter outlets or through a culvert end wall, etc.). It is suggested that batter outlets should be permanently marked in some way so they can be easily located by maintenance personnel.

The spacing of access chambers (pits) for stormwater varies depending on the size of the pipe (**Section 13.5.9**). However, the location of these chambers will influence the spacing of subsurface drainage pits and flushout risers.

Outlets should be in areas that are easily accessible and where possible, visible to personnel standing on the road surface. An outlet should not hinder road maintenance activities such as cleaning unlined table drains or grass cutting.

Outlets should be provided with some form of erosion protection commonly referred to as a splash zone.

Typically, this consists of either:

i. a masonry or concrete apron
ii. an area of large aggregate to dissipate the outflow energy.

A typical outlet is shown in **Figure 13.141**.

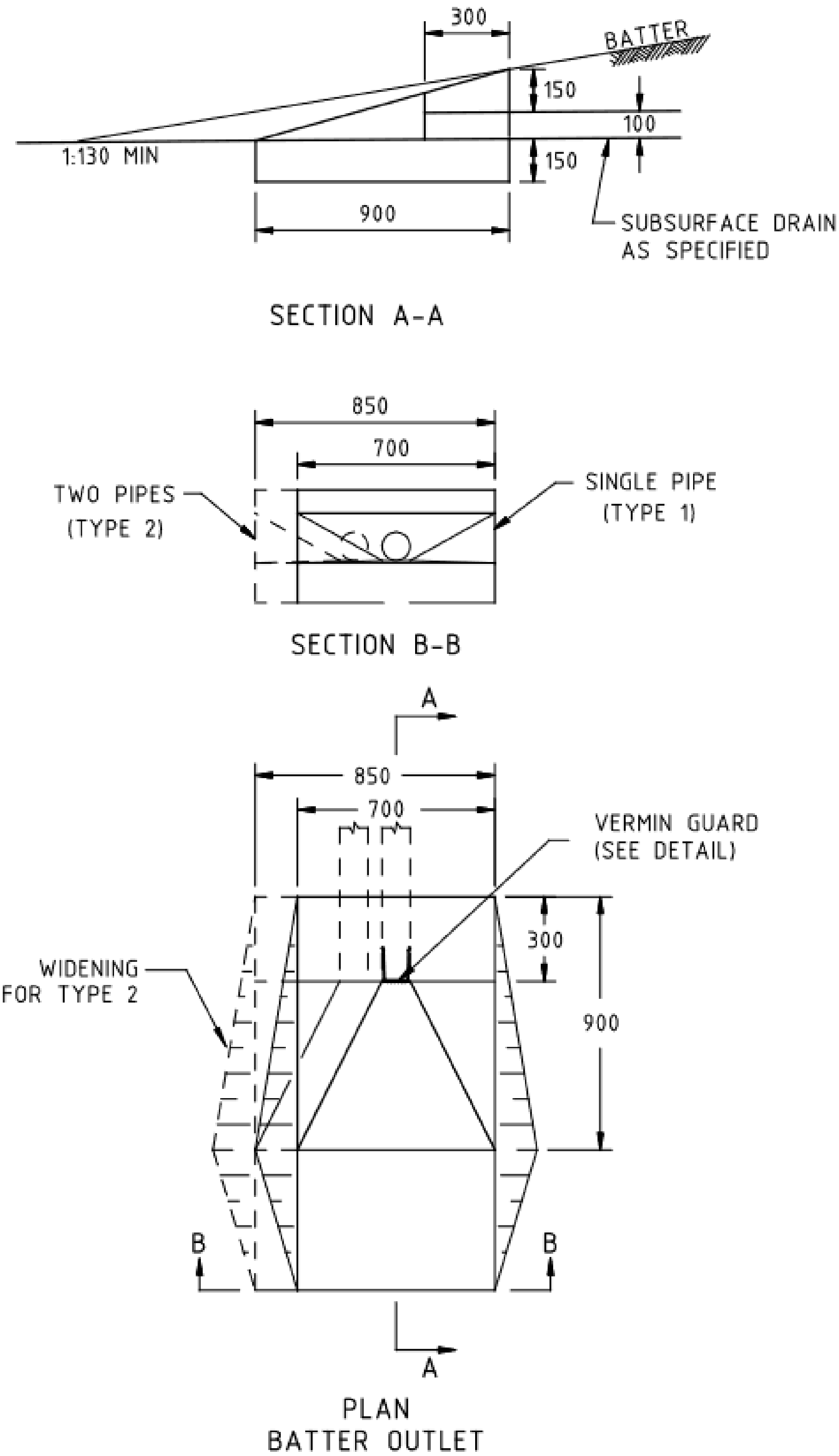


Figure 13.141 Typical subsurface drain outlet (all dimensions in mm)

13.9.7 SELECTION OF PAVEMENT DRAIN TYPE AND FILTER TYPE

Table 13.63 is an example guide for the initial selection of pavement drain type and filter type.

Table 13.63 Example guide to selection of subsurface drain type and filter type

Parent soil type	Permeability range (m/sec)	Type of pavement drain (Figure 13.132)	Filter material (refer to Table 13.64 & Table 13.65)
Homogenous clay with very low permeability	$<10^{-9}$	Type 3 or Type 4	Sand (Grade A1 to A3)
Silty or sandy clays and stratified clays with moderate low permeability	10^{-9} to 10^{-5}	Type 2, Type 3 or Type 4	Sand (Grade A4 to A6)
Clean sand or gravel with high permeability	$>10^{-5}$	Type 1 or Type 2	Aggregate (Grade B1 or B2)
Solid rock or clean broken rock with high permeability to permeable fissures	Not applicable	Type 1	Aggregate (Grade B3 or B4)

Note: Type 4 subsurface drains may be substituted for a Type 2 or Type 3 pavement drain if located in fill areas or within pavement layers such as at the interface between two pavement types.

Table 13.64 Type A (sands, uniformly graded fine aggregates and gravel) filter gradings

Type A	Percentage passing sieve					
Description of filter	A1	A2	A3	A4	A5	A6
	Dune (Fine) sand	Course washed sand		5 mm one size	6 – 8 mm one size	Sandy gravel
37.5 mm						100
26.5 mm						
19.0 mm					100	85 – 100
13.2 mm					90 – 100	
9.50 mm		100	100	100	70 – 100	65 – 100
4.75 mm		90 – 100	90 – 100	70 – 100	28 – 100	45 – 82
2.36 mm	100	75 – 100	70 – 100	0 – 50	0 – 28	30 – 60
1.18 mm	95 – 100	50 – 98	40 – 65	0 – 10	0 – 8	15 – 40
600 µm	70 – 98	30 – 80	12 – 40			5 – 25
300 µm	30 – 60	10 – 40	0 – 16	0 – 5	0 – 5	0 – 10
150 µm	0 – 12	0 – 7	0 – 4			0 – 5
75 µm	0	0 – 3	0 – 3	0 – 3	0 – 3	0 – 3
Parent soil	**Silt and friable clays**		**Sand silts**	**Fine to medium sands**	**Coarse sand**	**Sandy silts**
Maximum pipe slot width	0.4 mm	0.6 mm	1.5 mm	3.0 mm	3.3 mm	5.0 mm
Suitable second stage filter	B1	B2	B3	B4		B3 or B4

Table 13.65 Type B (uniformly graded aggregates) filter gradings

Type B	Percentage passing sieve			
Direction of Filter	B1	B2	B3	B4
	5 mm one size	6 – 8 mm one size	10 mm one size	19 mm one size
37.5 mm				
26.5 mm				100
19.0 mm		100	100	70 – 100
13.2 mm		90 – 100	90 – 100	0 – 30
9.50 mm	100	70 – 100	40 – 70	0 – 10
4.75 mm	70 – 100	28 – 100	0 – 15	
2.36 mm	0 – 50	0 – 28	0 – 5	0 – 5
1.18 mm	0 – 10	0 – 8		
600 µm				
300 µm	0 – 5	0 – 5		
150 µm				
75 µm	0 – 3	0 – 3	0 – 3	0 – 3
Proposed use	**With type A1**	**With type A2**	**With types A3, A6**	**With typesA4, A6**
Maximum Pipe Slot Size	3.0 mm	3.3 mm	9.0 mm	15.0 mm

13.9.8 DRAINAGE DETAILS

A. Size of Drain

The grade line of the drain is generally known as it follows the road geometry. If minimum grades cannot be achieved for subsurface drainage, independent grading needs to be considered. There are a number of design charts available which can be used to size the drain given the slope and the volume of water that the drains are required to carry. However, to avoid the system failing due to partial blocking of the drain, the drain should be designed to carry at least three times the expected flow.

Generally, the diameter of subsurface drains should be no less than 100 mm. In some cases a combined stormwater and subsurface drainage system may be used.

Under normal to dry conditions the pipe diameters and maximum pipe lengths (without outlets) given below are used if no design data are available:

- Diameter 100 mm, lengths up to 200 m;
- Diameter 150 mm, lengths up to 300 m.

In expressway/motorway drainage systems 100 mm diameter pipes are mainly used, but where high inflows of water are encountered, 150 mm diameter pipes are more suitable.

In cases where the maximum discharge is known, the pipe diameter is calculated according to **Figure 13.142**. A sub-surface drainage pipe should preferably not flow more than 70% full under maximum discharge conditions, so that excessive pressure conditions do not develop.

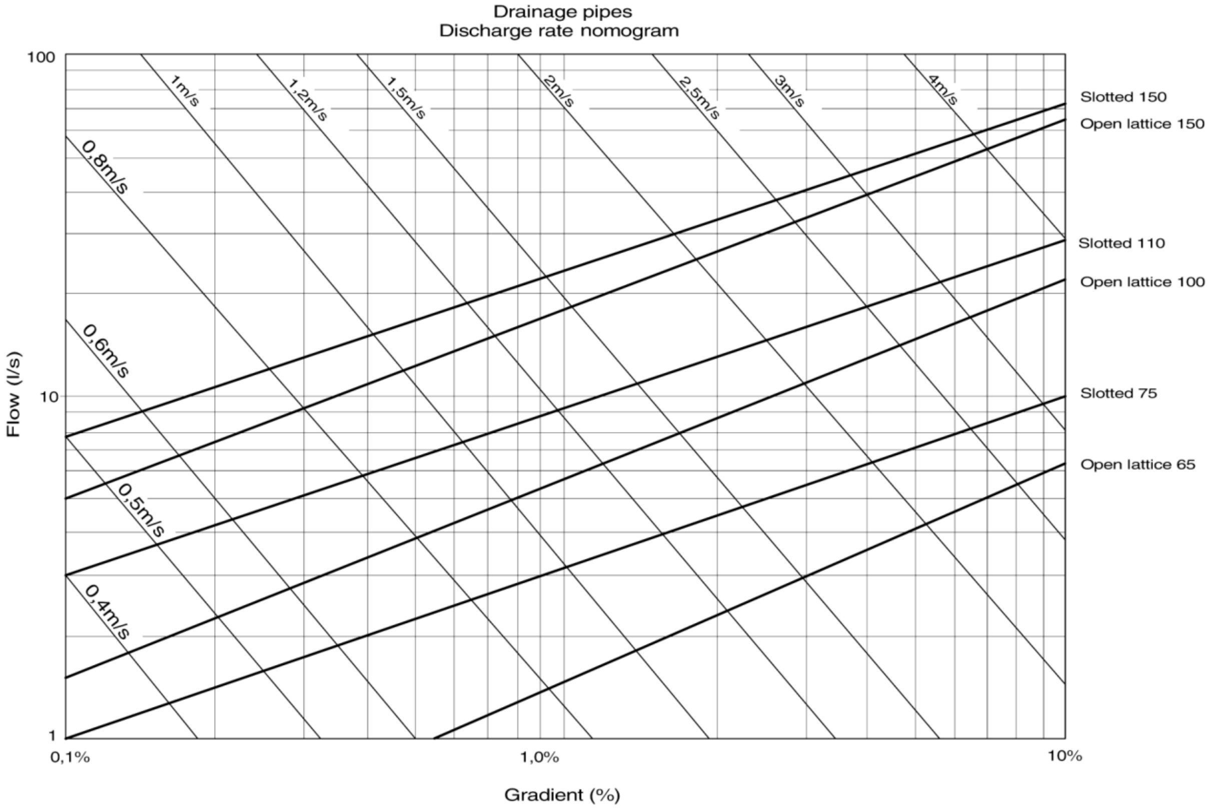


Figure 13.142 Nomogram for the discharge rate of drainage pipes

B. Materials

Subsurface drains are manufactured from a range of materials, but all require some form of perforation to allow subsurface water to enter the pipe.

The pipe perforations should satisfy the following requirements to prevent penetration of filter material:

- Diameter of openings, slots or perforations < D_{85} filter;
- 1.2 x width of openings, slots or perforations < D85 filter
  (Where D_{85} = soil particle size for which 85% of openings are smaller (mm))

Simply put, the smaller sized particles of the filter material should be larger than the largest size openings, slots or perforations of the pipe.

Corrugated polyethylene agricultural drain is generally the most cost-effective material. Pipe with a diameter of 90 mm is regarded as the minimum for roadwork.

Smooth polyvinyl chloride (PVC) pipe is used to convey flows across a pavement or may be used where longitudinal gradients are flatter than 0.5%. Since this material is expensive, ‘herringbone grading’ of corrugated pipe is typically applied. Pipe sizes generally range from 100 mm diameter to 300 mm diameter.

Prefabricated polyethylene (PE) drain, also known as fin drain, may be laid in batters or parallel to the pavement to intercept groundwater. It may be used across a pavement if the trench is backfilled with no-fines concrete. This material has less hydraulic capacity than the corresponding diameter of pipe, so this may have to be checked. Depths of the fin drains are typically 200 mm to 450 mm (**Section 13.9.4**).

Concrete pipes are typically used where groundwater flows require diameters not available in plastic pipes.

Pipe sizes in common use range from 300 mm to 750 mm in diameter. Concrete pipes (steel reinforced or fibre reinforced) may be used as outlet pipes or impervious collector pipes.

Smooth plastic may be slotted or unslotted. Smooth plastic pipe should be used where the subgrade drain gradient must be flatter than 0.5%. Unslotted pipe may be used to convey flows beneath pavement to outlet or as a collector pipe.

Perforated corrugated steel may be used for deep formation drains where the soil and water are not highly corrosive. They require specific structural design and proposals for use of this material should be approved by the road agency.

C. Filters

Whether the drainage system comprises a vertical trench or a horizontal blanket, it is generally accepted practice that either a granular filter or a synthetic filter fabric is used to supplement the subsurface drainage solution.

The design of the filter material should avoid clogging with fines from the adjacent material and ensure that the size of the filter material and the openings in the drainage pipe are such that entry of filter material into the drainage pipe will not occur.

D. Minimum Diameter

The minimum diameter used for subsurface drains is 90 mm. For formation drains, outlet from pits, and transverse drains, 150 mm diameter is preferred.

E. Minimum Cover

Subsoil pipes shall have a minimum cover of 0.3m (from top of pipe to formation level) if it is not subjected to vehicular loading. If there is vehicular loading minimum cover shall be 1.2m.

F. Minimum Grades

The minimum grade for subsurface drains in expansive clays is 1%, except where the trench is isolated from the natural clay by at least 100 mm of impermeable material, or by an impermeable membrane. The minimum grade for corrugated subsurface drain pipes is 0.5%. Smooth bore pipes may be laid as flat as 0.3%.

G. Sub-pavement Layers

Where springs occur under the road, a permeable layer of no-fines concrete or sandy material may be provided under the pavement. Detailed design of the layer should be done by a specialist designer. In expansive volcanic clays, it may be advantageous to place an impermeable layer beneath the pavement at least 100 mm thick which, in conjunction with the pavement drain, prevents entry of water into the moisture sensitive subgrade material.

13.9.9 DESIGN PROCEDURES

The following design procedure outlines the steps that a road designer should undertake in designing a subsurface drainage system.

A. Data Required

Before starting to detail the locations of subsurface drains, the road designer needs to know:

i. the extent of each flat area on the pavement, e.g., at superelevation changes
ii. the pavement composition, and whether full-width or boxed construction is proposed

iii. the relative permeabilities of the pavement materials and surrounding materials
iv. the places where groundwater levels are high, or springs occur.

Design of subsurface drainage filter zones usually requires laboratory and field testing of materials, groundwater investigations, and interaction with the design of the pavement. The filter type for a particular project should be designed and specified by a geotechnical expert.

B. Procedure

The steps involved in designing a subsoil drainage system are:

i. to decide on an appropriate material testing and site investigation program for the project
ii. after groundwater investigation, to carry out hydraulic design of cut-off drains
iii. at the road grading stage, to ensure that fills are high enough to inhibit capillary rise
iv. to arrange for pavement design. The pavement depth must be known to set subsurface drain levels
v. to obtain a copy of the road geometry
vi. to select the appropriate locations of subsurface drains
vii. if subsurface drains can be placed parallel to the road surface in the vertical plane, subsurface drain detailing can follow stormwater drainage design
viii. where the longitudinal grade of the road is very flat (less than 0.4% grade), and there is a need for independent grading of subsurface drains, both drainage systems could be designed concurrently if appropriate to do so
ix. to identify cut-to-fill lines and locate the transverse drain
x. to detail the locations of inlets and outlet pits where these do not coincide with stormwater pits.

Drainage facilities should be designed and constructed recognising that periodic inspection and repair will be required and provide for the safety of maintenance personnel as well as for road users. Investigation of potentially cracked or failed underground pipes should be carried out using a remote television camera to reduce the risk to inspection personnel.

13.9.10 SPECIALIST SUBSURFACE DRAINAGE TECHNIQUES

The following section identifies techniques for managing subsurface/groundwater issues. However, as these techniques require specialist knowledge they have been provided for information and general guidance only.

13.9.10.1 LOWERING OF GROUNDWATER TABLE

The lowering of a static water table is achieved through the use of a system of vertical cut-off drains below the road pavement (**Figure 13.143**).

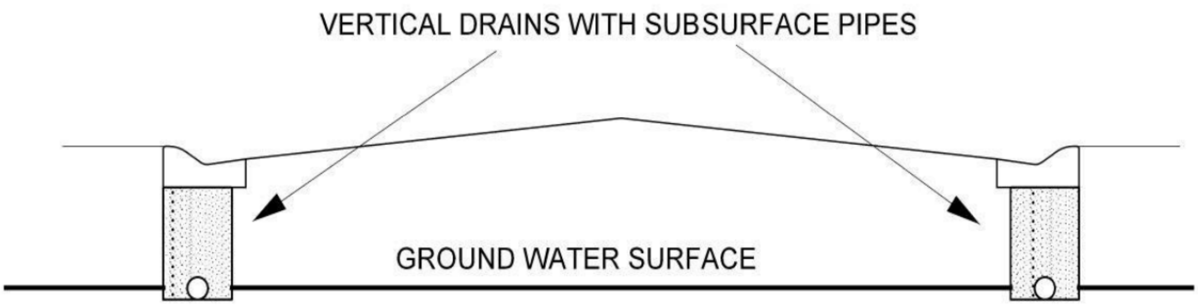


Figure 13.143 Typical groundwater drainage system

Table 13.66 is used for a preliminary assessment of the effectiveness of a proposed trench drainage system.

However, if the subgrade permeability is less than 1 x 10^{-7} m/s then vertical drains on both sides of the roadway are unlikely to be effective in lowering a water table. An alternative solution to subsoil drainage should be adopted, such as pavement design based on saturated subgrade strength.

Table 13.66 Effectiveness of trench drainage systems

Coefficient of permeability, k (m/s)	**Amount of lowering in (m) after different periods midway between two 1 m deep trenches (Initial water table 1 m above bottom of trenches)**					
	Trench spacing 3 m		**Trench spacing 10 m**		**Trench spacing 20 m**	
	3 months	**1 year**	**3 months**	**1 year**	**3 months**	**1 year**
1 x 10^{-5}	1.00	1.00	1.00	1.00	1.00	1.00
1 x 10^{-6}	1.00	1.00	0.93	1.00	0.48	0.93
1 x 10^{-7}	0.94	1.00	0.23	0.65	0.06	0.23
1 x 10^{-8}	0.25	0.68	0.03	0.10	0.00	0.03
1 x 10^{-9}	0.03	0.11	0.00	0.00	0.00	0.00
1 x 10^{-10}	0.00	0.00	0.00	0.00	0.00	0.00

13.9.10.2 SCHILFGAARDE’S METHOD

Schilfgaarde’s method (van Schilfgaarde 1963) can be used to determine the drain spacing that will lower the water table by $m_o - m$ (**Figure 13.144** and **Equation (13.162)**). The accuracy of the solution is extremely dependent on the accuracy of k and f (i.e. permeability and drainable pore space of the subgrade). The location of a water table that is likely to cause a drainage problem and the effect of a typical subsurface pipe arrangement is illustrated in **Figure 13.144**. The design procedure is shown in **Figure 13.145**.

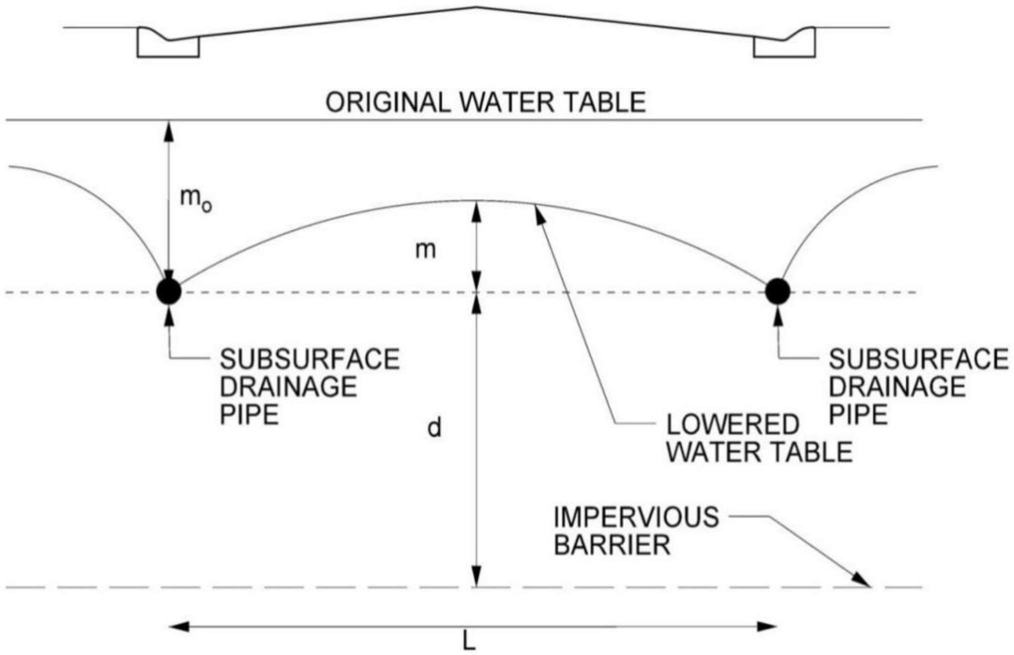


Figure 13.144 Geometry of the drainage problem and effect of subsurface drains

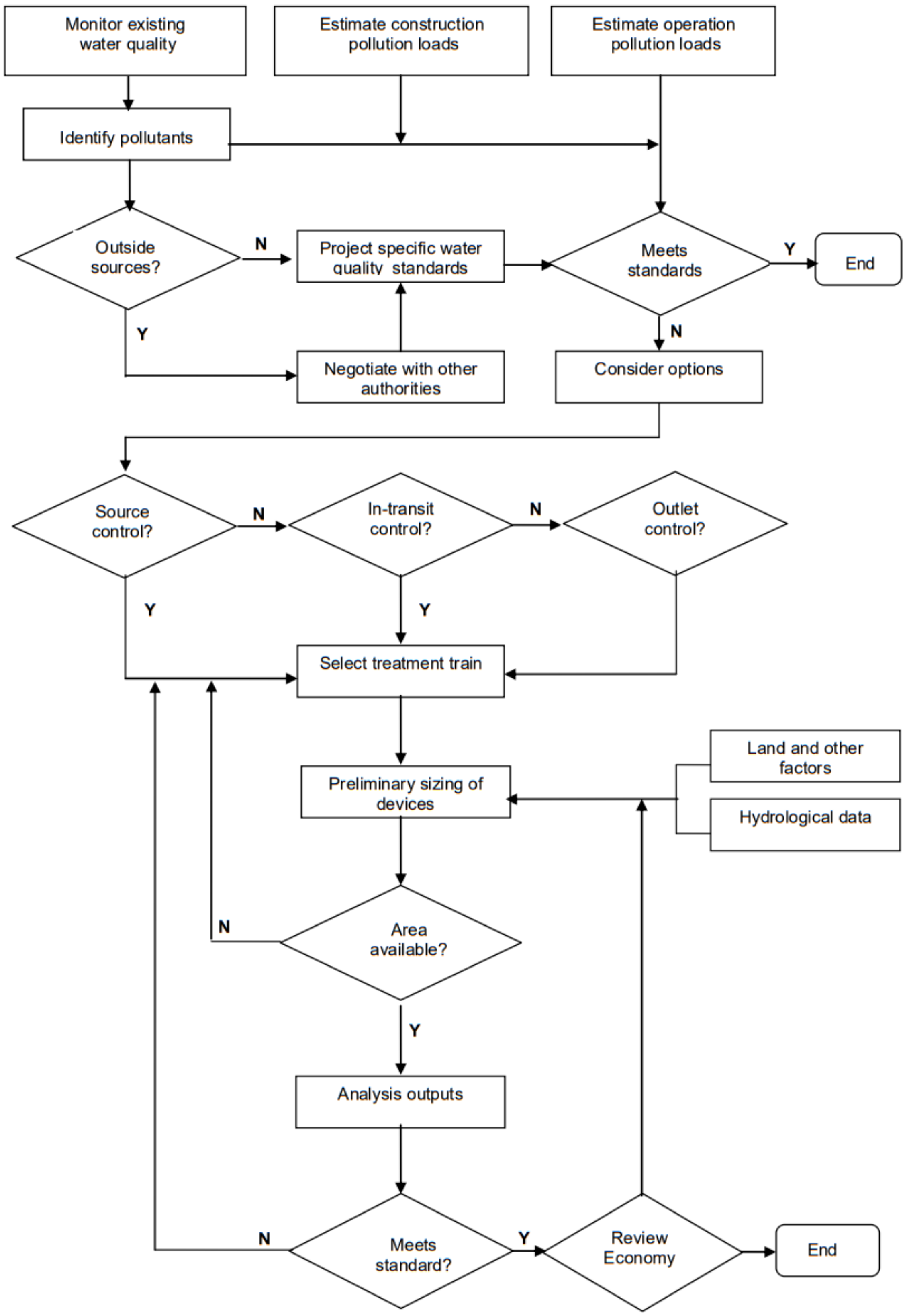


Figure 13.145 Flow chart of Schilfgaarde's method

$$S = 3j\left[\frac{k(d_e + m_0)(d_e + m)t}{2f(m_0 - m)}\right]^{1/2} \tag{13.162}$$

Where,

S = Spacing between drains (m)
j = Geometrical factor (determined from **Figure 13.147**)
k = Saturated permeability (m/s)
m_o = Depth of drain below original water table (m)
m = Depth of drain below the lowered water table (m)
$m_o - m$= Distance water table is lowered
d_e =Equivalent depth of drain to impervious barrier (m). Differs from ‘d’ because of convergence of the flow lines
f =Drainable pore surface, expressed as a fraction of total volume drained at 600mm tension (typically clays range from 0.0 to 0.11, well-structured loams from 0.10 to 0.15 and sands range from 0.18 to 0.35)
t = Time to lower water table (sec)

The solution requires that a starting estimate of L (drain spacing) be input with known values of d, m_o, m, k, t and f. The equivalent depth, d_e is estimated from **Figure 13.146**. This is then used to calculate a convergence factor as shown in **Equation (13.163)**.

$$Convergence\ Factor = \frac{d_e}{(d_e + m_0)} \quad (13.163)$$

The convergence factor estimate is then used with **Figure 13.146** to estimate ‘j’. ‘L’ is recalculated and if different from the initial estimate, a further iteration of calculations is commenced with the revised equivalent depth.

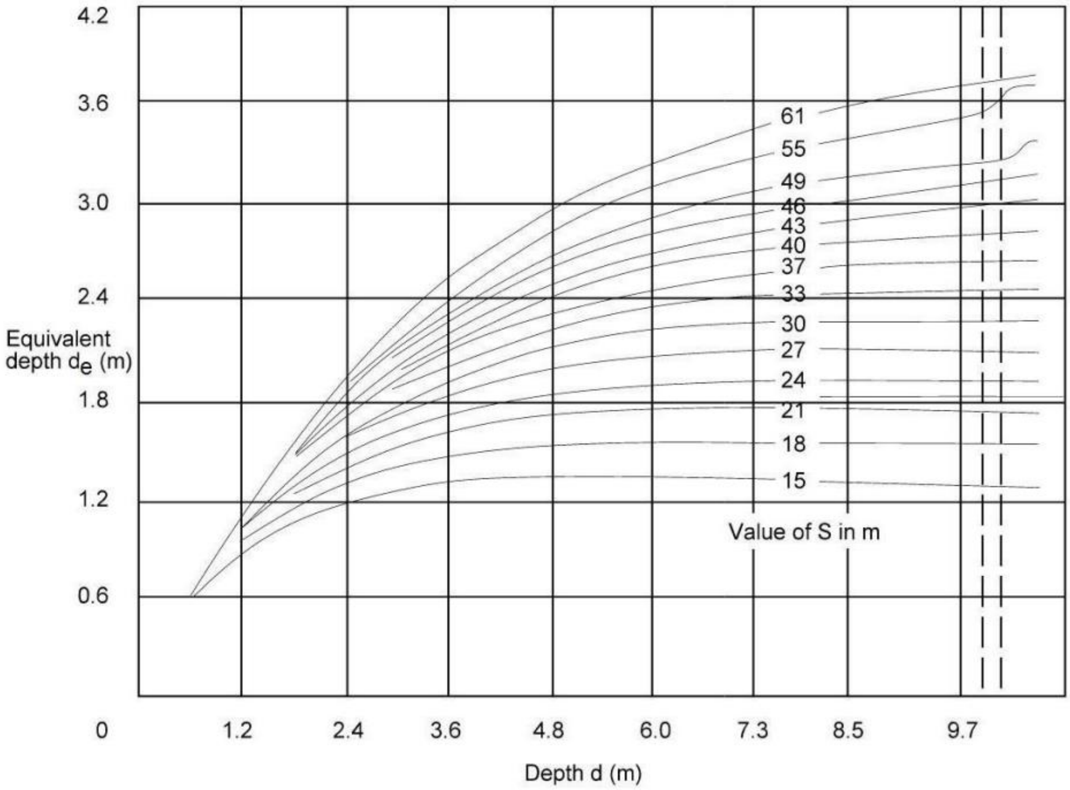


Figure 13.146 Equivalent depth for convergence correction

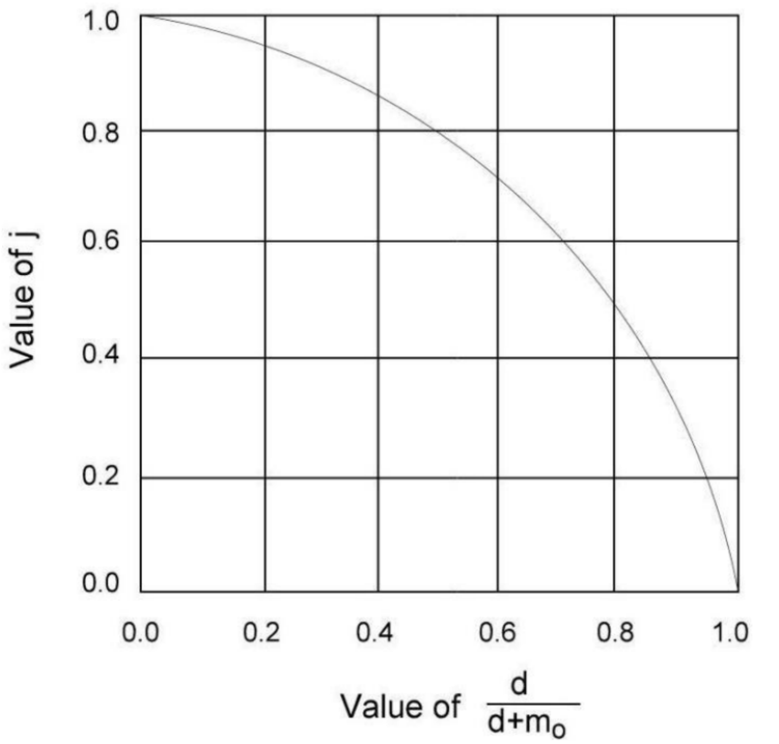


Figure 13.147 Dependence of factor j on depth to impervious layer

13.9.10.3 DRAINING AN INCLINED AQUIFER

The design of a subsurface drainage system which intercepts an inclined aquifer (**Figure 13.148**) is relatively straightforward. Darcy's law, shown as **Equation (13.164)**, governs the discharge from the aquifer.

$$q_m = kAi \qquad (13.164)$$

Where,

- q_m = Discharge per unit length of trench (m^3/m)
- k = Permeability of the aquifer (m/s)
- A = Area of aquifer (but this equation is equal to the thickness, since the discharge required is per unit metre of length (m^2))
- i = Slope of the aquifer (when the piezometric heads within the aquifer are equal) (m/m)

To ensure that the subsurface drainage systems intercept all of the seepage, the permeability of the filter material and the width of the trench need to be checked. This implies that the piezometric head must drop to zero within the trench filter material. The principle is shown mathematically in **Equation (13.165)** below.

Tan (piezometric gradient in trench)/Tan (slope of aquifer) = the ratio of the permeability of the aquifer material divided by the permeability of the filter material. The nomenclature used is shown in **Figure 13.148**.

$$\frac{Tan\ (B)}{Tan\ (A)} = \frac{k_a}{k_f} \qquad (13.165)$$

Where,

- $Tan\ (B)$ = Can be approximated by W/T (width of the trench divided by the thickness the aquifer)
- $Tan\ (A)$ = Can be approximated by the slope of the aquifer (shown as 's' in **Figure 13.148**)
- k_a = Permeability of aquifer material (m/s)
- k_f = Permeability of filter material (m/s)

There are two values, which can be altered by the drainage designer (W, or k_f) to balance **Equation (13.165)**.

However, trench width is normally fixed to a standard value (typically 300 mm) and so it then becomes a case of selecting filter material to ensure that the ratio Tan (B)/Tan (A) is less than k_a/k_f.

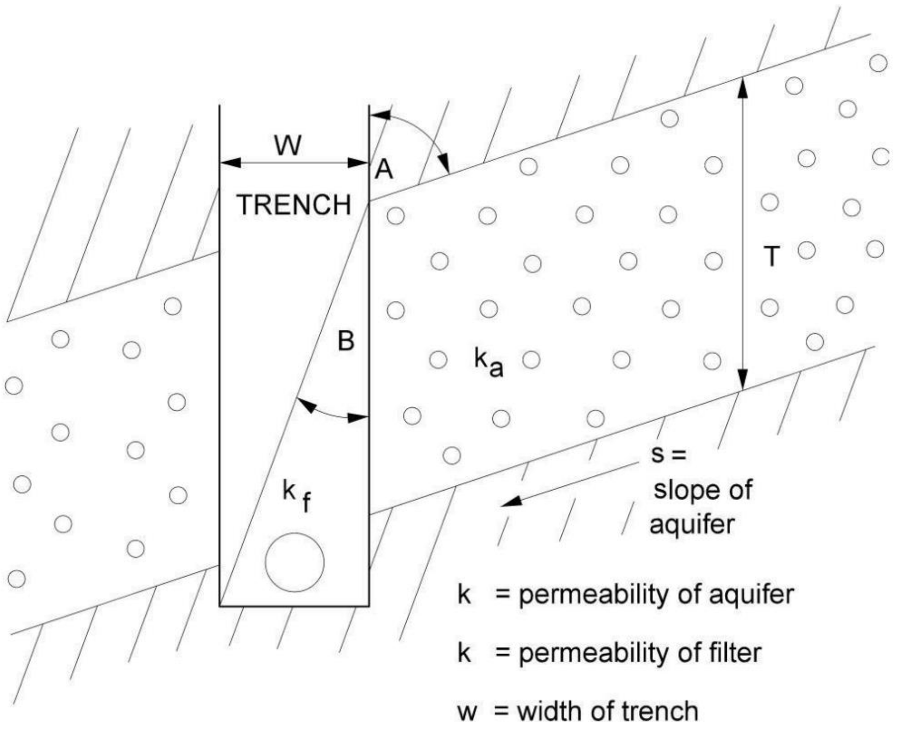


Figure 13.148 Trench excavated through an inclined aquifer

13.9.10.4 DESIGN OF A DRAINAGE BLANKET TO LOWER A WATER TABLE

Where a pavement is to be placed within or below the natural groundwater level it may be necessary to lower the water table. This can be achieved by placing a horizontal drainage blanket below the pavement. The design of the drainage blanket should be undertaken using analytical procedures such as flow net procedures and finite element methods. These analytical procedures are beyond the scope of this Guide and are usually undertaken by a geotechnical expert (see AGRD Part 1 (Austroads 2021).

13.9.10.5 DESIGN OF CUT-OFF (FORMATION) DRAINS

Simple analysis consisting of homogeneous layers of differing permeability rarely applies to natural conditions. Fissures, joints, faults and bedding planes in soil or rock structures can have large hydrostatic head differences over short distances that may vary rapidly. Strategic placement of piezometers and standpipes is therefore of the utmost importance. Theoretical models can give good results only if the ground conditions, input during design, are close to those in the field.

Road surfaces are more permeable than generally assumed, and the quantity of water entering a pavement (infiltration rate) may be estimated by multiplying the infiltration coefficient (**Table 13.67**) by the two-year, one hour rainfall intensity over the surface area.

Table 13.67 Surface infiltration coefficient

Surface type	Infiltration coefficient
Sprayed seal	0.2–0.25
Asphalt	0.2–0.4
Cement concrete	0.3–0.4
Unsealed shoulders	0.4–0.6

Once the infiltration rate is estimated (in m/s) and the coefficient of permeability has been determined by laboratory testing, the quantity of water entering the road or the inflow is determined by applying Darcy's law (**Equation (13.164)**), where:

q = Quantity of water entering the surface (m^3/s)

k = Coefficient of permeability, or infiltration rate (m/s)

A = Area of pavement (taken as one square metre in this application)

i =Hydraulic gradient, i.e., head of water divided by the length of drainage path (dimensionless)

A hydraulic gradient of unity is suggested for rain falling on a surface. With a hydraulic gradient of unity, the inflow as calculated from **Equation (13.165)** is equal to the infiltration rate multiplied by the surface area of the pavement.

13.9.10.6 DESIGN OF TRANSVERSE DRAINS (HERRINGBONE SYSTEM)

A. Spacing of laterals

For the general case the spacing of laterals may be determined with the aid of **Table 13.68**, which is self-explanatory.

Table 13.68 Recommended depth and spacing of laterals for different types of soil

<table>
<tr><th rowspan="2">Soil classification</th><th colspan="3">Soil composition</th><th colspan="4">Spacing of laterals (m)</th></tr>
<tr><th>% sand</th><th>% silt</th><th>% clay</th><th>0.9 m deep</th><th>1.2 m deep</th><th>1.5 m deep</th><th>1.8 m deep</th></tr>
<tr><td>Clean sand</td><td>80-100</td><td>0-20</td><td rowspan="3">0-20</td><td>33-45</td><td>45-60</td><td></td><td></td></tr>
<tr><td>Sandy loam</td><td>50-80</td><td>0-50</td><td>15-30</td><td>30-45</td><td></td><td></td></tr>
<tr><td>Loam</td><td>30-50</td><td>30-50</td><td>9-18</td><td>12-25</td><td>15-30</td><td>18-36</td></tr>
<tr><td>Clayey loam</td><td>20-50</td><td>20-50</td><td>20-30</td><td>6-12</td><td>8-15</td><td>9-18</td><td>12-25</td></tr>
<tr><td>Sandy clay</td><td>50-70</td><td>0-20</td><td rowspan="2">30-50</td><td>4-9</td><td>6-12</td><td>8-15</td><td>9-18</td></tr>
<tr><td>Silt/clay</td><td>0-20</td><td>50-70</td><td>3-8</td><td>4-9</td><td>6-12</td><td>8-16</td></tr>
<tr><td>Clay</td><td>0-50</td><td>0-50</td><td>30-100</td><td>4 max</td><td>6 max</td><td>8 max</td><td>12 max</td></tr>
</table>

In more complex cases, and if feasible, the spacing of the laterals may be designed according to the methods for steady or unsteady groundwater flow.

B. Design of laterals

The diameter of laterals may be read off **Figure 13.149**. However, if the drainage rate is known, the area that

could be drained by a pipe of a certain diameter may be calculated by means of **Equation (13.166)**.

$$A = \frac{(26.92 \times 10^6) d^{\frac{8}{3}} S_0^{\frac{1}{2}}}{nq}(0.7) \qquad (for\ 70\%\ of\ full\ capacity) \qquad (13.166)$$

Where:

S = spacing (m)
A = surface area (m) = S (L+0.5 S)
d = diameter of pipe (m)
L = length of the pipe (m)
q = drainage rate (mm/day)
n = Manning's n (s/m$^{1/3}$)
S_o = slope of the pipe (m/m)

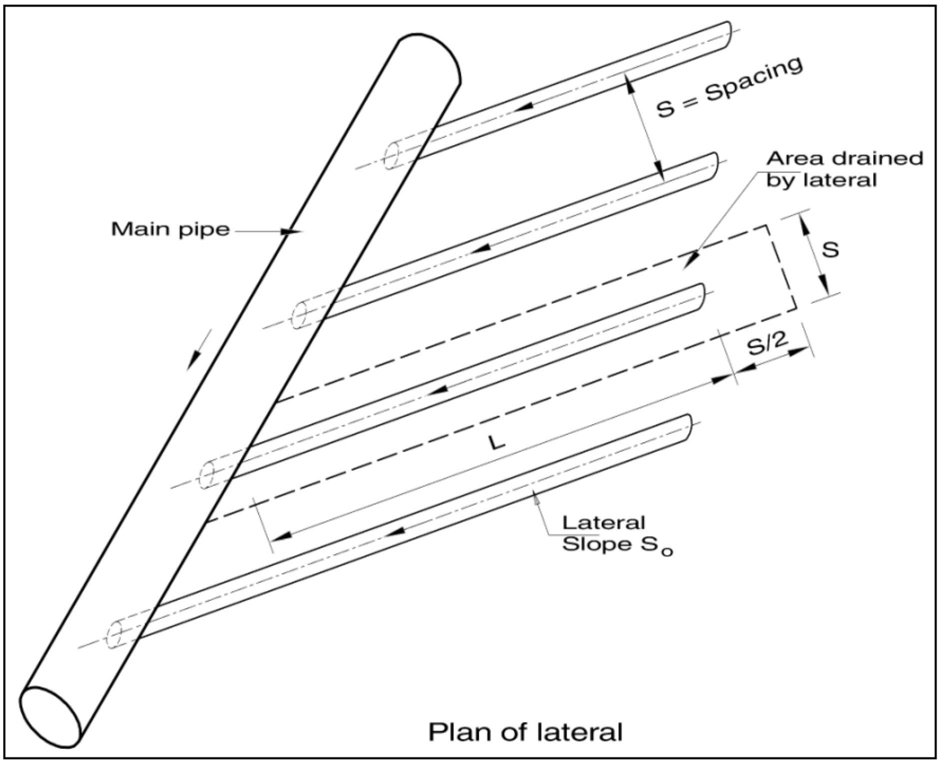


Figure 13.149 General view of a herringbone drainage system

C. Main drainage pipes

The capacity of the main pipe is not the sum of the capacities of the lateral pipes, unless non-porous pipes are used. The surface drained by a pipe of a certain diameter is calculated in the same way as for lateral pipes.

13.9.10.7 CAPILLARY RISE IN SOILS

Where a shallow formation is proposed over saturated ground, or fine-grained embankment material is used to cross swamps, the height of capillary rise of the groundwater should be calculated to ensure that excess water does not enter the pavement. The rise in capillary water can be calculated using **Equation (13.167)** and **Equation (13.168)**.

$$h_c = \frac{10C}{eD_{10}} \qquad (13.167)$$

Where,

h_c = Capillary rise (mm)

C =An empirical constant that depends on the shape of the grains and varies from 0.1 to 0.5 cm^2 (for perfect spheres, C = 0.1 cm^2)

eD_{10} =Allen Hazen's effective grain size, based on the sieve opening in cm that 10% of the material passes. The value is obtained from the grading curve.

$$e\ (void\ ratio) = \frac{V_v}{V - V_v} \tag{13.168}$$

Where,

V = Total volume (units)

V_v = Total volume of voids (units)

13.9.10.8 DESIGN OF EDGE DRAIN COLLECTOR SYSTEM WITH OUTLET PIPE

Edge drains are perhaps the most effective subsurface drainage systems for removing water infiltrating joints and cracks in PCC pavements (Jeffcoat et al., 1992). Since the effectiveness of any system can be highly site specific, it is essential that careful evaluation of site conditions be carried out when considering retrofitting edge drains because addition of edge drains in areas with highly erodible subgrade or base material may accelerate erosion problems. This is due to the fact that the fines can be lost through the edge drains (Gulden, 1983).

The longitudinal edge drains, when used in existing pavements, just like those installed during initial construction, can be grouped into three basic types known as pipe edge drains, PGED or “fin drains,” and aggregate trenches or “French drains.” Refer to **Figure 13.150** and **Figure 13.151**.

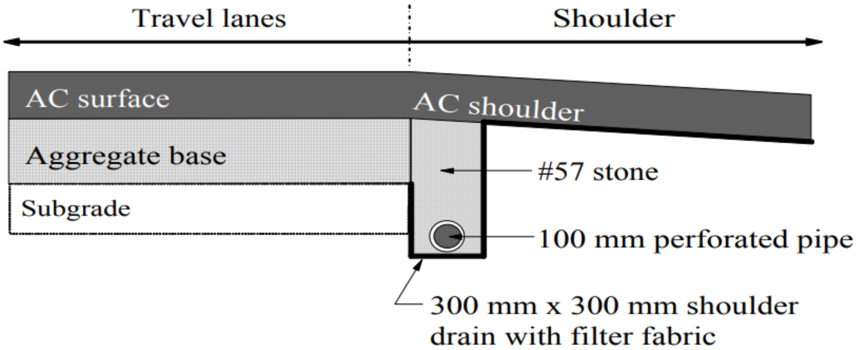


Figure 13.150 Typical AC pavement with pipe edge drains

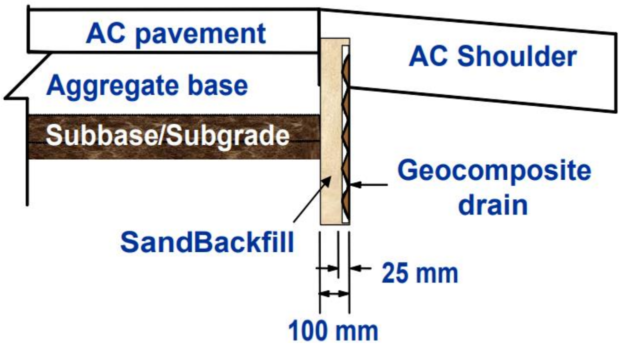


Figure 13.151 Typical AC pavement with geocomposite edgedrains

Providing an AC layer of adequate thickness above the permeable base is essential for obtaining good performance (Yu et al., 1998b). Dense-graded bases that are daylighted have been determined not suitable for providing drainage to newly constructed or reconstructed AC pavements. However, daylighting of the dense-graded bases will provide positive drainage and would hence be far superior to bathtub design (Kersten and Skok, 1968).

Another advantage of Dense-Graded Aggregate Base (DGAB) is that a daylighted permeable base is able to breathe, thus preventing build-up of water vapor pressures under the AC surface from hydrogenesis (Fehsenfeld, 1988). Asphalt concrete pavements with granular bases are particularly susceptible to hydrogenesis, which can lead to stripping (Hindermann, 1968).

13.9.10.8.1 EDGE DRAIN CAPACITY AND OUTLET SPACING

The goal of installing subsurface drainage systems in pavement structures is to remove water entering the base and subgrade layers as quickly as possible. It is imperative that the edge drain capacity should be designed so as not to be an impediment to the removal. A common recommendation is that the capacity of the edge drain system should always increase as the water flows through the system (FHWA, 1992). This would be accomplished if the combination of edge drain capacity and outlet spacing are adequate to handle the design flows.

The required pipe capacity and outlet spacing can be determined by one of three design approaches (FHWA, 1992). These are:

i. the pavement infiltration discharge rate (q_i)
ii. permeable base discharge rate, and
iii. time-to-drain discharge rate.

The engineer needs to select the design approach that meets the field conditions. The design pipe flow for this approach is determined by **Equation (13.169)**.

$$Q_p = q_i W L \tag{13.169}$$

Where,

Q = Discharge flow rate for pipe flow, m^3/day

q_i = Pavement infiltration, $m^3/day/m^2$

W = Width of permeable base, m

L = Outlet spacing, m

To determine the required pipe flow, the design discharge rate from the permeable base need to be adjusted. The resulting equation is (**Equation (13.170)**):

$$Q_p = kS_R HL\cos(A) \tag{13.170}$$

Where:

Q_p = Design flow rate for pipe flow, m^3/day
k = Coefficient of permeability, m/day
S_R = Resultant slope, m/m
H = Thickness of base, m
L = Outlet spacing, m
A = Angle between roadway cross slope and resultant slope.

In the time to drain discharge rate approach, the edge drain system is required to be capable of handling the flow generated by draining the permeable base. The pipe flow rate is determined by **Equation (13.171)**:

$$Q_p = (WLHN_eU)\left(\frac{1}{t_D}\right) \times 24 \tag{13.171}$$

Where:

Q_p = Design flow rate for pipe, m^3/day
W = Width of permeable base, m
L = outlet spacing, m
H = Thickness of base, m
N_e = Effective porosity, %
U = Percentage drained, expressed as a decimal
t_D = Drainage time period, hours

13.10 DRAINAGE CHARTS

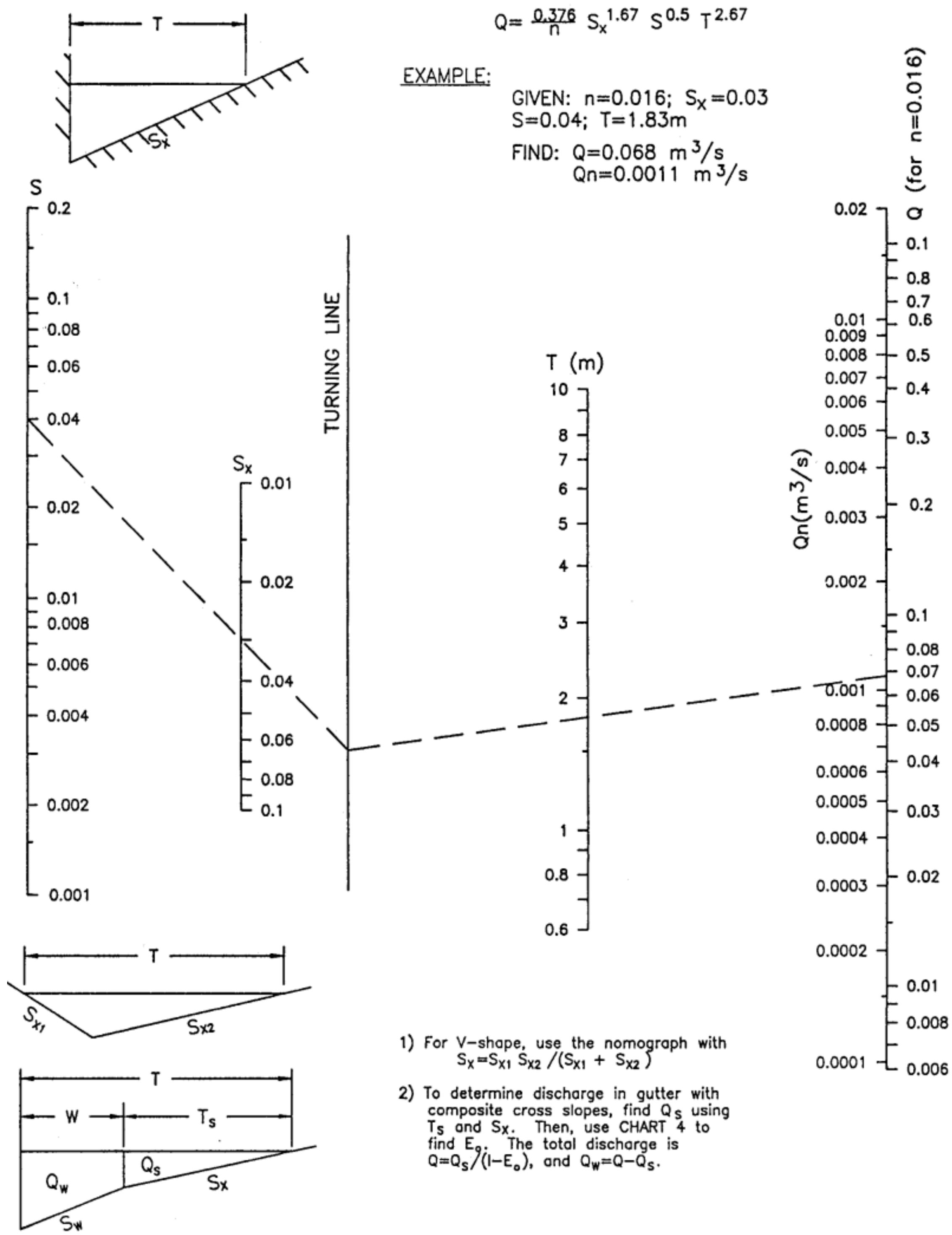


Chart 1 Flow in Triangular Guter Sections

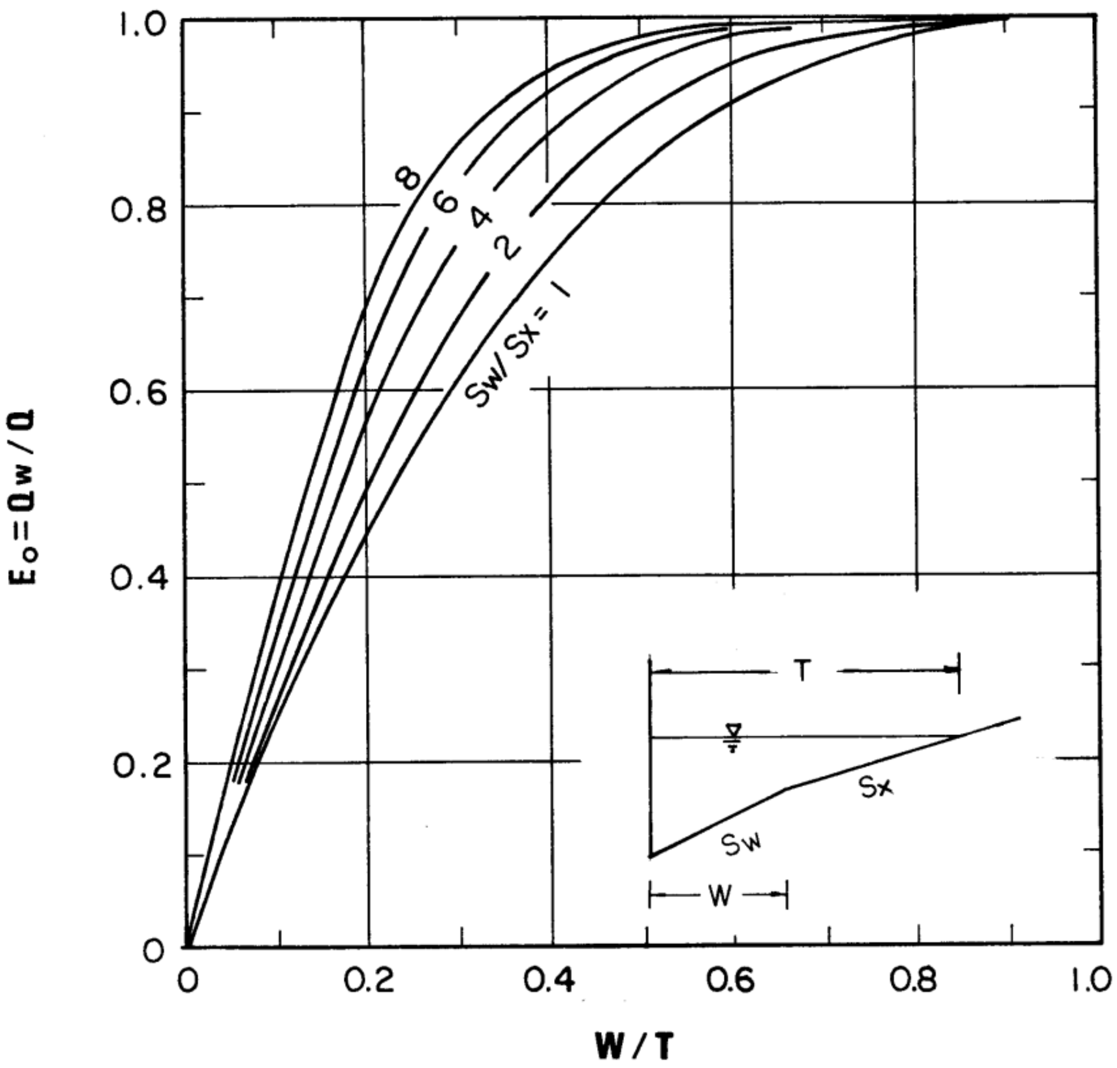


Chart 2 Ratio of frontal flow to total gutter flow

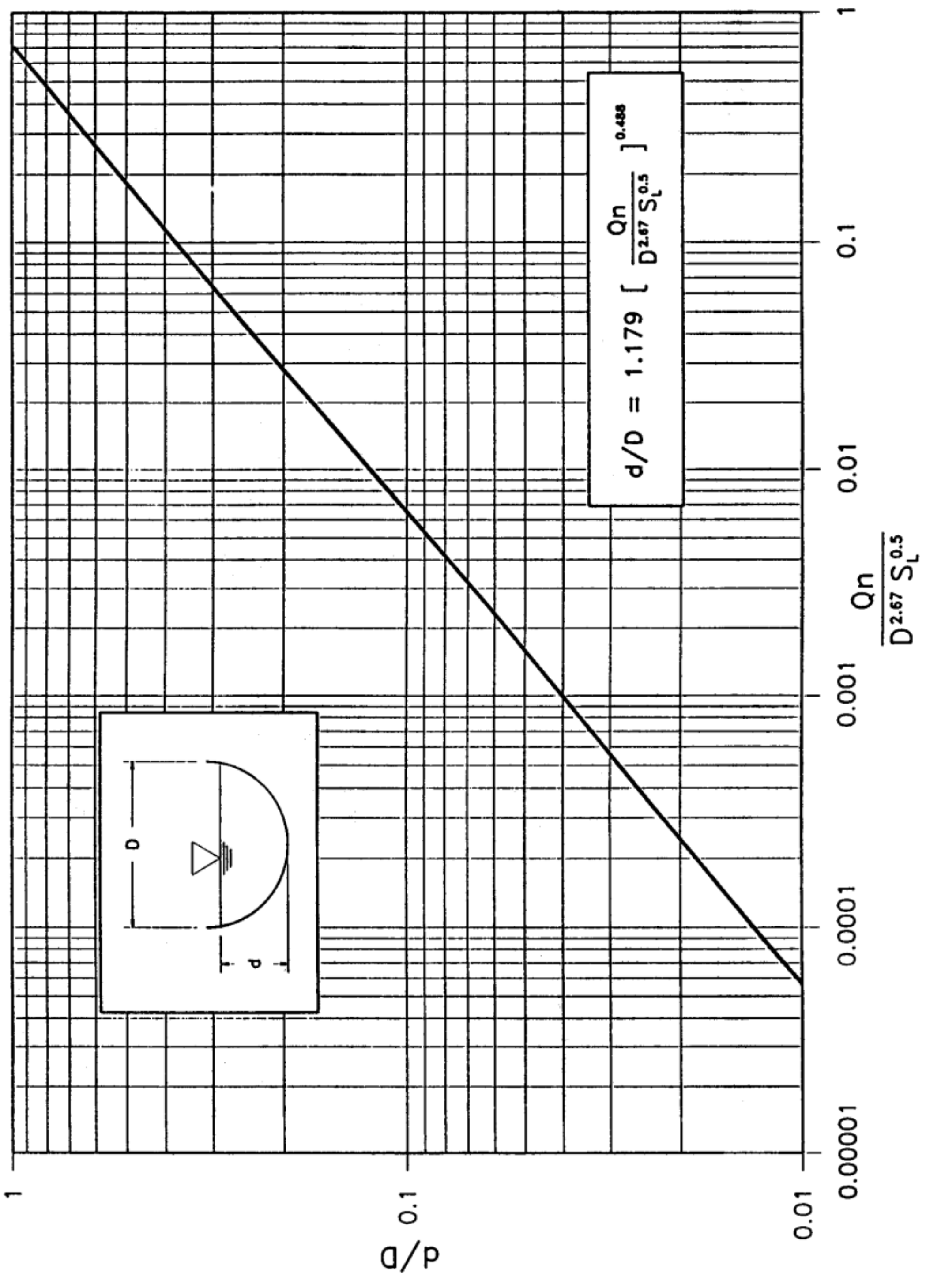


Chart 3 Conveyance in Circular Channels

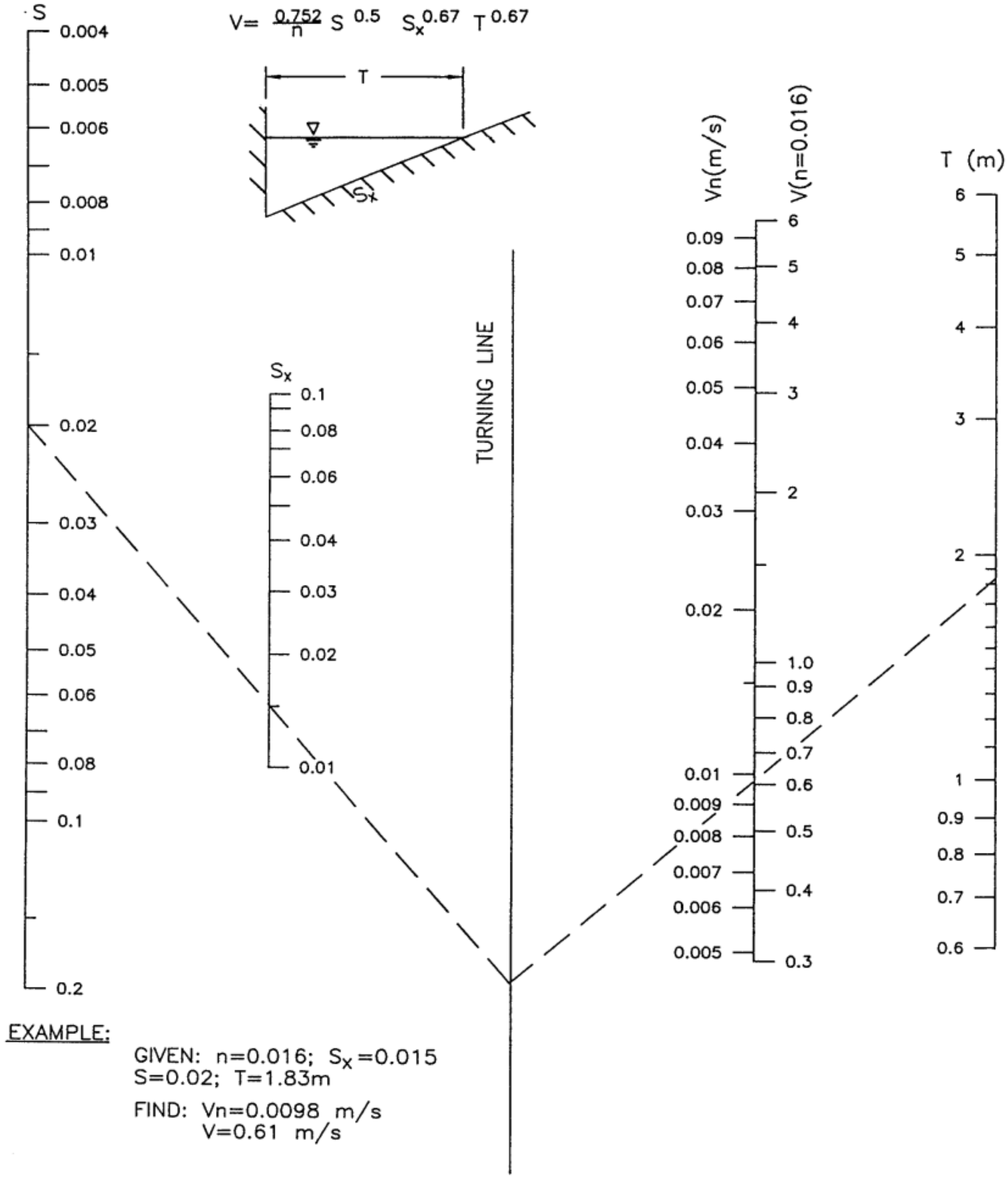


Chart 4 Velocity in Triangular Gutter Sections

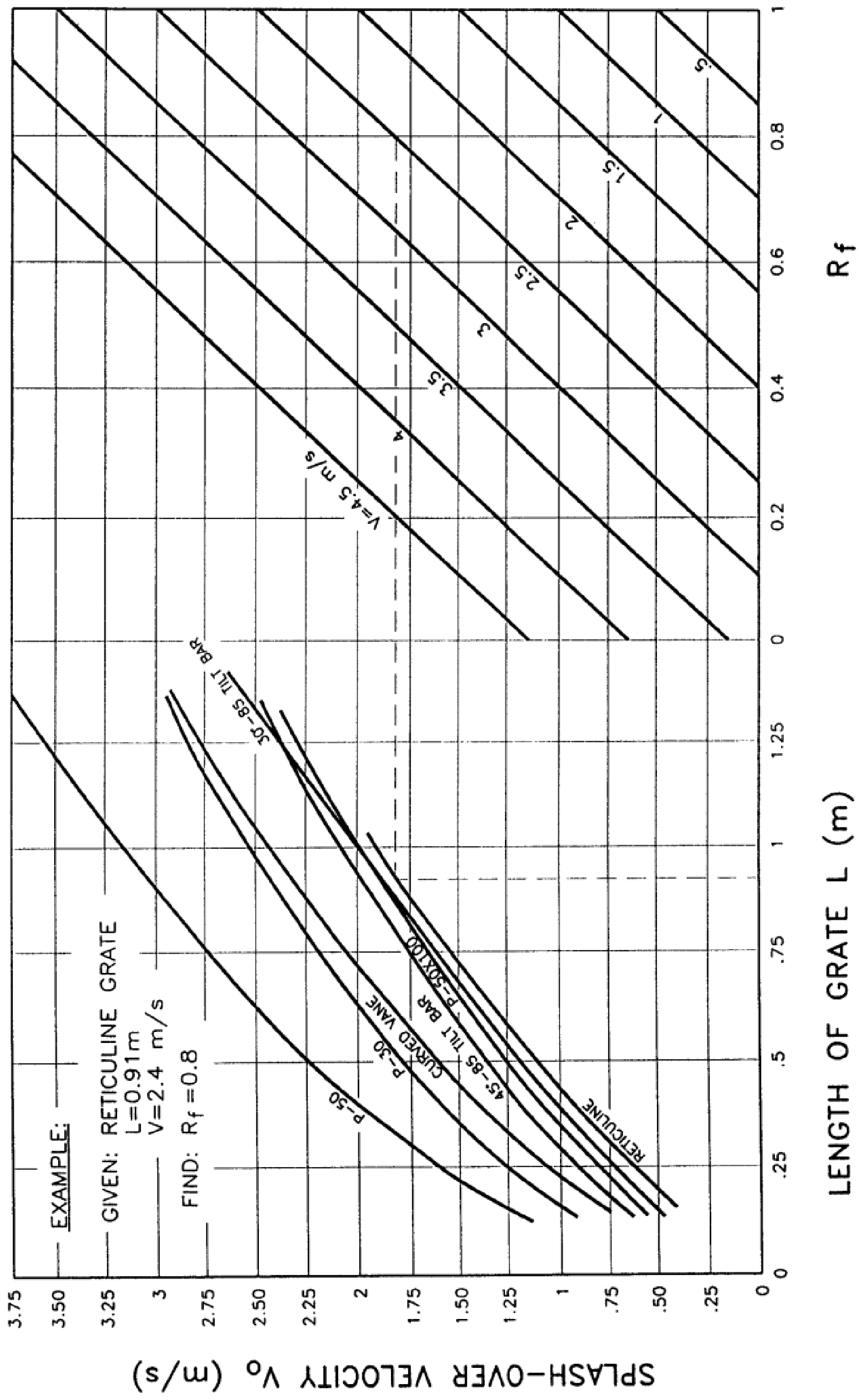


Chart 5 Grate Inlet Frontal Interception Efficiency

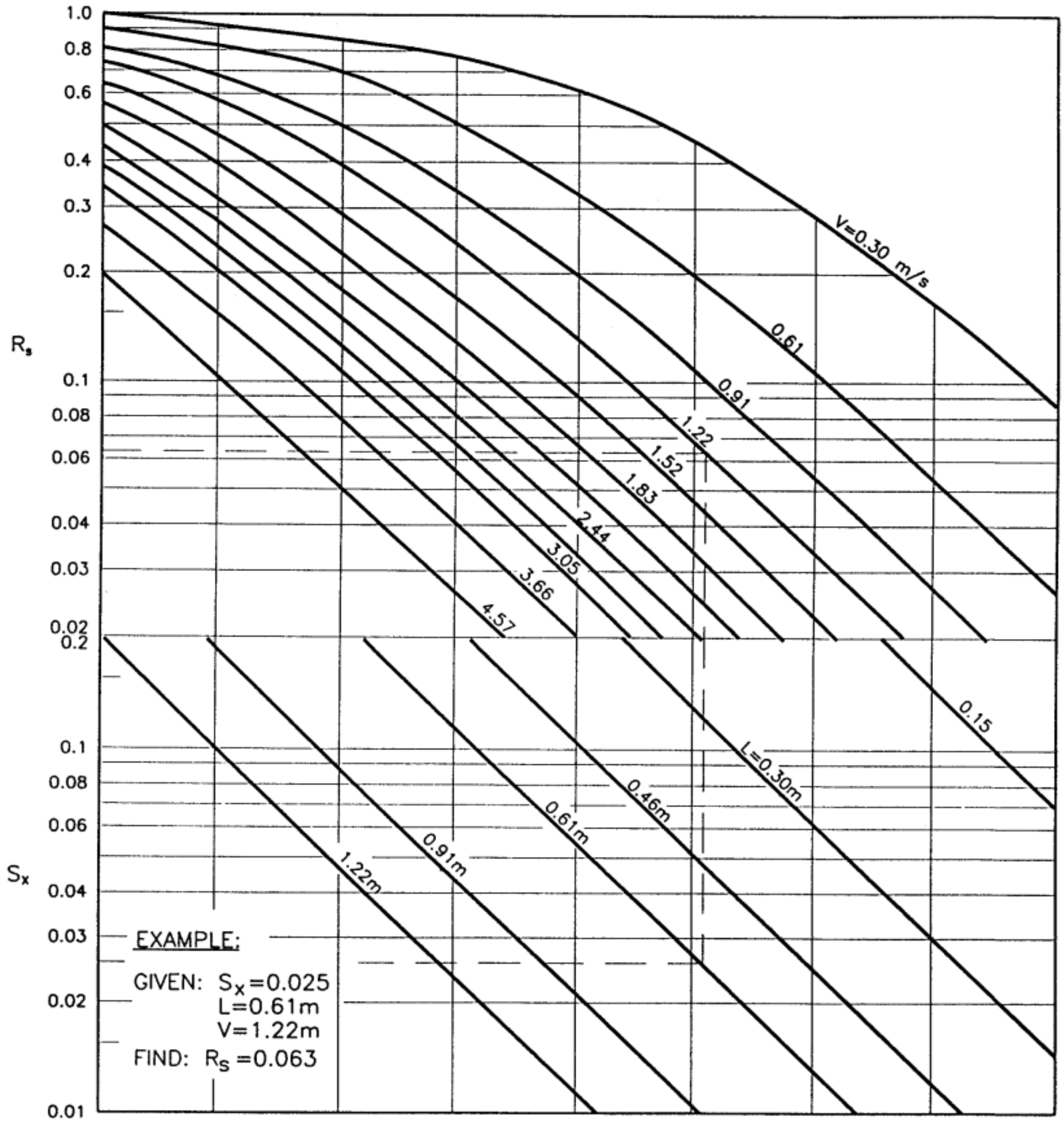


Chart 6 Grate Inlet Side Flow Intercept Efficiency

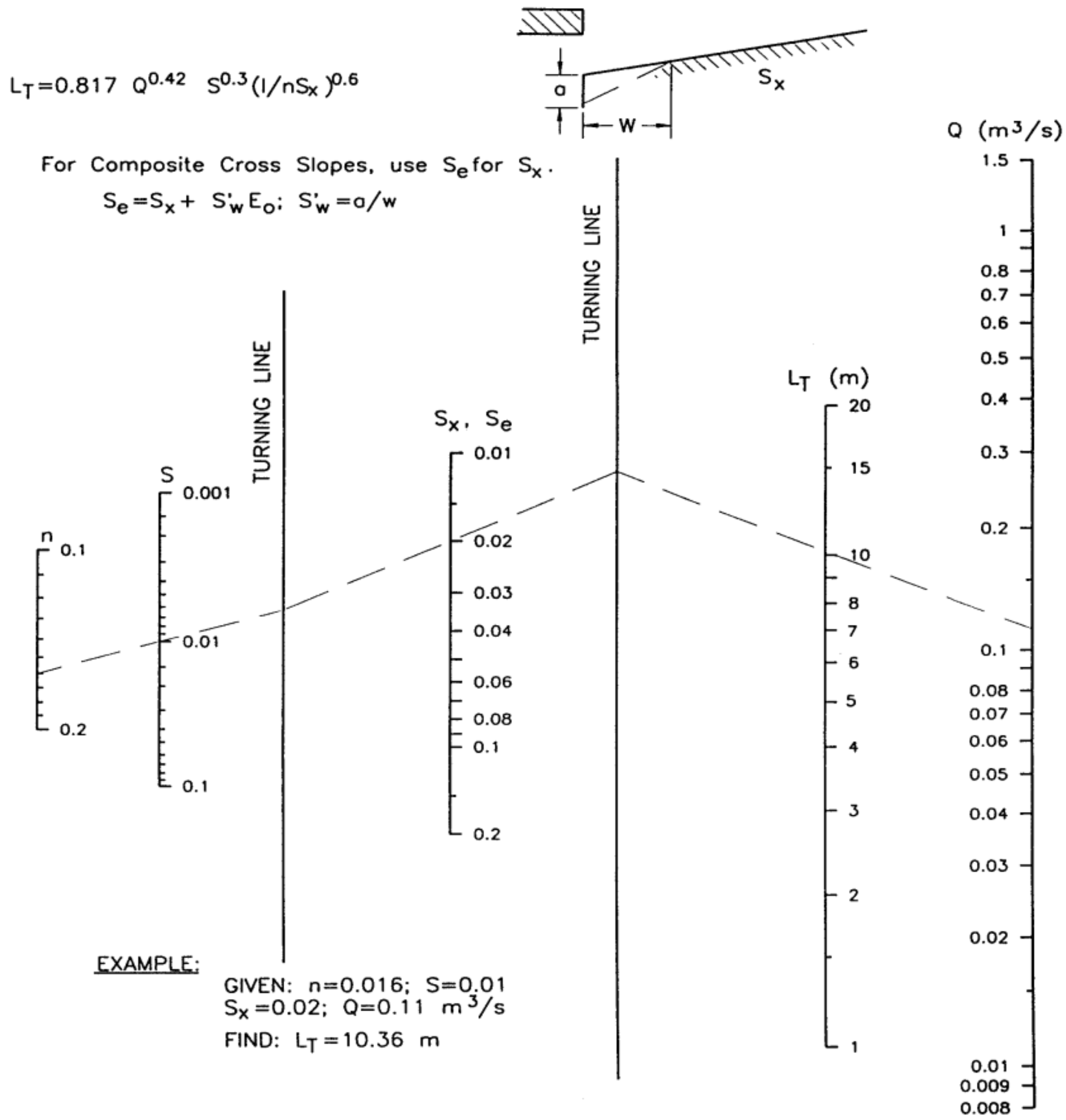


Chart 7 Kerb-opening & Slotted Drain Inlet Length For Total Interception

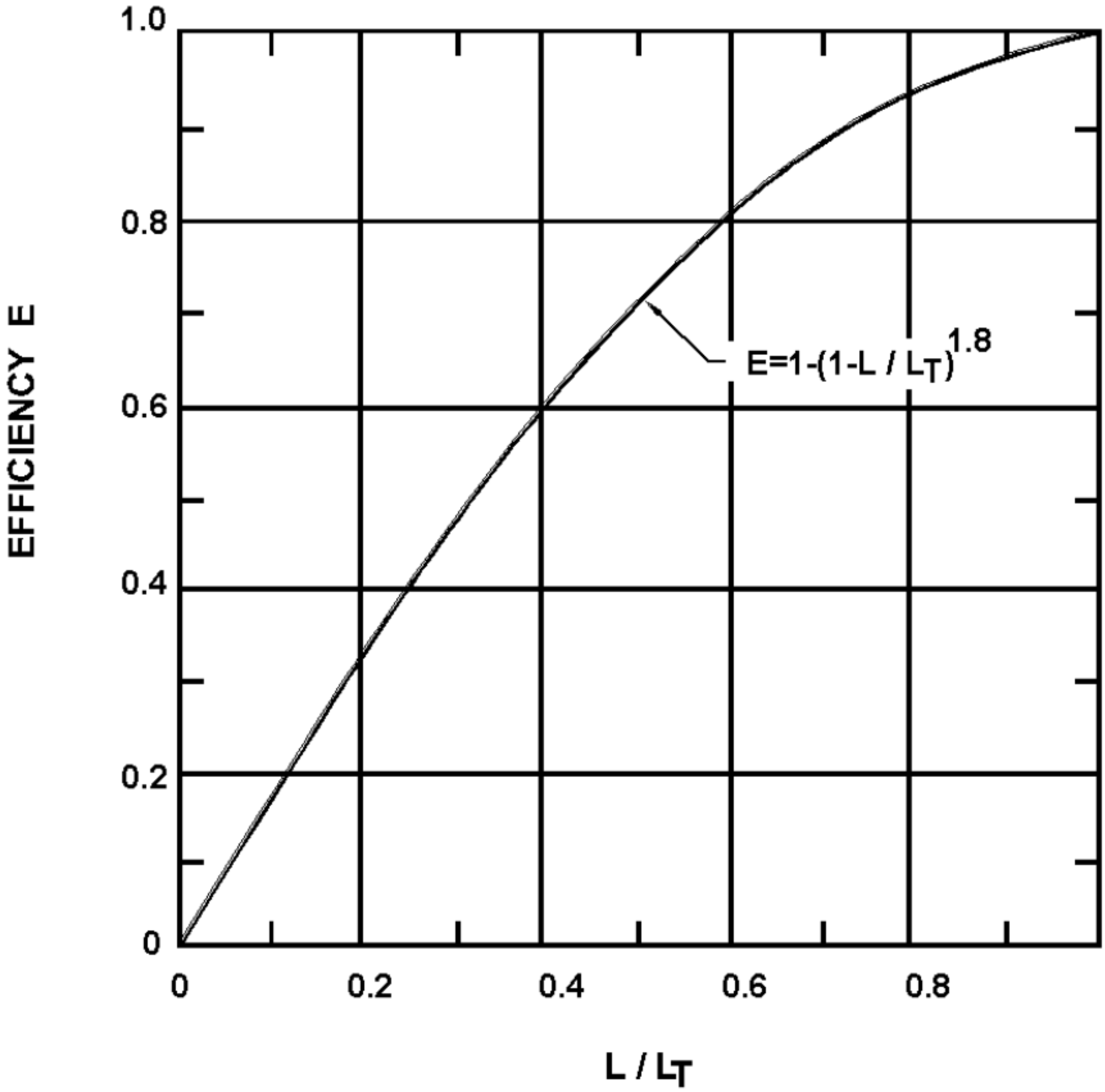


Chart 8 Kerb-opening & Slotted Drain Inlet Interception Efficiency

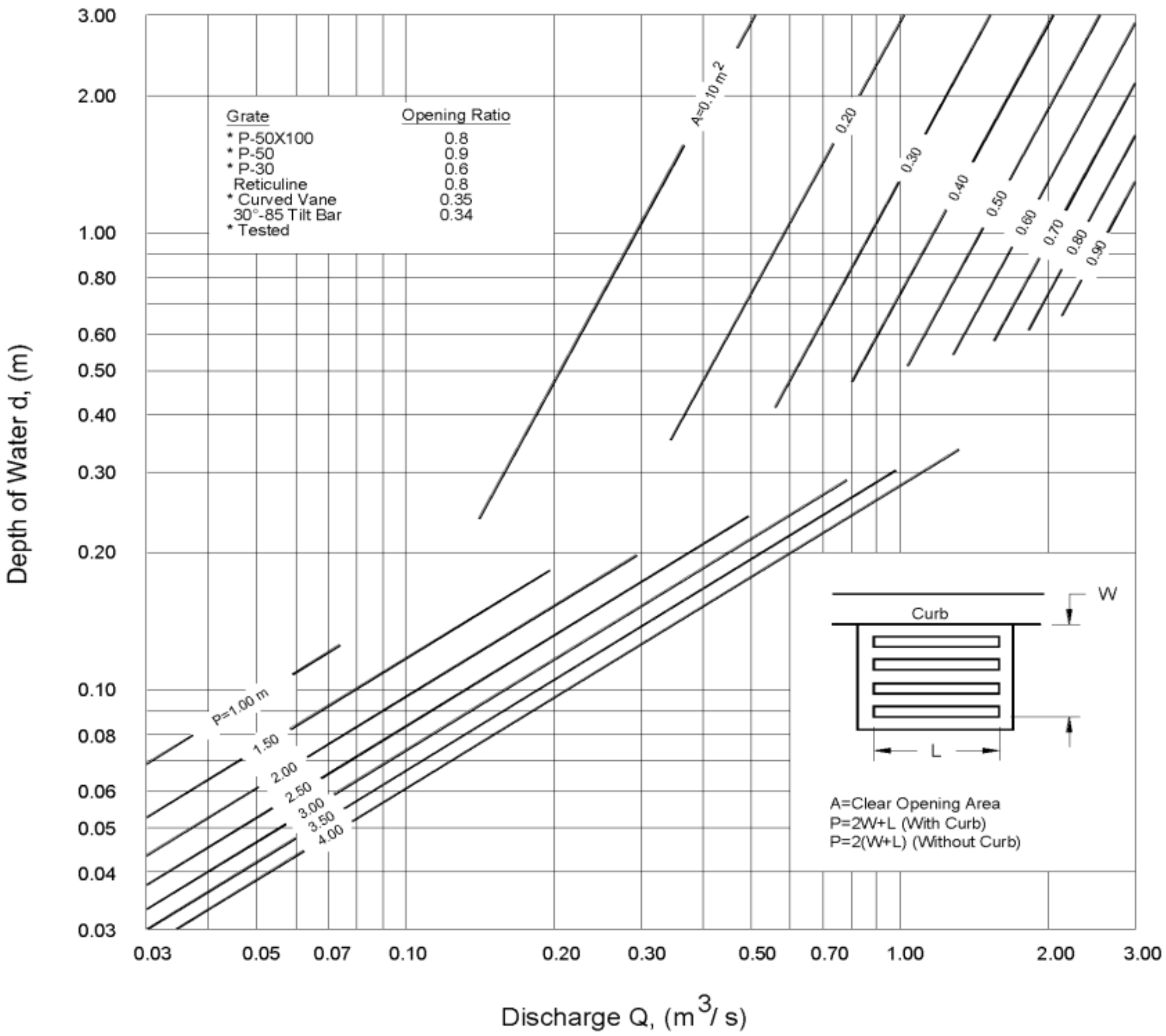


Chart 9 Grate Inlet Capacity In Sump Conditions

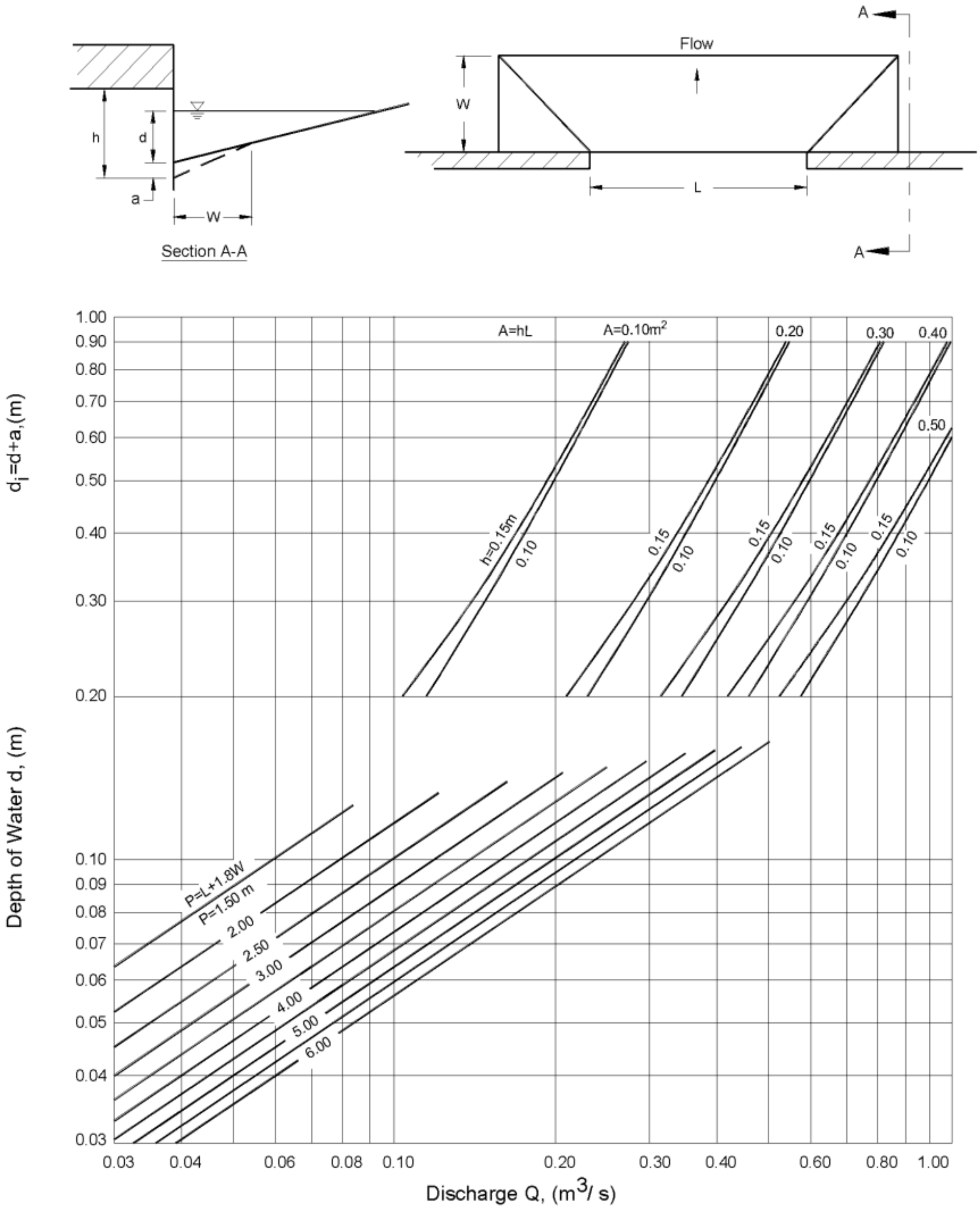


Chart 10 Depressed Kerb-opening Inlet Capacity In Sump Locations

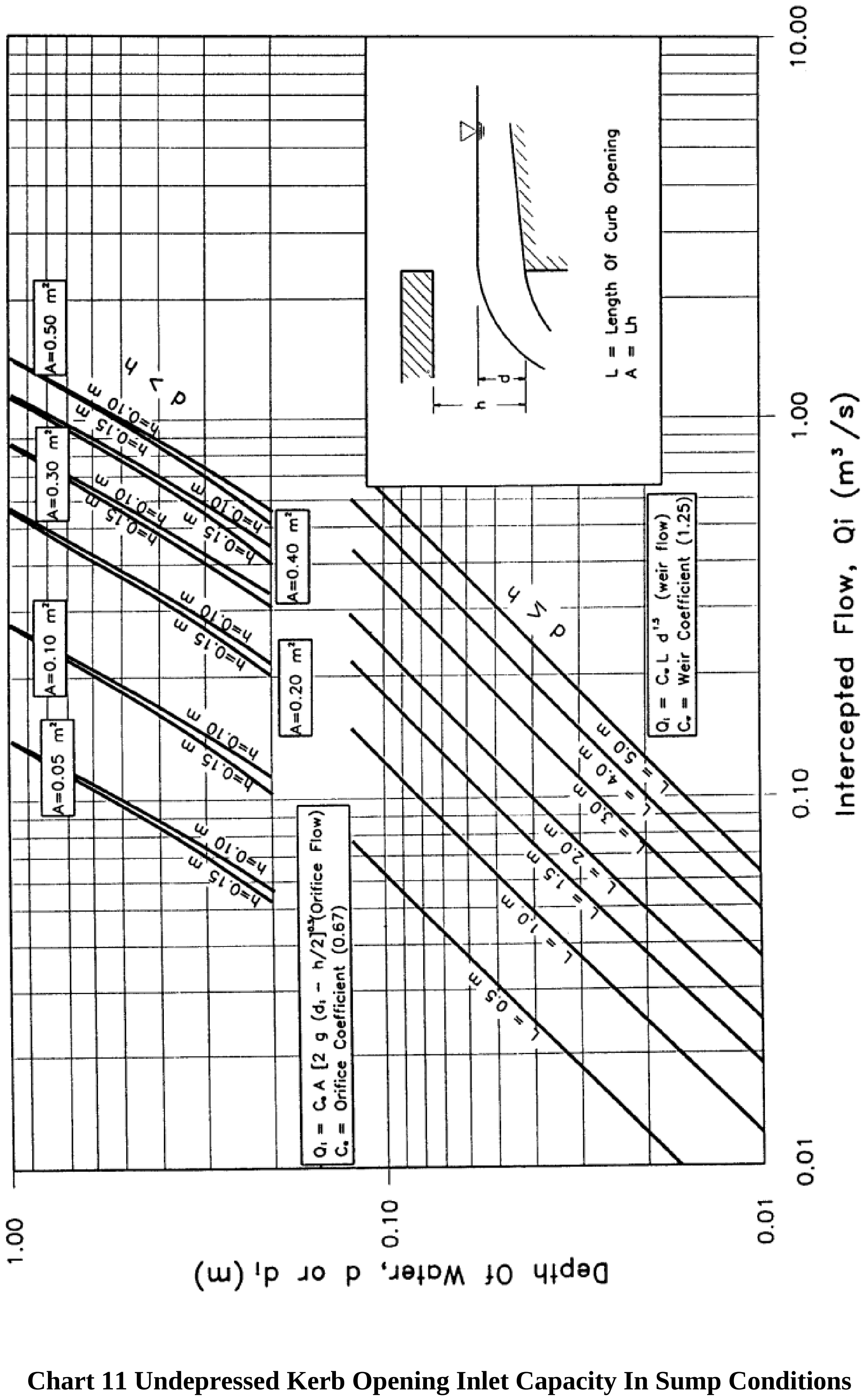


Chart 11 Undepressed Kerb Opening Inlet Capacity In Sump Conditions

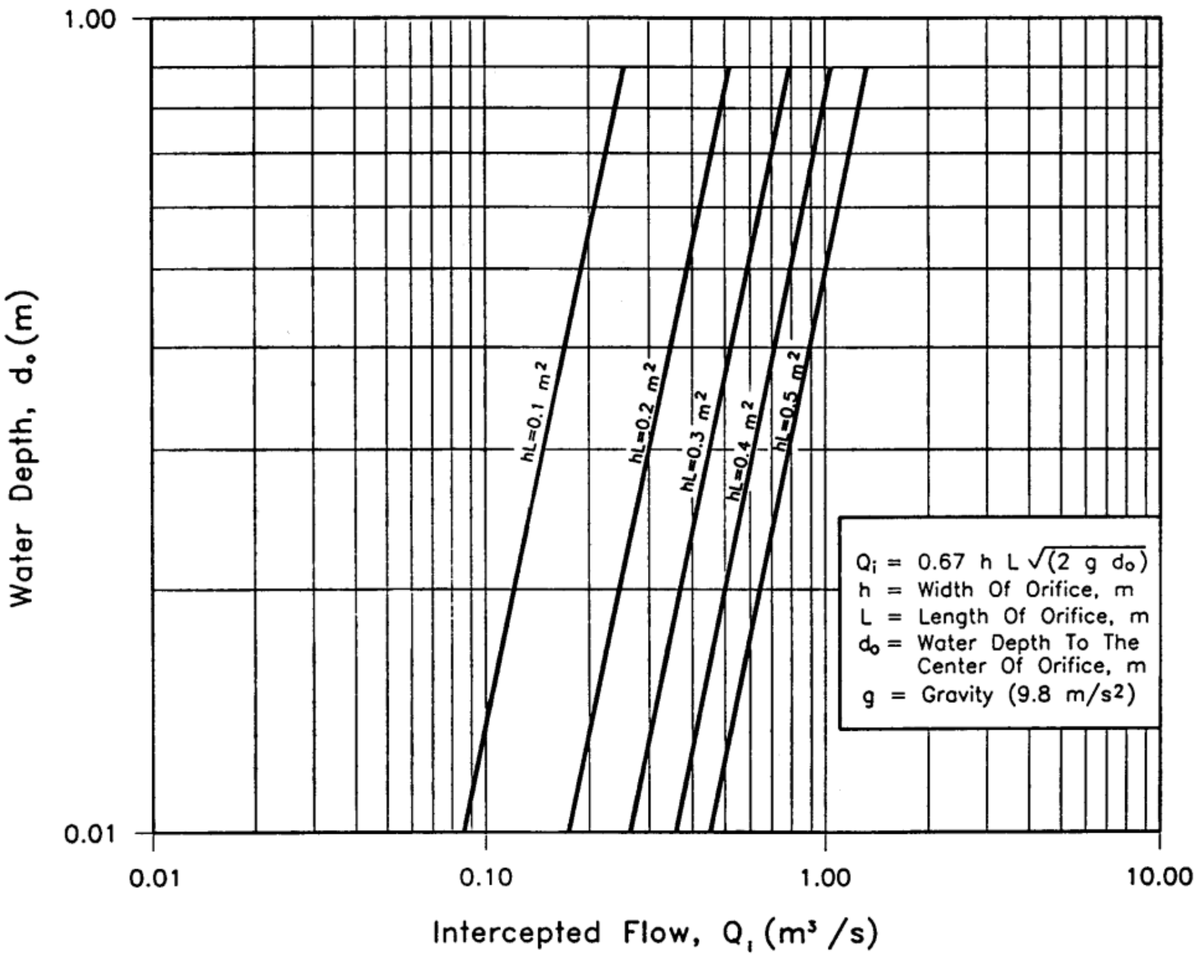


Chart 12 Kerb Opening Inlet Orifice Capacity For Inclined And Vertical Orifice Throats

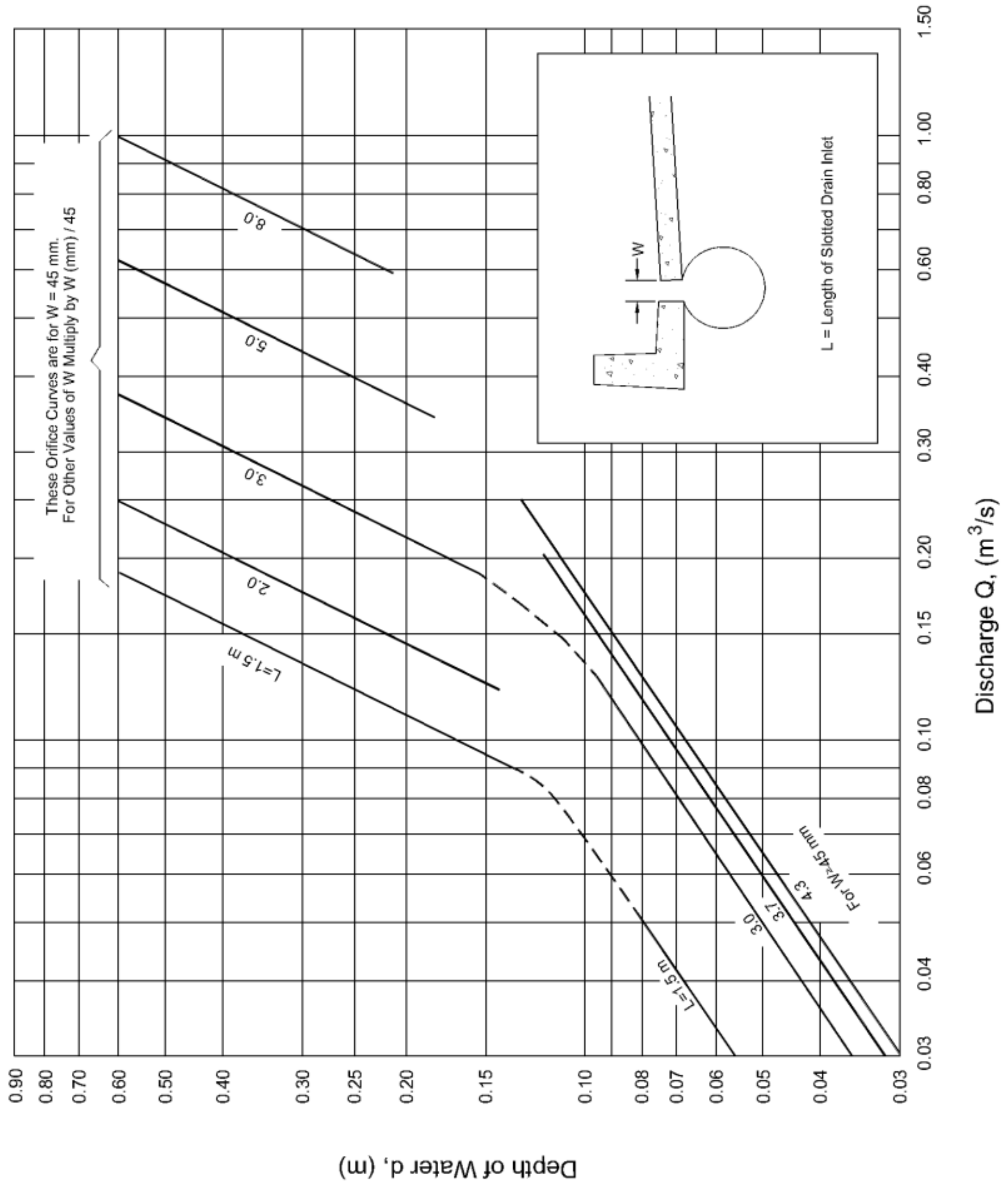


Chart 13 Slotted Drain Capacity In Sump Locations

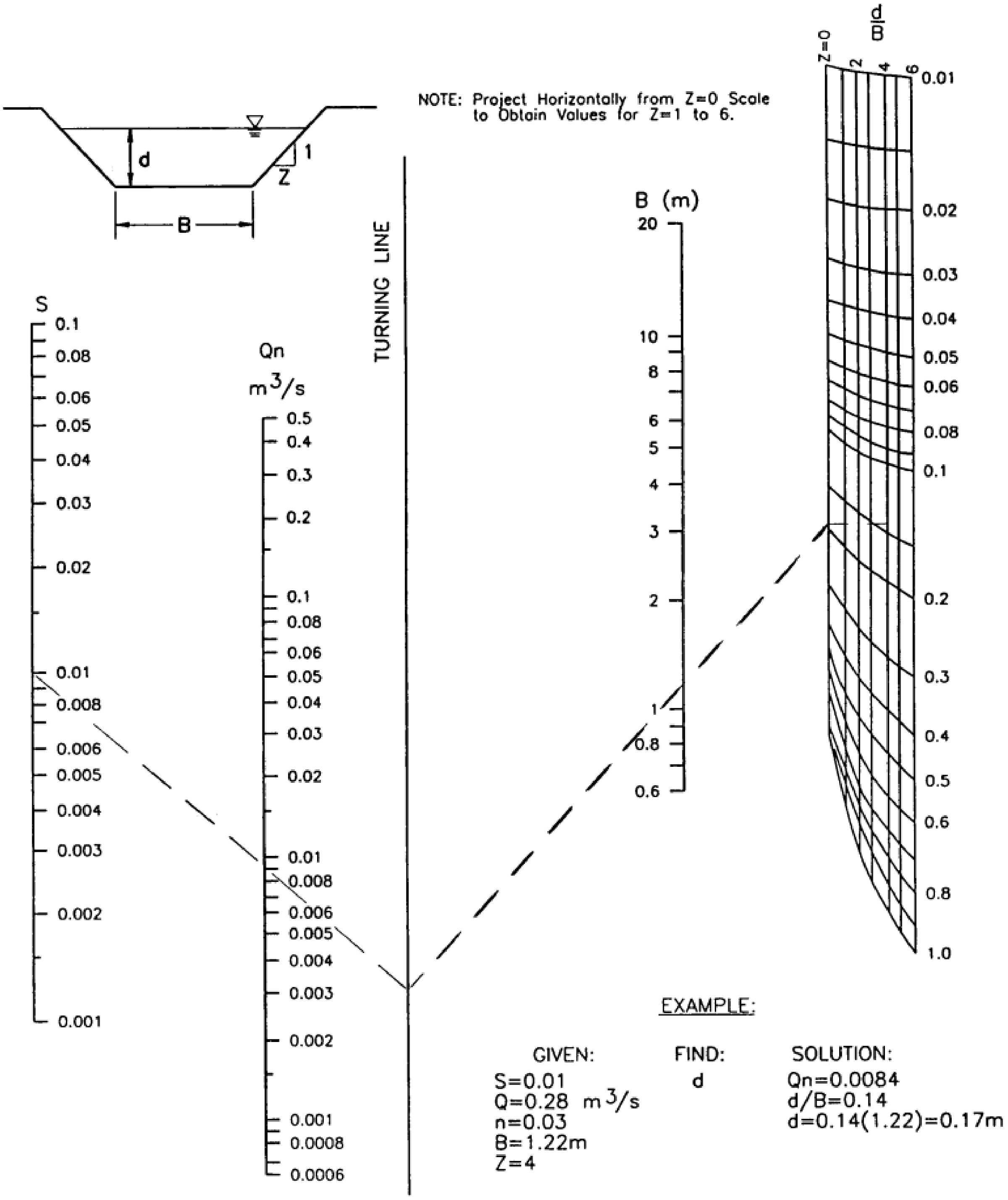


Chart 14 Solution In Manning's Equation For Channels Of Various Slopes

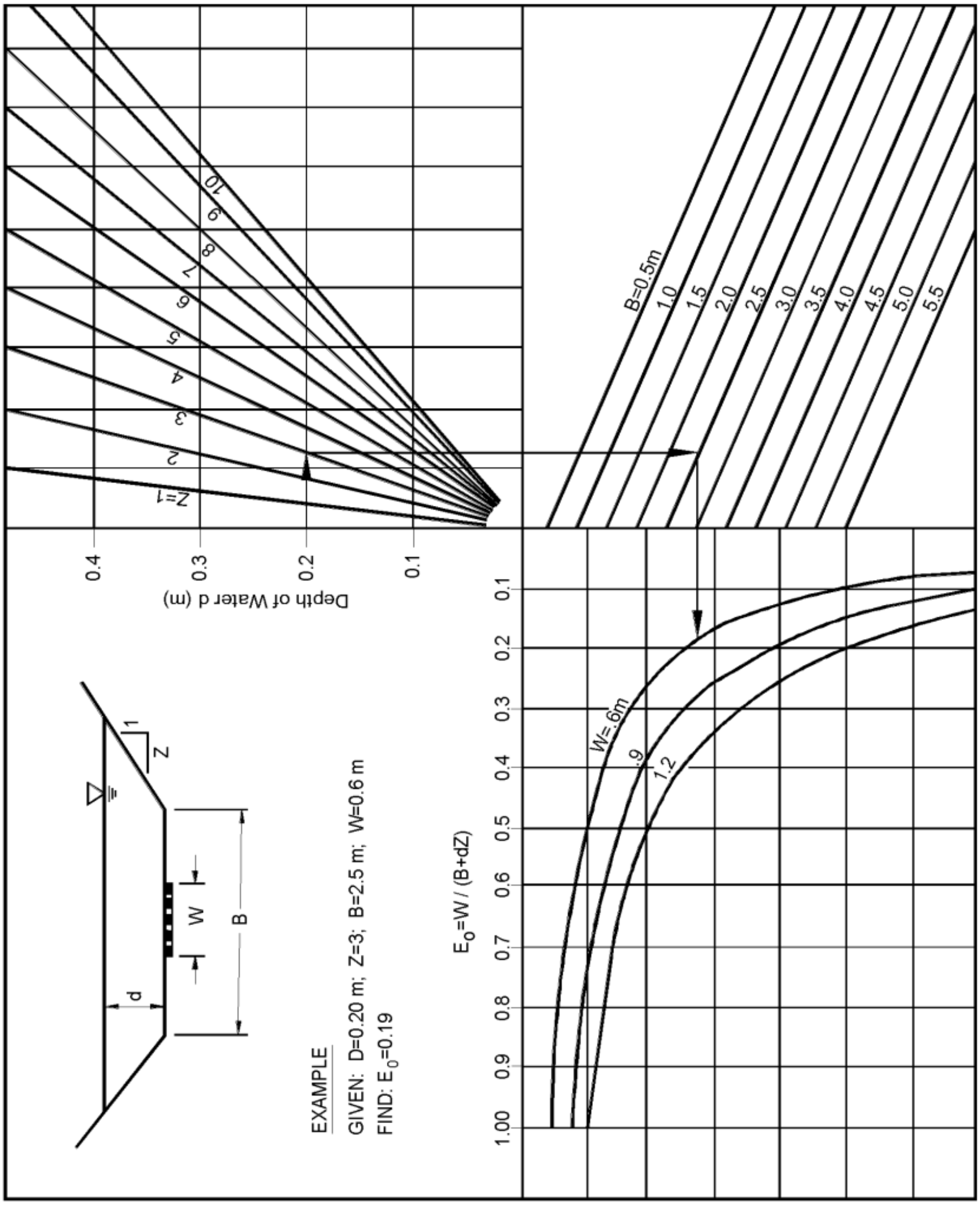


Chart 15 Ratio Of Frontal Flow To Total Flow In A Trapezoidal Channel

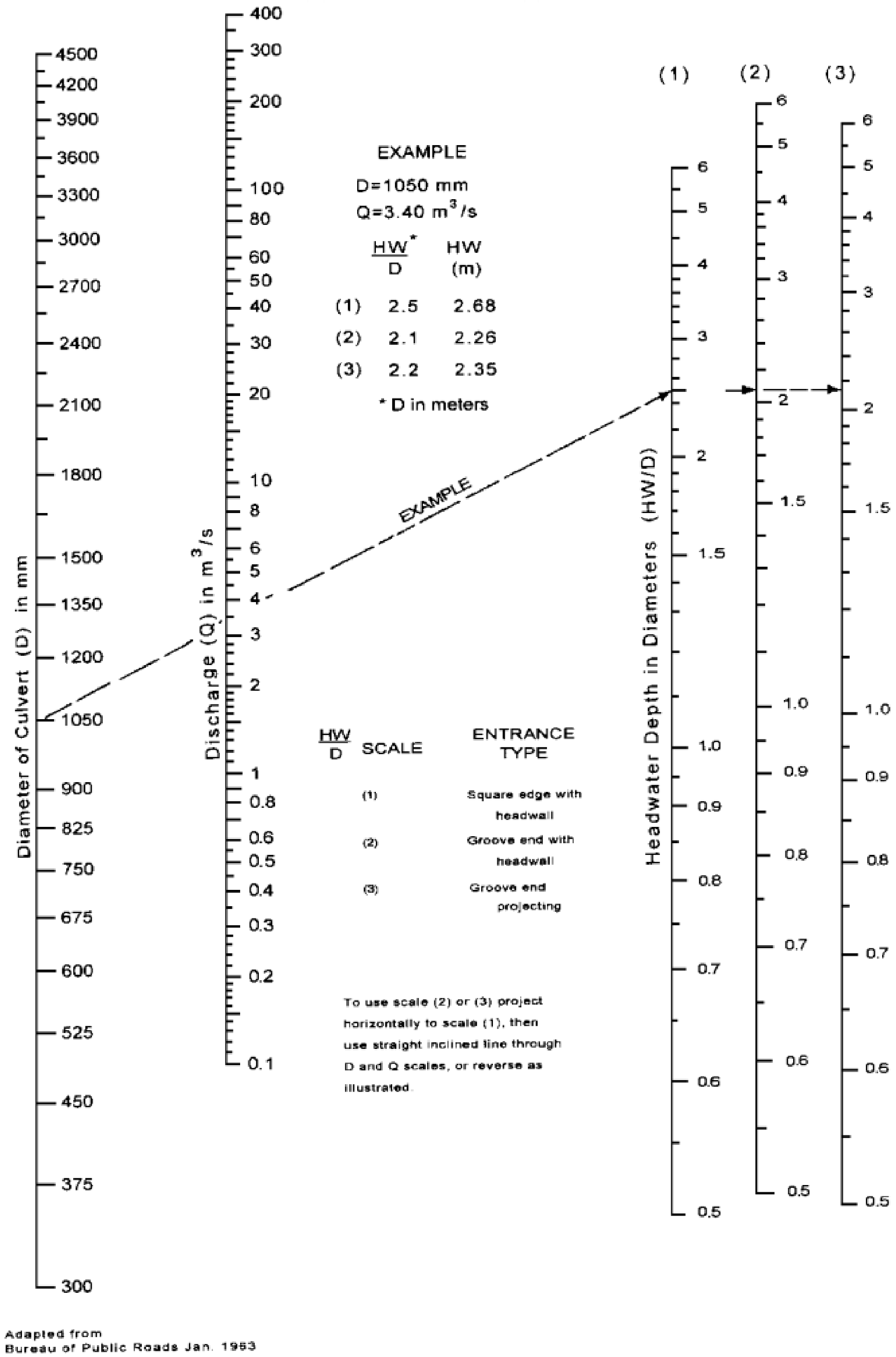
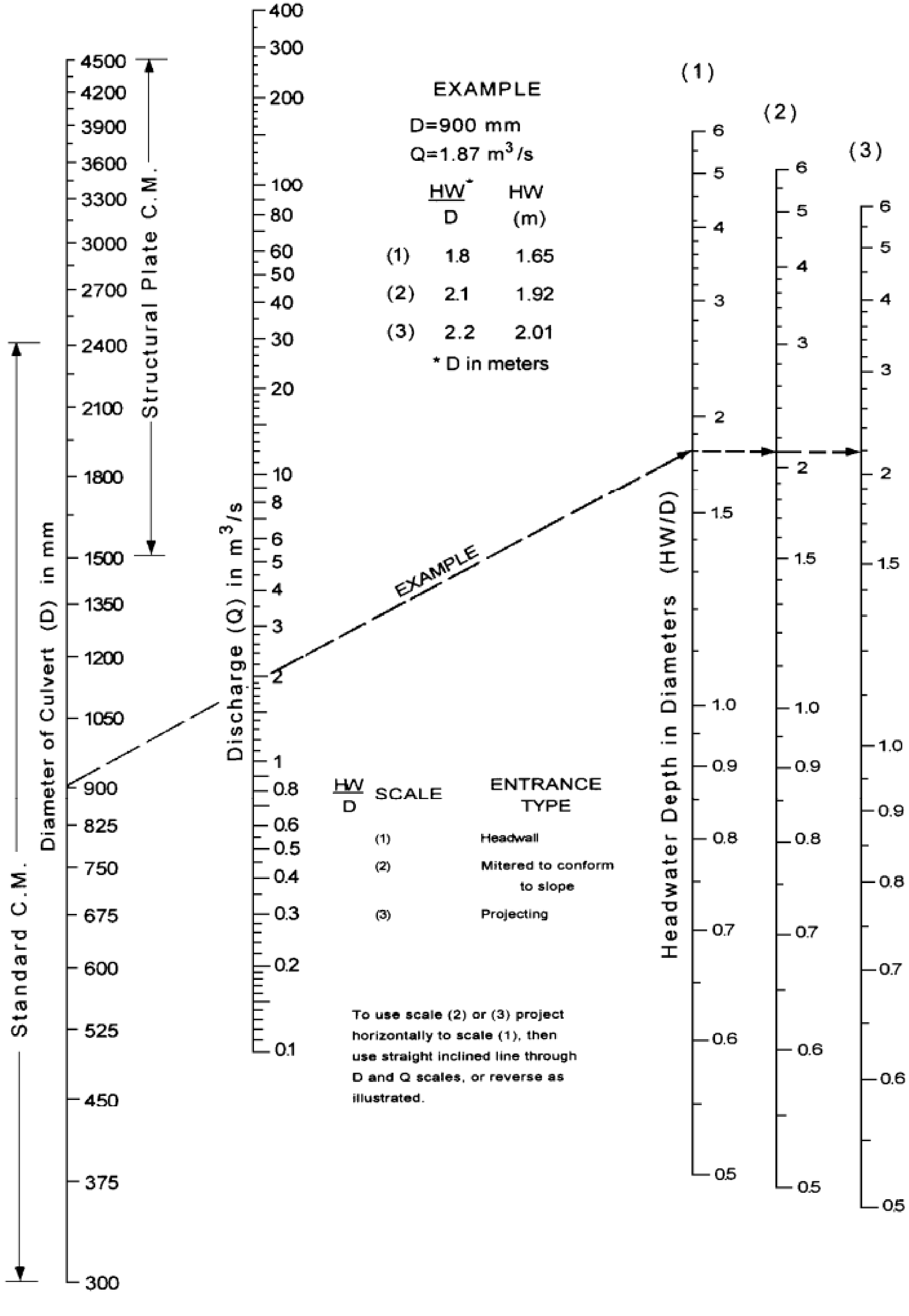
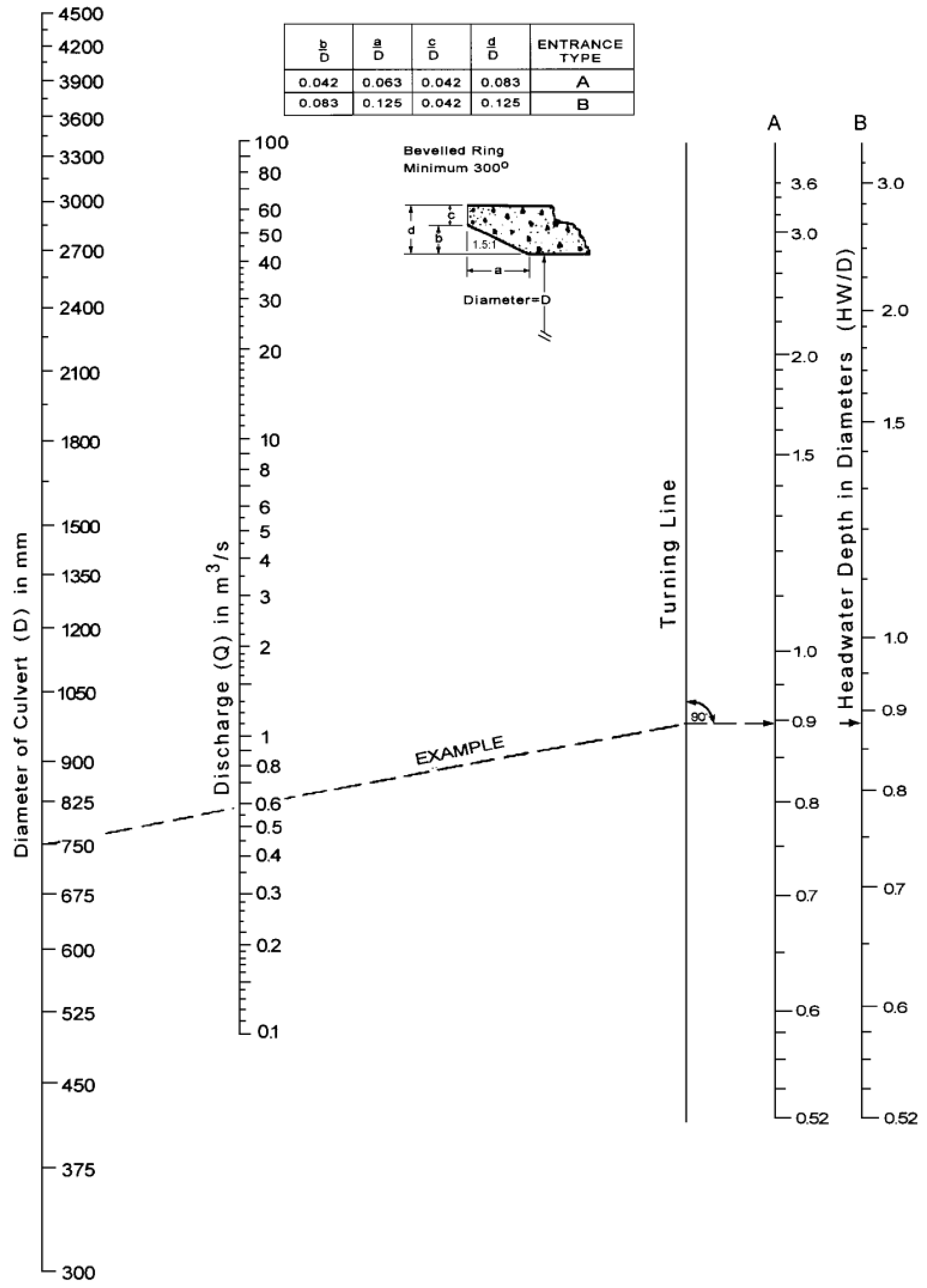


Chart 16 Headwater Depth For Concrete Pipe Culverts With Inlet Control



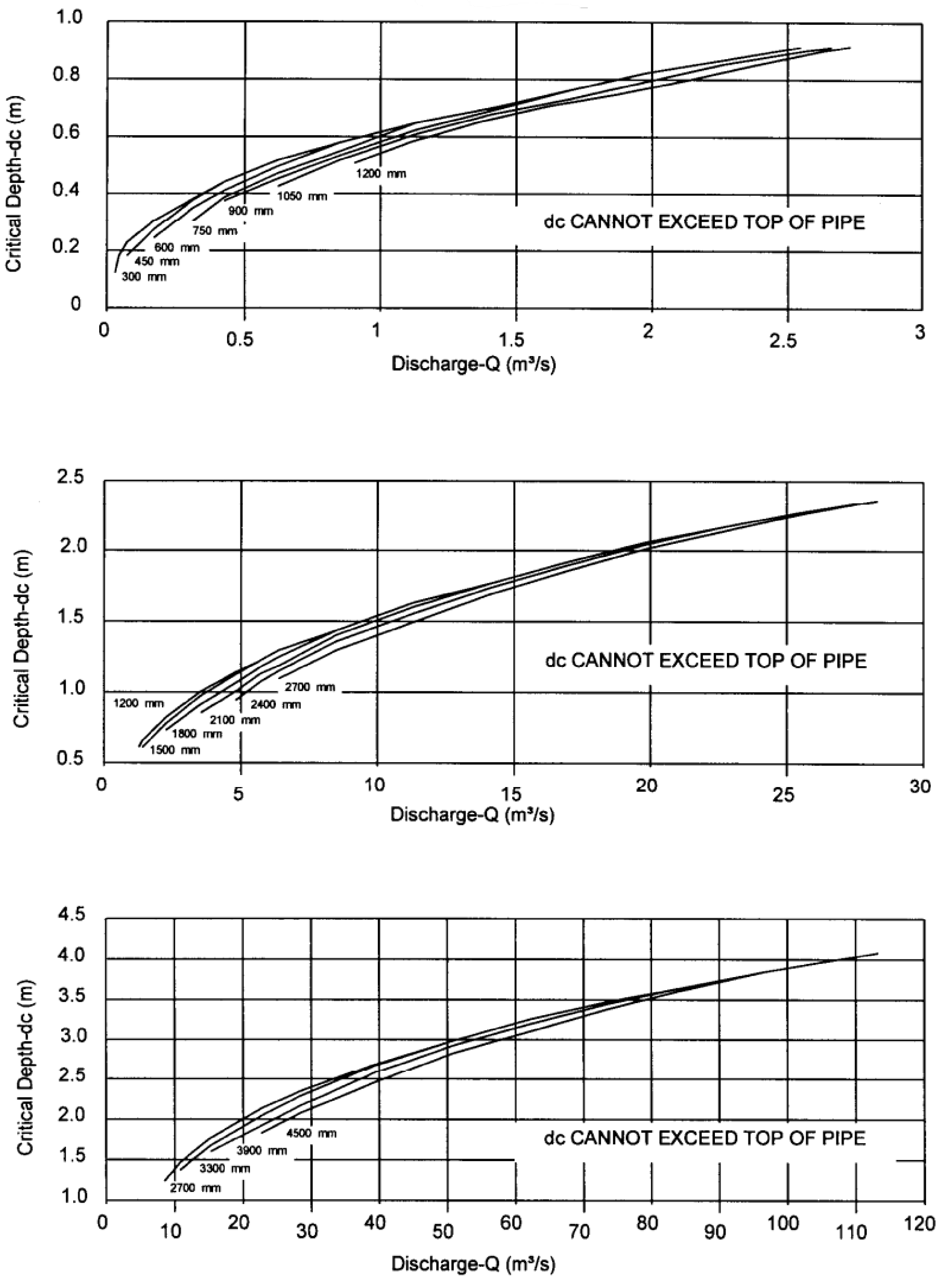
Adapted from
Bureau of Public Roads Jan. 1963

Chart 17 Headwater Depth For C.M. Pipe Culverts With Inlet Control



Adapted from
Federal Highway Administration May 1973

Chart 18 Headwater Depth For Circular Pipe Culverts With Bevelled Ring Inlet Control



Adapted from Bureau of Public Roads

Chart 19 Critical Depth - Circular Pipe

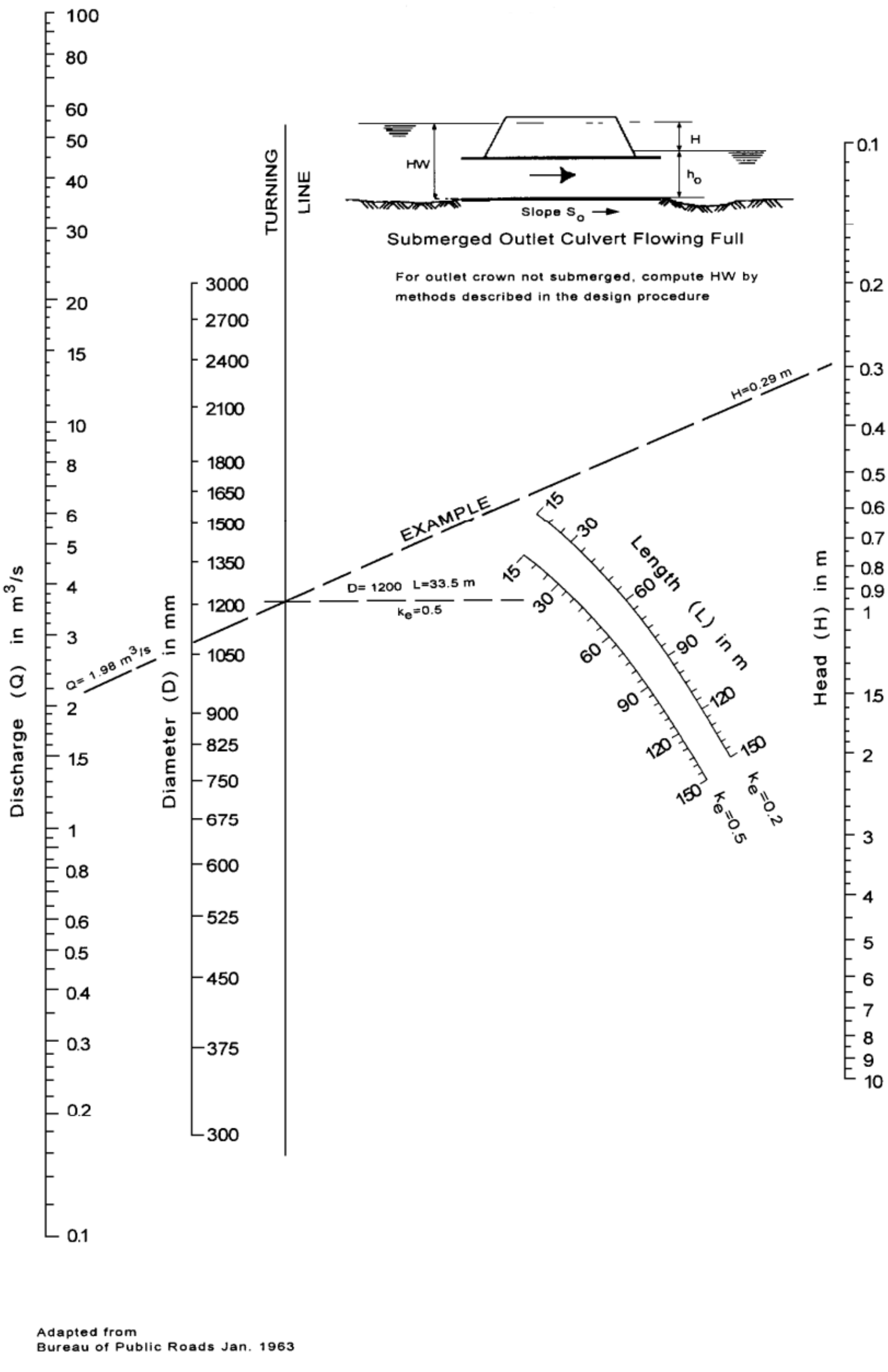


Chart 20 Head For Concrete Pipe Culverts Flowing Full (n=0.012)

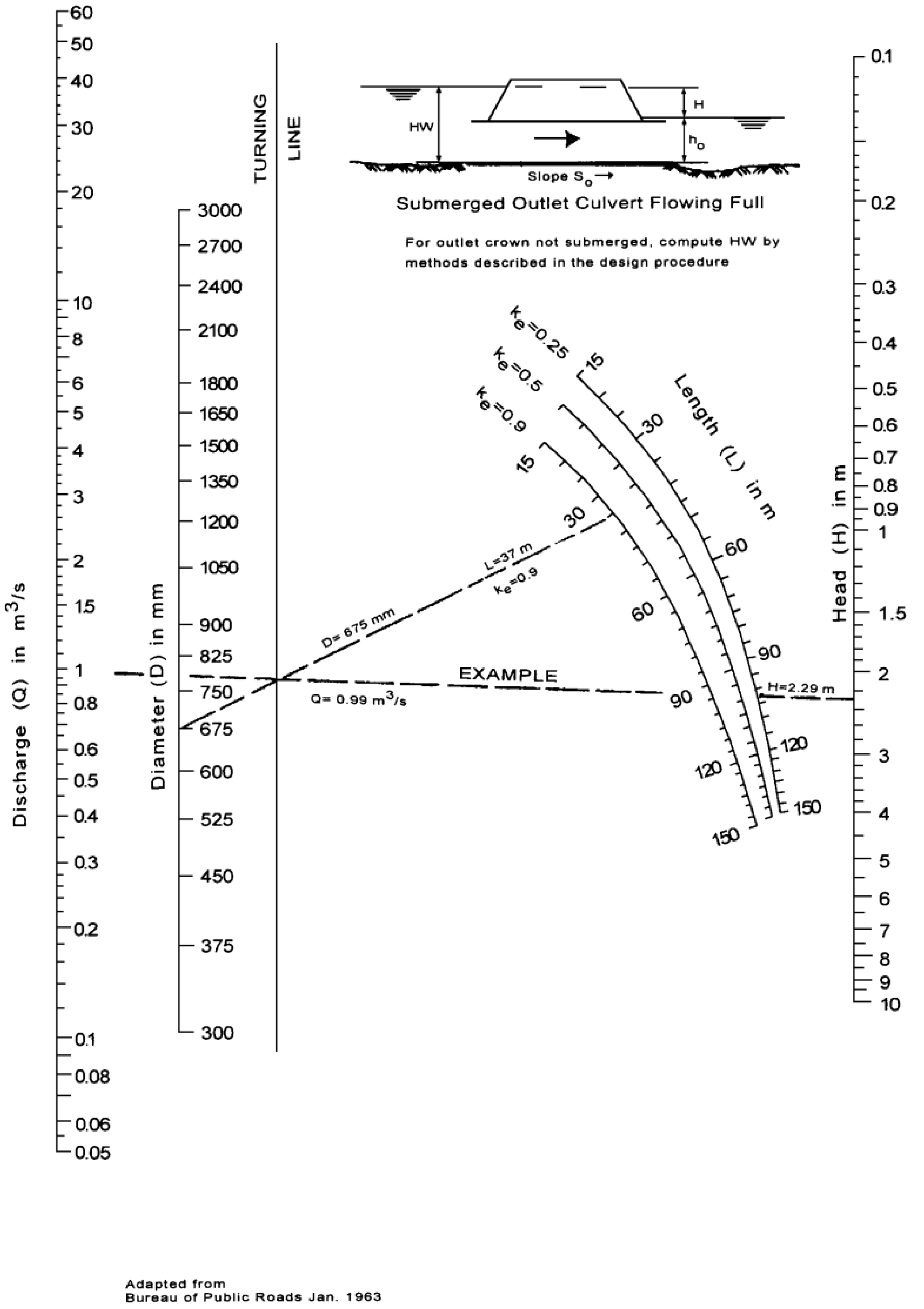
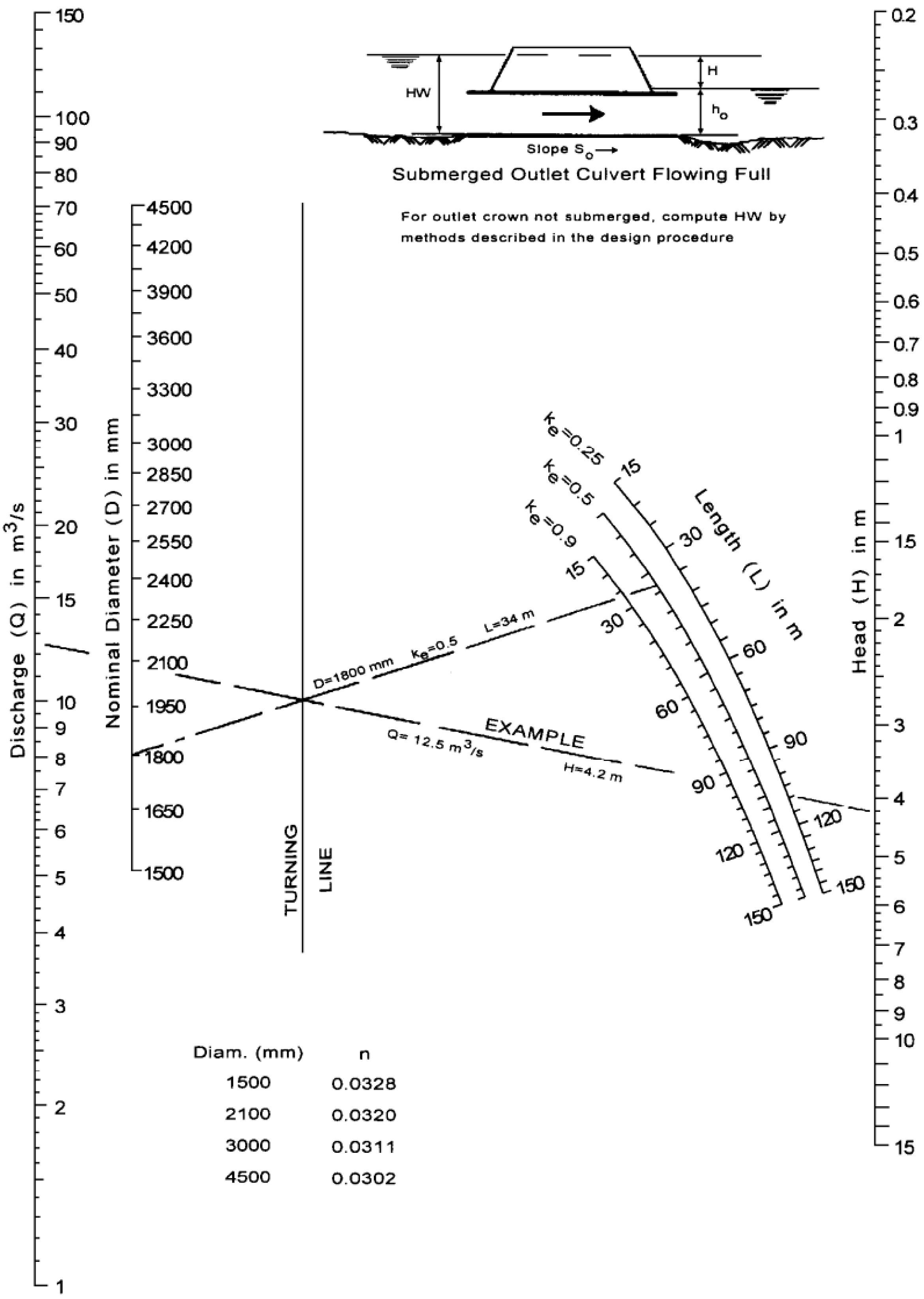


Chart 21 Head For Standard C.M. Pipe Culverts Flowing Full (n=0.024)



Adapted from
Bureau of Public Roads Jan. 1963

Chart 22 head for structural plate corr. metal pipe culverts flowing full (n=0.0328 - 0.0302)

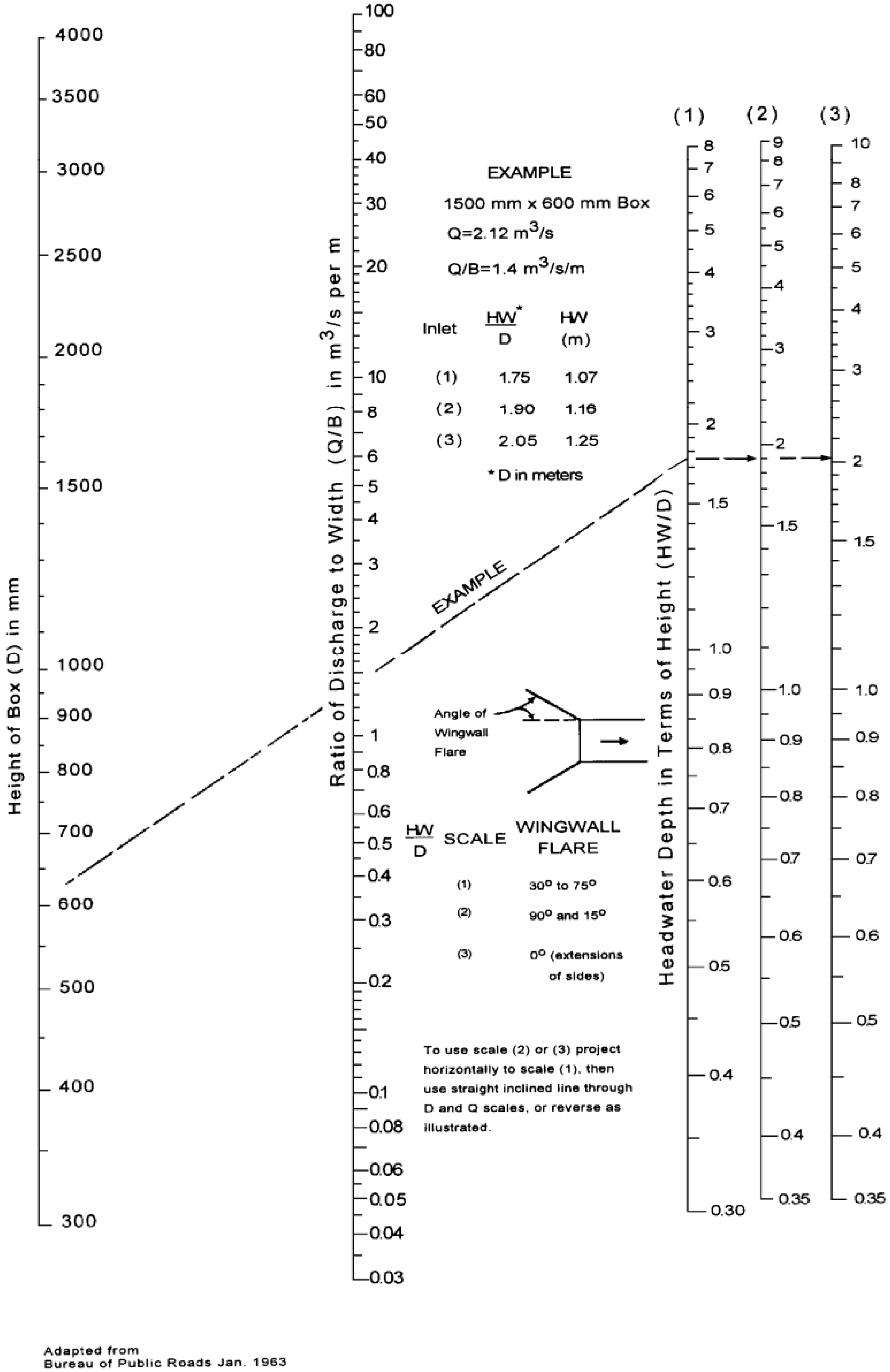
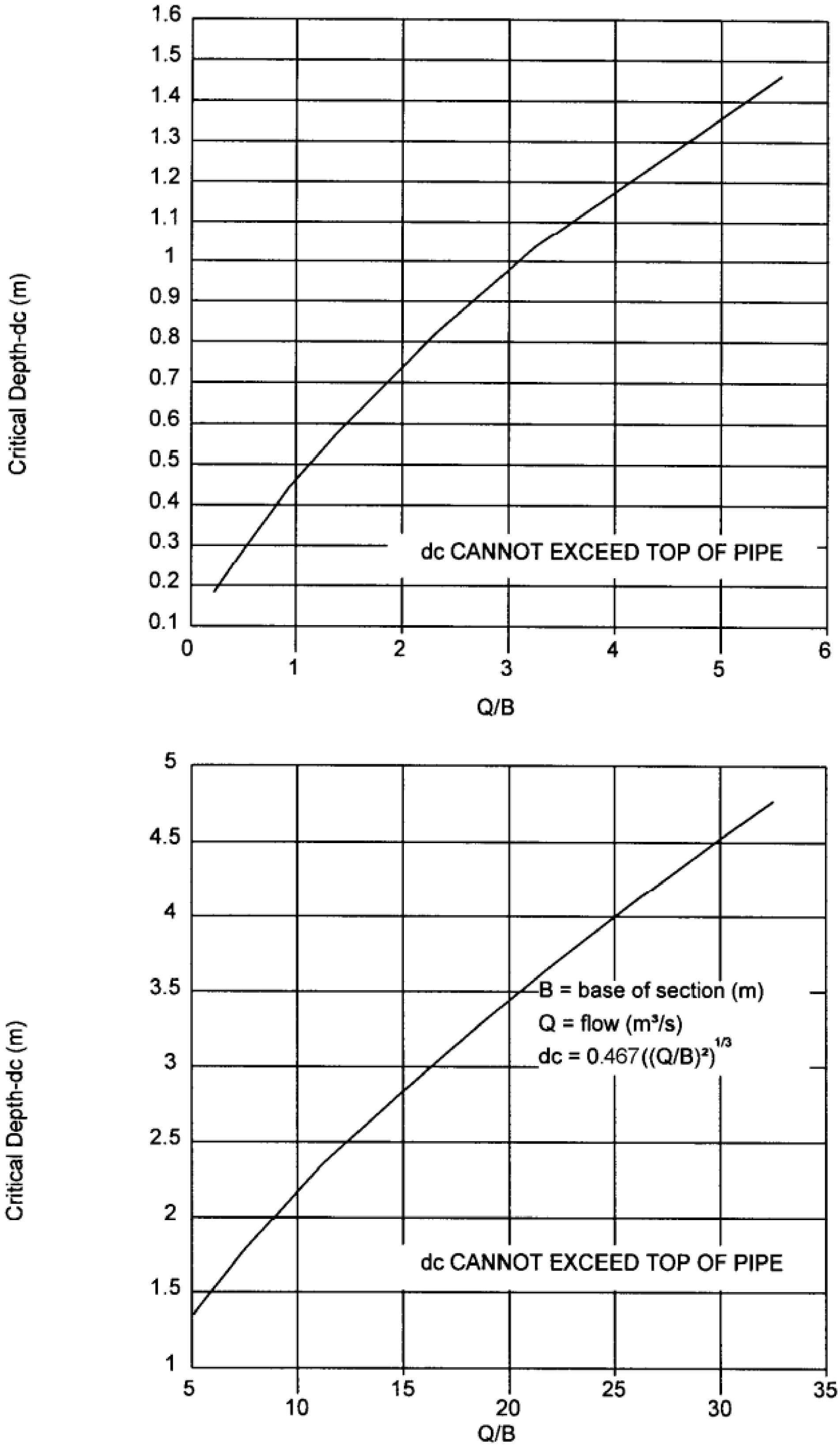
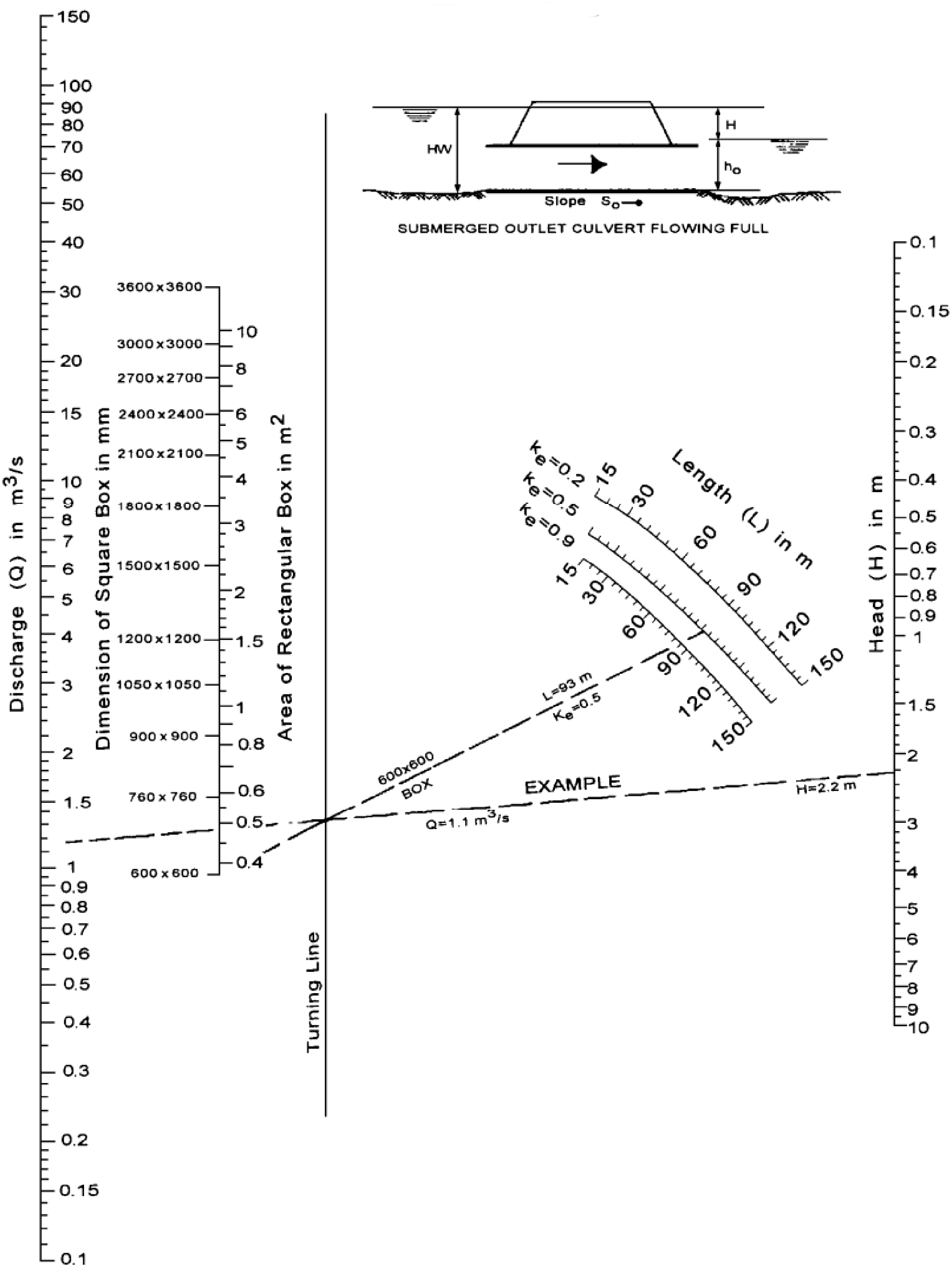


Chart 23 Headwater Depth For Culverts With Inlet Control



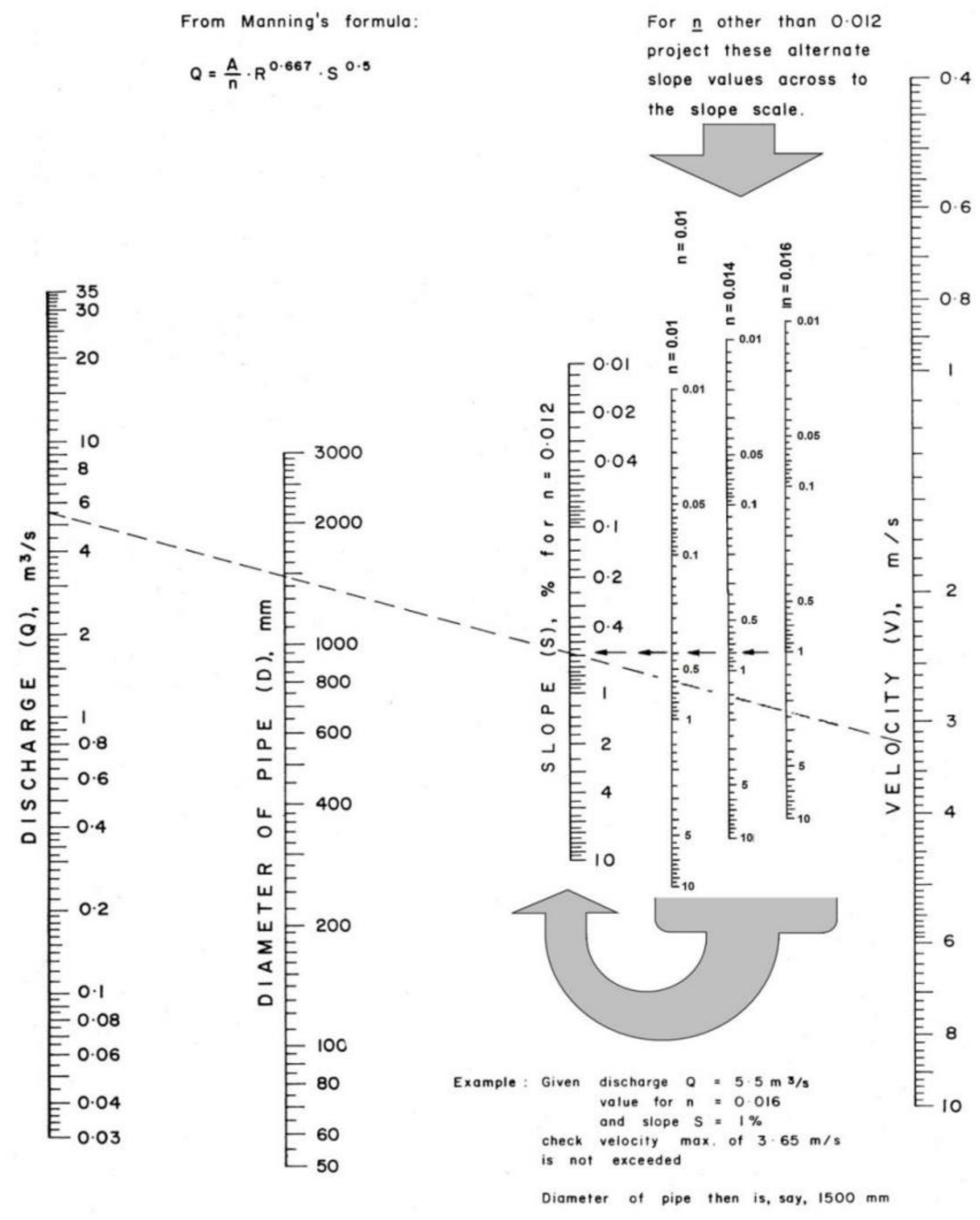
Adapted from Bureau of Public Roads

Chart 24 Critical Depth - Rectangular Section



Adapted from
Burea of Public Roads Jan. 1963

Chart 25 Head For Concrete Box Culverts Flowing Full (n=0.012)



Source: DTMR (2010).

Chart 26 Discharge And Velocity In Round Pipes Flowing Full

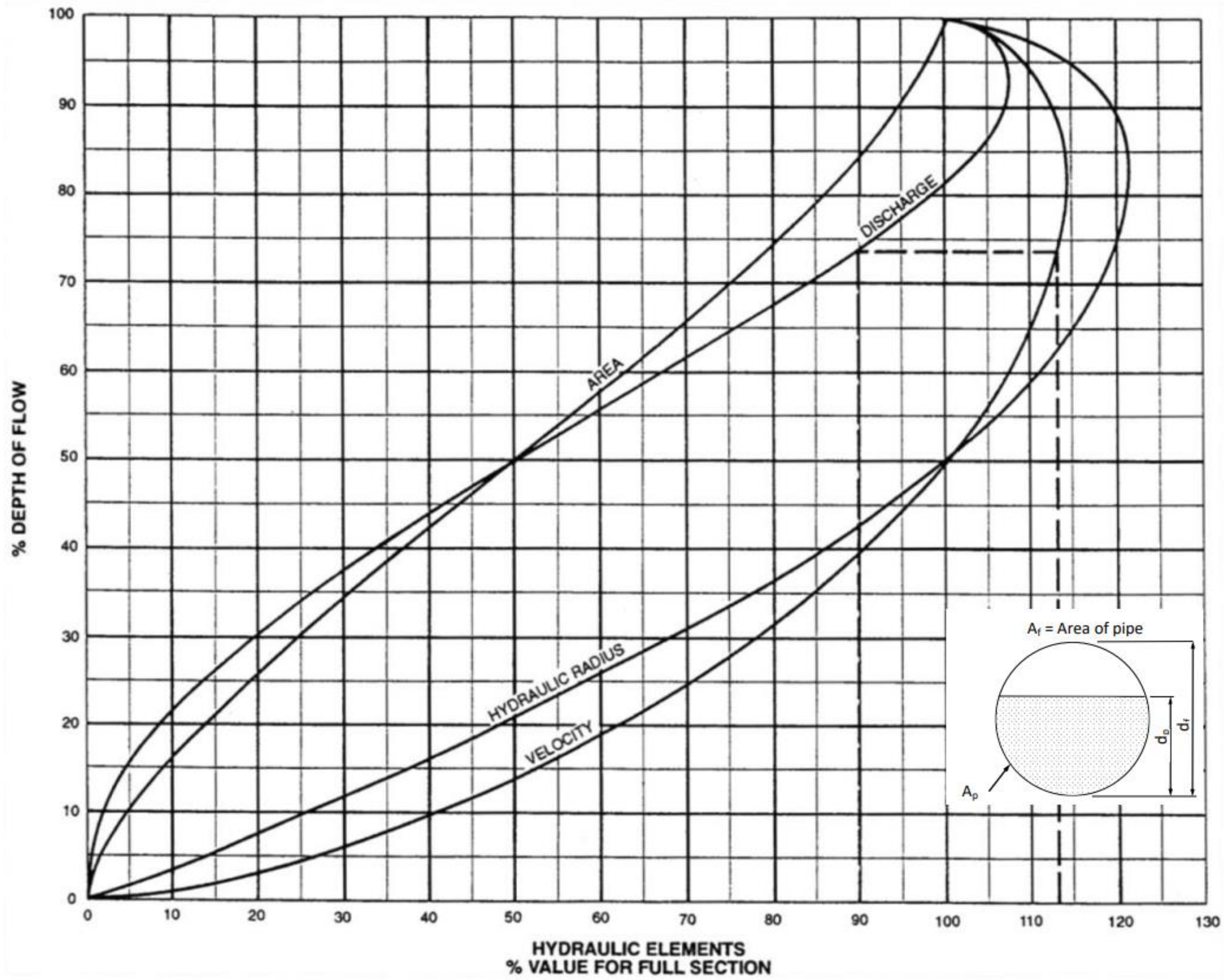


Chart 27 Velocity And Discharge In Part-Full Pipes

13.11 REFERENCES

1. Ghana Highway Authority Road Design Guide, 1991.
2. Japanese Road Structure Ordinance, April 2021.
3. Highway Drainage Manual, Department of Urban Roads, 2006.
4. Manual For Low Volume Roads Part C - Hydrology, Drainage Design, And Roadside Slope Stabilisation 2019
5. Ethiopian Roads Authority's Drainage Design Manual - 2013
6. Brown, Scott A., James Douglas Schall, Johnny L. Morris, Stuart Stein, and John C. Warner. Urban drainage design manual: hydraulic engineering circular 22. No. FHWA-NHI-10-009. National Highway Institute (US), 2009.
7. Schall, JD, EV Richardson, and JL MORRIS. "Hydraulic design series No. 4: introduction to highway hydraulics." (2008).
8. Jones, Neville, and C. Lawson. "The Queensland urban drainage manual." Local Government Engineers Association of Queensland Journal 9, no. 4th quarter (1991).
9. McCuen, Richard H., Peggy A. Johnson, and Robert M. Ragan. Highway hydrology: Hydraulic design series number 2. No. FHWA-NHI-02-001. National Highway Institute (US), 2002.
10. Transportation Officials. Task Force on Hydrology. Highway drainage guidelines. AASHTO, 2007.

11. Balkham, Matt, C. Fosbeary, A. Kitchen, and C. Rickard. "Culvert design and operation guide." Construction and industry research and information association, London, UK (2010).
12. SANRAL. (2007). Kruger, E.J. (Editor), Rooseboom, A. Van Vuuren, S.J., Van Dijk, M., Jansen van Vuuren, A.M., Pienaar, W.J., Pienaar, P.A., James, G.M., Maastrecht, J. and Stipp, D.W. (2006). Drainage Manual. 5th Fully revised. The South African National Roads Agency Ltd. (2007),
13. Manual, South Dakota Drainage. "Bridge Hydraulics." (2011).
14. Zevenbergen, L. W., L. A. Arneson, J. H. Hunt, and Arthur Carl Miller. Hydraulic design of safe bridges. No. FHWA-HIF-12-018. United States. Federal Highway Administration, 2012.
15. Ngo, Hanson. Guide to bridge technology, part 8: hydraulic design of waterway structures. No. AGBT08-18. 2018.
16. Weeks, W., M. Babister, and M. Retallick. Guide to road design part 5A: drainage: road surface, networks, basins and subsurface. No. AGRD05A-23. 2023.
17. Lagasse, P. F., P. E. Clopper, J. E. Pagan-Ortiz, L. W. Zevenbergen, L. A. Arneson, J. D. Schall, and L. G. Girard. Bridge scour and stream instability countermeasures: experience, selection, and design guidance: Volume 1. No. FHWA-NHI-09-111. National Highway Institute (US), 2009.
18. de Carteret, R., and L. COMPORT. "Guide to pavement technology part 10: subsurface drainage." (2009).
19. Arika, Caleb N., Dario J. Canelon, and John L. Nieber. "Subsurface Drainage Manual for Pavements in Minnesota." (2009).
20. Moulton, Lyle K. Highway subdrainage design. The Administration, 1980.
21. Uganda Road Design Manual Volume II: Drainage Design, 2010
22. Bradley, J.N., Revised 1978,"Hydraulics of Bridge Waterways," Hydraulic Design Series No. 1, Federal Highway Administration, U.S. Government Printing Office, Washington, D.C., FHWA-EPD86-101, NTIS PB86-181708, 2nd edition 1970, 111 pp.
23. AASHTO, "Roadside Design Guide," 1996.
24. FHWA. Drainage of Highway Pavements. Hydraulic Engineering Circular No. 12, TS-84-202, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1984.
25. Horton, R. E. Surface Runoff Phenomena, Part 1, Analysis of the Hydrograph. Vorheesville, N.Y., Horton Hydrol. Lab. Pub. 101, 73 p., 1935.
26. Schall, James Douglas. Hydraulic design of highway culverts. No. FHWA-HIF-12-026. United States. Federal Highway Administration, 2012.
27. FHWA, 1961, "Design Charts for Open Channel Flow, Hydraulic Design Series No. 3 Hydraulics Branch, Bridge Division, Office of Engineering, Washington, D.C. 20590.
28. FHWA, 1965, "Hydraulic Charts for the Selection of Highway Culverts," HEC No. 5, Hydraulics Branch, Bridge Division, Office of Engineering, FHWA, Washington, D.C. 20590 (L.A. Herr and H.G. Bossy).
29. FHWA, 1972a, "Hydraulic Design of Improved Inlets for Culverts," HEC No. 13, Hydraulics Branch, Bridge Division, Office of Engineering, FHWA, Washington, D.C. 20590, August (L.J. Harrison, J.L. Morris, J.M. Normann and F.L. Johnson).
30. FHWA, 1972b, "Computation of Uniform and Nonuniform Flow in Prismatic Channels," Office of Research (P.N. Zelensky).
31. FHWA, 1978, "Hydraulics of Bridge Waterways," Hydraulic Design Series No. 1, revised second edition (J.N. Bradley).

32. FHWA, 1979, "Design of Urban Highway Drainage - The State of the Art," FHWA-TS-79 225, Hydraulics Branch, Bridge Division, Office of Engineering, FHWA, Washington, D.C. 20590, August (S.W. Jens).
33. FHWA, 2009b, "Bridge Scour and Stream Instability Countermeasures - Experience, Selection, and Design Guidelines," Third Edition, Report FHWA NHI 09-111, Federal Highway Administration, Hydraulic Engineering Circular No. 23, U.S. Department of Transportation.
34. Austroads (2010) Guide to road design part 3: geometric design (superseded), AGRD03-10, Austroads, Sydney, NSW.
35. Austroads (2016) Guide to road design part 3: geometric design, AGRD03-16 (Edn 3.4 published February 2021), Austroads, Sydney, NSW.
36. Austroads (2017) Guide to road design part 6A: paths for walking and cycling, AGRD06A-17 (Edn 2.1 published February 2021), Austroads, Sydney, NSW.
37. Austroads (2021a) Guide to road design part 1: objectives of road design, AGRD01-21, Austroads, Sydney, NSW.
38. Austroads (2022) Guide to road design part 6: roadside design, safety and barriers, AGRD06-20, Austroads, Sydney, NSW.
39. Austroads (2023) Guide to road design part 5: drainage – general and hydrology considerations, AGRD0523, Austroads, Sydney, NSW.
40. Austroads (2023b) Guide to road design: part 5A: drainage – road surface, networks, basins and subsurface, AGRD05A-21, Austroads, Sydney, NSW.
41. Department of Transport and Main Roads (2010) Road drainage manual, DTMR, Brisbane, Qld.
42. Arneson, LA, Zevenbergen, LW, Lagasse, PF & Clopper, PE 2012, Evaluating scour at bridges, 5th edn, FHWA-HIF-12-003, Hydraulic Engineering Circular no. 18, Federal Highway Administration, Washington, DC, USA.
43. Karaki, S 1960, ‘Laboratory study of spur dikes for highway bridge protection’, Highway Research Board bulletin no. 286, Washington, DC, USA.
44. Briaud, JL, Chen, HC, Chang, KA, Oh, SJ, Chen, S, Wang, J, Li, Y, Kwak, K, Nartjaho, P, Gudaralli, R, Wei, W, Pergu, S, Cao, YW & Ting, F 2011, The SRICOS-EFA method: summary report, Texas A&M University, College Station, TX, USA.
45. Lagasse, PF, Clopper, PE, Pagan-Ortiz, JE, Zevenbergen, LW, Arneson, LA, Schall, JD & Girard, LG 2009, Bridge scour and stream instability countermeasures: experience, selection, and design guidance, 3rd edn, FHWA-NHI-09-111, Hydraulic Engineering Circular no. 23, Federal Highway Administration, Washington, DC, USA.
46. Lagasse, PF, Zevenbergen, LW, Spitz, WJ & Arneson, LA 2012, Stream stability at highway structures, 4th edn, FHWA-HIF-12-004, Hydraulic Engineering Circular no. 20, Federal Highway Administration, Washington, DC, USA.
47. Austroads (2021) Guide to road design: part 1: objectives of road design, AGRD01-21, Austroads, Sydney, NSW.
48. van Schilfgaarde J (1963) ‘Design of tile drainage for falling water table’, Journal of Irrigation and Drainage Division, ASCE, vol. 89, no. 2, pp. 1–11.
49. Richardson, Everett V., and Stanley R. Davis. Evaluating scour at bridges. No. HEC 18. United States. Federal Highway Administration. Office of Technology Applications, 1995.
50. Schall, James Douglas. Hydraulic design of highway culverts. No. FHWA-HIF-12-026. United States. Federal Highway Administration, 2012.
51. Rossmiller, Ronald L. "The rational formula revised." In International Symposium on Urban Storm Runoff, pp. 28-31. 1980.

52. Fowler, Gary, and P. E. Wouter Gulden. Investigation of Location of Dowel Bars Placed by Mechanical Implantation. No. FHWA/RD-82/153. United States. Federal Highway Administration, 1983.
53. Earley, P. C. 1979. Gully Inlet Spacing Design. Australia: University of Western Australia.
54. Australian Road Research Board (ARRB). 1987. Special Report No 35. Subsurface Drainage of Road Structures. Australia: Australian Road Research Board.
55. Gulden, W. 1983. Experience in Georgia with Drainage of Jointed Concrete Pavements Transportation Research Record 1440. Transportation Research Board, Washington, D.C.
56. Jeffcoat, H.H., F.A. Kilpatrick, J.B. Atkins, and J.A. Pearman. 1992. Effectiveness of Highway Edge Drains: Experimental Project No. 12, Concrete Pavement Drainage
57. Yu, H.T., L. Khazanovich, S.P. Rao, M.I. Darter, and H.V. Quintus. 1998b. Guidelines for Subsurface Drainage Based on Performance. NCHRP 1-34 Final Report. National Cooperative Highway Research Program., Washington, D.C.
58. Hindermann, W.L. 1968. The Swing to Full-Depth information series No. 146. Asphalt Institute, Lexington, KY.
59. Queensland Department of Transport and Main Roads 2013, Bridge scour manual, TMR, Brisbane, Qld.
60. Ettema, R, Nakato, T & Muste, M 2010, Estimation of scour depth at bridge abutments, NCHRP report 24-20, Transportation Research Board, Washington, DC, USA.
61. Kersten, M. S., and Skok, E. L., Jr. Application of AASHO Road Test Results to Design of Flexible Pavements in Minnesota. Highway Research Record 291, 1969, pp. 70-88.
62. USGS, 1968, "Measurement of peak discharge at culverts by indirect methods," U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chapter A3 (Bodhaine, G.L.).
63. FHWA, D.P. 1992. Drainable Pavement Systems, Participant Notebook Publication FHWA-SA-92-008. FHWA, United States Department of Transportation, Washington, D.C.
64. Williams, E. J., D. D. Parrish, M. P. Buhr, F. C. Fehsenfeld, and R. Fall. "Measurement of soil NO x emissions in central Pennsylvania." Journal of Geophysical Research: Atmospheres 93, no. D8 (1988): 9539-9546.
65. Manual For Low Volume Roads Part C - Hydrology, Drainage Design, And Roadside Slope Stabilisation 2019.

Volume VI

Appendix-A Details of Horizontal Curve

Appendix-B Details of Vertical Curve

Appendix-C Rainfall Intensity

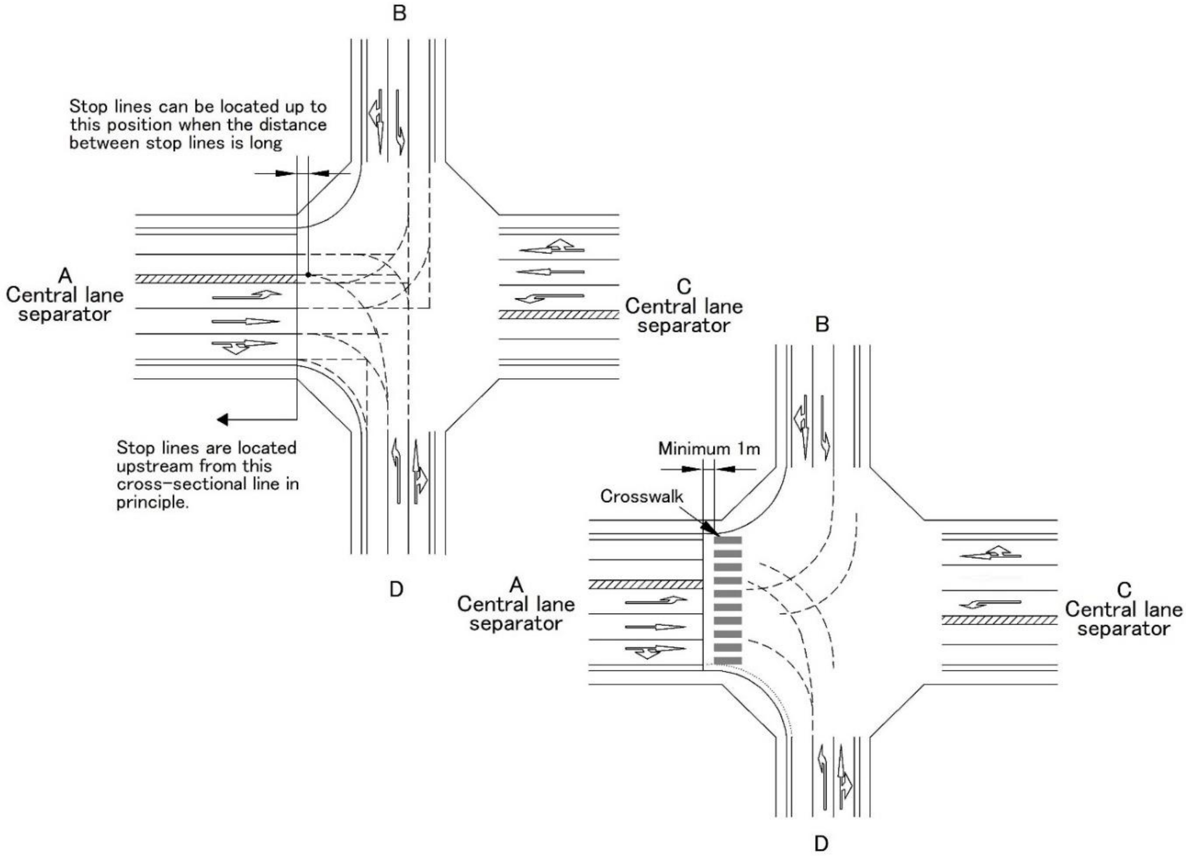
Appendix-D Toll Gates

Appendix-E Presentation of Drawings and Design Reports

Appendix-F Procedure for Determining Geometric Structures of At-Grade Intersections

Appendix-G Road Design Check Sheet

Appendix-H Glossary



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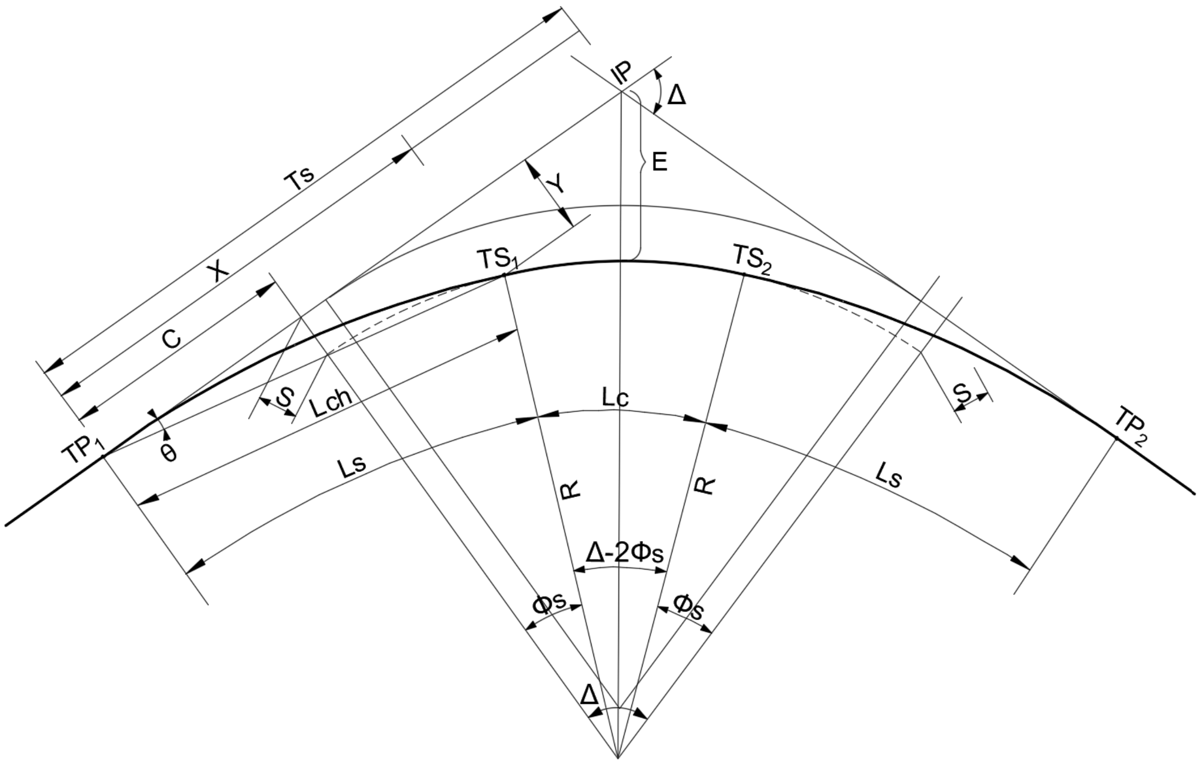
APPENDIX A DETAILS OF HORIZONTAL CURVE

SIMPLE CURVE WITH SPIRAL TRANSITIONS AT BOTH SIDES



(1) Spiral transition formula

$$\text{Length of spiral (Ls)} = \frac{v^3}{3.63^3 \text{CR}} \tag{A.1}$$

$$\text{Ls} = \frac{0.06\text{V}^3}{\text{R}} \tag{A.2}$$

$$\text{Superelevation (Se)} = \frac{0.7^2\text{V}^3}{127\text{R}} \tag{A.3}$$

$$\text{Se} = 0.003858\frac{\text{V}^2}{\text{R}} \tag{A.4}$$

$$\text{Shift (S)} = \frac{\text{Ls}^2}{24\text{R}} \tag{A .5}$$

$$\text{Tp to point of shift (C)} = \frac{\text{Ls}}{2} - \frac{\text{Ls}^3}{240\text{R}^2} \tag{A .6}$$

$$\frac{\text{Ls}}{2\text{R}}\text{radians}(\Phi\text{s}) = \frac{\text{LsD}}{200}\ \text{degrees} \tag{A .7}$$

$$\text{Degree of curve (D)} = \frac{5729.58}{R} \tag{A .8}$$

Curve with transition

$$\text{Apex Distance (Ts)} = (R + S)\left(\tan\frac{\Delta}{2}\right) + C \tag{A.9}$$

$$\text{Length of circular curve (Lc)} = \frac{\Delta - 2\Phi s}{D}100 \tag{A.10}$$

$$\text{Length of circular curve (Lc)} = R(\Delta - 2\Phi s)\frac{\pi}{180} \tag{A.11}$$

$$\text{External distance (E)} = (R + S)\left(\sec\frac{\Delta}{2} - 1\right) + S \tag{A.12}$$

TP_1 Ch= IP Ch - Ts (Apex Distance)
TS_1 Ch = TP_1 Ch + Ls (Length of spiral)
TS_2 Ch = TS_1 Ch + Lc (Length of circular curve)
TP_2 Ch = TS_2 Ch + Ls (Length of spiral)

Curve without transition

$$\text{Tangent length (T)} = R\tan\frac{\Delta}{2} \tag{A.13}$$

$$\text{Length of circular curve(Lc)} = \frac{\Delta}{D}100 \tag{A.14}$$

$$\text{Length of circular curve (Lc)} = R \times \Delta \times \frac{\pi}{180} \tag{A.15}$$

$$\text{External distance (E)} = R\left(\sec\frac{\Delta}{2} - 1\right) \tag{A.16}$$

TP_1 Ch = IP Ch - T (Length of circular curve)
TP_2 Ch = TP1 Ch+ Lc (length of circular curve)

$$X = Ls - \frac{ls\Phi s^2}{10} = Ls - \frac{Ls^3}{40R^2} \tag{A.17}$$

$$Y = \frac{Ls\Phi s}{3 \times 57.29} - \frac{Ls\Phi^3}{42(57.29)^3} = \frac{Ls^2}{6R} - \frac{Ls^4}{336R^3} \tag{A.18}$$

$$\text{Long Chord (Lch)} = \sqrt{X^2 + Y^2}$$

$$\text{Deflection angle from origin } (\theta) = \tan^{-1}\frac{Y}{X} \tag{A.19}$$

Where,

V: Design speed (km/h)
R: Radius (m)
Δ: intersection angle

C: $0.3572 m/s^3$ (the rate of gain of radial acceleration)
Φs: in radians (1 radian=57.29)

Example of spiral transition.

Find the horizontal curve elements for an intersection angle (Δ) $35^0 29'45''$, IP Ch 2+357.00m and design speed (V) 100 km/h.

Solution

(i) From the design chart.

D	; 0.5^0 (Assumed)	Lch	; 52.359m
R	; 1145.916m	θ	; $00^0 26'12''$
Ls	; 52.360m	Se	; 3%
2Φs	; $02^0 37'05''$	X	; 52.357m
S	; 0.10m	Y	; 0.399m
C	; 26.180m		

(ii) Calculate Ts, Lc, and E

Apex distance (Ts) ………. **Equation (A.9)**

$$\mathrm{Ts} = (\mathrm{R} + \mathrm{S})\,(\tan\frac{\Delta}{2}) + \mathrm{C}$$

$$\mathrm{Ts} = (1145.916 + 0.100)\,(\tan\frac{35°29'45''}{2}) + 26.180$$

$$\mathrm{Ts} = (1146.016)\,(0.3200624) + 26.180 = 392.977\mathrm{m}$$

Length of circular curve (Lc) ……… **Equation (A.10)**

$$\mathrm{Lc} = \frac{\Delta - 2\Phi s}{\mathrm{D}} 100$$

$$\mathrm{Lc} = \frac{35°29'45'' - 02°37'05''}{0.5°} 100$$

$$\mathrm{Lc} = \frac{32°52'40''}{0.5°} 100$$

$$\mathrm{Lc} = \frac{32.8778}{5} = 657.556\mathrm{m}$$

External distance (E) ………. **Equation (A.12)**

$$\mathrm{E} = (\mathrm{R} + \mathrm{S})\left(\sec\frac{\Delta}{2} - 1\right) + \mathrm{S}$$

$$\mathrm{E} = (1145.916 + 0.100)\left(\sec\frac{35°29'45''}{2} - 1\right) + 0.100$$

$$E = (1145.016)(1.0499714 - 1) + 0.100 = 57.368\text{m}$$

(iii) Calculate IP, TS.1, TS.2, and TP 2

TP_1 Ch	= IP Ch – Apex distance	
	= 2+357.00 – (392.977)	= 1+964.023
TS_1 Ch	= TP_1 Ch + Ls	
	= 1+964.023 + (57.368)	= 2+016.383
TS_2 Ch	= TS_1Ch + Lc	
	= 2+016.383 + (657.0556)	= 2+673.939
TP_2 Ch	= TS_2 Ch + Ls	
	= 2+673.939 + (57.368)	= 2+726.299

DESIGN SPEED V= 120.00 km/h. SPIRAL DATA										
RADIUS	**DEGREE OF CURVE**	**LENGTH OF SPIRAL**	**MINIMUM DEVIATION ANGLES**	**SHIFT**	**TP TO POINT OF SHIFT**	**LONG CHORD**	**DEF ANGLE FROM ORIGIN**	**CO-ORDS**		**SUPER ELEV**
(m)	**(D.M)**	**(m)**	**(D.MS)**	**(m)**	**(m)**	**(m)**	**(D.MS)**	**(m)**		**(m)**
R	**D**	**Ls**	**2∅s**	**S**	**C**	**Lch**	**∅**	**X**	**Y**	**e rise in 1m**
11459.160	0.30									
7639.440	0.45									
5729.580	1.00									0.025
4543.279	1.15									0.025
3819.720	1.30									0.025
3253.389	1.45									0.025
2864.790	2.00									0.025
2533.967	2.15									0.025
2281.691	2.30									0.025
2083.484	2.45									0.025
1909.860	3.00	54.287	1.3743	0.064	27.143	54.287	0.1617	54.287	0.257	0.030
1756.941	3.15	59.012	1.5528	0.083	29.506	59.013	0.1915	59.012	0.330	0.030
1631.842	3.30	63.536	2.1351	0.103	31.767	63.537	0.2218	63.536	0.412	0.030
1527.888	3.45	67.858	2.3241	0.126	33.929	67.860	0.2527	67.858	0.502	0.040
1432.395	4.00	72.382	2.5343	0.152	36.190	72.385	0.2857	72.382	0.610	0.040
1348.136	4.15	76.906	3.1607	0.183	38.452	76.910	0.3241	76.906	0.731	0.040
1270.104	4.30	81.631	3.4057	0.219	40.814	81.636	0.3649	81.631	0.874	0.040
1206.227	4.45	85.954	4.0458	0.255	42.975	85.960	0.4049	85.954	1.021	0.050
1145.916	5.00	90.478	4.3126	0.298	45.237	90.486	0.4514	90.478	1.191	0.050
1091.349	5.15	95.002	4.5915	0.345	47.498	95.012	0.4952	95.002	1.378	0.050
1039.642	5.30	99.727	5.2946	0.399	49.860	99.739	0.5457	99.727	1.594	0.050
996.449	5.45	104.050	5.5858	0.453	52.020	104.065	0.5949	104.049	1.810	0.060
954.930	6.00	108.573	6.3052	0.514	54.281	108.593	1.0507	108.573	2.057	0.060
916.733	6.15	113.097	7.0407	0.581	56.541	113.121	1.1039	113.097	2.325	0.060
879.970	6.30	117.822	7.4018	0.657	58.902	117.852	1.1641	117.822	2.628	0.060
848.287	6.45	122.145	8.1441	0.732	61.062	122.180	1.2224	122.145	2.928	0.070
818.511	7.00	126.669	8.5201	0.817	63.322	126.711	1.2837	126.669	3.266	0.070
790.287	7.15	131.193	9.3041	0.907	65.581	131.243	1.3503	131.193	3.628	0.070
762.814	7.30	135.918	10.1232	1.009	67.941	135.978	1.4200	135.918	4.034	0.070
739.301	7.45	140.241	10.5207	1.108	70.099	140.310	1.4835	140.240	4.431	0.080
716.198	8.00	144.765	11.3452	1.219	72.358	144.846	1.5541	144.764	4.873	0.080
694.495	8.15	149.288	12.1859	1.337	74.615	149.384	2.0301	149.288	5.344	0.080
674.068	8.30	153.812	13.0427	1.462	76.873	153.923	2.1033	153.812	5.844	0.080
653.979	8.45	158.537	13.5323	1.601	79.230	158.666	2.1841	158.537	6.399	0.080
636.620	9.00	162.860	14.3927	1.736	81.386	163.007	2.2619	162.860	6.936	0.090
619.414	9.15	167.384	15.2859	1.885	83.641	167.553	2.3431	171.907	7.529	0.090
603.114	9.30	171.908	16.1952	2.042	85.896	172.101	2.4257	176.632	8.155	0.090
586.980	9.45	176.633	17.1429	2.215	88.250	176.853	2.5159	180.955	8.844	0.090
572.958	10.00	180.956	18.0544	2.381	90.403	181.205	3.0028	185.479	9.508	0.090
558.983	10.15	185.480	19.0042	2.564	92.655	185.761	3.0933	185.479	10.237	0.090
545.674	10.30	190.003	19.5701	2.757	94.906	190.321	3.1851	190.002	11.003	0.090

DESIGN SPEED V= 100.00 km/hr. SPIRAL DATA										
RADIUS	DEGREE OF CURVE	LENGTH OF SPIRAL	MINIMUM DEVIATION ANGLES	SHIFT	TP TO POINT OF SHIFT	LONG CHORD	DEF ANGLE FROM ORIGIN	CO-ORDS		SUPER ELEV
(m)	(D.M)	(m)	(D.MS)	(m)	(m)	(m)	(D.MS)	(m)		(m)
R	D	Ls	2øs	S	C	Lch	ø	X	Y	e rise in 1m
11459.160	0.30									
7639.440	0.45									
5729.580	1.00									
4543.279	1.15									0.025
3819.720	1.30									0.025
3253.389	1.45									0.025
2864.790	2.00									0.025
2533.967	2.15									0.025
2281.691	2.30									0.025
2083.484	2.45									0.025
1909.860	3.00									0.025
1756.941	3.15									0.025
1631.842	3.30									0.025
1527.888	3.45									0.025
1432.395	4.00									0.025
1348.136	4.15	44.506	1.5329	0.061	22.253	44.507	0.1855	44.506	0.245	0.030
1270.104	4.30	47.240	2.0752	0.073	23.620	47.241	0.2119	47.240	0.293	0.030
1206.227	4.45	49.742	2.2146	0.085	24.871	49.743	0.2338	49.742	0.342	0.030
1145.916	5.00	52.360	2.3705	0.100	26.179	52.361	0.2611	52.360	0.399	0.030
1091.349	5.15	54.978	2.5311	0.115	27.488	54.980	0.2852	54.978	0.462	0.040
1039.642	5.30	57.712	3.1050	0.133	28.855	57.715	0.3148	57.712	0.534	0.040
996.449	5.45	60.214	3.2744	0.152	30.106	60.217	0.3437	60.214	0.606	0.040
954.930	6.00	62.832	3.4612	0.172	31.415	62.836	0.3742	62.832	0.689	0.040
916.733	6.15	65.450	4.0526	0.195	32.724	65.454	0.4054	65.450	0.779	0.040
897.970	6.30	68.184	4.2622	0.220	34.090	68.190	0.4423	68.184	0.880	0.040
848.827	6.45	70.686	4.4617	0.245	35.341	70.693	0.4742	70.686	0.981	0.050
818.511	7.00	73.304	5.0753	0.274	36.649	73.312	0.5118	73.304	1.094	0.050
790.287	7.15	75.922	5.3016	0.304	37.958	75.932	0.5502	75.922	1.215	0.050
762.814	7.30	78.656	5.5429	0.338	39.325	78.668	0.5904	78.656	1.351	0.050
739.301	7.45	81.158	6.1723	0.371	40.575	81.171	1.0253	81.158	1.485	0.050
716.198	8.00	83.776	6.4207	0.408	41.883	83.792	1.0660	83.776	1.633	0.050
694.495	8.15	86.394	7.0739	0.448	43.191	86.412	1.1115	86.394	1.791	0.060
674.068	8.30	89.012	7.3358	0.490	44.499	89.412	1.1537	89.012	1.958	0.060
653.979	8.45	91.746	8.0217	0.536	45.866	91.771	1.2020	91.746	2.144	0.060
636.620	9.00	94.248	8.2856	0.581	47.115	94..276	1.2446	94.248	2.325	0.060
619.414	9.15	96.866	8.5736	0.631	48.423	96.899	1.2932	96.866	2.524	0.060
603.114	9.30	99.484	9.2703	0.684	49.731	99.521	1.3426	99.484	2.734	0.060
586.980	9.45	102.218	9.5839	0.742	51.096	102.261	1.3942	102.216	2.965	0.070
572.958	10.00	104.720	10.2819	0.797	52.345	104.768	1.4438	104.720	3.188	0.070
558.983	10.15	107.338	11.0006	0.859	53.652	107.392	1.4955	107.338	3.433	0.070
545.674	10.30	109.956	11.3243	0.923	54.959	110.017	1.5520	109.955	3.690	0.070
532.434	10.45	112.690	12.0736	0.994	56.324	112.760	2.0107	112.690	3.972	0.070
520.871	11.00	115.192	12.4016	1.061	57.572	115.270	2.0633	115.191	4.242	0.070
509.296	11.15	117.810	13.1513	1.135	58.879	117.897	2.1221	117.809	4.538	0.080
498.224	11.30	120.428	13.5057	1.213	60.185	120.525	2.1816	120.427	4.846	0.080
487.163	11.45	123.162	14.2907	1.297	61.548	123.271	2.2436	123.162	5.184	0.080
477.465	12.00	125.664	15.0447	1.378	62.796	125.784	2.3031	125.663	5.505	0.080
467.721	12.15	128.282	15.4252	1.466	64.101	128.415	2.3650	128.281	5.856	0.080
458.366	12.30	130.900	16.2145	1.558	65.405	131.047	2.4316	130.899	6.221	0.080
448.988	12.45	133.634	17.0311	1.657	66.768	133.797	2.5007	133.633	6.619	0.090
440.737	13.00	136.136	17.4151	1.752	68.014	136.315	2.5631	136.135	6.996	0.090
432.421	13.15	138.754	18.2305	1.855	69.317	138.950	3.0320	138.753	7.407	0.090
424.413	13.30	141.372	19.0507	1.962	70.620	141.588	3.1017	141.371	7.833	0.090

DESIGN SPEED V= 100.00 km/hr. SPIRAL DATA										
RADIUS	**DEGREE OF CURVE**	**LENGTH OF SPIRAL**	**MINIMUM DEVIATION ANGLES**	**SHIFT**	**TP TO POINT OF SHIFT**	**LONG CHORD**	**DEF ANGLE FROM ORIGIN**	**CO-ORDS**		**SUPER ELEV**
(m)	**(D.M)**	**(m)**	**(D.MS)**	**(m)**	**(m)**	**(m)**	**(D.MS)**	**(m)**		**(m)**
416.360	13.45	144.106	19.4950	2.078	71.981	144.343	3.1740	144.105	8.295	0.090
409.256	14.00	146.608	20.3130	2.188	73.225	146.866	3.2432	146.606	8.733	0.090
402.076	14.15	149.226	21.1553	2.308	74.527	149.508	3.3151	149.244	9.208	0.090
395.143	14.30	151.844	22.0102	2.431	75.828	152.152	3.3918	151.842	9.699	0.090
388.154	14.45	154.578	22.4903	2.565	77.187	154.915	3.4712	154.576	10.231	0.090
381.972	15.00	157.080	23.3343	2.692	78.429	157.444	3.5433	157.078	10.734	0.090
375.710	15.15	159.698	24.2114	2.828	79.729	160.093	4.0221	159.696	11.277	0.090

DESIGN SPEED V= 80.00 km/hr. SPIRAL DATA										
RADIUS	**DEGREE OF CURVE**	**LENGTH OF SPIRAL**	**MINIMUM DEVIATION ANGLES**	**SHIFT**	**TP TO POINT OF SHIFT**	**LONG CHORD**	**DEF ANGLE FROM ORIGIN**	**CO-ORDS**		**SUPER ELEV**
(m)	**(D.M)**	**(m)**	**(D.MS)**	**(m)**	**(m)**	**(m)**	**(D.MS)**	**(m)**		**(m)**
R	**D**	**Ls**	**2∅s**	**S**	**C**	**Lch**	**∅**	**X**	**Y**	**e rise in 1m**
3819.720	1.30									0.025
2864.790	2.00									0.025
2281.691	2.30									0.025
1631.842	3.30									0.025
1432.395	4.00									0.025
1270.104	4.30									0.025
1145.916	5.00									0.025
1039.642	5.30									0.025
954.930	6.00									0.025
879.970	6.30	34.910	2.1623	0.058	17.455	34.911	0.2244	34.910	0.231	0.030
818.511	7.00	37.532	2.3738	0.072	18.765	37.533	0.2616	37.532	0.287	0.030
762.840	7.30	40.272	3.0130	0.089	20.136	40.274	0.3015	40.272	0.354	0.030
716.198	8.00	42.893	3.2553	0.107	21.446	42.895	0.3419	42.893	0.428	0.030
674.068	8.30	45.574	3.5226	0.128	22.786	45.577	0.3844	45.574	0.514	0.040
636.620	9.00	48.255	4.2035	0.152	24.126	48.259	0.4325	48.255	0.610	0.040
603.114	9.30	50.936	4.5020	0.179	25.466	50.941	0.4823	50.936	0.717	0.040
572.958	10.00	53.616	5.2142	0.209	26.806	53.623	0.5336	53.616	0.836	0.040
545.674	10.30	56.297	5.5440	0.242	28.146	56.306	0.5906	56.297	0.168	0.050
520.871	11.00	58.978	6.2915	0.278	29.486	58.989	1.0451	58.978	1.113	0.050
498.224	11.30	61.659	7.0527	0.316	30.826	61.672	1.1053	61.659	1.271	0.050
477.465	12.00	64.340	7.4315	0.361	32.165	64.356	1.1710	64.340	1.445	0.050
458.366	12.30	67.021	8.2239	0.408	33.504	67.040	1.2344	67.021	1.633	0.050
440.737	13.00	69.701	9.0340	0.459	34.843	69.726	1.3033	69.701	1.836	0.060
424.413	13.30	72.382	9.4618	0.514	36.182	72.411	1.3738	72.382	2.056	0.060
409.256	14.00	75.063	10.3032	0.574	37.521	75.098	1.4460	75.063	2.293	0.060
395.143	14.30	77.744	11.1622	0.637	38.859	77.785	1.5237	77.744	2.548	0.060
381.972	15.00	80.425	12.0349	0.706	40.198	80.474	2.0030	80.425	2.820	0.060
369.650	15.30	83.106	12.5253	0.778	41.535	83.163	2.0838	83.105	3.111	0.070
358.099	16.00	85.786	13.4333	0.856	42.873	85.854	2.1703	85.786	3.422	0.070
347.247	16.30	88.467	14.3550	0.939	44.210	88.546	2.2543	88.467	3.752	0.070
337.034	17.00	91.148	15.2943	1.027	45.546	91.240	2.3439	91.148	4.103	0.070
327.405	17.30	93.829	16.2512	1.120	46.882	93.935	2.4350	93.828	4.475	0.080
318.310	18.00	96.510	17.2216	1.219	48.218	96.632	2.5317	96.509	4.869	0.080
309.707	18.30	99.191	18.2101	1.324	49.553	99.330	3.0260	99.190	3.285	0.080
301.557	19.00	101.871	19.2120	1.434	50.887	102.031	3.1258	101.870	5.724	0.080
293.825	19.30	104.552	20.2316	1.550	52.221	104.734	3.2311	104.551	6.186	0.080
286.479	20.00	107.233	21.2648	1.672	53.554	107.439	3.3339	107.232	6.673	0.090
279.492	20.30	109.914	22.3156	1.801	54.886	110.147	3.4423	109.912	7.184	0.090
272.834	21.00	112.595	23.3842	1.936	56.217	112.857	3.5522	112.593	7.721	0.090
266.492	21.30	115.275	24.4703	2.078	57.548	115.571	4.0636	115.273	8.283	0.090
260.435	22.00	117.956	25.5701	2.226	58.877	118.287	4.1804	117.954	8.871	0.090
254.648	22.30	120.637	27.0836	2.381	60.206	121.007	4.2948	120.634	9.487	0.090
249.112	23.00	123.318	28.2147	2.544	61.533	123.730	4.4146	123.315	10.130	0.090
243.812	23.30	125.999	29.3635	2.713	62.859	126.457	4.5358	125.995	10.801	0.090
238.733	24.00	128.680	30.5259	2.890	64.184	129.189	5.0625	128.676	11.500	0.090
233.860	24.30	131.360	32.1060	3.074	65.508	131.924	5.1907	131.356	12.226	0.090

DESIGN SPEED V= 60.00 km/hr. SPIRAL DATA										
RADIUS	**DEGREE OF CURVE**	**LENGTH OF SPIRAL**	**MINIMUM DEVIATION ANGLES**	**SHIFT**	**TP TO POINT OF SHIFT**	**LONG CHORD**	**DEF ANGLE FROM ORIGIN**	**CO-ORDS**		**SUPER ELEV**
(m)	**(D.M)**	**(m)**	**(D.MS)**	**(m)**	**(m)**	**(m)**	**(D.MS)**	**(m)**		**(m)**
R	**D**	**Ls**	**2Øs**	**S**	**C**	**Lch**	**Ø**	**X**	**Y**	**e rise in 1m**
5729.580	1.00									
3819.720	1.30									
2864.790	2.00									
2281.691	2.30									
1909.860	3.00									0.025
1631.842	3.30									0.025
1432.395	4.00									0.025
1270.104	4.30									0.025
1145.916	5.00									0.025
1039.642	5.30									0.025
954.930	6.00									0.025
879.970	6.30									0.025
818.511	7.00									0.025
762.814	7.30									0.025
716.198	8.00									0.025
674.068	8.30									0.025
636.620	9.00									0.025
603.114	9.30									0.025
572.958	10.00									0.025
545.674	10.30									0.025
520.871	11.00									0.025
498.224	11.30	26.012	2.5929	0.057	13.006	26.013	0.2955	26.012	0.226	0.030
477.465	12.00	27.143	3.1526	0.064	13.571	27.145	0.3234	27.143	0.257	0.030
458.366	12.30	28.274	3.3203	0.073	14.137	28.276	0.3520	28.274	0.291	0.030
440.737	13.00	29.405	3.4922	0.082	14.702	29.407	0.3813	29.405	0.327	0.030
424.413	13.30	30.536	4.0721	0.092	15.267	30.538	0.4113	30.536	0.366	0.030
409.256	14.00	31.667	4.2600	0.102	15.833	31.670	0.4420	31.667	0.408	0.030
395.143	14.30	32.798	4.4521	0.113	16.398	32.801	0.4733	32.798	0.454	0.040
381.972	15.00	33.929	5.0522	0.126	16.963	33.933	0.5053	33.929	0.502	0.040
369.650	15.30	35.060	5.2604	0.139	17.529	35.065	0.5420	35.060	0.554	0.040
358.099	16.00	36.191	5.4726	0.152	18.094	36.196	0.5753	36.191	0.609	0.040
347.247	16.30	37.322	6.0929	0.167	18.659	37.328	1.0134	37.322	0.668	0.040
337.034	17.00	38.453	6.3213	0.183	19.224	38.460	1.0521	38.453	0.731	0.040
327.405	17.30	39.584	6.5538	0.199	19.790	39.592	1.0915	39.584	0.797	0.040
318.310	18.00	40.715	7.1943	0.217	20.355	40.724	1.1315	40.715	0.868	0.040
309.707	18.30	41.846	7.4429	0.236	20.920	41.857	1.1723	41.846	0.942	0.040
301.557	19.00	42.977	8.0956	0.255	21.485	42.989	1.2137	42.977	1.020	0.050
293.825	19.30	44.108	8.3604	0.276	22.050	44.122	1.2557	44.108	1.103	0.050
286.479	20.00	45.239	9.0252	0.298	22.615	45.254	1.3025	45.239	1.190	0.050
279.492	20.30	46.370	9.3021	0.321	23.180	46.387	1.3459	46.370	1.282	0.050
272.837	21.00	47.501	9.5831	0.345	23.744	47.521	1.3940	47.501	1.378	0.050
266.492	21.30	48.632	10.2721	0.370	24.309	48.654	1.4428	48.632	1.478	0.050
260.435	22.00	49.763	10.5652	0.396	24.872	49.788	1.4922	49.763	1.584	0.050
254.648	22.30	50.894	11.2704	0.424	25.438	50.922	1.5423	50.894	1.694	0.050
249.112	23.00	52.025	11.5756	0.453	26.003	52.056	1.5931	52.025	1.809	0.060
243.812	23.30	53.156	12.2930	0.483	26.567	53.190	2.0445	53.155	1.930	0.060
283.733	24.00	54.287	13.0144	0.514	27.132	54.325	2.1006	54.286	2.056	0.060
233.860	24.30	55.418	13.3438	0.547	27.696	55.460	2.1534	55.417	2.187	0.060

DESIGN SPEED V= 60.00 km/hr. SPIRAL DATA											
RADIUS	**DEGREE OF CURVE**	**LENGTH OF SPIRAL**	**MINIMUM DEVIATION ANGLES**	**SHIFT**	**TP TO POINT OF SHIFT**	**LONG CHORD**	**DEF ANGLE FROM ORIGIN**	**CO-ORDS**		**SUPER ELEV**
(m)	**(D.M)**	**(m)**	**(D.MS)**	**(m)**	**(m)**	**(m)**	**(D.MS)**	**(m)**		**(m)**
229.183	25.00	56.549	14.0814	0.581	28.260	56.596	2.2108	56.548	2.323	0.060
224.659	25.30	57.680	14.4230	0.617	28.824	57.732	2.2649	57.679	2.465	0.060
220.368	26.00	58.811	15.1727	0.654	29.388	58.868	2.3237	58.810	2.613	0.060
216.211	26.30	59.942	15.5304	0.692	29.952	60.005	2.3831	59.941	2.766	0.060
212.207	27.00	61.073	16.2922	0.732	30.515	61.142	2.4432	61.072	2.925	0.070
208.348	27.30	62.204	17.0621	0.774	31.079	62.280	2.5039	62.203	3.090	0.070
204.628	28.00	63.334	17.4401	0.817	31.642	63.418	2.5653	63.334	3.262	0.070
201.038	28.30	64.465	18.2222	0.861	32.205	64.556	3.0313	64.465	3.439	0.070
197.572	29.00	65.596	19.0123	0.907	32.768	65.695	3.0940	65.596	3.623	0.070
194.223	29.30	66.727	19.4104	0.955	33.331	66.835	3.1613	66.726	3.813	0.070
190.986	30.00	67.858	20.2127	1.005	33.893	67.976	3.2253	67.857	4.009	0.070
187.855	30.30	68.989	21.0230	1.056	34.456	69.117	3.2939	68.988	4.213	0.070
184.825	31.00	70.120	21.4414	1.108	35.018	70.258	3.3632	70.119	4.422	0.080
181.891	31.30	71.251	22.2639	1.163	35.580	71.401	3.4331	71.250	4.639	0.080
179.049	32.00	72.382	23.0944	1.219	36.142	72.544	3.5036	72.381	4.863	0.080
176.235	32.30	73.538	23.5429	1.279	36.716	73.713	3.5758	73.537	5.098	0.080
173.624	33.00	74.644	24.3757	1.337	37.265	74.832	4.0506	74.642	5.331	0.080
170.976	33.30	75.800	25.2405	1.400	37.838	76.003	4.1240	75.798	5.581	0.080
168.517	34.00	76.906	26.0853	1.462	38.386	77.124	4.2001	76.904	5.828	0.080
166.021	34.30	78.062	26.5625	1.529	38.959	78.297	4.2748	78.060	6.093	0.080
163.702	35.00	79.168	27.4232	1.595	39.507	79.420	4.3521	79.165	6.354	0.080
161.346	35.30	80.324	28.3126	1.666	40.079	80.595	4.4321	80.321	6.635	0.090
159.155	36.00	81.430	29.1853	1.736	40.626	81.719	4.5106	81.427	6.911	0.090
156.927	36.30	82.586	30.0911	1.811	41.198	82.896	4.5918	82.583	7.208	0.090
154.854	37.00	83.692	30.5758	1.885	41.744	84.023	5.0714	83.688	7.499	0.090
152.744	37.30	84.848	31.4939	1.964	42.315	85.203	5.1539	84.844	7.812	0.090
150.778	38.00	85.954	32.3945	2.042	42.861	86.332	5.2347	85.949	8.119	0.090
148.777	38.30	87.110	33.3249	2.125	43.431	87.514	5.3224	87.105	8.449	0.090
146.912	39.00	88.216	34.2415	2.207	43.975	88.646	5.4044	88.210	8.772	0.090
145.012	39.30	89.372	35.1843	2.295	44.545	89.830	5.4932	89.366	9.118	0.090
143.240	40.00	90.478	36.1128	2.381	45.089	90.964	5.5803	90.472	9.457	0.090
141.432	40.30	91.634	37.0719	2.474	45.657	92.152	6.0704	91.627	9.821	0.090
139.746	41.00	92.740	38.0124	2.564	46.200	93.289	6.1546	92.732	10.177	0.090
138.025	41.30	93.896	38.5838	2.661	46.767	94.480	6.2458	93.888	10.558	0.090
136.419	42.00	95.002	39.5403	2.757	47.309	95.620	6.3351	94.993	10.931	0.090
134.778	42.30	96.158	40.5240	2.858	47.875	96.814	6.4314	96.149	11.330	0.090
133.246	43.00	97.264	41.4924	2.958	48.416	97.958	6.5218	97.254	11.720	0.090
131.681	43.30	98.420	42.4925	3.065	48.981	99.155	7.0153	98.409	12.138	0.090
130.218	44.00	99.526	43.4729	3.169	49.512	100.302	7.1107	99.514	12.546	0.090

DESIGN SPEED V= 50.00 km/hr. SPIRAL DATA										
RADIUS	**DEGREE OF CURVE**	**LENGTH OF SPIRAL**	**MINIMUM DEVIATION ANGLES**	**SHIFT**	**TP TO POINT OF SHIFT**	**LONG CHORD**	**DEF ANGLE FROM ORIGIN**	**CO-ORDS**		**SUPER ELEV**
(m)	**(D.M)**	**(m)**	**(D.MS)**	**(m)**	**(m)**	**(m)**	**(D.MS)**	**(m)**		**(m)**
R	**D**	**Ls**	**2∅s**	**S**	**C**	**Lch**	**∅**	**X**	**Y**	**e rise in 1m**
5729.580	1.00									
3819.720	1.30									
2864.790	2.00									
2281.691	2.30									
1909.860	3.00									
1631.842	3.30									
1432.395	4.00									
1270.104	4.30									0.025
1145.916	5.00									0.025
1039.642	5.30									0.025
954.930	6.00									0.025
879.970	6.30									0.025
818.511	7.00									0.025
762.814	7.30									0.025
716.198	8.00									0.025
674.068	8.30									0.025
636.620	9.00									0.025
603.114	9.30									0.025
572.958	10.00									0.025
545.674	10.30									0.025
520.871	11.00									0.025
498.224	11.30									0.025
477.465	12.00									0.025
458.366	12.30									0.025
440.737	13.00									0.025
424.413	13.30									0.025
409.256	14.00									0.025
395.143	14.30									0.025
381.972	15.00									0.025
369.650	15.30									0.025
358.099	16.00									0.025
347.247	16.30	21.598	3.3349	0.056	10.799	21.600	0.3538	21.598	0.224	0.030
337.034	17.00	22.253	3.4659	0.061	11.126	22.254	0.3750	22.253	0.245	0.030
327.405	17.30	22.907	4.0032	0.067	11.453	22.909	0.4005	22.907	0.267	0.030
318.310	18.00	23.562	4.1428	0.073	11.780	23.564	0.4224	23.562	0.291	0.030
309.707	18.30	24.216	4.2848	0.079	12.108	24.218	0.4448	24.216	0.316	0.030
301.557	19.00	24.871	4.4332	0.085	12.435	24.873	0.4715	24.871	0.342	0.030
293.825	19.30	25.525	4.5839	0.092	12.762	25.528	0.4946	25.525	0.370	0.030
286.479	20.00	26.180	5.1410	0.100	13.089	26.183	0.5221	26.180	0.399	0.030
279.492	20.30	26.834	5.3004	0.107	13.416	26.838	0.5460	26.834	0.429	0.030
272.837	21.00	27.489	5.4622	0.115	13.743	27.493	0.5743	27.489	0.462	0.040
266.492	21.30	28.143	6.0303	0.124	14.070	28.148	1.0029	28.143	0.495	0.040
260.435	22.00	28.798	6.2008	0.133	14.397	28.803	1.0320	28.798	0.531	0.040
254.648	22.30	29.452	6.3736	0.142	14.725	29.458	1.0615	29.452	0.568	0.040
249.112	23.00	30.107	6.5529	0.152	15.052	30.113	1.0913	30.107	0.606	0.040
243.812	23.30	30.761	7.1344	0.162	15.379	30.768	1.1215	30.761	0.647	0.040
238.733	24.00	31.416	7.3225	0.172	15.706	31.423	1.1522	31.316	0.689	0.040
233.860	24.30	32.070	7.5126	0.183	16.033	32.079	1.1832	32.070	0.733	0.040

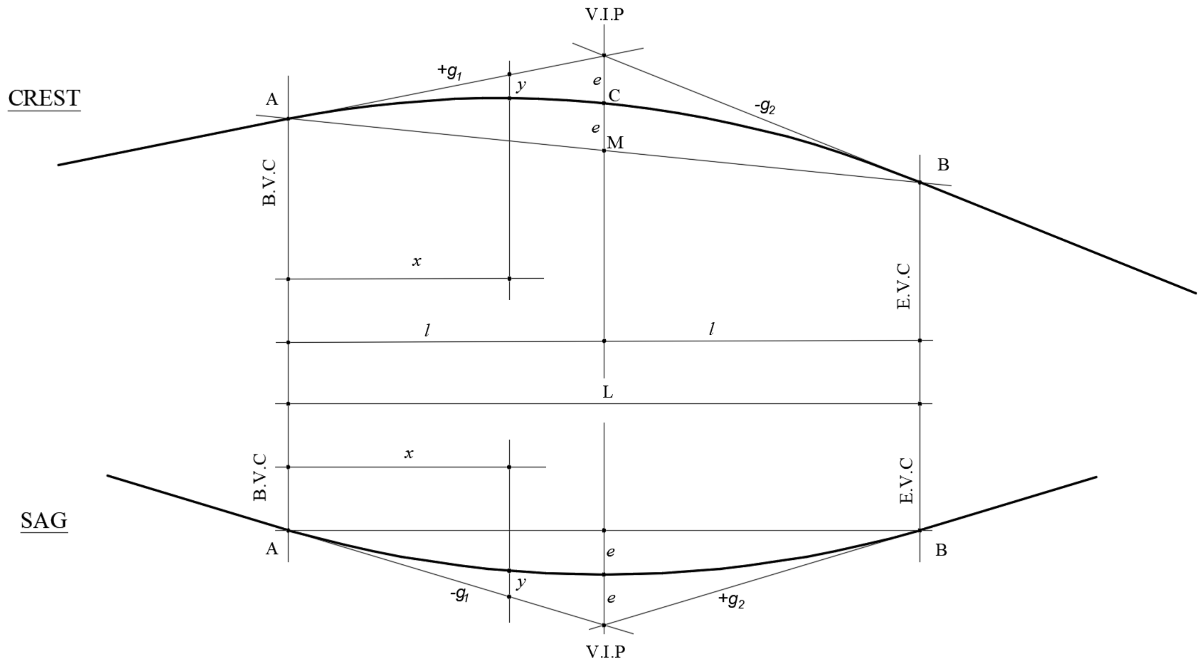
DESIGN SPEED V= 50.00 km/hr. SPIRAL DATA										
RADIUS	**DEGREE OF CURVE**	**LENGTH OF SPIRAL**	**MINIMUM DEVIATION ANGLES**	**SHIFT**	**TP TO POINT OF SHIFT**	**LONG CHORD**	**DEF ANGLE FROM ORIGIN**	**CO-ORDS**		**SUPER ELEV**
(m)	**(D.M)**	**(m)**	**(D.MS)**	**(m)**	**(m)**	**(m)**	**(D.MS)**	**(m)**		**(m)**
229.183	25.00	32.725	8.1052	0.195	16.360	32.734	1.2146	32.725	0.779	0.040
224.689	25.30	33.379	8.3042	0.207	16.687	33.390	1.2504	33.379	0.826	0.040
220.368	26.00	34.034	8.5056	0.219	17.014	34.045	1.2826	34.034	0.876	0.040
216.211	26.30	34.688	9.1133	0.232	17.340	34.701	1.3152	34.688	0.927	0.040
212.207	27.00	35.343	9.3233	0.245	17.667	35.356	1.3521	35.343	0.981	0.050
208.348	27.30	35.997	9.5357	0.259	17.994	36.012	1.3855	35.997	1.036	0.050
204.626	28.00	36.652	10.1545	0.274	18.321	36.668	1.4232	36.652	1.094	0.050
201.038	28.30	37.306	10.3756	0.288	18.648	37.324	1.4613	37.306	1.153	0.050
197.572	29.00	37.961	11.0031	0.304	18.975	37.980	1.4959	37.961	1.215	0.050
194.223	29.30	38.615	11.2330	0.320	19.301	38.636	1.5336	38.615	1.279	0.050
190.986	30.00	39.270	11.4651	0.336	19.628	39.293	1.5741	39.270	1.345	0.050
187.855	30.30	39.924	12.1037	0.354	19.955	39.949	2.0137	39.924	1.413	0.050
184.825	31.00	40.579	12.3446	0.371	20.281	40.606	2.0538	40.579	1.484	0.050
181.891	31.30	41.233	12.5919	0.389	20.608	41.262	2.0942	41.233	1.556	0.050
179.049	32.00	41.888	13.2415	0.408	20.934	41.919	2.1351	41.888	1.632	0.050
176.235	32.30	42.557	13.5009	0.426	21.268	42.359	2.1808	42.557	1.711	0.050
173.624	33.00	43.197	14.1518	0.448	21.587	43.234	2.2219	43.196	1.789	0.060
170.976	33.30	43.866	14.4160	0.469	21.921	43.905	2.2644	43.866	1.874	0.060
168.517	34.00	44.506	15.0755	0.490	22.240	44.548	2.3103	44.505	1.957	0.060
166.021	34.30	45.175	15.3525	0.512	22.574	45.221	2.3536	45.174	2.046	0.060
163.702	35.00	45.815	16.0207	0.534	22.892	45.864	2.4001	45.814	2.134	0.060
161.346	35.30	46.484	16.3025	0.558	23.226	46.537	2.4442	46.483	2.229	0.060
159.155	36.00	47.124	16.5753	0.581	23.545	47.180	2.4915	47.123	2.322	0.060
156.927	36.30	47.793	17.2659	0.606	23.878	47.854	2.5404	47.792	2.422	0.060
154.854	37.00	48.433	17.5513	0.631	24.197	48.498	2.5844	48.432	2.520	0.060
152.744	37.30	49.102	18.2507	0.658	24.530	49.171	3.0340	49.101	2.626	0.060
150.778	38.00	49.742	18.5407	0.684	24.848	49.816	3.0828	49.741	2.730	0.060
148.777	38.30	50.411	19.2450	0.712	25.181	50.290	3.1332	50.410	2.841	0.060
146.912	39.00	51.051	19.5435	0.739	25.500	51.135	3.1827	51.050	2.950	0.070
145.012	39.30	51.720	20.2606	0.769	25.833	51.810	3.2339	51.719	3.067	0.070
143.240	40.00	52.360	20.5638	0.797	26.151	52.455	3.2841	52.359	3.182	0.070
141.432	40.30	53.029	21.2857	0.828	26.483	53.131	3.3401	53.028	3.305	0.070
139.746	41.00	53.669	22.0015	0.859	26.801	53.777	3.3901	53.667	3.426	0.070
138.025	41.30	54.338	22.3323	0.891	27.135	54.453	3.4437	54.336	3.555	0.070
136.419	42.00	54.978	23.0526	0.923	27.452	55.099	3.4954	54.976	3.682	0.070
134.778	42.30	55.647	23.3922	0.953	27.784	55.776	3.5529	55.645	3.818	0.070
133.246	43.00	56.287	24.1212	0.991	28.102	56.423	4.0053	56.285	3.950	0.070
131.681	43.30	56.956	24.4656	1.026	28.434	57.101	4.0635	56.954	4.092	0.070
130.218	44.00	56.596	25.2832	1.061	28.751	57.749	4.1206	57.594	4.231	0.070
128.722	44.30	58.265	25.5604	1.099	29.083	58.427	4.1755	58.263	4.379	0.070
127.324	45.00	58.905	26.3026	1.135	29.400	59.067	4.2333	58.902	4.525	0.080
125.894	45.30	59.574	27.0646	1.175	29.731	59.755	4.2930	59.571	4.608	0.080
124.556	46.00	60.214	27.4154	1.213	30.048	60.405	4.3515	60.213	4.831	0.080
123.187	46.30	60.883	28.1902	1.254	30.379	61.084	4.4119	60.880	4.993	0.080
121.906	47.00	61.523	28.5457	1.294	30.696	61.735	4.4711	61.520	5.151	0.080
120.595	47.30	62.192	29.3253	1.336	31.027	62.416	4.5323	62.188	5.320	0.080
119.366	48.00	62.832	30.0933	1.378	31.343	63.067	4.5922	62.828	5.485	0.080
118.109	48.30	63.501	30.4818	1.423	31.674	63.749	5.0540	63.497	5.661	0.080
116.930	49.00	64.141	31.2544	1.466	31.990	64.401	5.1196	64.137	5.831	0.080
115.723	49.30	64.810	32.0517	1.512	32.320	65.084	5.1812	64.805	6.015	0.080

DESIGN SPEED V= 50.00 km/hr. SPIRAL DATA										
RADIUS	**DEGREE OF CURVE**	**LENGTH OF SPIRAL**	**MINIMUM DEVIATION ANGLES**	**SHIFT**	**TP TO POINT OF SHIFT**	**LONG CHORD**	**DEF ANGLE FROM ORIGIN**	**CO-ORDS**		**SUPER ELEV**
(m)	**(D.M)**	**(m)**	**(D.MS)**	**(m)**	**(m)**	**(m)**	**(D.MS)**	**(m)**		**(m)**
114.592	50.00	65.450	32.4330	1.558	32.636	65.738	5.2424	65.445	6.194	0.080
113.432	50.30	66.119	33.2351	1.606	32.966	66.421	5.3057	66.114	6.384	0.090
112.345	51.00	66.759	34.0249	1.653	33.281	67.076	5.3716	66.754	6.570	0.090
111.230	51.30	67.426	34.4358	1.703	33.611	67.761	5.4356	67.422	6.768	0.090
110.184	52.00	68.068	35.2343	1.752	33.926	68.417	5.5021	68.062	6.961	0.090
109.112	52.30	68.737	36.0540	1.804	34.255	69.103	5.5708	68.731	7.166	0.090
108.105	53.00	69.377	36.4611	1.855	34.569	69.760	6.0340	69.370	7.366	0.090
107.073	53.30	70.046	37.2856	1.909	34.898	70.448	6.1033	70.039	7.579	0.090
106.103	54.00	70.686	38.1013	1.962	35.212	71.106	6.1712	70.678	7.786	0.090
105.105	54.30	71.355	38.5347	2.018	35.540	71.795	6.2412	71.347	8.007	0.090
104.174	55.00	71.995	39.3550	2.073	35.854	72.455	6.3057	71.987	8.222	0.090
103.215	55.30	72.664	40.2011	2.131	36.182	73.145	6.3803	72.655	8.450	0.090
102.314	56.00	73.304	41.0300	2.188	36.495	73.806	6.4454	73.295	8.673	0.090
101.389	56.30	73.973	41.4810	2.249	36.822	74.498	6.5207	73.963	8.910	0.090
100.519	57.00	74.613	42.3145	2.308	37.135	75.160	6.5905	74.603	9.140	0.090
99.626	57.30	75.282	43.1743	2.370	37.462	75.854	7.0624	75.271	9.384	0.090
98.786	58.00	75.922	44.0205	2.431	37.774	76.518	7.1327	75.910	9.622	0.090
97.932	58.30	76.519	44.4851	2.496	38.100	77.213	7.2053	76.579	9.875	0.090
97.112	59.00	77.231	45.3358	2.559	38.412	77.879	7.2802	77.218	10.121	0.090
96.277	59.30	77.900	46.2133	2.626	38.737	78.576	7.3554	77.887	10.382	0.090
95.493	60.00	78.540	47.0726	2.692	39.049	79.243	7.4249	78.526	10.636	0.090
94.686	60.30	79.209	47.5548	2.761	39.373	79.942	7.5026	79.194	10.906	0.090
93.928	61.00	79.849	48.4228	2.828	39.684	80.611	7.5747	79.833	11.167	0.090
93.147	61.30	80.518	49.3139	2.900	40.008	81.311	8.0531	80.502	11.445	0.090
92.413	62.00	81.158	50.1904	2.970	40.318	81.982	8.1257	81.141	11.715	0.090
91.657	62.30	81.827	51.0903	3.044	40.642	82.685	8.2046	81.809	12.002	0.090
90.946	63.00	82.467	51.5715	3.116	40.951	83.358	8.2817	82.448	12.280	0.090
90.214	63.30	83.136	52.4802	3.192	41.274	84.062	8.3612	83.116	12.575	0.090
89.525	64.00	83.776	53.3659	3.267	41.582	84.737	8.4349	83.755	12.862	0.090
88.815	64.30	84.445	54.2835	3.345	41.904	85.444	8.5149	84.423	13.166	0.090
88.147	65.00	85.085	55.1818	3.422	42.212	86.121	8.5931	85.062	13.460	0.090
87.460	65.30	85.754	53.1042	3.503	42.533	86.830	9.0737	85.730	13.773	0.090
86.812	66.00	86.394	57.0112	3.582	42.840	87.509	9.1523	86.369	14.076	0.090
86.145	66.30	87.063	57.5423	3.666	43.161	88.220	9.2334	87.037	14.398	0.090
85.516	67.00	87.703	58.4539	3.748	43.467	88.901	9.3126	87.676	14.709	0.090

DESIGN SPEED V= 40.00 km/hr. SPIRAL DATA										
RADIUS	DEGREE OF CURVE	LENGTH OF SPIRAL	MINIMUM DEVIATION ANGLES	SHIFT	TP TO POINT OF SHIFT	LONG CHORD	DEF ANGLE FROM ORIGIN	CO-ORDS		SUPER ELEV
(m)	(D.M)	(m)	(D.MS)	(m)	(m)	(m)	(D.MS)	(m)		(m)
R	D	Ls	2∅s	S	C	Lch	∅	X	Y	e rise in 1m
1145.916	5.00									
762.814	7.30									0.025
572.958	10.00									0.025
458.366	12.30									0.025
381.972	15.00									0.025
327.405	17.30									0.025
286.479	20.00									0.025
254.648	22.30									0.025
229.183	25.00									0.025
208.348	27.30	18.4307	5.0406	0.068	9.215	18.433	0.5040	18.431	0.272	0.030
190.986	30.00	20.1062	6.0155	0.088	10.052	20.109	1.0018	20.106	0.353	0.030
176.235	32.30	21.7891	7.0502	0.112	10.893	21.794	1.1049	21.789	0.449	0.040
163.702	35.00	23.4572	8.1236	0.140	11.727	23.464	1.2203	23.457	0.560	0.040
152.744	37.30	25.1402	9.2549	0.172	12.567	25.150	1.3414	25.140	0.689	0.040
143.240	40.00	26.8082	10.4324	0.209	13.400	26.821	1.4708	26.808	0.836	0.040
134.778	42.30	28.4912	12.0643	0.251	14.240	28.509	2.0058	28.491	1.003	0.050
127.324	45.00	30.1593	13.3418	0.298	15.073	30.182	2.1531	30.159	1.189	0.050
120.595	47.30	31.8422	15.0743	0.330	15.912	31.873	2.3100	31.842	1.400	0.050
114.592	50.00	33.5103	16.4519	0.408	16.743	33.549	2.4710	33.510	1.631	0.050
109.112	52.30	35.1933	18.2849	0.473	17.581	35.243	3.0417	35.192	1.888	0.060
104.174	55.00	36.8613	20.1625	0.543	18.411	36.924	3.2203	36.860	2.169	0.060
99.626	57.30	38.5443	22.1002	0.621	19.248	38.622	3.4047	38.543	2.479	0.060
95.493	60.00	40.2124	24.0739	0.706	20.076	40.309	4.0008	40.211	2.813	0.060
91.657	62.30	41.8953	26.1121	0.798	20.911	42.013	4.2026	41.893	3.180	0.070
88.147	65.00	43.5634	28.1858	0.897	21.737	43.707	4.4119	43.560	3.573	0.070
84.869	67.30	45.2464	30.3247	1.005	22.570	45.419	5.0309	45.243	4.000	0.070
81.851	70.00	46.9144	32.5024	1.120	23.393	47.121	5.2532	46.910	4.455	0.080
79.017	72.30	48.5974	35.1419	1.245	24.222	48.843	5.4851	48.592	4.948	0.080
76.394	75.00	50.2655	37.4157	1.378	25.042	50.555	6.1240	50.258	5.470	0.080
73.919	77.30	51.9484	40.1557	1.521	25.083	52.289	6.3724	51.940	6.031	0.080
71.620	80.00	53.6165	42.5335	1.672	26.683	54.014	7.0235	53.606	6.623	0.090
69.440	82.30	55.2995	45.3741	1.835	27.504	55.761	7.2839	55.287	7.257	0.090
67.407	85.00	56.9675	48.2521	2.006	28.314	57.501	7.5508	56.952	7.922	0.090
65.473	87.30	58.6505	51.1932	2.189	29.129	59.264	8.2227	58.633	8.631	0.090
63.662	90.00	60.3186	54.1712	2.381	29.934	61.021	8.5007	60.297	9.372	0.090
61.934	92.30	62.0015	57.2130	2.586	30.742	62.804	9.1835	61.976	10.160	0.090
60.311	95.00	63.6696	60.2910	2.801	31.539	64.508	9.4719	63.640	10.980	0.090
58.758	97.30	65.3526	63.4334	3.029	32.339	66.384	10.1648	65.318	11.843	0.090
57.296	100.00	67.0206	67.0114	3.267	33.128	68.183	10.4629	66.981	12.747	0.090
55.892	102.30	68.7056	70.2544	3.519	33.919	70.010	11.1652	68.657	13.695	0.090
54.567	105.00	70.3717	73.5325	3.781	34.698	71.833	11.4721	70.318	14.676	0.090
53.293	107.30	72.0546	77.2800	4.059	35.478	73.686	12.1827	71.993	15.707	0.090
52.083	110.00	73.7227	81.0542	4.348	36.248	75.537	12.4934	73.652	16.769	0.090
50.925	112.30	75.4056	84.5023	4.652	37.014	77.418	13.2114	75.324	17.881	0.090

APPENDIX B DETAILS OF VERTICAL CURVE

(1) Symmetrical vertical curve



(i) Formula of symmetrical vertical curve

$$e = \frac{l \times G}{400} \tag{B.1}$$

$$y = \left(\frac{x}{l}\right)^2 \times e \tag{B.2}$$

(ii) Formula of turning point (Highest or lowest point).

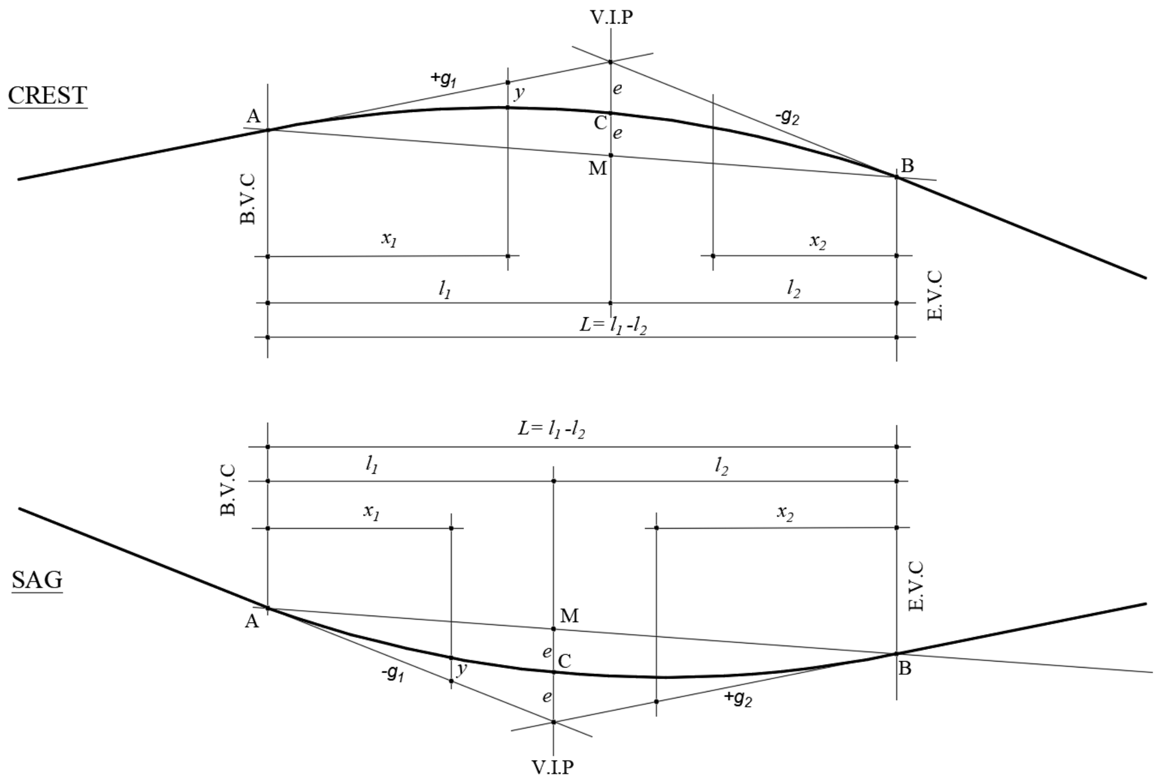
$$x = \frac{g_1 \times L}{g_1 - g_2} \tag{B.3}$$

$$y = \left(\frac{x}{l}\right)^2 \times e \tag{B.4}$$

Where,

e: offset from V.I.P (m)
l: ½ length of V.C (m)
g_1, g_2: Gradients of two tangents (%)
G: Algebraic difference of grades (g_2 - g_1)
x: Distance from B.V.C (m)
y; Tangent offset at a distance ‘x’.

(2) Unsymmetrical vertical curve



(i) Formula of unsymmetrical vertical curve,

$$e = \frac{l_1 \times l_2}{2L} \times \frac{G}{100} \tag{B.5}$$

$$y_1 = \left(\frac{x_1}{l_1}\right)^2 \times e \tag{B.6}$$

$$y_2 = \left(\frac{x_2}{l_2}\right)^2 \times e \tag{B.7}$$

(ii) Formula of turning point (Highest or lowest point).

$$x_1 = \frac{l_1}{l_2} \times \frac{g_1 \times L}{g_1 - g_2} \tag{B.8}$$

$$x_2 = \frac{l_2}{l_1} \times \frac{g_2 \times L}{g_2 - g_1} \tag{B.9}$$

Where,

- e: offset from V.I.P (m)
- l_1, l_2: Length of curve AC and BC(m)
- g_1, g_2: Gradients of two tangents (%)
- G: Algebraic difference of grades (g_2 - g_1)
- x_1, x_2: Distance from B.V.C and E.V.C respectively
- y_1, yy_2: Tangent offset (m)

LENGTH OF VERTICAL CURVES (CREST CURVES)

Algebraic difference of grade (%)	**Design Speed (km/h)**							
	120	**100**	**80**	**60**	**50**	**40**	**30**	**20**
	K (m/%)							
	111	**64**	**30**	**14**	**8**	**4**	**2**	**1**
0.5	100	85	70	50	40	35	25	20
1.0	111	85	70	50	40	35	25	20
1.5	167	96	70	50	40	35	25	20
2.0	222	128	70	50	40	35	25	20
2.5	278	160	75	50	40	35	25	20
3.0	333	192	90	50	40	35	25	20
3.5	389	224	105	50	40	35	25	20
4.0	444	256	120	56	40	35	25	20
4.5	500	288	135	63	40	35	25	20
5.0	555	320	150	70	40	35	25	20
5.5	611	352	165	77	44	35	25	20
6.0	666	384	180	84	48	35	25	20
6.5	722	416	195	91	52	35	25	20
7.0	777	448	210	98	56	35	25	20
7.5	833	480	225	105	60	35	25	20
8.0	888	512	240	112	64	35	25	20
8.5	944	544	255	119	68	35	25	20
9.0	999	576	270	126	72	36	25	20
9.5	1055	608	285	133	76	38	25	20
10.0	1110	640	300	140	80	40	25	20
10.5	1166	672	315	147	84	42	25	20
11.0	1221	704	330	154	88	44	25	20
11.5		736	345	161	92	46	25	20
12.0		768	360	168	96	48	25	20
12.5		800	375	175	100	50	25	20
13.0		832	390	182	104	52	26	20
13.5			405	189	108	54	27	20
14.0			420	196	112	56	28	20
14.5			435	203	116	58	29	20
15.0			450	210	120	60	30	20
15.5				217	124	62	31	20
16.0				224	128	64	32	20
16.5				231	132	66	33	20
17.0				238	136	68	34	20
17.5					140	70	35	20
18.0					144	72	36	20
18.5					148	74	37	20
19.0					152	76	38	20
19.5						78	39	20
20.0						80	40	20
20.5						82	41	20.5
21.5						86	43	21.5
22.0							44	22
22.5							45	22.5
23.0							46	23

LENGTH OF VERTICAL CURVES (SAG CURVES)

Algebraic difference of grade (%)	**Design Speed (km/h)**							
	120	**100**	**80**	**60**	**50**	**40**	**30**	**20**
	K (where street lighting is provided), m/%							
	40	**28**	**18**	**10**	**7**	**5**	**3**	**1**
0.5	100	85	70	50	40	35	25	20
1.0	100	85	70	50	40	35	25	20
1.5	100	85	70	50	40	35	25	20
2.0	100	85	70	50	40	35	25	20
2.5	100	85	70	50	40	35	25	20
3.0	120	85	70	50	40	35	25	20
3.5	140	98	70	50	40	35	25	20
4.0	160	112	72	50	40	35	25	20
4.5	180	126	81	50	40	35	25	20
5.0	200	140	90	50	40	35	25	20
5.5	220	154	99	55	40	35	25	20
6.0	240	168	108	60	42	35	25	20
6.5	260	182	117	65	46	35	25	20
7.0	280	196	126	70	49	35	25	20
7.5	300	210	135	75	53	38	25	20
8.0	320	224	144	80	56	40	25	20
8.5	340	238	153	85	60	43	26	20
9.0	360	252	162	90	63	45	27	20
9.5	380	266	171	95	67	48	29	20
10.0	400	280	180	100	70	50	30	20
10.5	420	294	189	105	74	53	32	20
11.0	440	308	198	110	77	55	33	20
11.5		322	207	115	81	58	35	20
12.0		336	216	120	84	60	36	20
12.5		350	225	125	88	63	38	20
13.0		364	234	130	91	65	39	20
13.5			243	135	95	68	41	20
14.0			252	140	98	70	42	20
14.5			261	145	102	73	44	20
15.0			270	150	105	75	45	20
15.5				155	109	78	47	20
16.0				160	112	80	48	20
16.5				165	116	83	50	20
17.0				170	119	85	51	20
17.5					123	88	53	20
18.0					126	90	54	20
18.5						93	56	20
19.0						95	57	20
19.5						98	59	20
20.0						100	60	20
20.5						103	62	21
21.0						105	63	21
21.5							65	22
22.0							66	22
22.5							68	23

LENGTH OF VERTICAL CURVES (SAG CURVES)

Algebraic difference of grade (%)	Design Speed (km/h)							
	120	100	80	60	50	40	30	20
	K (where street lighting is not provided), m/%							
	52	38	24	15	10	6	4	2
0.5	100	85	70	50	40	35	25	20
1.0	100	85	70	50	40	35	25	20
1.5	100	85	70	50	40	35	25	20
2.0	104	85	70	50	40	35	25	20
2.5	130	95	70	50	40	35	25	20
3.0	156	114	72	50	40	35	25	20
3.5	182	133	84	53	40	35	25	20
4.0	208	152	96	60	40	35	25	20
4.5	234	171	108	68	45	35	25	20
5.0	260	190	120	75	50	35	25	20
5.5	286	209	132	83	55	35	25	20
6.0	312	228	144	90	60	36	25	20
6.5	338	247	156	98	65	39	26	20
7.0	364	266	168	105	70	42	28	20
7.5	390	285	180	113	75	45	30	20
8.0	416	304	192	120	80	48	32	20
8.5	442	323	204	128	85	51	34	20
9.0	468	342	216	135	90	54	36	20
9.5	494	361	228	143	95	57	38	20
10.0	520	380	240	150	100	60	40	20
10.5	546	399	252	158	105	63	42	21
11.0	572	418	264	165	110	66	44	22
11.5		437	276	173	115	69	46	23
12.0		456	288	180	120	72	48	24
12.5		475	300	188	125	75	50	25
13.0		494	312	195	130	78	52	26
13.5			324	203	135	81	54	27
14.0			336	210	140	84	56	28
14.5			348	218	145	87	58	29
15.0			360	225	150	90	60	30
15.5				233	155	93	62	31
16.0				240	160	96	64	32
16.5				248	165	99	66	33
17.0				255	170	102	68	34
17.5					175	105	70	35
18.0					180	108	72	36
18.5						111	74	37
19.0						114	76	38
19.5						117	78	39
20.0						120	80	40
20.5						123	82	41
21.0						126	84	42
21.5							86	43
22.0							88	44
22.5							90	45

Example 1 (Symmetrical Curve)

Find the vertical curve elements for a crest curve with the following Data:

g_1= +1. 03%, g_2 = - 1. 47%, V. I.P. Ch= 0 + 320. The level of V.I.P. = 45.237m.

Design Speed = 100km/h

Solution

G = Algebraic difference in Grades % = g_2 - g_1 = - 1. 47 - 1.03 = - 2.50 %

From the chart for Length of Vertical Curves in metres (minimum) for Crest Curves, for

G = 2.5 % and V=100 km /h, L =160m (min).

So let us assume L = 240m (*l*=120m)

i. Determine e (offset from the V.I.P) from **Equation B.1**

$$e = \frac{l \times G}{400}$$

$$e = \frac{120 \times 2.5}{400} = 0.750m$$

Now let us calculate the offsets at every 20m (**Equation B.2**)

$$y = \left(\frac{x}{l}\right)^2 \times e$$

$$\text{For x} \ = \ 20 \text{ m}, y = \left(\frac{20}{120}\right)^2 \times 0.750m = 0.021m$$

$$\text{For x} \ = \ 40 \text{ m}, y = \left(\frac{40}{120}\right)^2 \times 0.750m = 0.083m$$

$$\text{For x} \ = \ 60 \text{ m}, y = \left(\frac{60}{120}\right)^2 \times 0.750m = 0.1875m$$

$$\text{For x} \ = \ 80 \text{ m}, y = \left(\frac{80}{120}\right)^2 \times 0.750m = 0.333m$$

$$\text{For x} \ = \ 1000 \text{ m}, y = \left(\frac{100}{120}\right)^2 \times 0.750m = 0.521m$$

ii. Position of the Highest Point

$$x = \frac{g_1 \times l}{g_1 - g_2} \qquad \text{(B.3)}$$

$$y = \left(\frac{x}{l}\right)^2 \times e \qquad \text{(B.4)}$$

$$x = \frac{1.30 \times 240}{2.5} = 98.88m$$

$$y = \left(\frac{98.880}{120}\right)^2 \times 0.750 = 0.509m$$

iii. Determine levels on Grade line in metre

At Ch. 0+200 (BVC) = Level at V.I.P. - $(\frac{1.03}{100} \times 120) = 15.237 - 1.236 = 44.001$

At Ch. 0 + 220 = Level at V. I.P. - $(\frac{1.03}{100} \times 120) = 45.231 - 1.030 = 44.207$

At Ch. 0 + 240 = Level at V.I.P. - $(\frac{1.03}{100} \times 80) = 45.237 - 0.824 = 44.413$

At Ch. 0 + 260 = 45.237 - $(\frac{1.03}{100} \times 60) = 45.237 - 0.618 = 44.619$

At Ch. 0 + 280 = 45.237 - $(\frac{1.03}{100} \times 40) = 45.237 - 0.412 = 44.825$

At Ch. 0 + 298.880 (turning point) = 44.001 + $(\frac{1.03}{100} \times 98.880) = 45.019$

At Ch. 0 + 300 = 45.237 - $(\frac{1.03}{100} \times 20) = 45.237 - 0.206 = 45.031$

At Ch. 0 + 320 (VIP) = 45.237

At Ch. 0 + 340 = Level at V.I.P. - $(\frac{1.47}{100} \times 20) = 45.237 - 0.294 = 44.943$

At Ch. 0 + 360 = Level at V.I.P. - $(\frac{1.47}{100} \times 40) = 45.237 - 0.588 = 44.649$

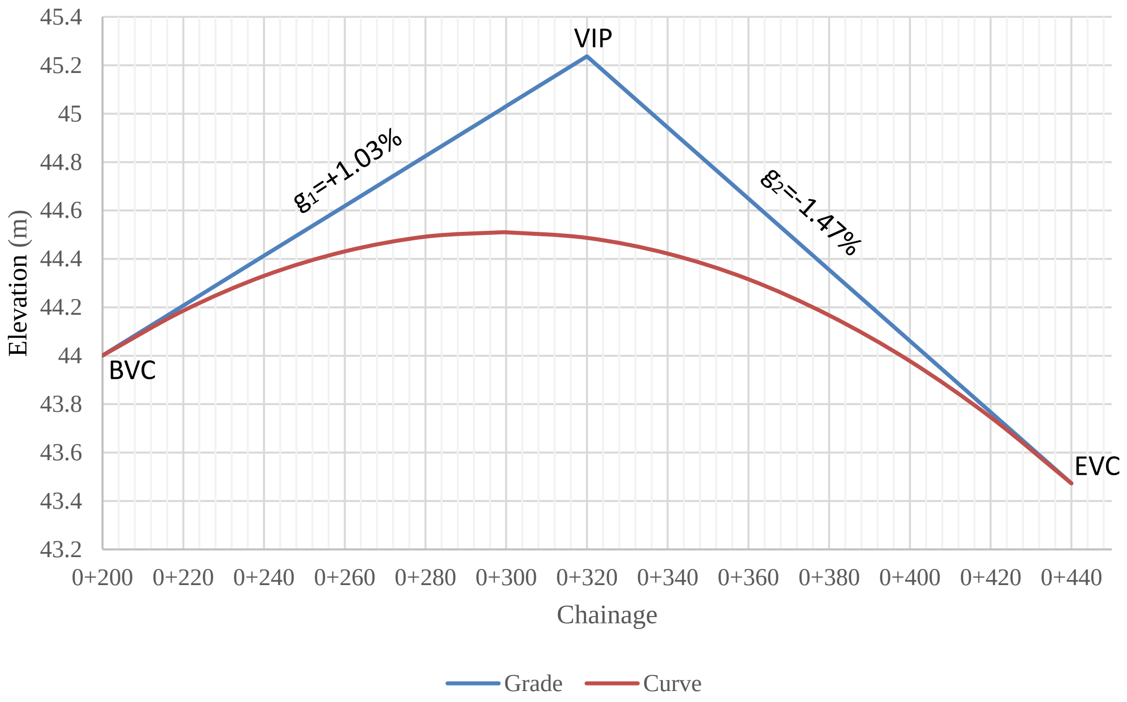
At Ch. 0 + 380 = Level at V.I.P. - $(\frac{1.47}{100} \times 60) = 45.237 - 0.882 = 44.355$

At Ch. 0 + 400 = Level at V.I.P. - $(\frac{1.47}{100} \times 80) = 45.237 - 1.176 = 44.061$

At Ch. 0 + 420 = Level at V.I.P. - $(\frac{1.47}{100} \times 100)\ 45.237 - 1.470 = 43.767$

At Ch. 0 + 440 (EVC) = Level at V.I.P. - $(\frac{1.47}{100} \times 120) = 45.237 - 1.764\ 43.473$

CHAINAGE	LEVEL ON GRADELINE	OFFSET(y)	LEVEL ON CURVE
m	**m**	**m**	**m**
0+200 (BVC)	44.001	0	44.001
0+220	44.207	0.021	44.186
0+240	44.413	0.083	44.330
0+260	44.619	0.188	44.431
0+280	44.825	0.333	44.492
0+299 (turning point)	45.019	0.509	44.510
0+300	45.031	0.521	44.510
0+320	45.237	0.75	44.487
0+340	44.943	0.521	44.422
0+360	44.649	0.333	44.316
0+380	44.355	0.188	44.167
0+400	44.061	0.083	43.978
0+420	43.767	0.021	43.746
0+440 (EVC)	43.473	0	43.473



Example 2 (Unsymmetrical curve)

Find the vertical curve elements for an unsymmetrical crest curve with the following data:

g_1 = +2.00 %, g_2 = -3.00%, V.I.P. Ch = 0+540.00, the level of V. I. P. = 25.000m, Design speed = 80km/h.

There is a constraint that the Vertical Curve should not be extended by more than 100m on the g_2 grade.

Solution

G = Algebraic difference in Grades % = $g_2 - g_1$ = -3.00 - 2.00 = -5.0 %

From the chart for length of Vertical Crest Curves in metres (minimum), for G=5.0% and V=80 km/h, L = 150 m (min).

Let us assume L=240m. Now length of Curve on g_2 grade = 100m. So, Length of Curve on g_1 grade = 140 m.

i. Determine e (offset from the V.I.P) from **Equation B.5**

$$e = \frac{l_1 \times l_2}{2L} \times \frac{G}{100}$$

$$e = \frac{140 \times 100}{2 \times 240} \times \frac{5}{100} = 1.458m$$

Now, let us calculate the offsets at every 20m (**Equations B.6** & **B.7**).

$$y_1 = \left(\frac{x_1}{l_1}\right)^2 \times e$$

$$y_2 = \left(\frac{x_2}{l_2}\right)^2 \times e$$

For $\mathrm{x_1 = \ 20m, y_1 = \left(\frac{20}{140}\right)^2 \times 1.458 = 0.030}$

For $\mathrm{x_1 = \ 40m, y_1 = \left(\frac{40}{140}\right)^2 \times 1.458 = 0.119}$

For $\mathrm{x_1 = \ 60m, y_1 = \left(\frac{60}{140}\right)^2 \times 1.458 = 0.268}$

For $\mathrm{x_1 = \ 80m, y_1 = \left(\frac{80}{140}\right)^2 \times 1.458 = 0.476}$

For $\mathrm{x_1 = \ 100m, y_1 = \left(\frac{100}{140}\right)^2 \times 1.458 = 0.744}$

For $\mathrm{x_1 = \ 120m, y_1 = \left(\frac{120}{140}\right)^2 \times 1.458 = 1.071}$

For $\mathrm{x_2 = \ 20m, y_2 = \left(\frac{20}{100}\right)^2 \times 1.458 = 0.058}$

For $\mathrm{x_2 = \ 40m, y_2 = \left(\frac{40}{100}\right)^2 \times 1.458 = 0.233}$

For $\mathrm{x_2 = \ 60m, y_2 = \left(\frac{60}{100}\right)^2 \times 1.458 = 0.5\ 25}$

For $\mathrm{x_2 = \ 80m, y_2 = \left(\frac{80}{100}\right)^2 \times 1.458 = 0.933}$

Position of the Highest Point (**Equation B.8**)

$$x_1 = \frac{l_1}{l_2} \times \frac{g_1 \times L}{g_1 - g_2}$$

$$x_1 = \frac{140}{100} \times \frac{2 \times 240}{5} = 134.4\mathrm{m\ i.e\ CH0} + 534.400$$

$$Where\ y_1 = \left(\frac{134.4}{140}\right) \times 1.458 = 1.344\mathrm{m}$$

ii. Determine levels on Grade line in metre

At Ch. 0 + 400 (BVC) = Level at V. I P - $(\frac{2.00}{100} \times 140) = 25.000 - 2800 = 22.200$

At Ch. 0 + 420 = Level at V. I P -$\left(\frac{2.00}{100} \times 120\right) = 25.000 - 2.400 = 22.600$

At Ch. 0 + 440 = Level at V. I P - ($\frac{2.00}{100} \times 100) = 25.000 - 2.000 = 23.000$

At Ch. 0 + 460 = Level at V. I P - ($\frac{2.00}{100} \times 80) = 25.000 - 1.600 = 23.400$

At Ch. 0 + 4,80 = Level at V. I P - ($\frac{2.00}{100}$ ×60) = 25.000 - 1.200 = 23.800

At Ch. 0 + 500 = Level at V. I P - ($\frac{2.00}{100} \times 40) = 25.000 - 0.800 = 24.200$

At Ch. 0. 520 = Level at V. I P - ($\frac{2.00}{100} \times 20) = 25.000 - 0.400 = 24.600$

At Ch. 0 + 540 (V.I.P) = 25.000

At Ch. 0+534.4 (turning point) =22.200 + ($\frac{200}{100} \times 134.4) = 24.888$

At Ch. 0 + 560 = Level at V. I P - ($\frac{3.00}{100} \times 20) = 25.000 - 0.600 = 24.400$

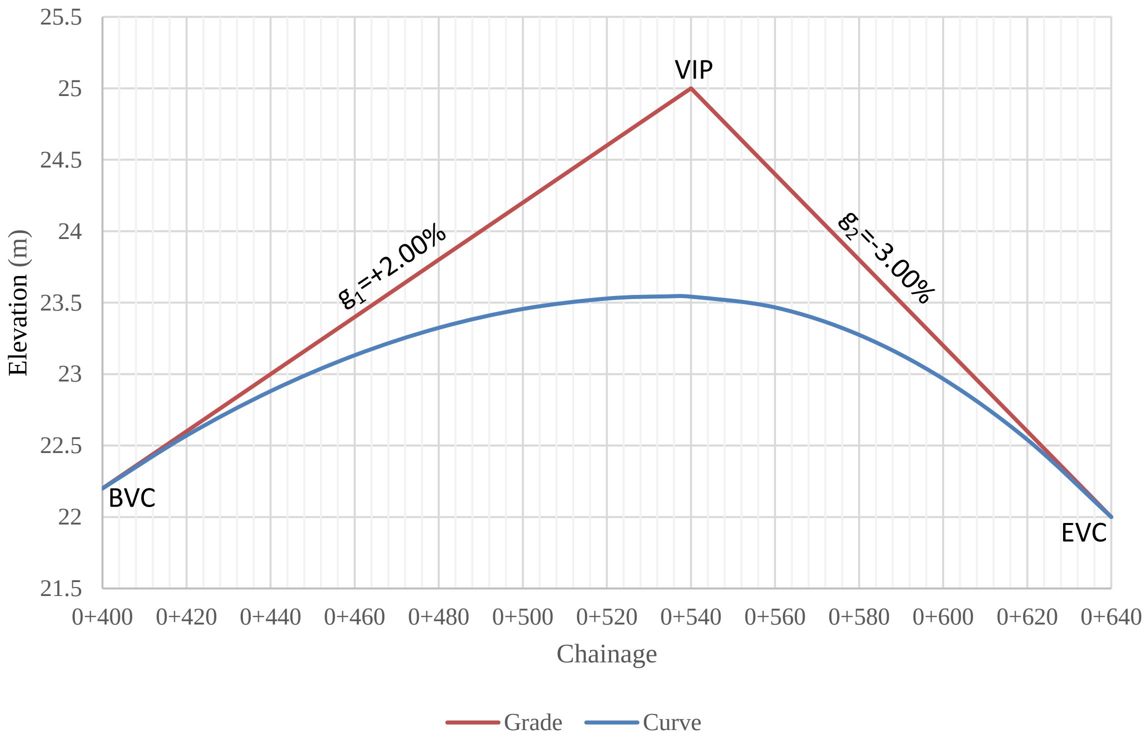
At Ch. 0 + 580 = Level at V. I P - ($\frac{3.00}{100} \times 40) = 25.000 - 1.200 = 23.800$

At Ch. 0 + 600 = Level at V. I P - ($\frac{3.00}{100} \times 60) = 25.000 - 1.800 = 23.200$

At Ch. 0 + 620 = Level at V. I P - ($\frac{3.00}{100} \times 80) = 25.000 - 2.400 = 22.600$

At Ch. 0 = 640 = Level at V. I P - ($\frac{3.00}{100} \times 100) = 25.000 - 3.000 = 22.000$

CHAINAGE	LEVEL ON GRADELINE	OFFSET(y)	LEVEL ON CURVE
m	m	m	m
0+400 (BVC)	22.2	0	22.2
0+420	22.6	0.03	22.57
0+440	23.0	0.119	22.881
0+460	23.4	0.268	23.132
0+480	23.8	0.476	23.324
0+500	24.2	0.764	23.456
0+520	24.6	1.071	23.529
0+534 (turning point)	24.888	1.344	23.544
0+540	25.0	1.458	23.542
0+560	24.4	0.933	23.467
0+580	23.8	0. 525	23.275
0+600	23.2	0. 233	22.967
0+620	22.6	0.058	22.542
0+640 (EVC)	22.0	0.000	22


25.5
25
24.5
24
23.5
23
22.5
22
21.5
Elevation (m)
VIP
g_1=+2.00%
g_2=-3.00%
BVC
EVC
0+400
0+420
0+440
0+460
0+480
0+500
0+520
0+540
0+560
0+580
0+600
0+620
0+640
Chainage
Grade
Curve

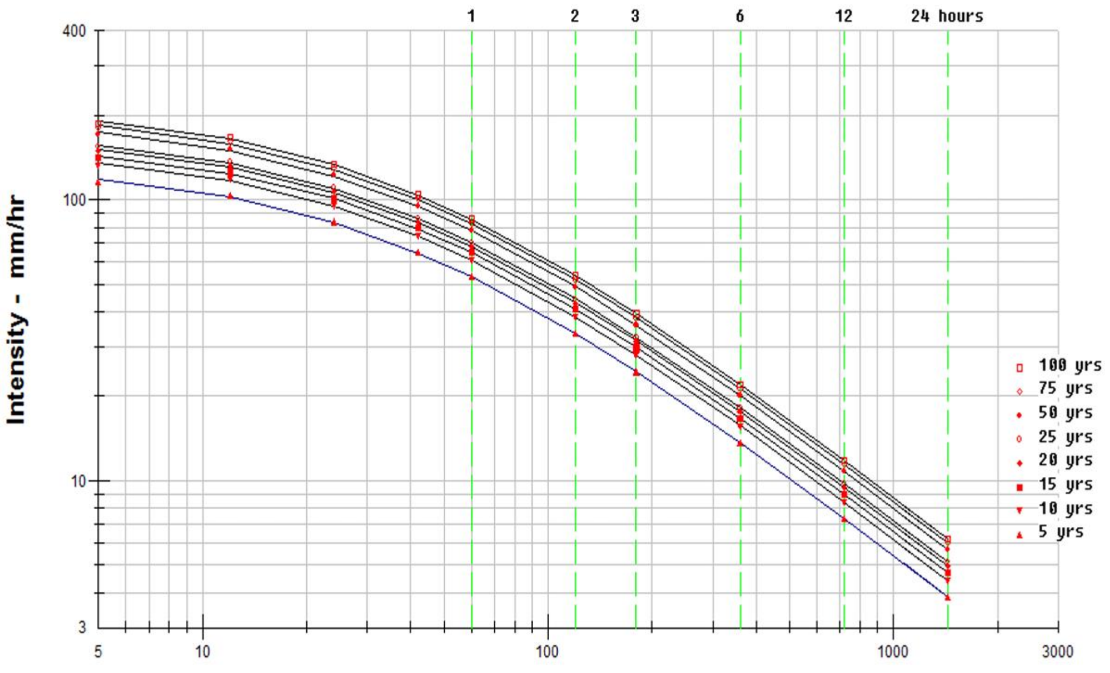
APPENDIX C RAINFALL INTENSITY

Weather Station: Abetifi

Longitude: 0°44'46.15"W

Latitude: 6°40'16.57"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	103.56	118.42	126.24	132.94	137.41	151.94	160.87	166.46
0.4	83.59	95.59	101.90	107.31	110.92	122.64	129.85	134.36
0.7	64.98	74.30	79.21	83.42	86.22	95.33	100.94	104.44
1	53.24	60.87	64.89	68.34	70.64	78.10	82.70	85.57
2	33.40	38.20	40.72	42.88	44.32	49.01	51.89	53.69
3	24.44	27.95	29.79	31.37	32.43	35.86	37.97	39.28
6	13.66	15.62	16.65	17.53	18.12	20.04	21.22	21.95
12	7.34	8.40	8.95	9.42	9.74	10.77	11.40	11.80
24	3.86	4.42	4.71	4.96	5.13	5.67	6.00	6.21



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

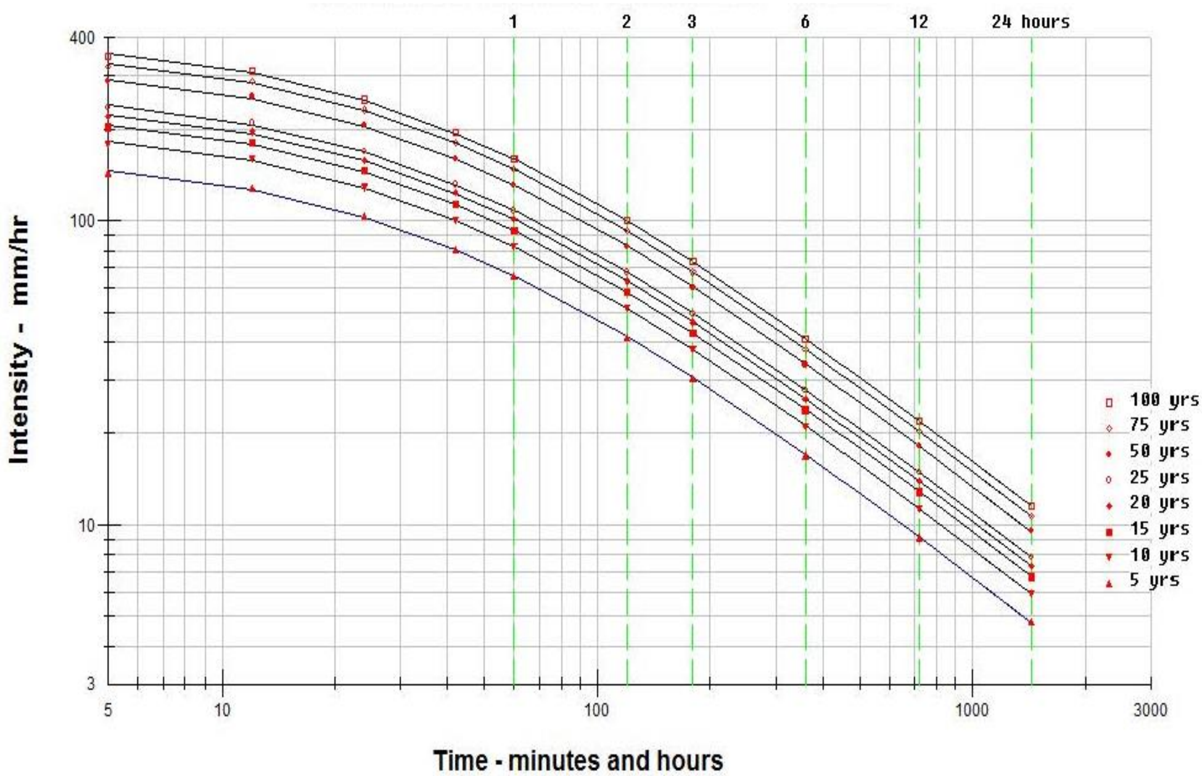
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Accra

Longitude: 0°11'13.07"W

Latitude: 5°36'13.38"N

Duration (hrs)	**Return period (yrs)**							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	128.47	159.76	180.98	196.62	210.03	255.83	287.11	310.57
0.4	103.70	128.95	146.08	158.71	169.53	206.50	231.75	250.69
0.7	80.61	100.24	113.56	123.37	131.78	160.52	180.15	194.87
1	66.04	82.12	93.03	101.08	107.97	131.51	147.59	159.65
2	41.44	51.53	58.38	63.42	67.74	82.52	92.61	100.17
3	30.32	37.70	42.71	46.40	49.57	60.38	67.76	73.30
6	16.94	21.07	23.87	25.93	27.70	33.74	37.87	40.96
12	9.11	11.33	12.83	13.94	14.89	18.14	20.35	22.02
24	4.79	5.96	6.75	7.33	7.83	9.54	10.71	11.58



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

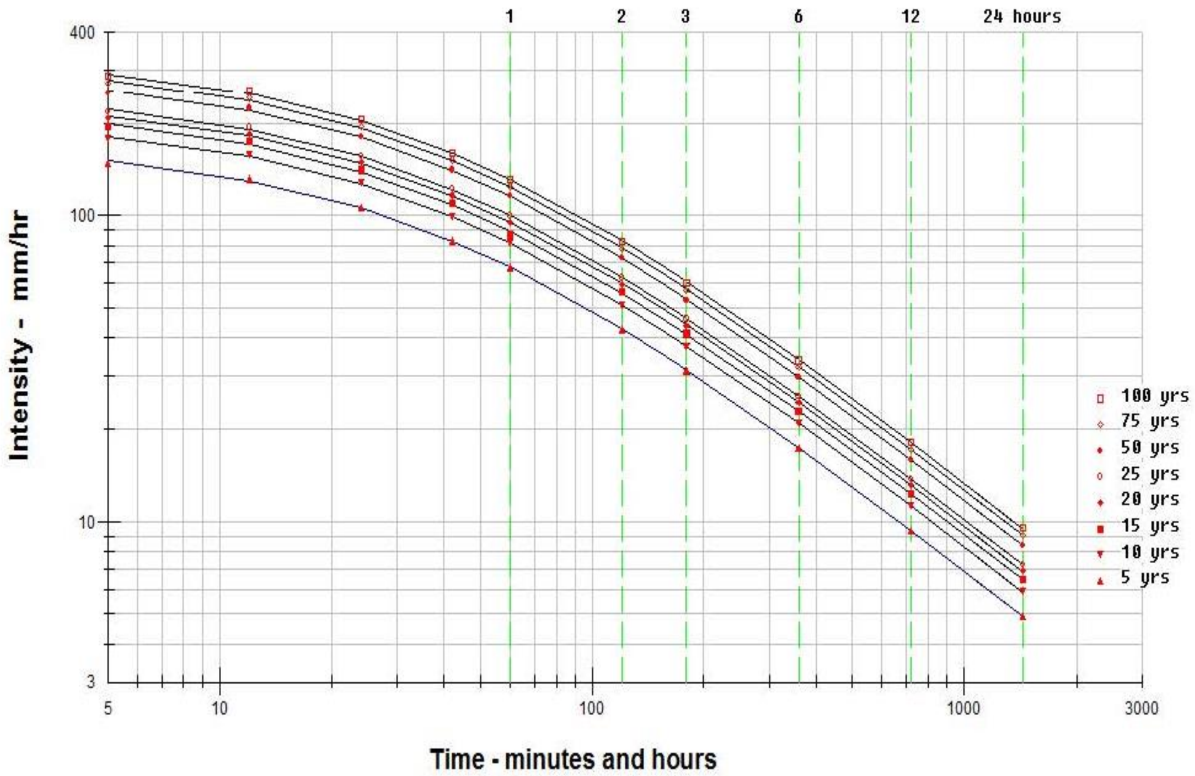
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Ada Foah

Longitude: 0°37'4.75"E

Latitude: 5°46'46.84"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	131.83	158.64	174.28	185.45	194.39	224.55	242.43	255.83
0.4	106.41	128.05	140.67	149.69	156.90	181.25	195.68	206.50
0.7	82.71	99.54	109.35	116.36	121.97	140.90	152.11	160.52
1	67.77	81.55	89.59	95.33	99.93	115.43	124.62	131.51
2	42.52	51.17	56.21	59.82	62.70	72.43	78.19	82.52
3	31.11	37.44	41.13	43.77	45.88	52.99	57.21	60.38
6	17.39	20.92	22.99	24.46	25.64	29.62	31.97	33.74
12	9.35	11.25	12.36	13.15	13.78	15.92	17.19	18.14
24	4.92	5.92	6.50	6.92	7.25	8.38	9.04	9.54



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

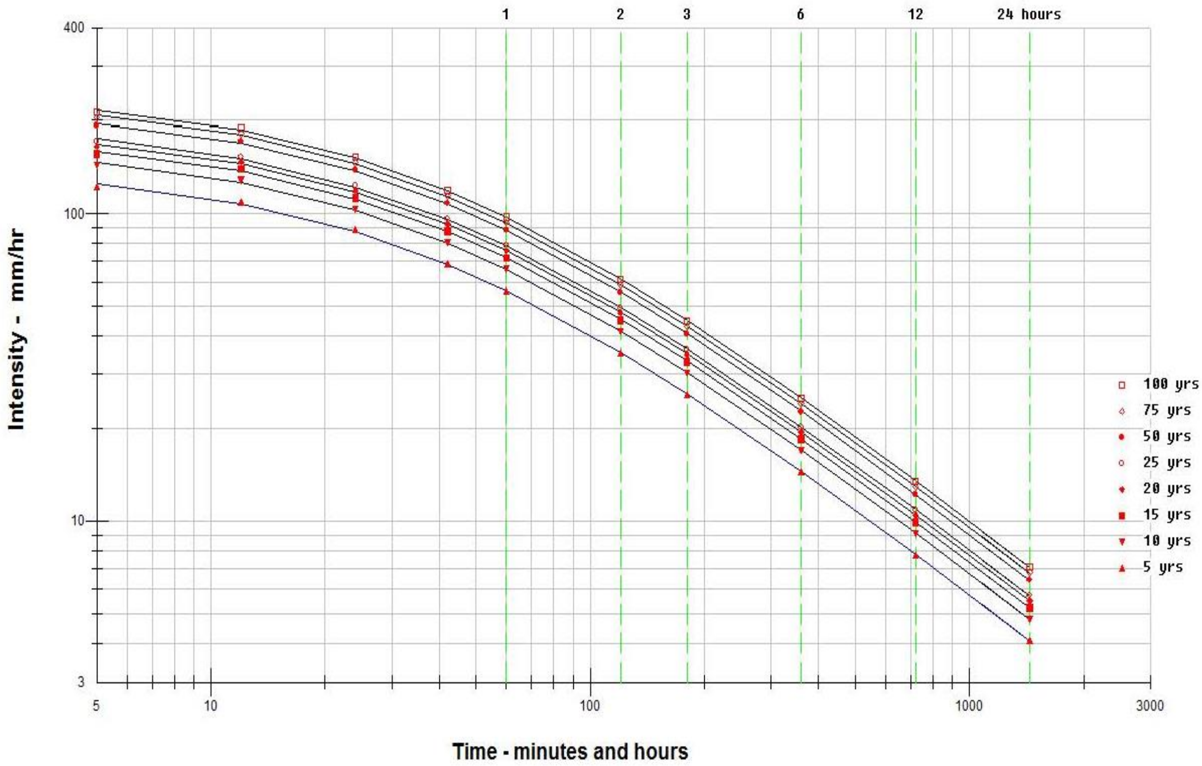
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Adidome

Longitude: 0°30'41.78"E

Latitude: 6° 4'22.71"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	109.37	128.47	139.65	147.47	153.05	172.04	182.10	189.92
0.4	88.28	103.70	112.72	119.03	123.54	138.87	146.99	153.30
0.7	68.63	80.61	87.62	92.53	96.03	107.95	114.26	119.17
1	56.22	66.04	71.79	75.81	78.68	88.44	93.61	97.63
2	35.28	41.44	45.04	47.56	49.37	55.49	58.74	61.26
3	25.81	30.32	32.96	34.80	36.12	40.60	42.98	44.82
6	14.42	16.94	18.42	19.45	20.19	22.69	24.02	25.05
12	7.75	9.11	9.90	10.45	10.85	12.20	12.91	13.46
24	4.08	4.79	5.21	5.50	5.71	6.42	6.79	7.08



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

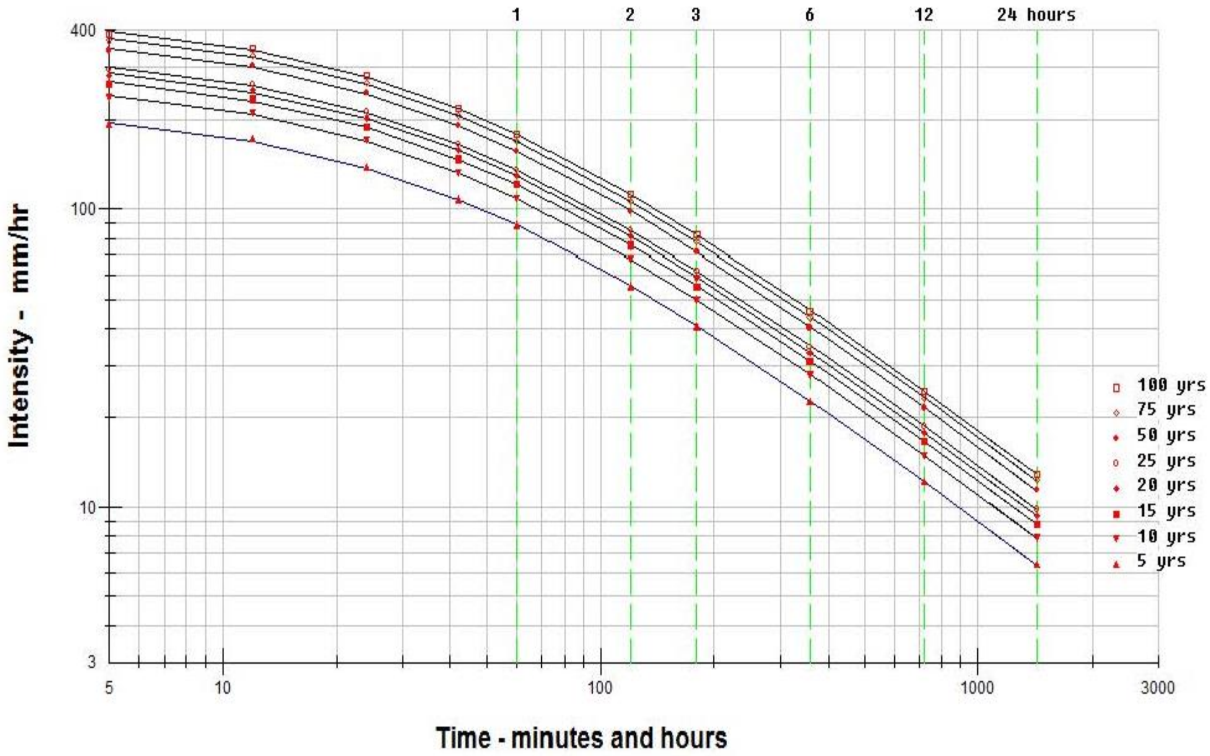
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Agona Nkwanta

Longitude: 1°45'60.00"W

Latitude: 4°53'60.00"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	172.04	211.15	234.61	251.36	263.65	304.99	329.57	347.44
0.4	138.87	170.43	189.37	202.89	212.81	246.18	266.02	280.44
0.7	107.95	132.48	147.20	157.72	165.43	191.37	206.79	218.00
1	88.44	108.54	120.60	129.22	135.53	156.78	169.42	178.60
2	55.49	68.10	75.67	81.08	85.04	98.37	106.30	112.07
3	40.60	49.83	55.37	59.32	62.22	71.98	77.78	82.00
6	22.69	27.85	30.94	33.15	34.77	40.22	43.47	45.82
12	12.20	14.97	16.63	17.82	18.69	21.62	23.36	24.63
24	6.42	7.88	8.75	9.38	9.83	11.38	12.29	12.96



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

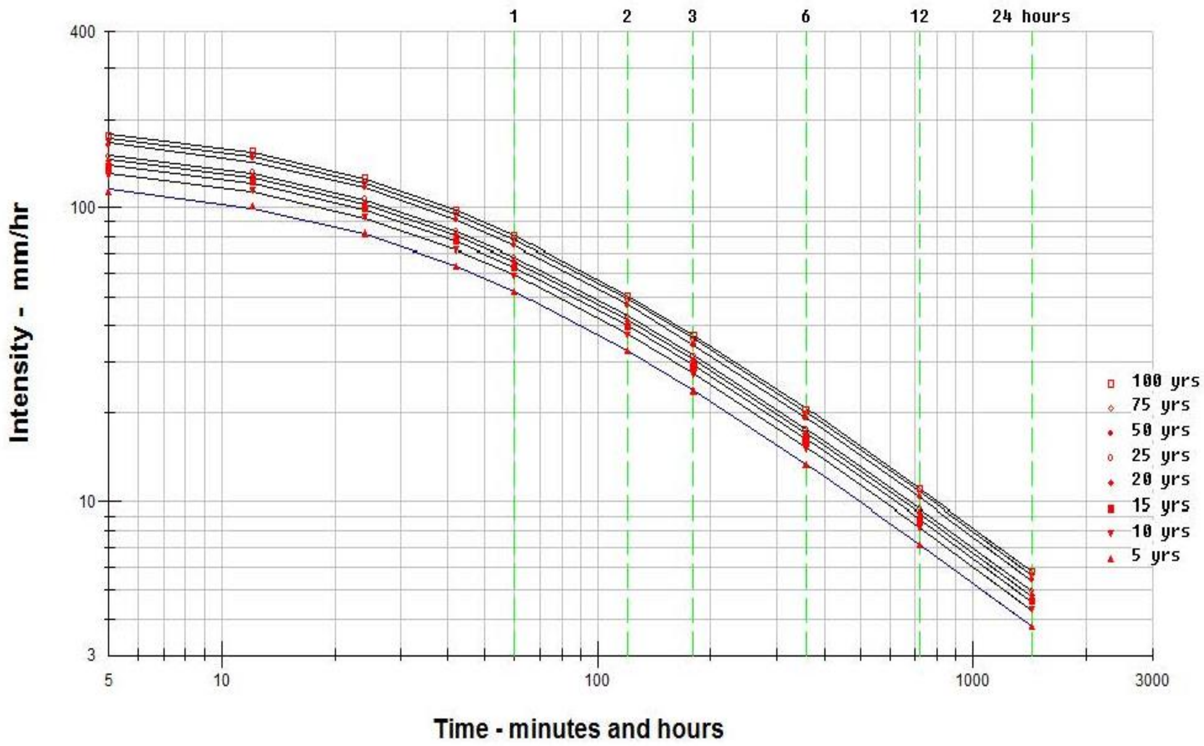
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Akatsi

Longitude: 0°47'54.45"E

Latitude: 6° 7'41.71"N

Duration (hrs)	Return period (yrs)								
	2	5	10	15	20	25	50	75	100
0.2	79.21	101.33	115.07	122.89	128.47	132.94	145.23	151.94	156.40
0.4	63.93	81.79	92.88	99.19	103.70	107.31	117.23	122.64	126.25
0.7	49.70	63.58	72.20	77.11	80.61	83.42	91.13	95.33	98.14
1	40.72	52.09	59.15	63.17	66.04	68.34	74.66	78.10	80.40
2	25.55	32.68	37.11	39.64	41.44	42.88	46.84	49.01	50.45
3	18.69	23.91	27.16	29.00	30.32	31.37	34.28	35.86	36.91
6	10.45	13.36	15.18	16.21	16.94	17.53	19.15	20.04	20.63
12	5.62	7.18	8.16	8.71	9.11	9.42	10.30	10.77	11.09
24	2.95	3.78	4.29	4.58	4.79	4.96	5.42	5.67	5.83



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

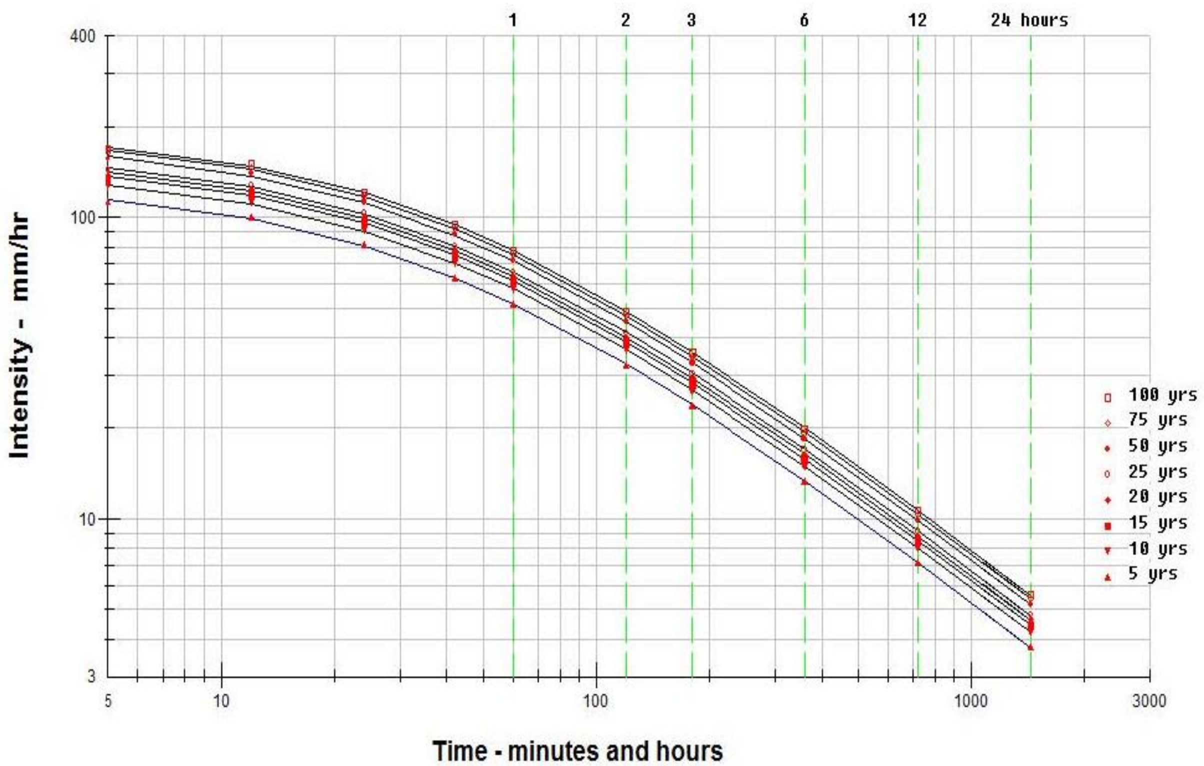
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Akim Oda

Longitude: 0°58'21.03"W

Latitude: 5°55'46.40"N

Duration (hrs)	**Return period (yrs)**							
	2	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	82.78	112.83	119.54	124.01	128.47	139.65	146.35	150.82
0.4	66.82	91.08	96.49	100.09	103.70	112.72	118.13	121.74
0.7	51.94	70.80	75.00	77.81	80.61	87.62	91.83	94.63
1	42.55	58.00	61.45	63.75	66.04	71.79	75.23	77.53
2	26.70	36.39	38.56	40.00	41.44	45.04	47.20	48.65
3	19.54	26.63	28.21	29.27	30.32	32.96	34.54	35.59
6	10.92	14.88	15.77	16.35	16.94	18.42	19.30	19.89
12	5.87	8.00	8.47	8.79	9.11	9.90	10.38	10.69
24	3.09	4.21	4.46	4.63	4.79	5.21	5.46	5.63



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

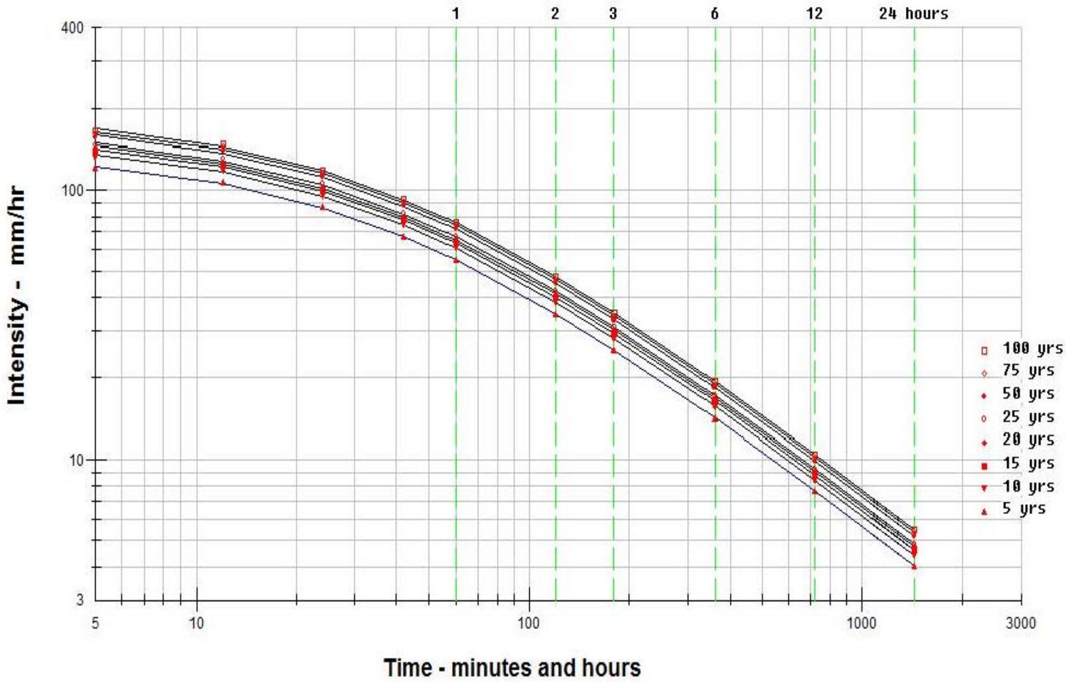
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Akosombo

Longitude: 0° 2'39.56"E

Latitude: 6°16'0.49"N

Duration (hrs)	Return period (yrs)								
	2	5	10	15	20	25	50	75	100
0.2	89.93	107.58	118.42	124.01	127.36	130.71	139.65	144.12	147.47
0.4	72.59	86.84	95.59	100.09	102.80	105.50	112.72	116.33	119.03
0.7	56.43	67.50	74.30	77.81	79.91	82.01	87.62	90.43	92.53
1	46.23	55.30	60.87	63.75	65.47	67.19	71.79	74.08	75.81
2	29.01	34.70	38.20	40.00	41.08	42.16	45.04	46.48	47.56
3	21.22	25.39	27.95	29.27	30.06	30.85	32.96	34.01	34.80
6	11.86	14.19	15.62	16.35	16.80	17.24	18.42	19.01	19.45
12	6.38	7.63	8.40	8.79	9.03	9.27	9.90	10.22	10.45
24	3.35	4.01	4.42	4.63	4.75	4.88	5.21	5.38	5.50



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

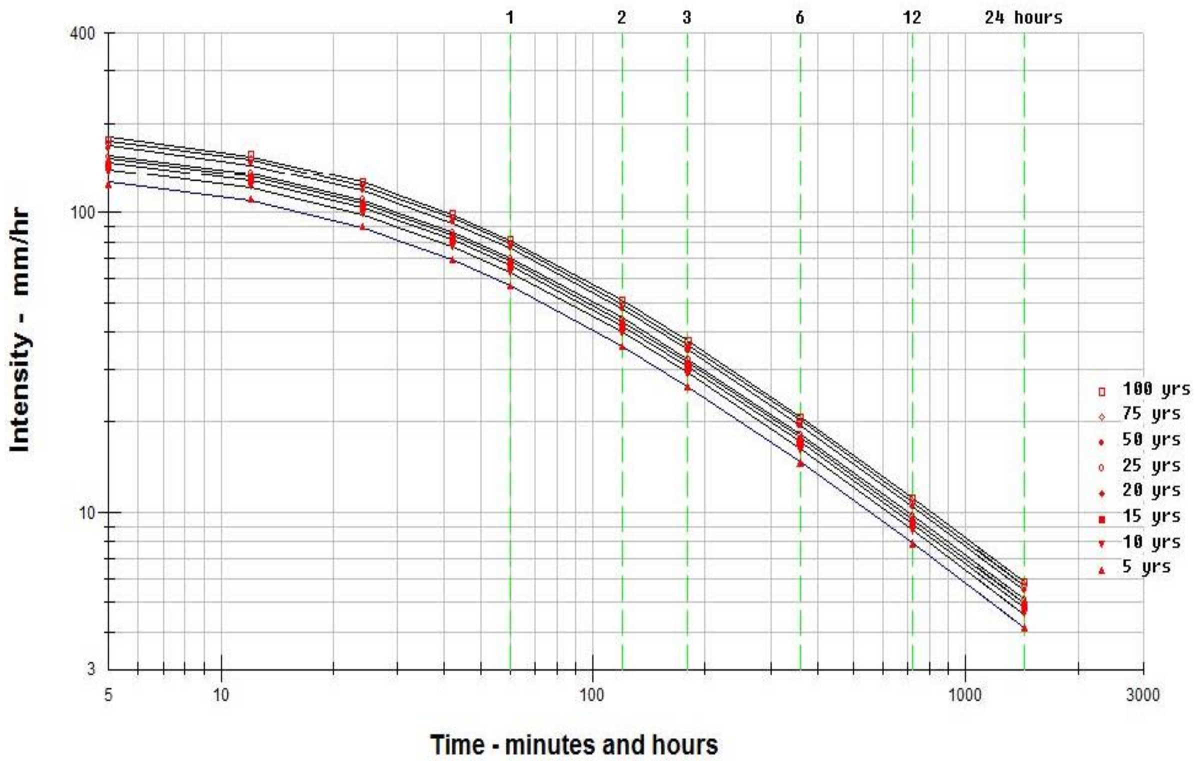
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Akuse

Longitude: 0° 7'39.62"E

Latitude: 6° 6'6.30"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	111.16	122.89	129.59	134.06	137.41	147.47	153.05	157.52
0.4	89.72	99.19	104.60	108.21	110.92	119.03	123.54	127.15
0.7	69.75	77.11	81.31	84.12	86.22	92.53	96.03	98.84
1	57.14	63.17	66.62	68.91	70.64	75.81	78.68	80.97
2	35.85	39.64	41.80	43.24	44.32	47.56	49.37	50.81
3	26.23	29.00	30.58	31.64	32.43	34.80	36.12	37.18
6	14.66	16.21	17.09	17.68	18.12	19.45	20.19	20.78
12	7.88	8.71	9.19	9.50	9.74	10.45	10.85	11.17
24	4.15	4.58	4.83	5.00	5.13	5.50	5.71	5.88



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

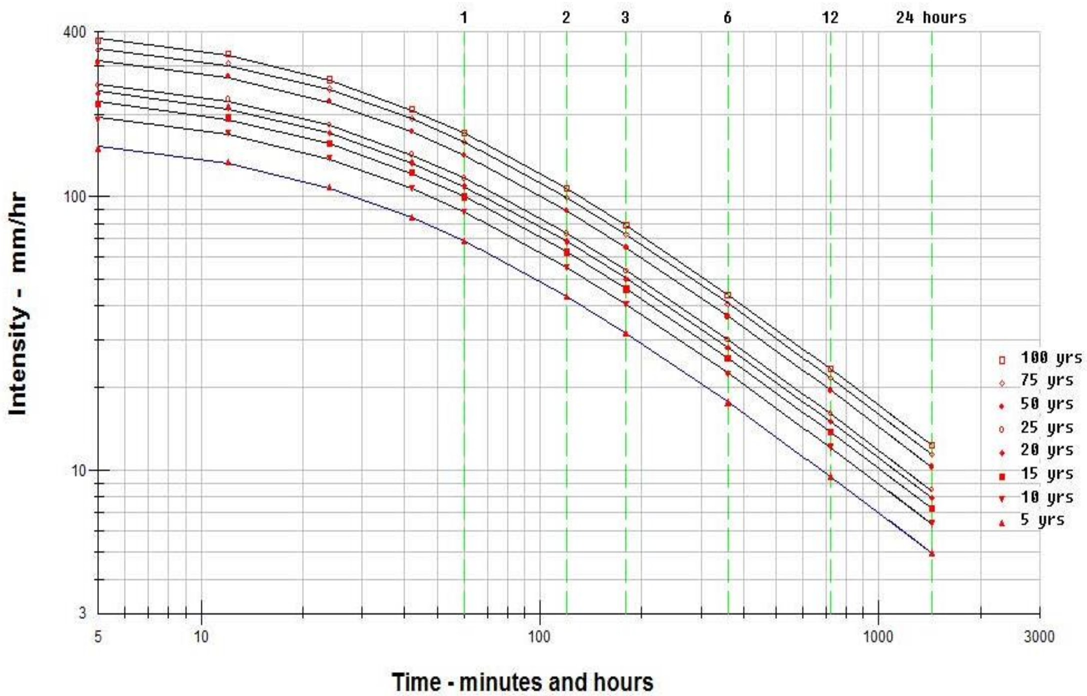
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Apam

Longitude: 0°44'20.49"W

Latitude: 5°17'38.87"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	134.06	170.93	194.39	212.26	226.79	275.94	307.22	331.80
0.4	108.21	137.97	156.90	171.33	183.06	222.73	247.98	267.82
0.7	84.12	107.25	121.97	133.18	142.30	173.14	192.77	208.19
1	68.91	87.87	99.93	109.12	116.58	141.85	157.93	170.56
2	43.24	55.13	62.70	68.46	73.15	89.00	99.09	107.02
3	31.64	40.34	45.88	50.09	53.52	65.12	72.51	78.31
6	17.68	22.54	25.64	27.99	29.91	36.39	40.52	43.76
12	9.50	12.12	13.78	15.05	16.08	19.56	21.78	23.52
24	5.00	6.38	7.25	7.92	8.46	10.29	11.46	12.38



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

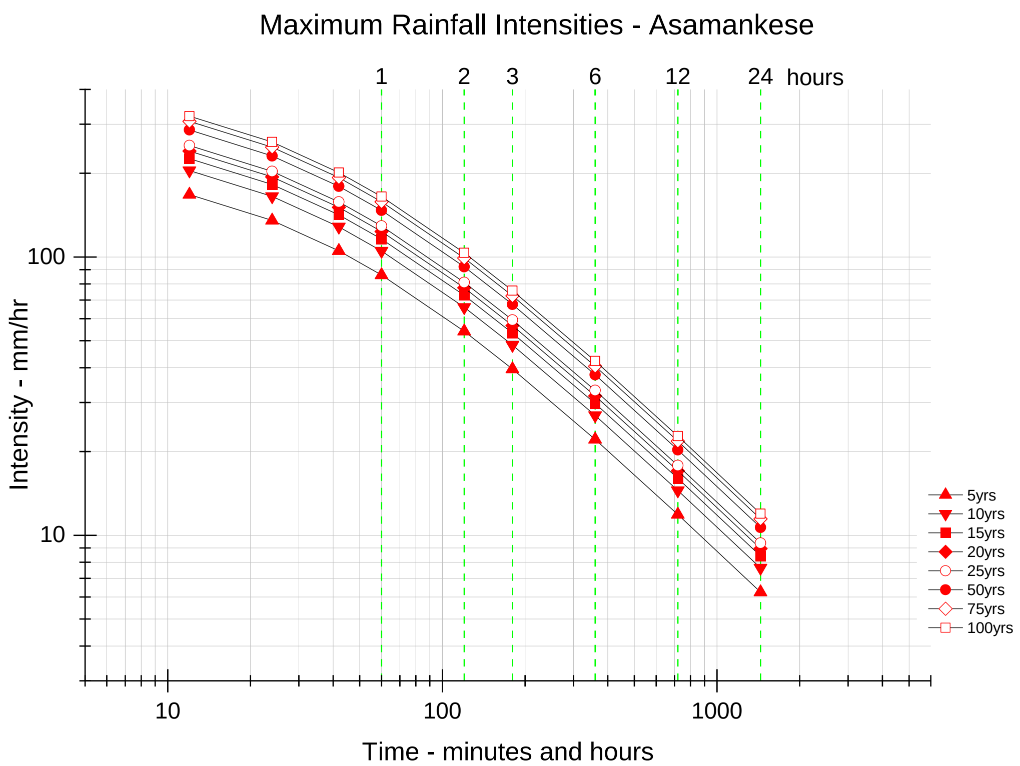
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Asamankese

Longitude:

Latitude:

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	167.58	204.44	225.67	240.19	251.36	286.00	307.22	320.63
0.4	135.26	165.02	182.15	193.88	202.89	230.85	247.98	258.80
0.7	105.15	128.28	141.60	150.71	157.72	179.45	192.77	201.18
1	86.14	105.10	116.01	123.47	129.22	147.02	157.93	164.82
2	54.05	65.94	72.79	77.47	81.08	92.25	99.09	103.42
3	39.55	48.25	53.26	56.69	59.32	67.50	72.51	75.67
6	22.10	26.96	29.76	31.68	33.15	37.72	40.52	42.29
12	11.88	14.49	16.00	17.03	17.82	20.28	21.78	22.73
24	6.25	7.63	8.42	8.96	9.38	10.67	11.46	11.96



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

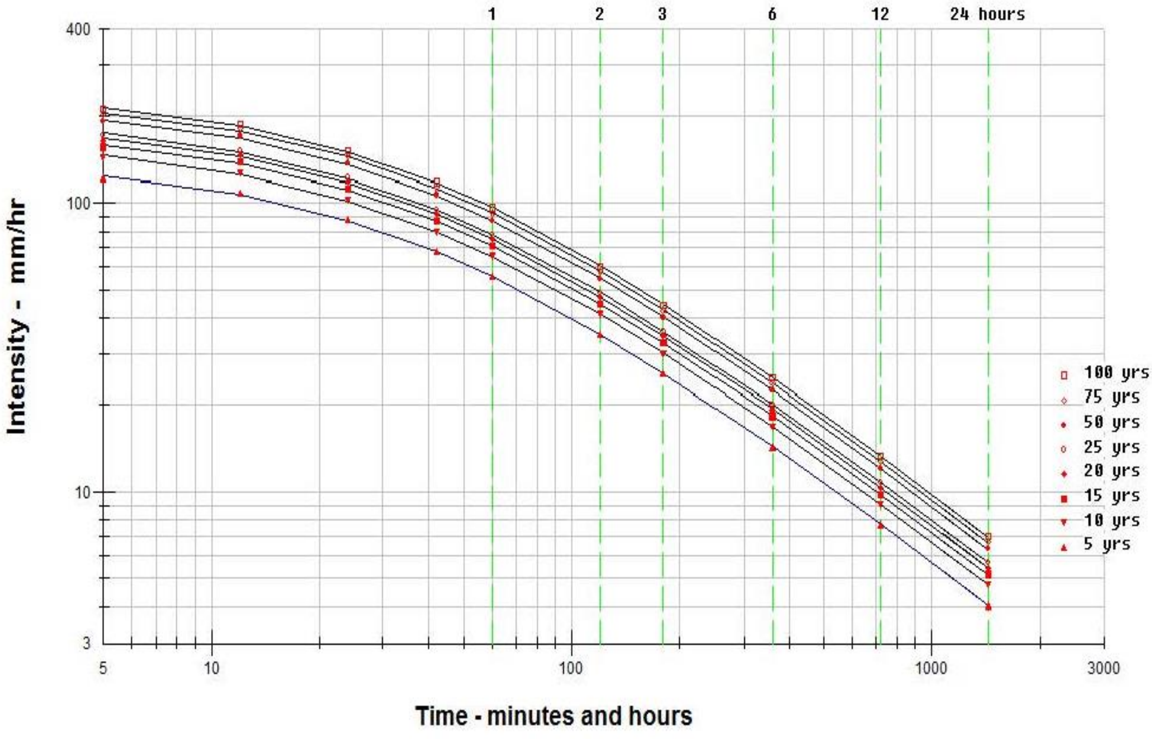
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Asankragua

Longitude: 2°26'8.51"W

Latitude: 5°48'51.62"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	108.70	127.36	138.53	146.35	151.94	169.81	179.86	187.68
0.4	87.74	102.80	111.82	118.13	122.64	137.07	145.18	151.49
0.7	68.20	79.91	86.92	91.83	95.33	106.55	112.86	117.76
1	55.88	65.47	71.21	75.23	78.10	87.29	92.46	96.48
2	35.06	41.08	44.68	47.20	49.01	54.77	58.01	60.54
3	25.65	30.06	32.69	34.54	35.86	40.08	42.45	44.29
6	14.34	16.80	18.27	19.30	20.04	22.40	23.72	24.75
12	7.71	9.03	9.82	10.38	10.77	12.04	12.75	13.31
24	4.05	4.75	5.17	5.46	5.67	6.33	6.71	7.00



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

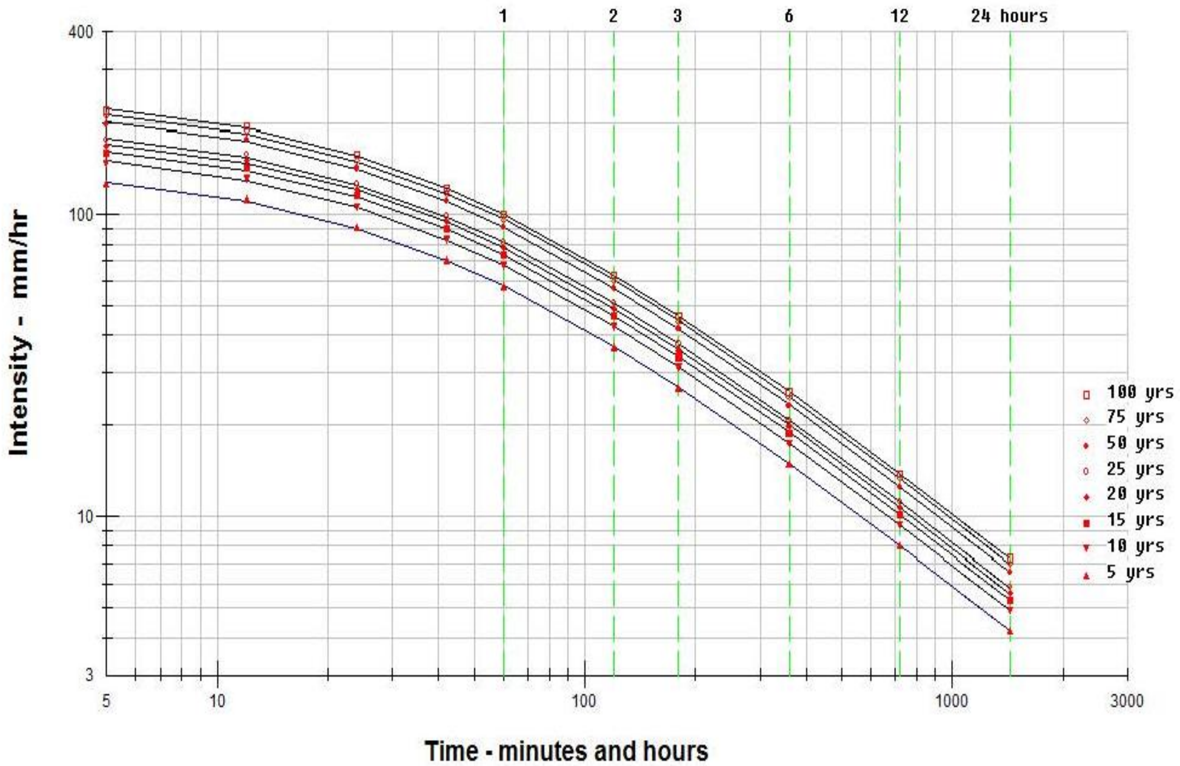
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Asante Bekwai

Longitude: 1°35'1.85"W

Latitude: 6°27'11.61"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	112.83	131.83	143.00	150.82	157.52	176.51	187.68	195.51
0.4	91.08	106.41	115.42	121.74	127.15	142.48	151.49	157.81
0.7	70.80	82.71	89.72	94.63	98.84	110.75	117.76	122.67
1	58.00	67.77	73.51	77.53	80.97	90.74	96.48	100.50
2	36.39	42.52	46.12	48.65	50.81	56.93	60.54	63.06
3	26.63	31.11	33.75	35.59	37.18	41.66	44.29	46.14
6	14.88	17.39	18.86	19.89	20.78	23.28	24.75	25.78
12	8.00	9.35	10.14	10.69	11.17	12.51	13.31	13.86
24	4.21	4.92	5.33	5.63	5.88	6.58	7.00	7.29



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

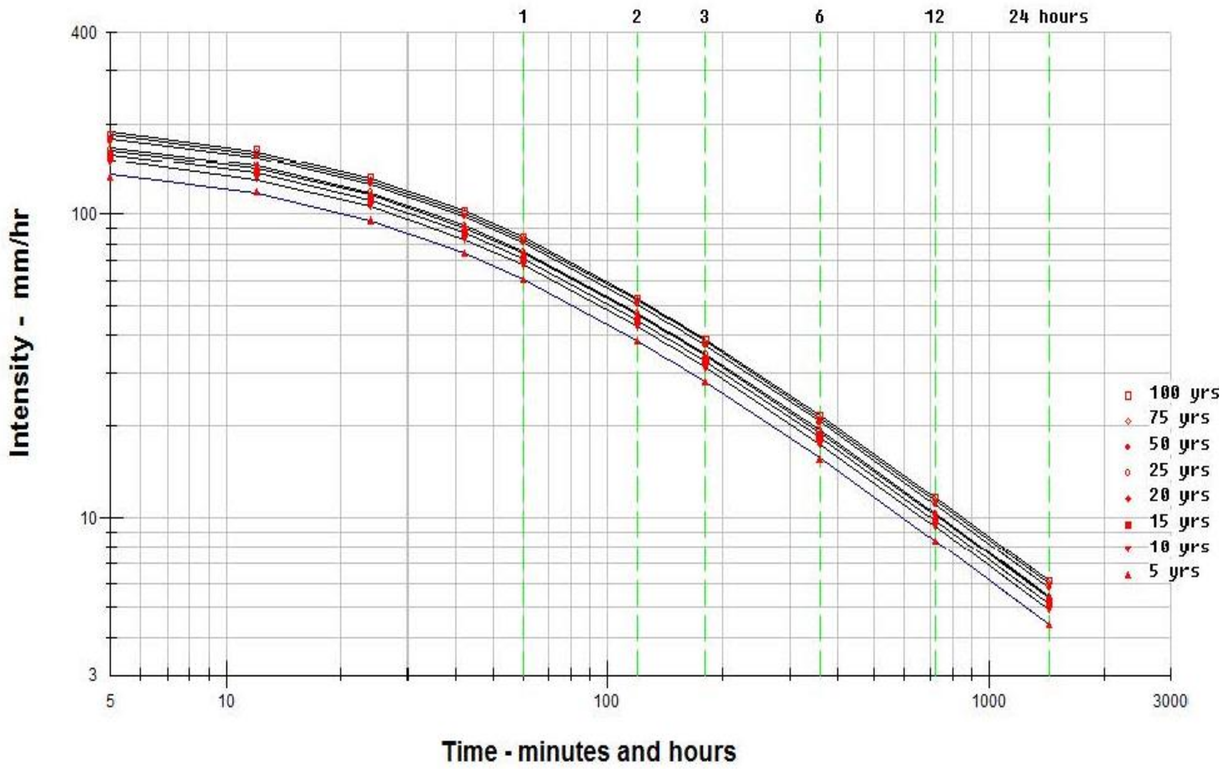
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Asante Mampong

Longitude: 1°23'59.99"W

Latitude: 7° 3'11.29"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	118.42	131.83	138.53	144.12	146.35	156.40	160.87	164.22
0.4	95.59	106.41	111.82	116.33	118.13	126.25	129.85	132.56
0.7	74.30	82.71	86.92	90.43	91.83	98.14	100.94	103.04
1	60.87	67.77	71.21	74.08	75.23	80.40	82.70	84.42
2	38.20	42.52	44.68	46.48	47.20	50.45	51.89	52.97
3	27.95	31.11	32.69	34.01	34.54	36.91	37.97	38.76
6	15.62	17.39	18.27	19.01	19.30	20.63	21.22	21.66
12	8.40	9.35	9.82	10.22	10.38	11.09	11.40	11.64
24	4.42	4.92	5.17	5.38	5.46	5.83	6.00	6.13



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

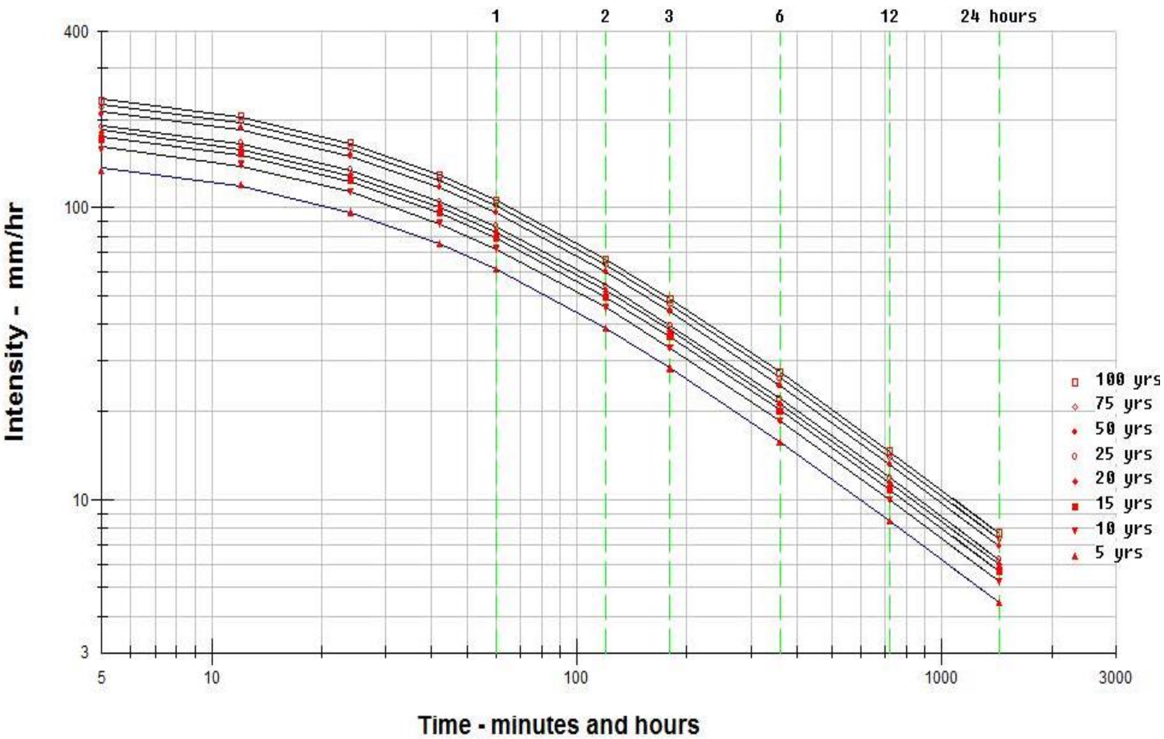
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Asesewa

Longitude: 0° 8'32.92"W

Latitude: 6°23'59.03"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	119.54	140.76	153.05	160.87	167.58	186.57	197.74	206.68
0.4	96.49	113.62	123.54	129.85	135.26	150.59	159.61	166.82
0.7	75.00	88.32	96.03	100.94	105.15	117.06	124.07	129.68
1	61.45	72.36	78.68	82.70	86.14	95.91	101.65	106.24
2	38.56	45.40	49.37	51.89	54.05	60.18	63.78	66.66
3	28.21	33.22	36.12	37.97	39.55	44.03	46.67	48.78
6	15.77	18.56	20.19	21.22	22.10	24.61	26.08	27.26
12	8.47	9.98	10.85	11.40	11.88	13.23	14.02	14.65
24	4.46	5.25	5.71	6.00	6.25	6.96	7.38	7.71



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

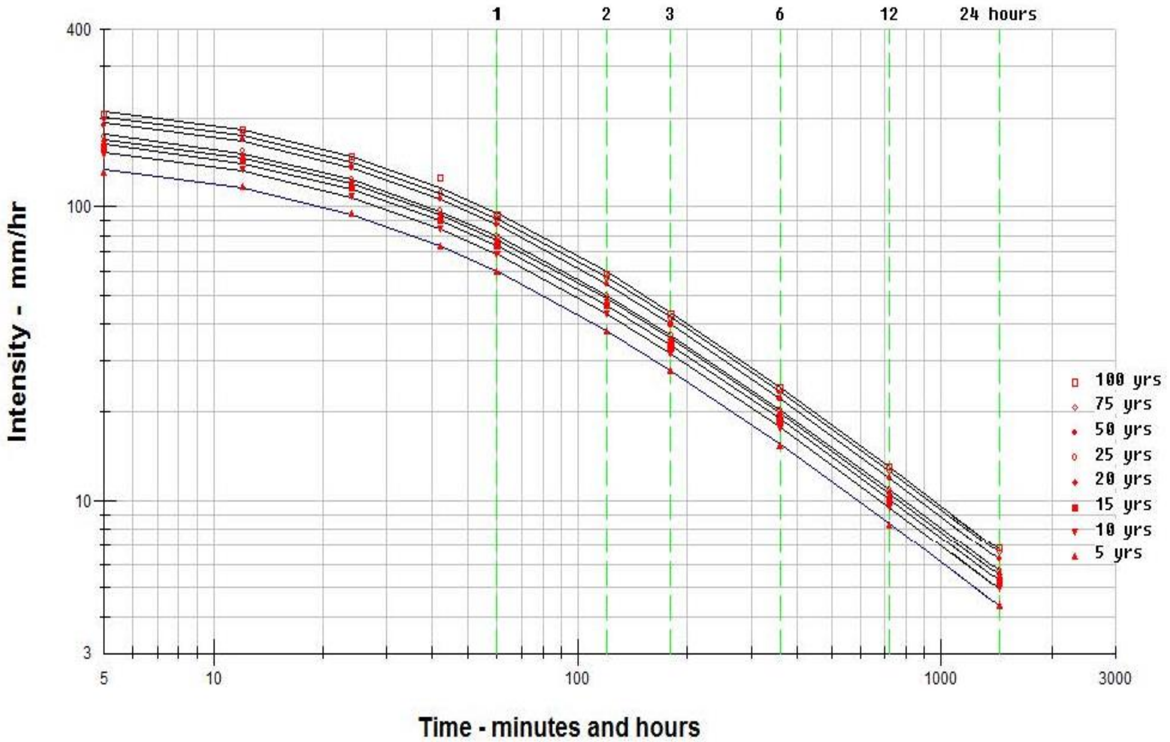
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Assin Fosu

Longitude: 1°17'3.44"W

Latitude: 5°42'8.05"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	117.30	134.06	143.00	149.70	154.17	168.69	177.63	183.22
0.4	94.68	108.21	115.42	120.83	124.44	136.16	143.38	147.89
0.7	73.60	84.12	89.72	93.93	96.73	105.85	111.45	114.96
1	60.30	68.91	73.51	76.95	79.25	86.72	91.31	94.18
2	37.84	43.24	46.12	48.29	49.73	54.41	57.29	59.10
3	27.68	31.64	33.75	35.33	36.38	39.81	41.92	43.24
6	15.47	17.68	18.86	19.74	20.33	22.25	23.43	24.16
12	8.32	9.50	10.14	10.61	10.93	11.96	12.59	12.99
24	4.38	5.00	5.33	5.58	5.75	6.29	6.63	6.83



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

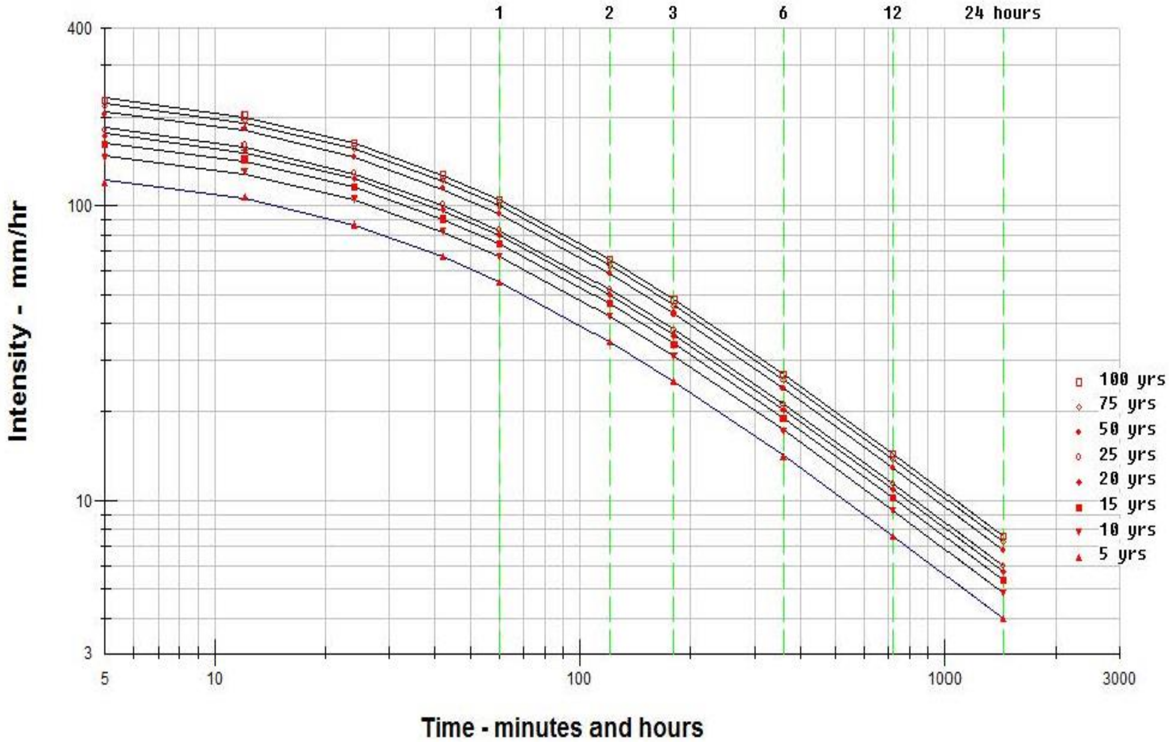
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Asutsuare

Longitude: 0°11'43.10"E

Latitude: 6° 5'33.14"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.08	125.36	152.78	168.45	180.20	188.04	212.85	227.21	237.66
0.2	107.25	130.71	144.12	154.17	160.87	182.10	194.39	203.33
0.4	86.57	105.50	116.33	124.44	129.85	146.99	156.90	164.12
0.7	67.29	82.01	90.43	96.73	100.94	114.26	121.97	127.58
1	55.13	67.19	74.08	79.25	82.70	93.61	99.93	104.52
2	34.59	42.16	46.48	49.73	51.89	58.74	62.70	65.58
3	25.31	30.85	34.01	36.38	37.97	42.98	45.88	47.99
6	14.14	17.24	19.01	20.33	21.22	24.02	25.64	26.82
12	7.60	9.27	10.22	10.93	11.40	12.91	13.78	14.41
24	4.00	4.88	5.38	5.75	6.00	6.79	7.25	7.58



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

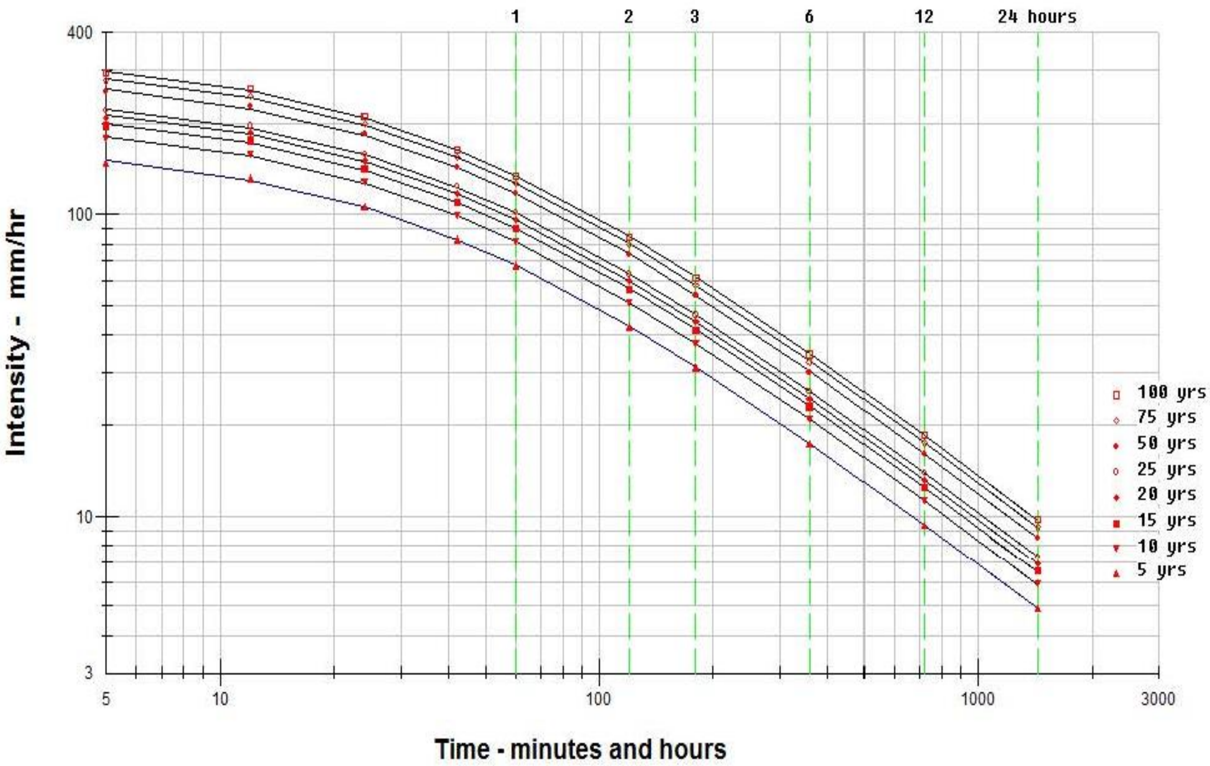
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Atebubu

Longitude: 0°59'4.04"W

Latitude: 7°45'14.84"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	131.83	158.64	175.40	186.57	196.62	227.90	246.90	261.42
0.4	106.41	128.05	141.57	150.59	158.71	183.96	199.29	211.01
0.7	82.71	99.54	110.05	117.06	123.37	143.00	154.91	164.03
1	67.77	81.55	90.16	95.91	101.08	117.16	126.92	134.38
2	42.52	51.17	56.57	60.18	63.42	73.51	79.64	84.32
3	31.11	37.44	41.39	44.03	46.40	53.79	58.27	61.70
6	17.39	20.92	23.13	24.61	25.93	30.06	32.56	34.48
12	9.35	11.25	12.43	13.23	13.94	16.16	17.50	18.53
24	4.92	5.92	6.54	6.96	7.33	8.50	9.21	9.75



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

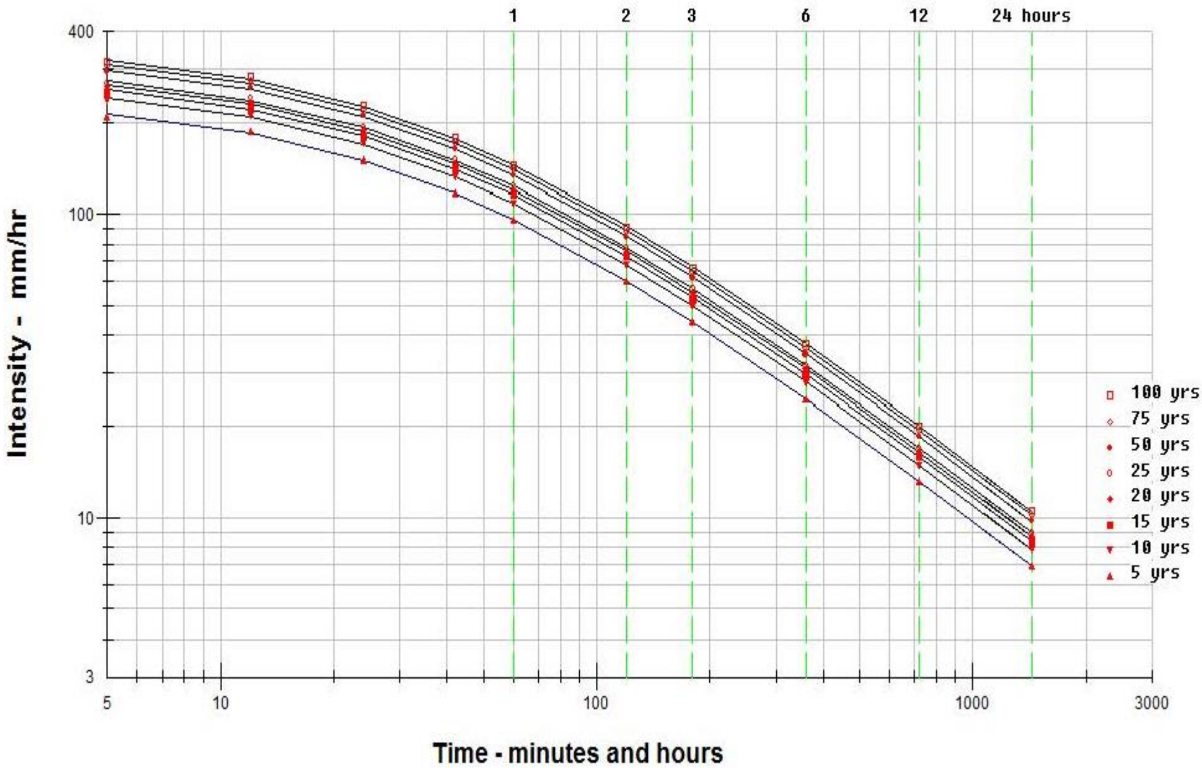
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Axim

Longitude: 2°14'27.20"W

Latitude: 4°51'59.43"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	186.57	211.15	224.55	233.49	240.19	261.42	273.71	282.64
0.4	150.59	170.43	181.25	188.47	193.88	211.01	220.93	228.14
0.7	117.06	132.48	140.90	146.50	150.71	164.03	171.74	177.35
1	95.91	108.54	115.43	120.03	123.47	134.38	140.70	145.30
2	60.18	68.10	72.43	75.31	77.47	84.32	88.28	91.17
3	44.03	49.83	52.99	55.10	56.69	61.70	64.60	66.70
6	24.61	27.85	29.62	30.79	31.68	34.48	36.10	37.28
12	13.23	14.97	15.92	16.55	17.03	18.53	19.40	20.04
24	6.96	7.88	8.38	8.71	8.96	9.75	10.21	10.54



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

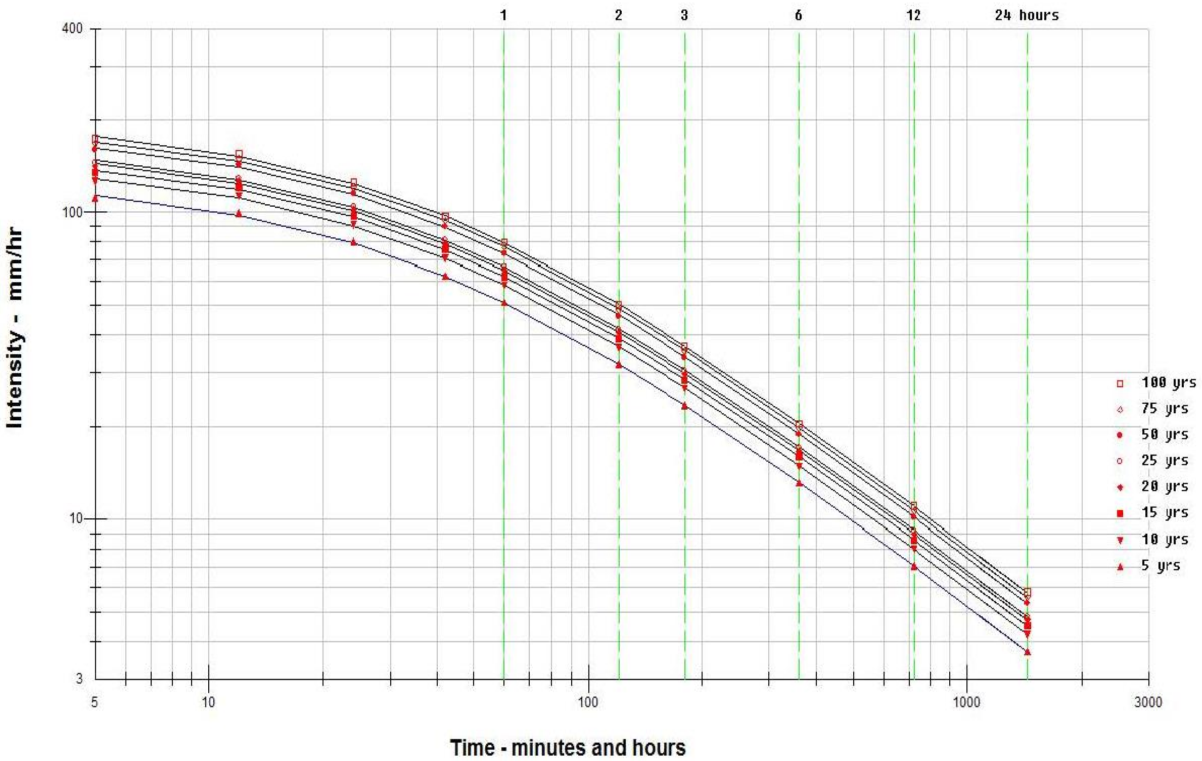
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Babile

Longitude: 2°50'6.98"W

Latitude: 10°31'11.35"N

Duration (hrs)	**Return period (yrs)**							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	99.32	112.83	120.65	126.24	129.59	143.00	149.70	155.29
0.4	80.17	91.08	97.39	101.90	104.60	115.42	120.83	125.34
0.7	62.32	70.80	75.71	79.21	81.31	89.72	93.93	97.44
1	51.05	58.00	62.02	64.89	66.62	73.51	76.95	79.83
2	32.03	36.39	38.92	40.72	41.80	46.12	48.29	50.09
3	23.44	26.63	28.47	29.79	30.58	33.75	35.33	36.65
6	13.10	14.88	15.91	16.65	17.09	18.86	19.74	20.48
12	7.04	8.00	8.55	8.95	9.19	10.14	10.61	11.01
24	3.70	4.21	4.50	4.71	4.83	5.33	5.58	5.79



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

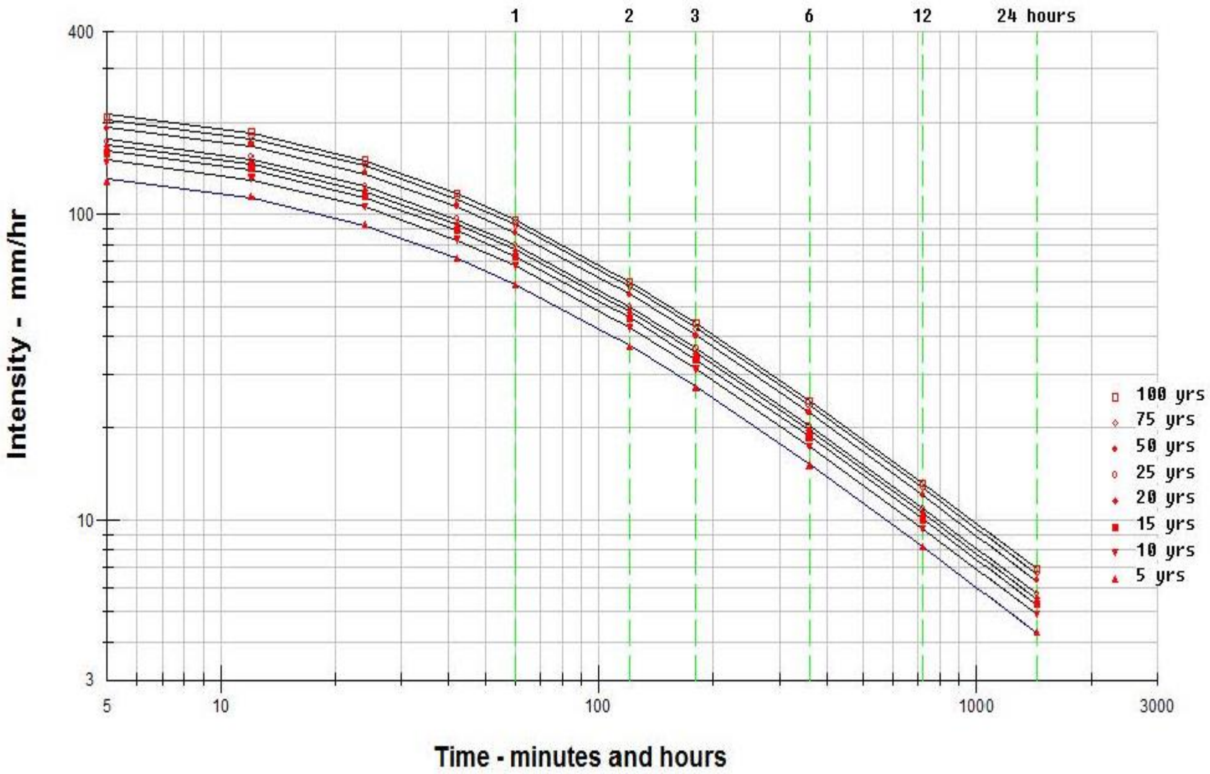
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Bechem

Longitude: 2° 1'47.16"W

Latitude: 7° 5'8.82"N

Duration (hrs)	Return period (yrs)							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	115.07	131.83	141.88	148.58	154.17	169.81	179.86	186.57
0.4	92.88	106.41	114.52	119.93	124.44	137.07	145.18	150.59
0.7	72.20	82.71	89.02	93.23	96.73	106.55	112.86	117.06
1	59.15	67.77	72.93	76.38	79.25	87.29	92.46	95.91
2	37.11	42.52	45.76	47.93	49.73	54.77	58.01	60.18
3	27.16	31.11	33.48	35.07	36.38	40.08	42.45	44.03
6	15.18	17.39	18.71	19.60	20.33	22.40	23.72	24.61
12	8.16	9.35	10.06	10.53	10.93	12.04	12.75	13.23
24	4.29	4.92	5.29	5.54	5.75	6.33	6.71	6.96



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

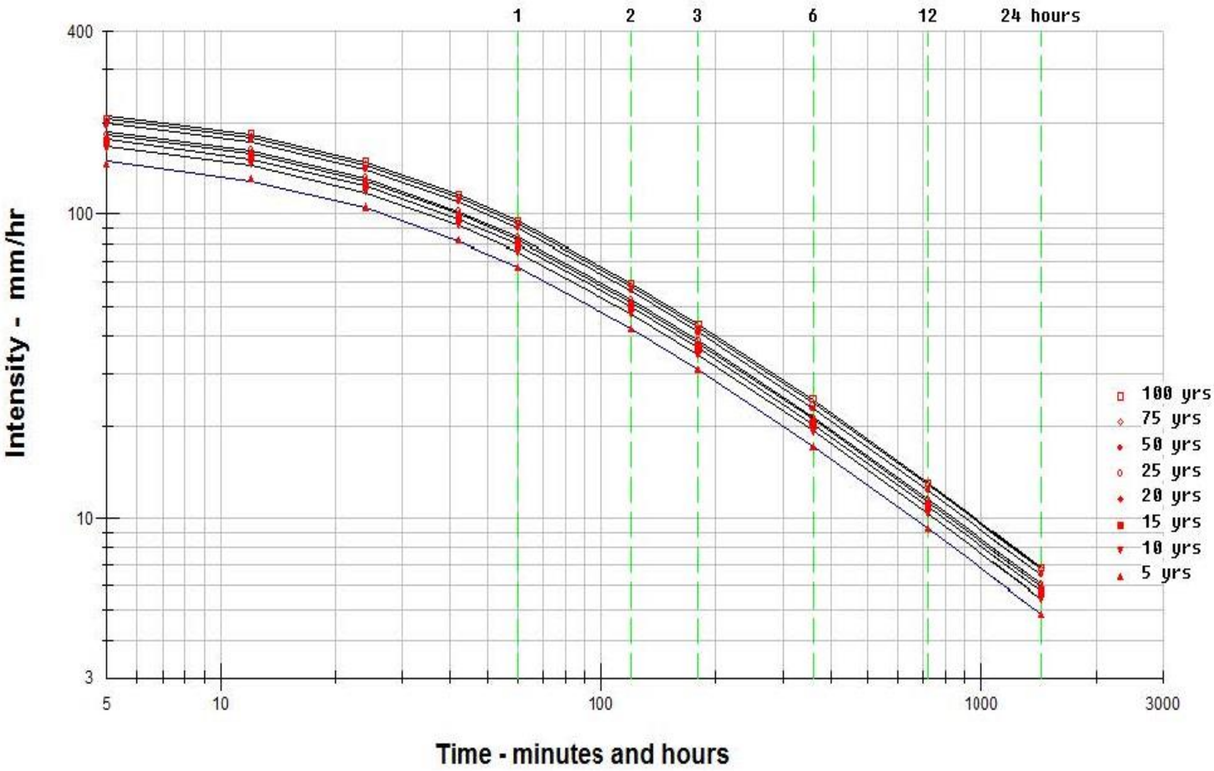
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Benso

Longitude: 1°53'38.58"W

Latitude: 5° 9'26.45"N

Duration (hrs)	Return period (yrs)							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	130.71	146.35	154.17	159.76	164.22	175.40	182.10	185.45
0.4	105.50	118.13	124.44	128.95	132.56	141.57	146.99	149.69
0.7	82.01	91.83	96.73	100.24	103.04	110.05	114.26	116.36
1	67.19	75.23	79.25	82.12	84.42	90.16	93.61	95.33
2	42.16	47.20	49.73	51.53	52.97	56.57	58.74	59.82
3	30.85	34.54	36.38	37.70	38.76	41.39	42.98	43.77
6	17.24	19.30	20.33	21.07	21.66	23.13	24.02	24.46
12	9.27	10.38	10.93	11.33	11.64	12.43	12.91	13.15
24	4.88	5.46	5.75	5.96	6.13	6.54	6.79	6.92



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

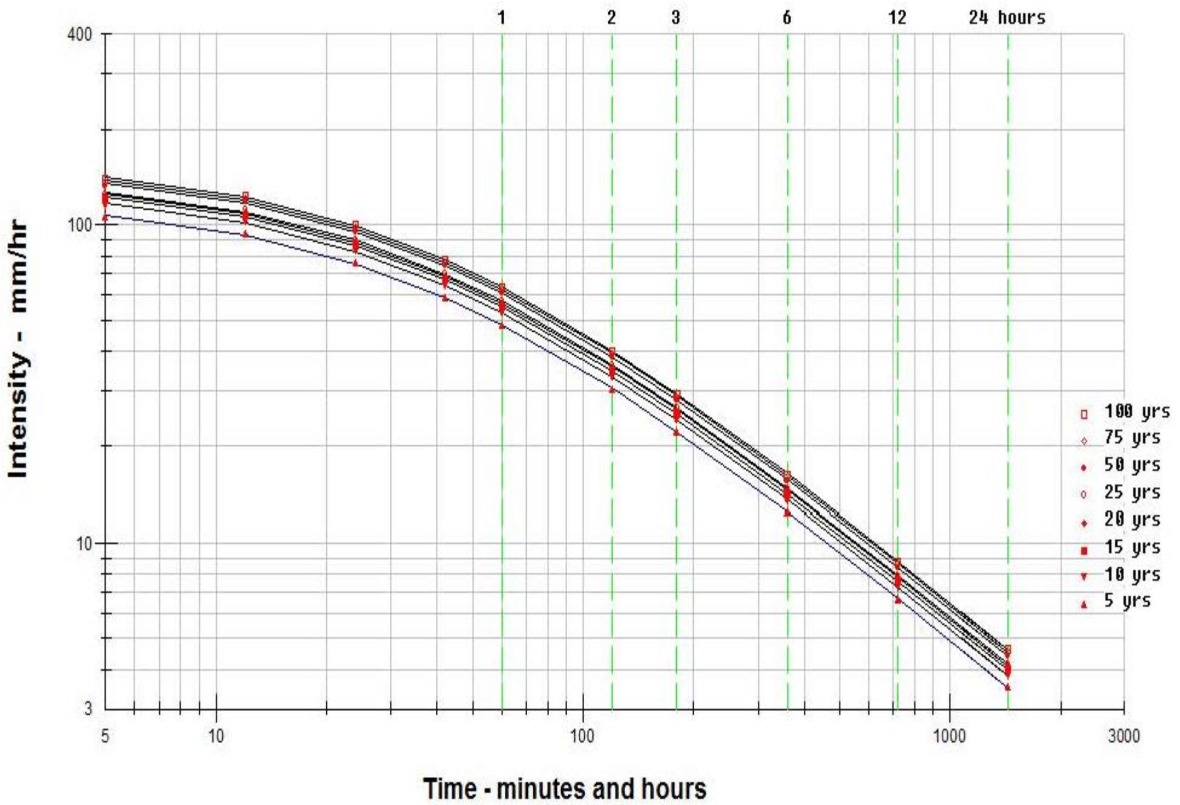
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Berekum

Longitude: 2°35'3.14"W

Latitude: 7°27'27.63"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	94.18	102.67	107.14	110.04	111.72	118.42	121.77	124.01
0.4	76.02	82.87	86.48	88.82	90.18	95.59	98.29	100.09
0.7	59.09	64.42	67.22	69.05	70.10	74.30	76.41	77.81
1	48.41	52.78	55.07	56.57	57.43	60.87	62.60	63.75
2	30.38	33.12	34.56	35.49	36.03	38.20	39.28	40.00
3	22.23	24.23	25.28	25.97	26.37	27.95	28.74	29.27
6	12.42	13.54	14.13	14.51	14.73	15.62	16.06	16.35
12	6.68	7.28	7.60	7.80	7.92	8.40	8.63	8.79
24	3.51	3.83	4.00	4.10	4.17	4.42	4.54	4.63



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

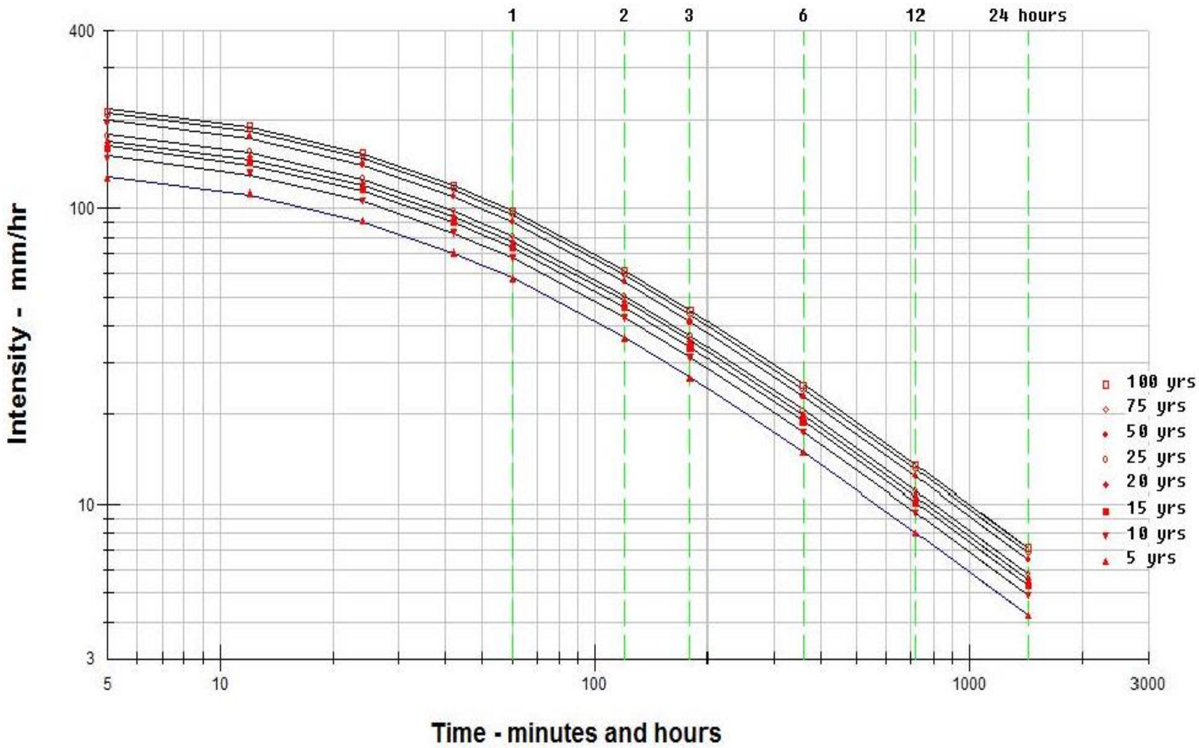
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Bimbila

Longitude: 0° 3'38.39"E

Latitude: 8°51'10.12"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	112.83	131.83	143.00	149.70	156.40	174.28	184.33	191.04
0.4	91.08	106.41	115.42	120.83	126.25	140.67	148.79	154.20
0.7	70.80	82.71	89.72	93.93	98.14	109.35	115.66	119.87
1	58.00	67.77	73.51	76.95	80.40	89.59	94.76	98.20
2	36.39	42.52	46.12	48.29	50.45	56.21	59.46	61.62
3	26.63	31.11	33.75	35.33	36.91	41.13	43.50	45.09
6	14.88	17.39	18.86	19.74	20.63	22.99	24.31	25.20
12	8.00	9.35	10.14	10.61	11.09	12.36	13.07	13.54
24	4.21	4.92	5.33	5.58	5.83	6.50	6.88	7.13



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

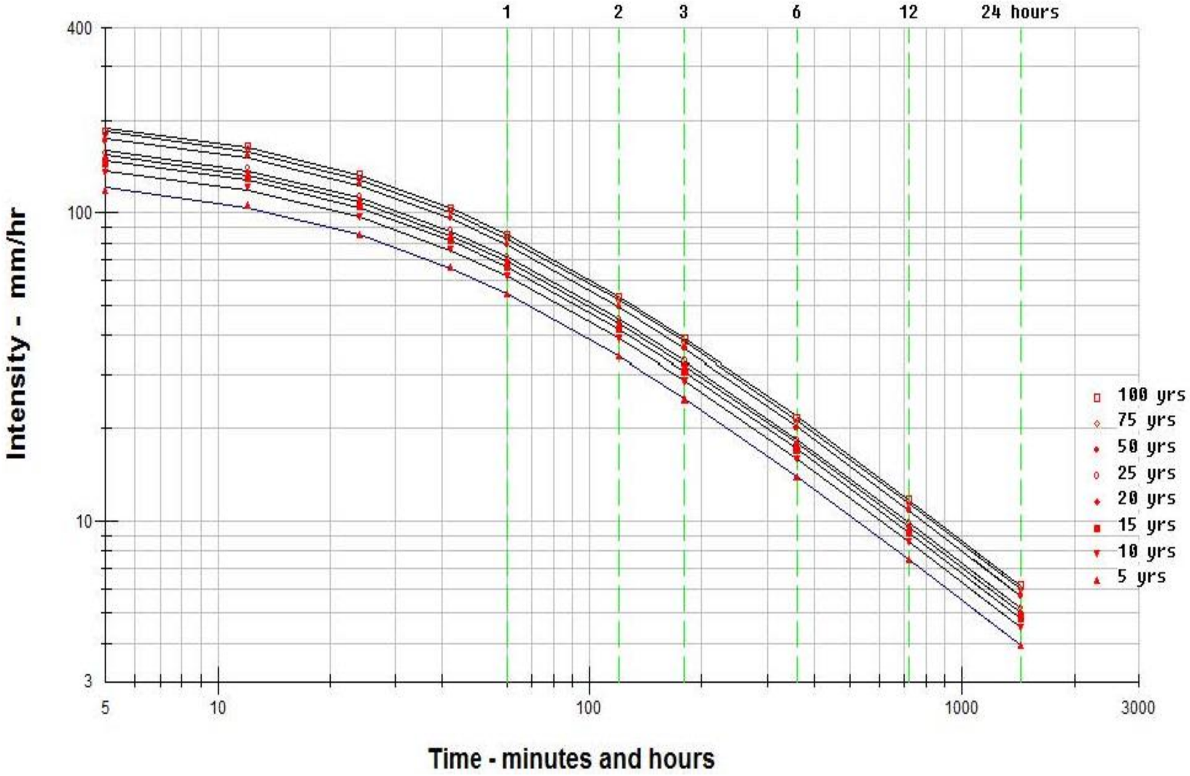
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Bole

Longitude: 2°29'6.53"W

Latitude: 9° 1'55.79"N

Duration (hrs)	**Return period (yrs)**							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	105.80	120.65	129.59	135.18	139.65	153.05	160.87	165.34
0.4	85.40	97.39	104.60	109.11	112.72	123.54	129.85	133.46
0.7	66.38	75.71	81.31	84.82	87.62	96.03	100.94	103.74
1	54.39	62.02	66.62	69.49	71.79	78.68	82.70	84.99
2	34.12	38.92	41.80	43.60	45.04	49.37	51.89	53.33
3	24.97	28.47	30.58	31.90	32.96	36.12	37.97	39.02
6	13.95	15.91	17.09	17.83	18.42	20.19	21.22	21.81
12	7.50	8.55	9.19	9.58	9.90	10.85	11.40	11.72
24	3.95	4.50	4.83	5.04	5.21	5.71	6.00	6.17



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

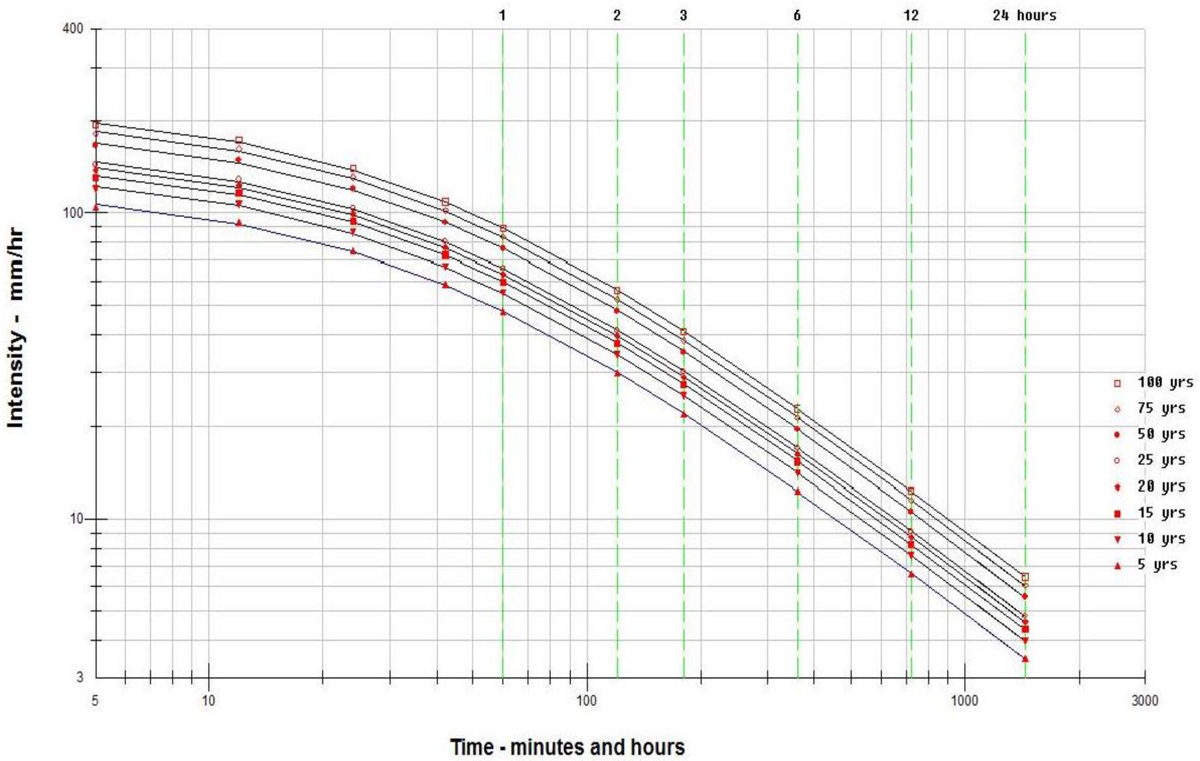
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Damango

Longitude: 1°49'37.05"W

Latitude: 9° 5'28.58"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	93.06	106.69	116.19	122.89	128.47	148.58	161.99	173.16
0.4	75.12	86.12	93.78	99.19	103.70	119.93	130.75	139.77
0.7	58.39	66.94	72.90	77.11	80.61	93.23	101.64	108.65
1	47.84	54.84	59.73	63.17	66.04	76.38	83.27	89.01
2	30.02	34.41	37.48	39.64	41.44	47.93	52.25	55.85
3	21.96	25.18	27.42	29.00	30.32	35.07	38.23	40.87
6	12.27	14.07	15.32	16.21	16.94	19.60	21.36	22.84
12	6.60	7.56	8.24	8.71	9.11	10.53	11.48	12.28
24	3.47	3.98	4.33	4.58	4.79	5.54	6.04	6.46



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

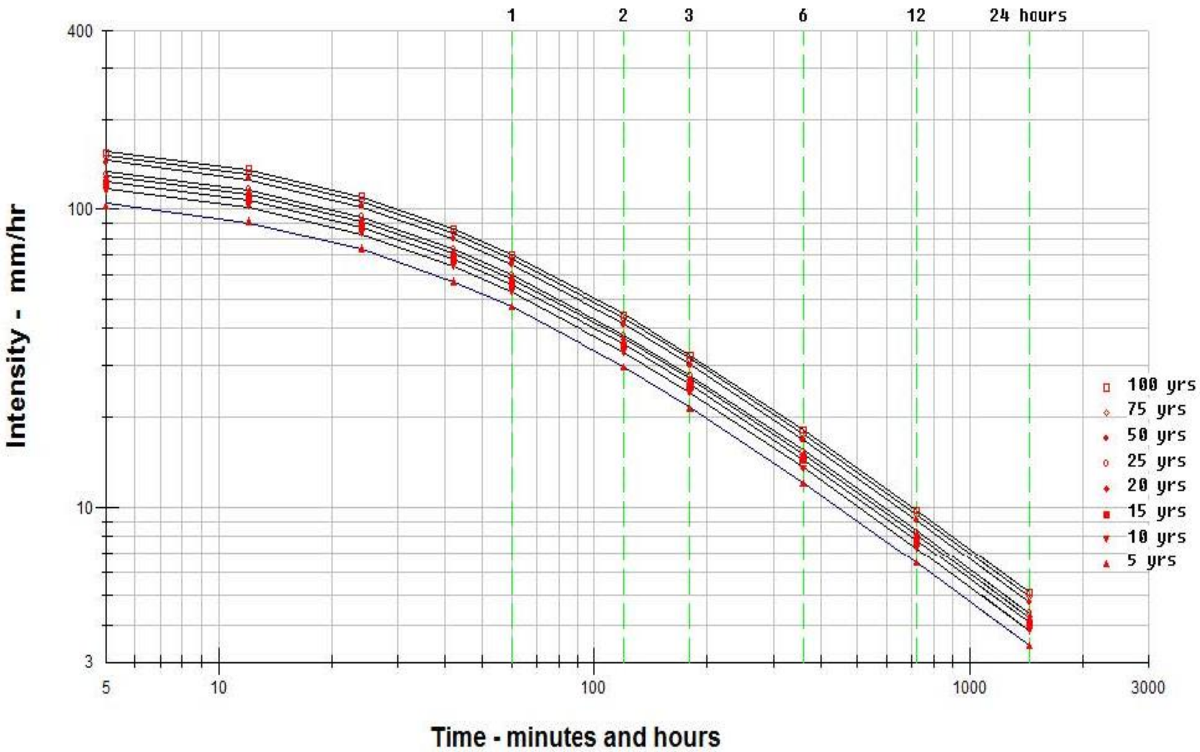
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Duayaw Nkwanta

Longitude: 2° 6'3.05"W

Latitude: 7°10'46.69"N

Duration (hrs)	**Return period (yrs)**							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	91.50	102.67	109.04	113.95	117.30	127.36	132.94	137.41
0.4	73.85	82.87	88.01	91.98	94.68	102.80	107.31	110.92
0.7	57.41	64.42	68.41	71.50	73.60	79.91	83.42	86.22
1	47.03	52.78	56.05	58.58	60.30	65.47	68.34	70.64
2	29.51	33.12	35.17	36.75	37.84	41.08	42.88	44.32
3	21.59	24.23	25.73	26.89	27.68	30.06	31.37	32.43
6	12.07	13.54	14.38	15.03	15.47	16.80	17.53	18.12
12	6.49	7.28	7.73	8.08	8.32	9.03	9.42	9.74
24	3.41	3.83	4.07	4.25	4.38	4.75	4.96	5.13



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

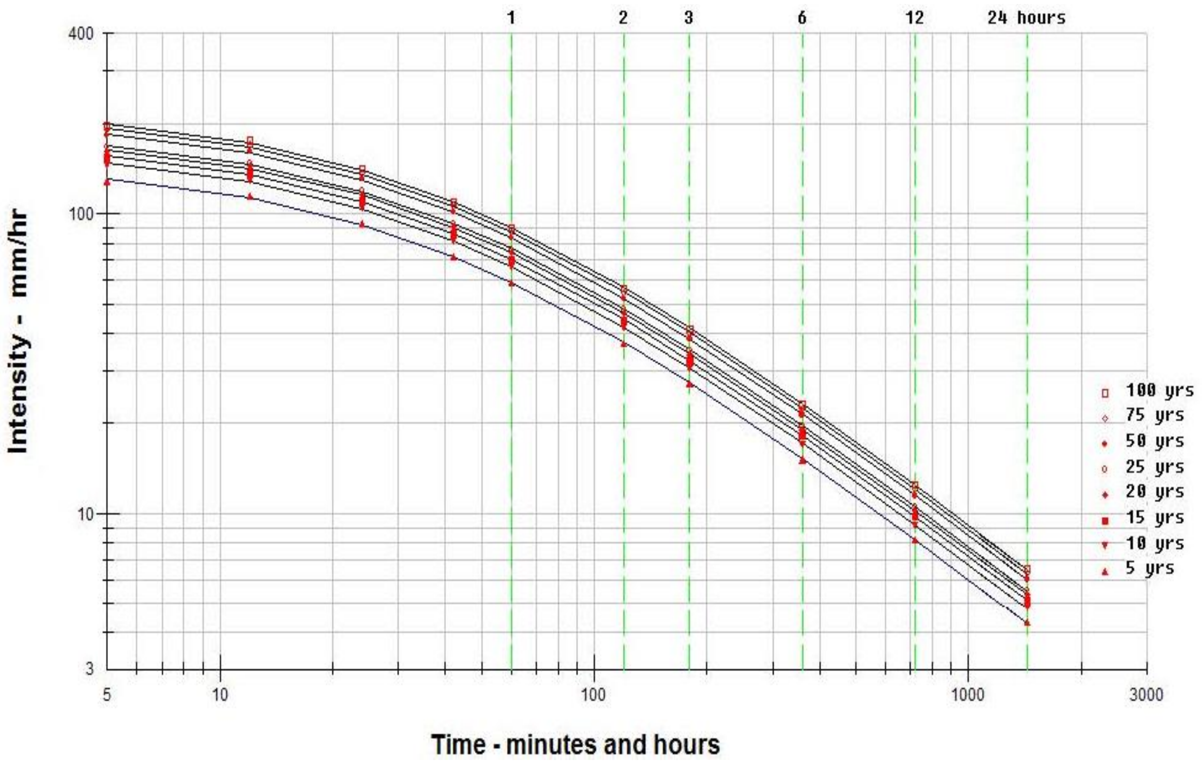
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Dunkwa on Offin

Longitude: 1°46'59.15"W

Latitude: 5°58'11.32"N

Duration (hrs)	**Return period (yrs)**							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	115.07	129.59	137.41	144.12	148.58	161.99	169.81	175.40
0.4	92.88	104.60	110.92	116.33	119.93	130.75	137.07	141.57
0.7	72.20	81.31	86.22	90.43	93.23	101.64	106.55	110.05
1	59.15	66.62	70.64	74.08	76.38	83.27	87.29	90.16
2	37.11	41.80	44.32	46.48	47.93	52.25	54.77	56.57
3	27.16	30.58	32.43	34.01	35.07	38.23	40.08	41.39
6	15.18	17.09	18.12	19.01	19.60	21.36	22.40	23.13
12	8.16	9.19	9.74	10.22	10.53	11.48	12.04	12.43
24	4.29	4.83	5.13	5.38	5.54	6.04	6.33	6.54



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

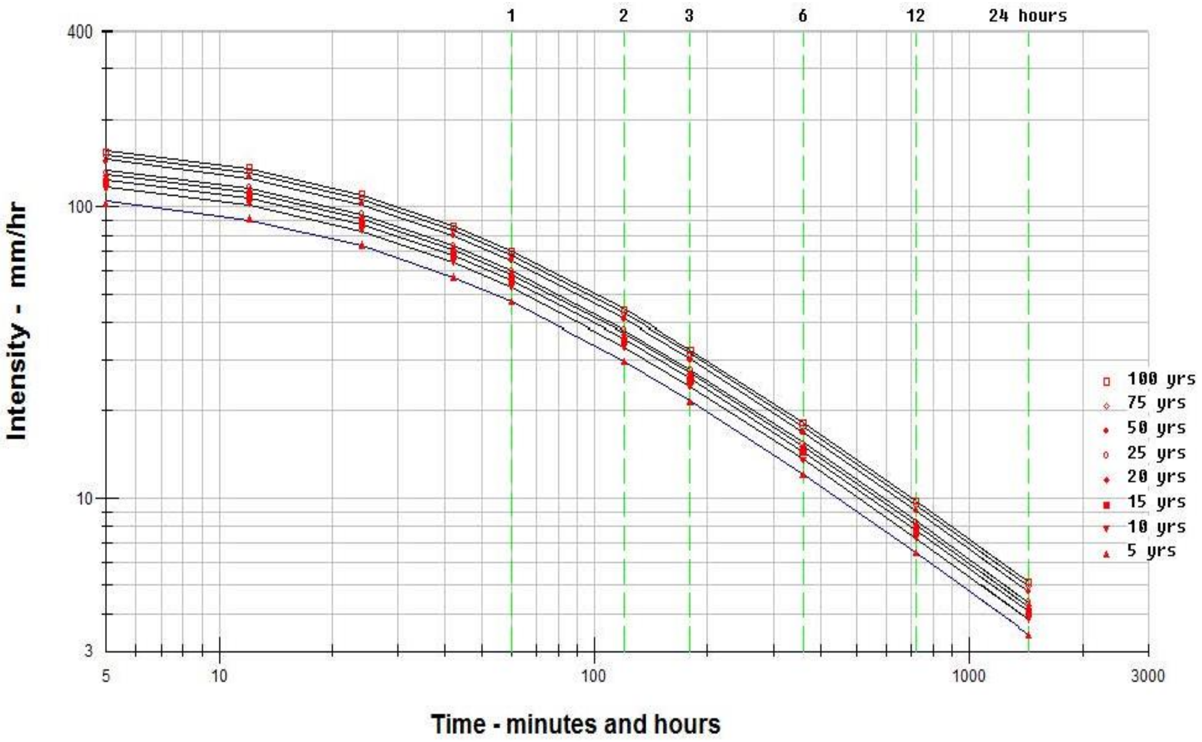
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Enchi

Longitude:

Latitude:

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	116.19	128.47	134.06	138.53	140.76	148.58	151.94	155.29
0.4	93.78	103.70	108.21	111.82	113.62	119.93	122.64	125.34
0.7	72.90	80.61	84.12	86.92	88.32	93.23	95.33	97.44
1	59.73	66.04	68.91	71.21	72.36	76.38	78.10	79.83
2	37.48	41.44	43.24	44.68	45.40	47.93	49.01	50.09
3	27.42	30.32	31.64	32.69	33.22	35.07	35.86	36.65
6	15.32	16.94	17.68	18.27	18.56	19.60	20.04	20.48
12	8.24	9.11	9.50	9.82	9.98	10.53	10.77	11.01
24	4.33	4.79	5.00	5.17	5.25	5.54	5.67	5.79



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

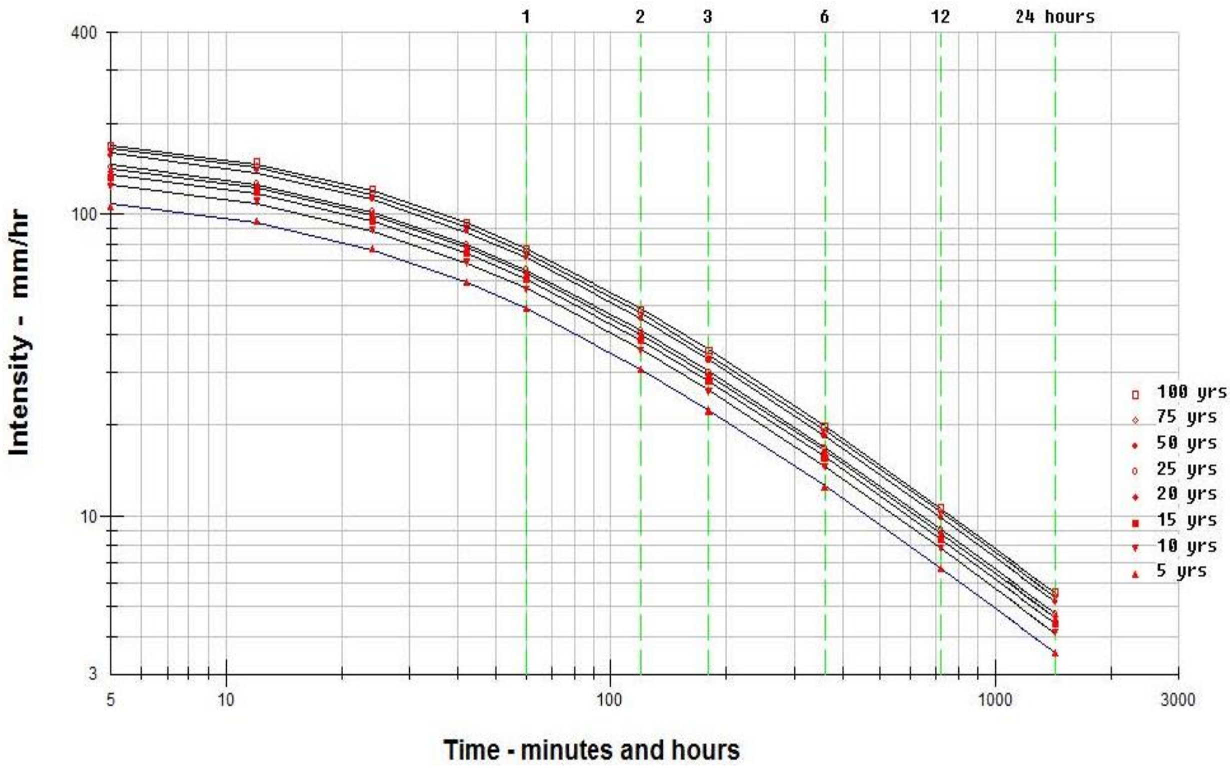
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Gambaga

Longitude: 0°26'18.41"W

Latitude: 10°31'55.63"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	94.85	110.15	118.42	124.01	127.36	139.65	145.23	149.70
0.4	76.56	88.91	95.59	100.09	102.80	112.72	117.23	120.83
0.7	59.51	69.12	74.30	77.81	79.91	87.62	91.13	93.93
1	48.76	56.62	60.87	63.75	65.47	71.79	74.66	76.95
2	30.59	35.53	38.20	40.00	41.08	45.04	46.84	48.29
3	22.38	26.00	27.95	29.27	30.06	32.96	34.28	35.33
6	12.51	14.53	15.62	16.35	16.80	18.42	19.15	19.74
12	6.72	7.81	8.40	8.79	9.03	9.90	10.30	10.61
24	3.54	4.11	4.42	4.63	4.75	5.21	5.42	5.58



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

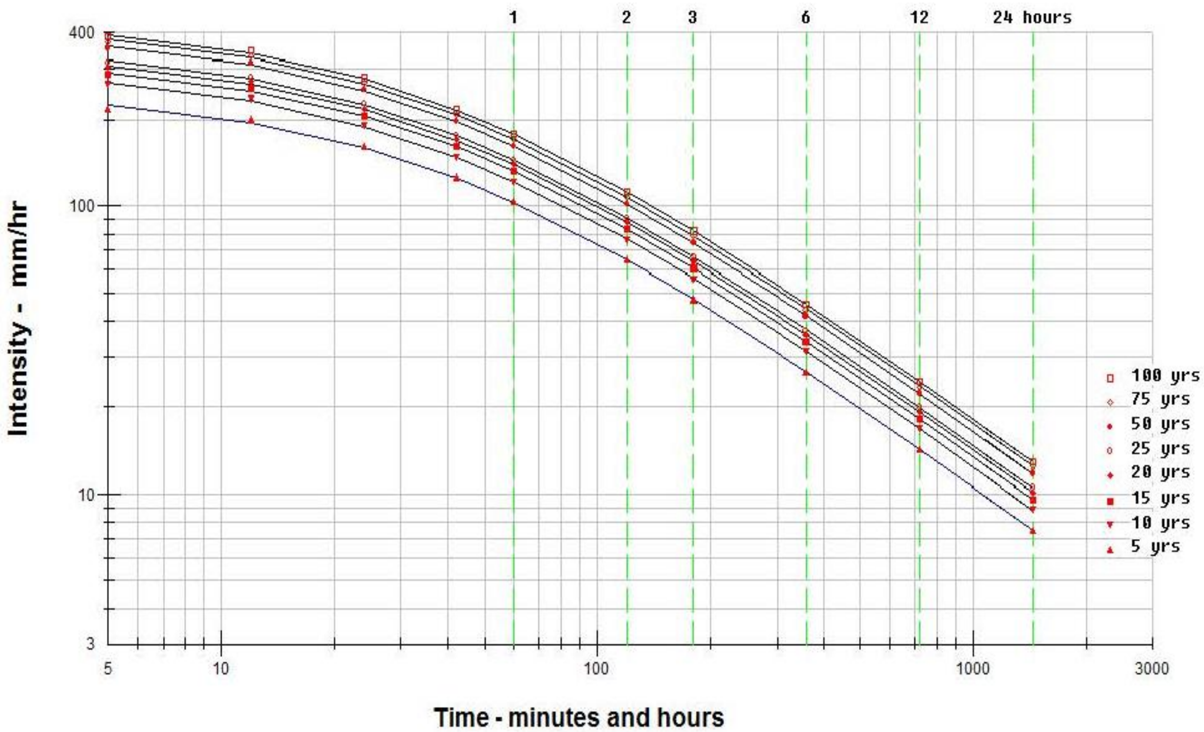
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Half Assini

Longitude: 2°52'48.57"W

Latitude: 5° 3'3.28"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	201.09	236.84	256.95	271.47	281.53	315.04	332.92	346.32
0.4	162.32	191.17	207.40	219.13	227.24	254.29	268.72	279.54
0.7	126.18	148.61	161.22	170.34	176.65	197.67	208.89	217.30
1	103.37	121.75	132.09	139.55	144.72	161.95	171.14	178.03
2	64.86	76.39	82.88	87.56	90.81	101.62	107.38	111.71
3	47.46	55.89	60.64	64.07	66.44	74.35	78.57	81.73
6	26.52	31.24	33.89	35.80	37.13	41.55	43.91	45.68
12	14.26	16.79	18.22	19.25	19.96	22.33	23.60	24.55
24	7.50	8.83	9.58	10.13	10.50	11.75	12.42	12.92



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

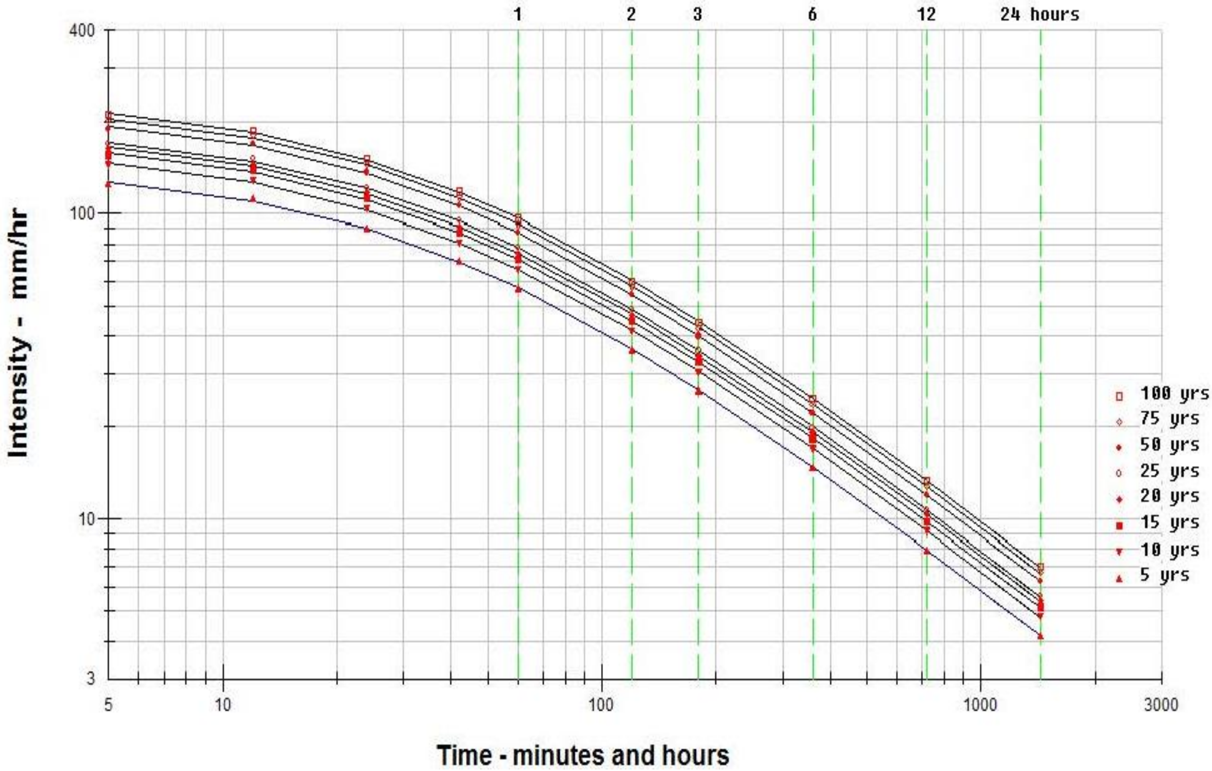
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Ho

Longitude: 0°28'42.36"E

Latitude: 6°36'36.35"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	111.72	128.47	138.53	145.23	150.82	168.69	179.86	187.68
0.4	90.18	103.70	111.82	117.23	121.74	136.16	145.18	151.49
0.7	70.10	80.61	86.92	91.13	94.63	105.85	112.86	117.76
1	57.43	66.04	71.21	74.66	77.53	86.72	92.46	96.48
2	36.03	41.44	44.68	46.84	48.65	54.41	58.01	60.54
3	26.37	30.32	32.69	34.28	35.59	39.81	42.45	44.29
6	14.73	16.94	18.27	19.15	19.89	22.25	23.72	24.75
12	7.92	9.11	9.82	10.30	10.69	11.96	12.75	13.31
24	4.17	4.79	5.17	5.42	5.63	6.29	6.71	7.00



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

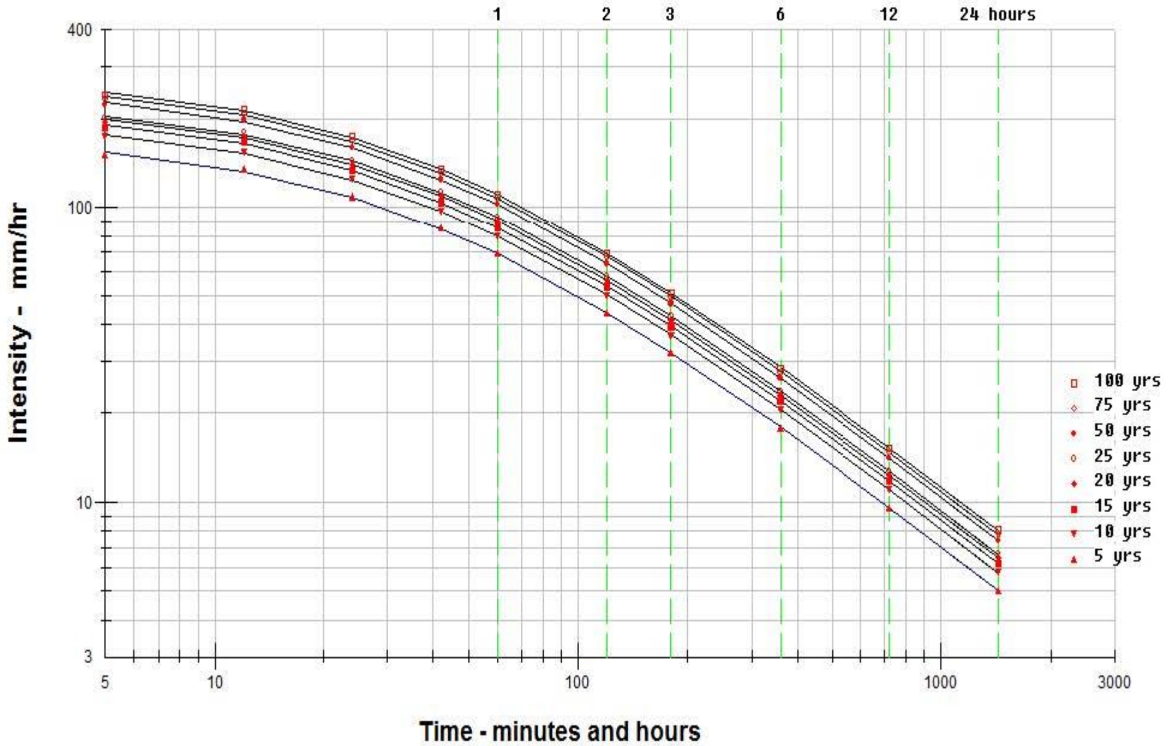
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Kete Krachi

Longitude: 0° 3'4.77"W

Latitude: 7°48'5.20"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	135.18	155.29	166.46	174.28	179.86	198.86	208.91	215.61
0.4	109.11	125.34	134.36	140.67	145.18	160.51	168.63	174.04
0.7	84.82	97.44	104.44	109.35	112.86	124.77	131.08	135.29
1	69.49	79.83	85.57	89.59	92.46	102.22	107.39	110.84
2	43.60	50.09	53.69	56.21	58.01	64.14	67.38	69.55
3	31.90	36.65	39.28	41.13	42.45	46.93	49.30	50.89
6	17.83	20.48	21.95	22.99	23.72	26.23	27.55	28.44
12	9.58	11.01	11.80	12.36	12.75	14.10	14.81	15.29
24	5.04	5.79	6.21	6.50	6.71	7.42	7.79	8.04



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

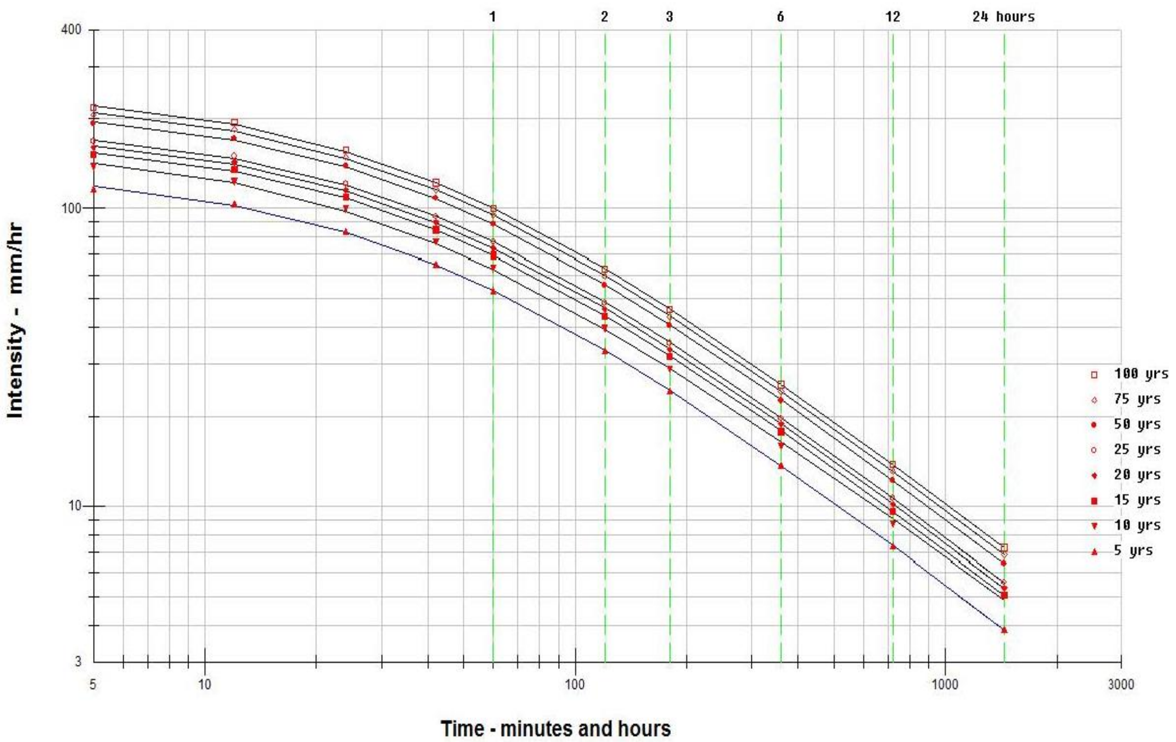
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Kibi

Longitude: 0°32'57.91"W

Latitude: 6°10'5.97"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	103.56	122.89	135.18	143.00	149.70	172.04	184.33	194.39
0.4	83.59	99.19	109.11	115.42	120.83	138.87	148.79	156.90
0.7	64.98	77.11	84.82	89.72	93.93	107.95	115.66	121.97
1	53.24	63.17	69.49	73.51	76.95	88.44	94.76	99.93
2	33.40	39.64	43.60	46.12	48.29	55.49	59.46	62.70
3	24.44	29.00	31.90	33.75	35.33	40.60	43.50	45.88
6	13.66	16.21	17.83	18.86	19.74	22.69	24.31	25.64
12	7.34	8.71	9.58	10.14	10.61	12.20	13.07	13.78
24	3.86	4.58	5.04	5.33	5.58	6.42	6.88	7.25



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

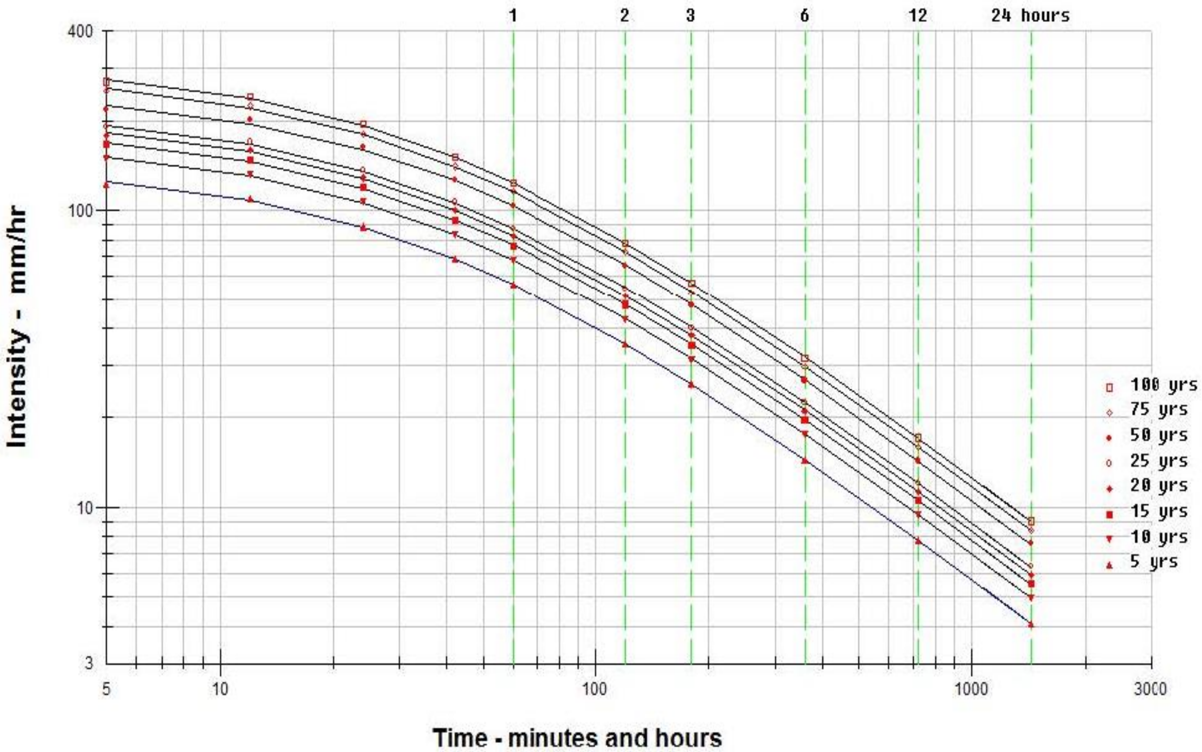
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Kintampo

Longitude: 1°43'46.38"W

Latitude: 8° 3'33.34"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	109.59	132.94	148.58	159.76	169.81	202.21	224.55	241.31
0.4	88.46	107.31	119.93	128.95	137.07	163.22	181.25	194.78
0.7	68.77	83.42	93.23	100.24	106.55	126.88	140.90	151.41
1	56.34	68.34	76.38	82.12	87.29	103.95	115.43	124.05
2	35.35	42.88	47.93	51.53	54.77	65.22	72.43	77.83
3	25.86	31.37	35.07	37.70	40.08	47.72	52.99	56.95
6	14.45	17.53	19.60	21.07	22.40	26.67	29.62	31.83
12	7.77	9.42	10.53	11.33	12.04	14.34	15.92	17.11
24	4.09	4.96	5.54	5.96	6.33	7.54	8.38	9.00



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

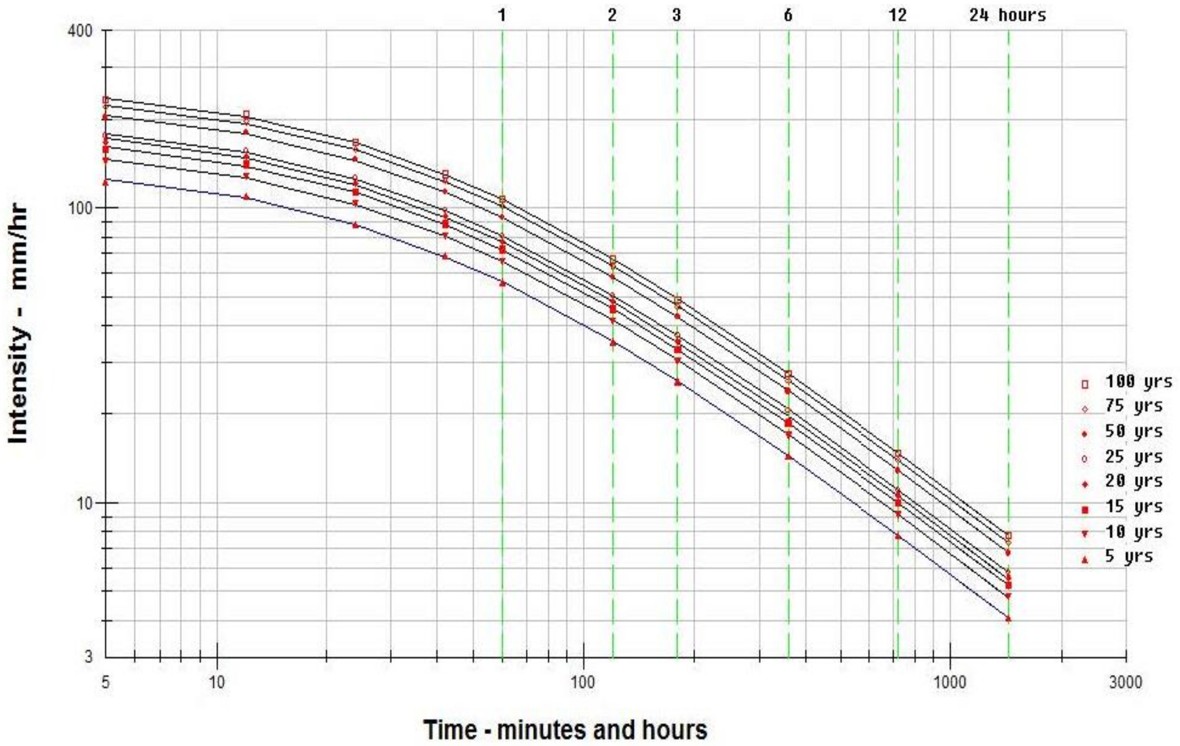
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Koforidua

Longitude: 0°16'17.02"W

Latitude: 6° 4'42.39"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	109.26	128.47	140.76	149.70	156.40	180.98	196.62	207.79
0.4	88.19	103.70	113.62	120.83	126.25	146.08	158.71	167.73
0.7	68.56	80.61	88.32	93.93	98.14	113.56	123.37	130.38
1	56.17	66.04	72.36	76.95	80.40	93.03	101.08	106.82
2	35.24	41.44	45.40	48.29	50.45	58.38	63.42	67.02
3	25.79	30.32	33.22	35.33	36.91	42.71	46.40	49.04
6	14.41	16.94	18.56	19.74	20.63	23.87	25.93	27.41
12	7.75	9.11	9.98	10.61	11.09	12.83	13.94	14.73
24	4.08	4.79	5.25	5.58	5.83	6.75	7.33	7.75



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

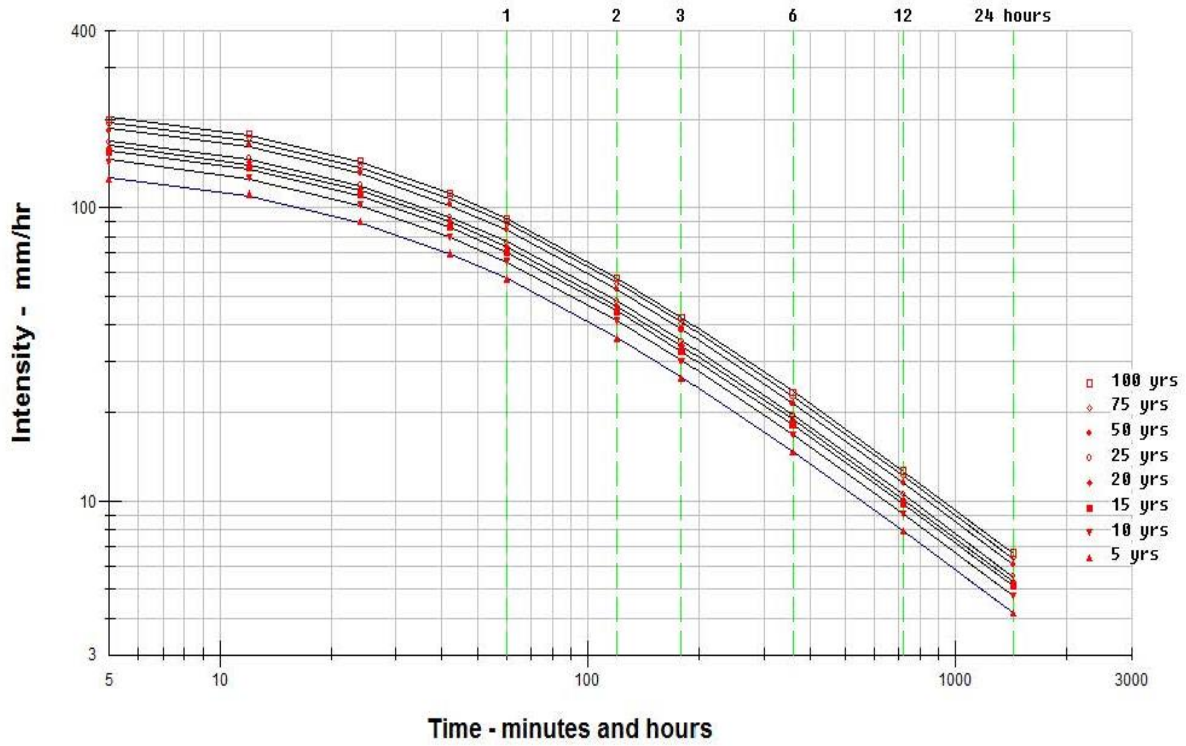
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Konongo

Longitude: 1°12'26.37"W

Latitude: 6°37'29.80"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.08	130.45	148.86	160.61	167.14	173.67	190.65	201.09	208.93
0.2	111.61	127.36	137.41	143.00	148.58	163.11	172.04	178.75
0.4	90.08	102.80	110.92	115.42	119.93	131.66	138.87	144.28
0.7	70.03	79.91	86.22	89.72	93.23	102.34	107.95	112.16
1	57.37	65.47	70.64	73.51	76.38	83.85	88.44	91.89
2	36.00	41.08	44.32	46.12	47.93	52.61	55.49	57.65
3	26.34	30.06	32.43	33.75	35.07	38.49	40.60	42.18
6	14.72	16.80	18.12	18.86	19.60	21.51	22.69	23.57
12	7.91	9.03	9.74	10.14	10.53	11.56	12.20	12.67
24	4.16	4.75	5.13	5.33	5.54	6.08	6.42	6.67



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

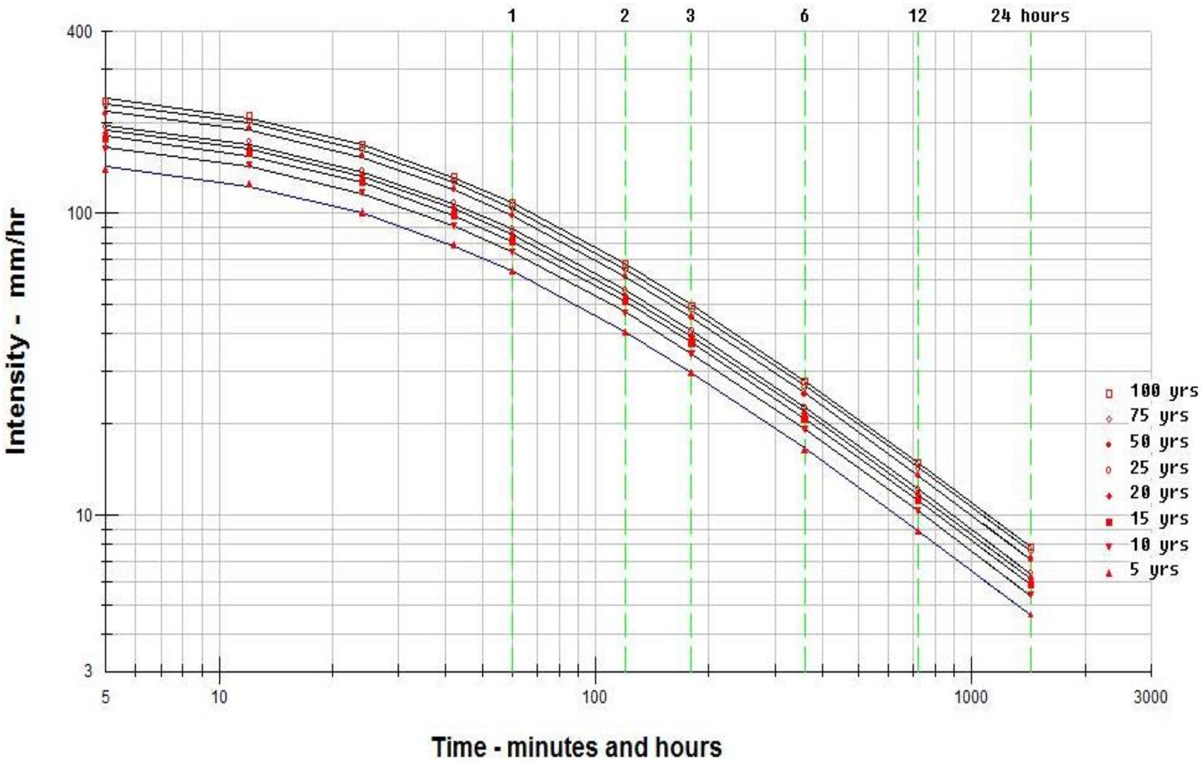
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Kpando

Longitude: 0°17'41.57"E

Latitude: 6°59'26.26"N

Duration (hrs)	Return period (yrs)							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	125.12	145.23	157.52	165.34	172.04	191.04	202.21	210.03
0.4	101.00	117.23	127.15	133.46	138.87	154.20	163.22	169.53
0.7	78.51	91.13	98.84	103.74	107.95	119.87	126.88	131.78
1	64.32	74.66	80.97	84.99	88.44	98.20	103.95	107.97
2	40.36	46.84	50.81	53.33	55.49	61.62	65.22	67.74
3	29.53	34.28	37.18	39.02	40.60	45.09	47.72	49.57
6	16.50	19.15	20.78	21.81	22.69	25.20	26.67	27.70
12	8.87	10.30	11.17	11.72	12.20	13.54	14.34	14.89
24	4.67	5.42	5.88	6.17	6.42	7.13	7.54	7.83



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

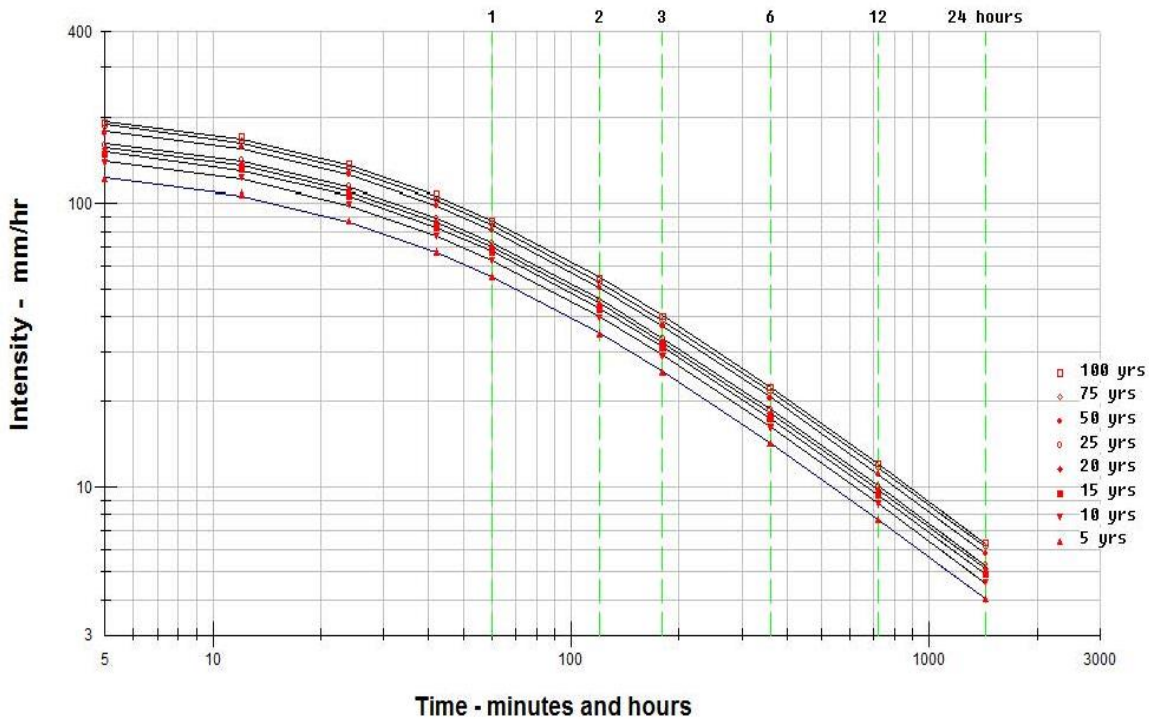
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Kpeve

Longitude: 0°20'1.18"E

Latitude: 0°20'1.18"E

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	107.92	122.89	131.83	137.41	141.88	156.40	164.22	169.81
0.4	87.11	99.19	106.41	110.92	114.52	126.25	132.56	137.07
0.7	67.71	77.11	82.71	86.22	89.02	98.14	103.04	106.55
1	55.48	63.17	67.77	70.64	72.93	80.40	84.42	87.29
2	34.81	39.64	42.52	44.32	45.76	50.45	52.97	54.77
3	25.47	29.00	31.11	32.43	33.48	36.91	38.76	40.08
6	14.23	16.21	17.39	18.12	18.71	20.63	21.66	22.40
12	7.65	8.71	9.35	9.74	10.06	11.09	11.64	12.04
24	4.03	4.58	4.92	5.13	5.29	5.83	6.13	6.33



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

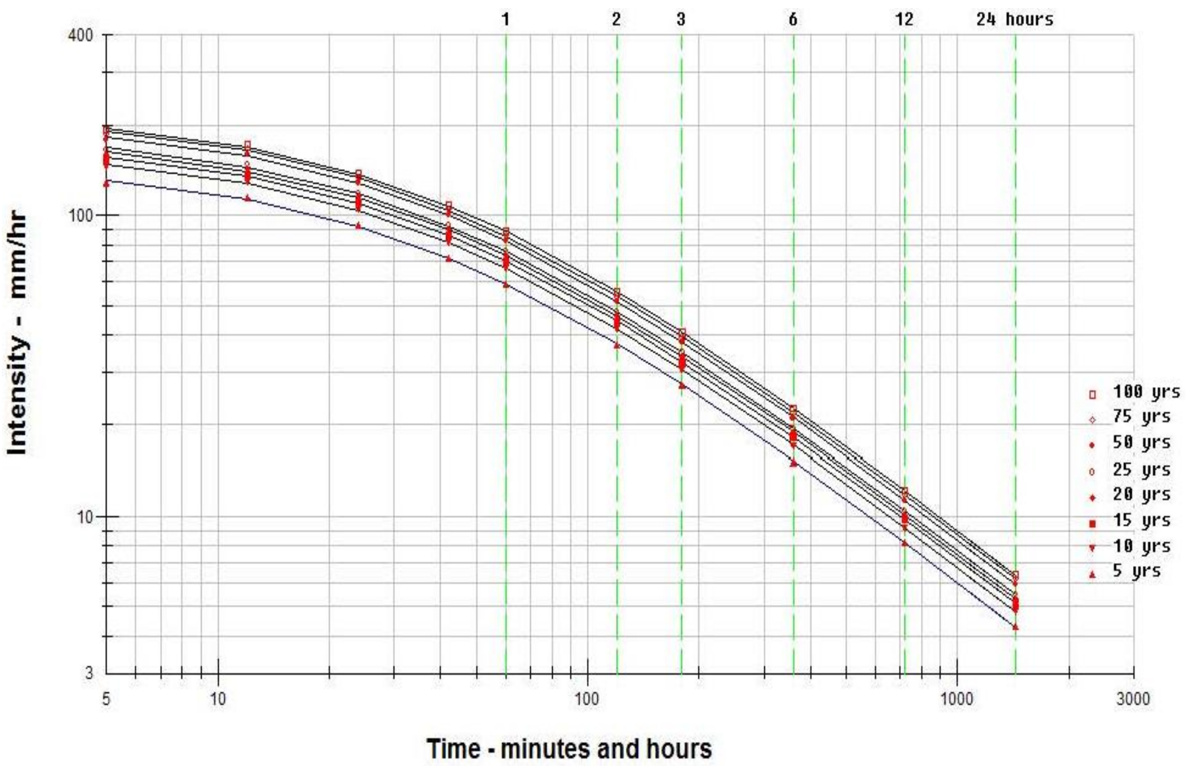
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Kumasi

Longitude: 1°36'58.58"W

Latitude: 6°39'59.76"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	115.07	129.59	137.41	143.00	147.47	159.76	166.46	172.04
0.4	92.88	104.60	110.92	115.42	119.03	128.95	134.36	138.87
0.7	72.20	81.31	86.22	89.72	92.53	100.24	104.44	107.95
1	59.15	66.62	70.64	73.51	75.81	82.12	85.57	88.44
2	37.11	41.80	44.32	46.12	47.56	51.53	53.69	55.49
3	27.16	30.58	32.43	33.75	34.80	37.70	39.28	40.60
6	15.18	17.09	18.12	18.86	19.45	21.07	21.95	22.69
12	8.16	9.19	9.74	10.14	10.45	11.33	11.80	12.20
24	4.29	4.83	5.13	5.33	5.50	5.96	6.21	6.42



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

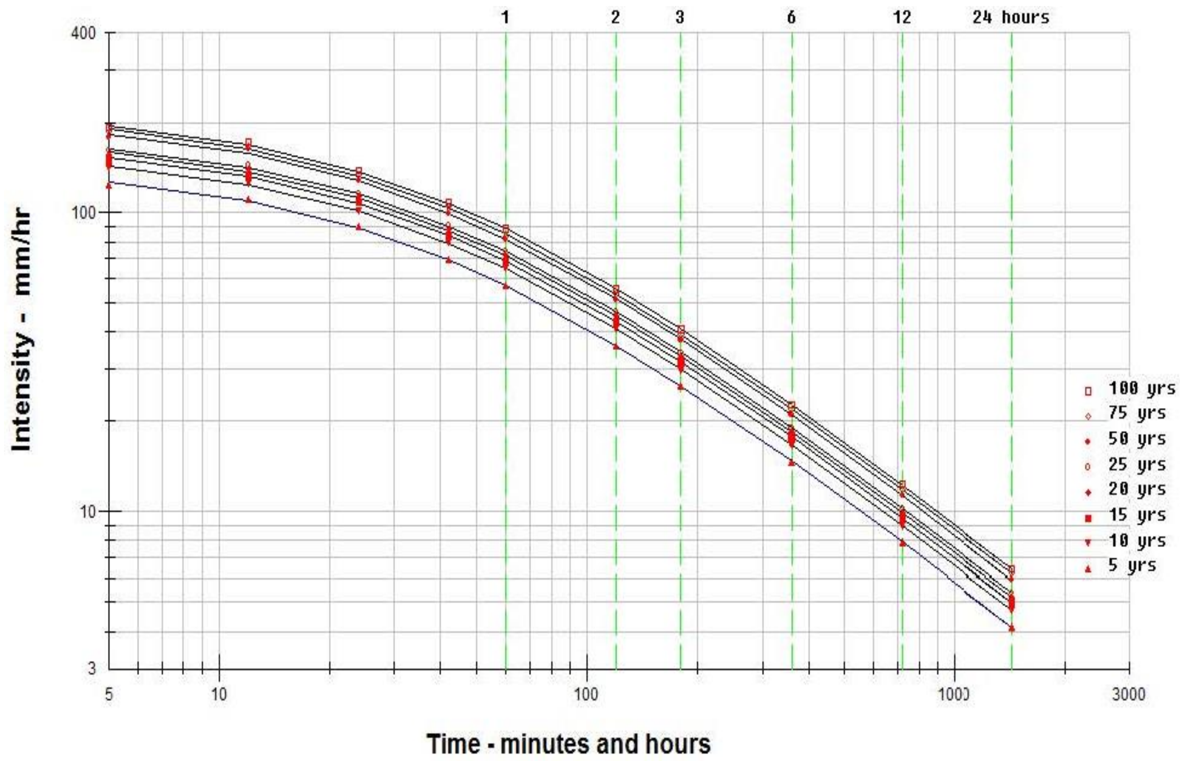
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Kusi

Longitude: 0°51'41.59"W

Latitude: 6° 2'49.62"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	111.05	126.24	134.06	139.65	144.12	158.64	166.46	172.04
0.4	89.63	101.90	108.21	112.72	116.33	128.05	134.36	138.87
0.7	69.68	79.21	84.12	87.62	90.43	99.54	104.44	107.95
1	57.08	64.89	68.91	71.79	74.08	81.55	85.57	88.44
2	35.82	40.72	43.24	45.04	46.48	51.17	53.69	55.49
3	26.21	29.79	31.64	32.96	34.01	37.44	39.28	40.60
6	14.65	16.65	17.68	18.42	19.01	20.92	21.95	22.69
12	7.87	8.95	9.50	9.90	10.22	11.25	11.80	12.20
24	4.14	4.71	5.00	5.21	5.38	5.92	6.21	6.42



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

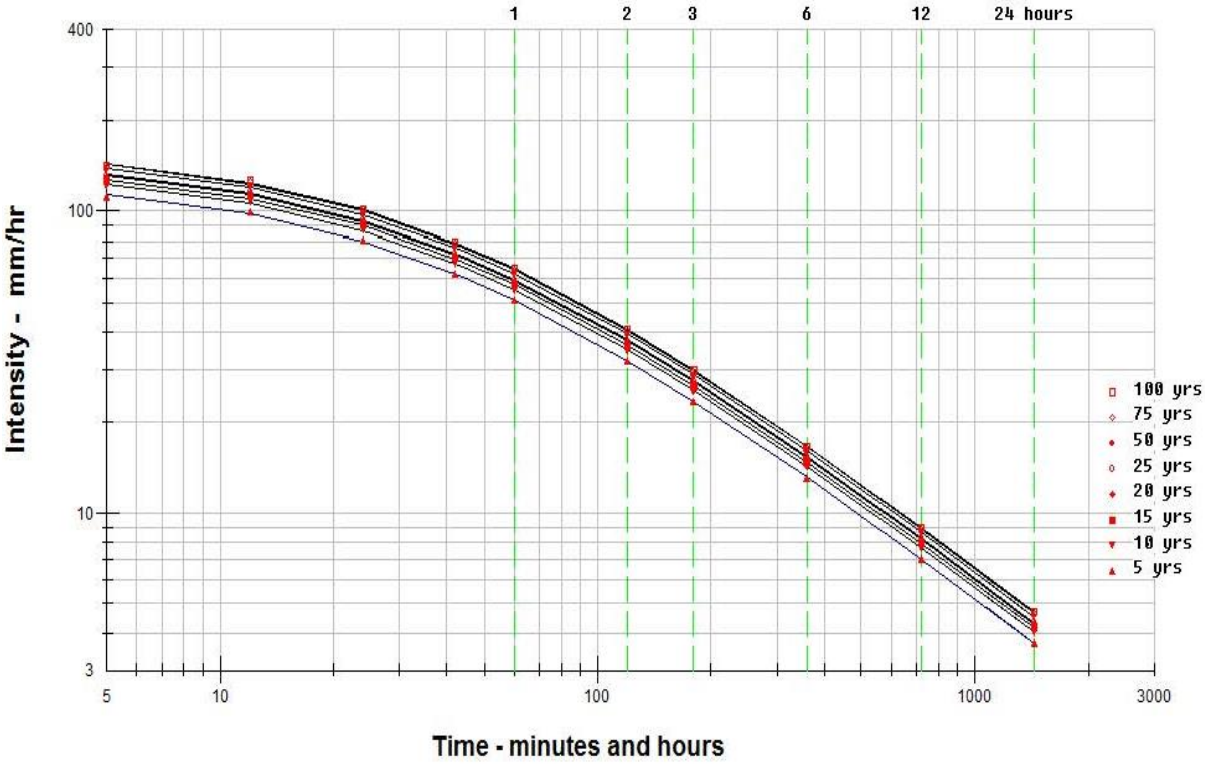
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Kwadaso

Longitude: 1°38'57.67"W

Latitude: 6°41'25.70"N

Duration (hrs)	Return period (yrs)								
	2	**5**	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	83.01	99.32	107.81	111.72	115.07	116.19	121.77	125.12	126.24
0.4	67.00	80.17	87.02	90.18	92.88	93.78	98.29	101.00	101.90
0.7	52.08	62.32	67.64	70.10	72.20	72.90	76.41	78.51	79.21
1	42.67	51.05	55.42	57.43	59.15	59.73	62.60	64.32	64.89
2	26.77	32.03	34.77	36.03	37.11	37.48	39.28	40.36	40.72
3	19.59	23.44	25.44	26.37	27.16	27.42	28.74	29.53	29.79
6	10.95	13.10	14.22	14.73	15.18	15.32	16.06	16.50	16.65
12	5.88	7.04	7.64	7.92	8.16	8.24	8.63	8.87	8.95
24	3.10	3.70	4.02	4.17	4.29	4.33	4.54	4.67	4.71



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

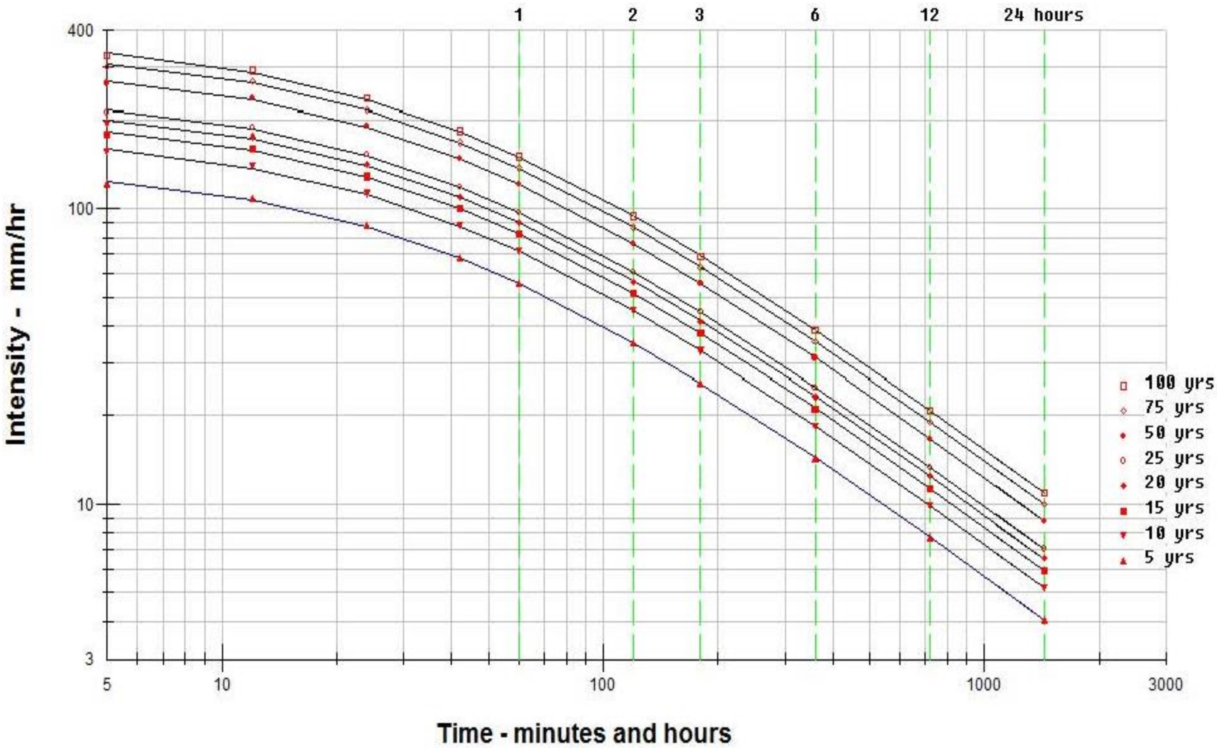
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Kwame Danso

Longitude: 0°40'46.84"W

Latitude: 7°43'58.75"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	108.70	139.65	159.76	175.40	188.80	235.72	268.12	292.70
0.4	87.74	112.72	128.95	141.57	152.40	190.27	216.42	236.26
0.7	68.20	87.62	100.24	110.05	118.46	147.91	168.23	183.65
1	55.88	71.79	82.12	90.16	97.06	121.18	137.83	150.46
2	35.06	45.04	51.53	56.57	60.90	76.03	86.48	94.41
3	25.65	32.96	37.70	41.39	44.56	55.63	63.28	69.08
6	14.34	18.42	21.07	23.13	24.90	31.09	35.36	38.60
12	7.71	9.90	11.33	12.43	13.38	16.71	19.01	20.75
24	4.05	5.21	5.96	6.54	7.04	8.79	10.00	10.92



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

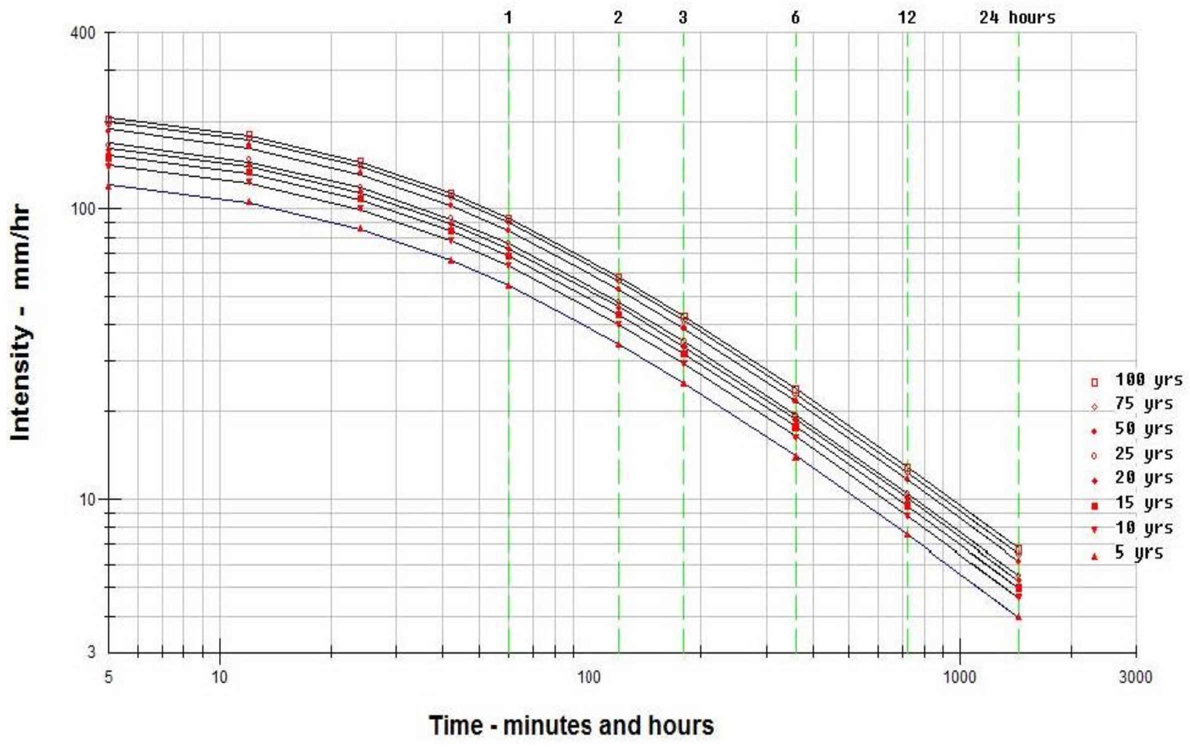
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Manga Bawku

Longitude: 0°15'51.67"W

Latitude: 11° 1'2.38"N

Duration (hrs)	**Return period (yrs)**							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	106.47	124.01	134.06	141.88	147.47	164.22	174.28	180.98
0.4	85.94	100.09	108.21	114.52	119.03	132.56	140.67	146.08
0.7	66.80	77.81	84.12	89.02	92.53	103.04	109.35	113.56
1	54.73	63.75	68.91	72.93	75.81	84.42	89.59	93.03
2	34.34	40.00	43.24	45.76	47.56	52.97	56.21	58.38
3	25.13	29.27	31.64	33.48	34.80	38.76	41.13	42.71
6	14.04	16.35	17.68	18.71	19.45	21.66	22.99	23.87
12	7.55	8.79	9.50	10.06	10.45	11.64	12.36	12.83
24	3.97	4.63	5.00	5.29	5.50	6.13	6.50	6.75



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

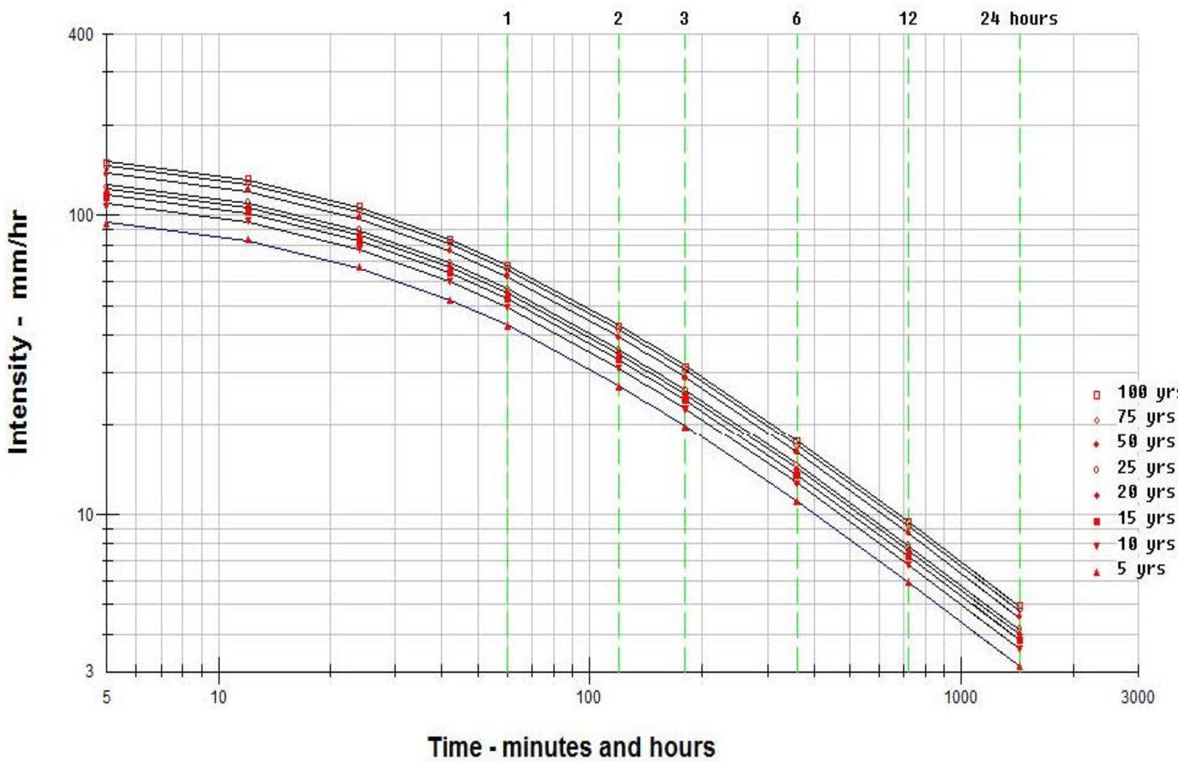
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Nadawli

Longitude: 2°39'48.98"W

Latitude: 10°22'0.82"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	83.56	95.85	102.56	107.25	110.94	121.77	128.47	132.94
0.4	67.45	77.37	82.78	86.57	89.54	98.29	103.70	107.31
0.7	52.43	60.14	64.35	67.29	69.61	76.41	80.61	83.42
1	42.96	49.27	52.72	55.13	57.03	62.60	66.04	68.34
2	26.95	30.92	33.08	34.59	35.78	39.28	41.44	42.88
3	19.72	22.62	24.20	25.31	26.18	28.74	30.32	31.37
6	11.02	12.64	13.53	14.14	14.63	16.06	16.94	17.53
12	5.92	6.80	7.27	7.60	7.86	8.63	9.11	9.42
24	3.12	3.58	3.83	4.00	4.14	4.54	4.79	4.96



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

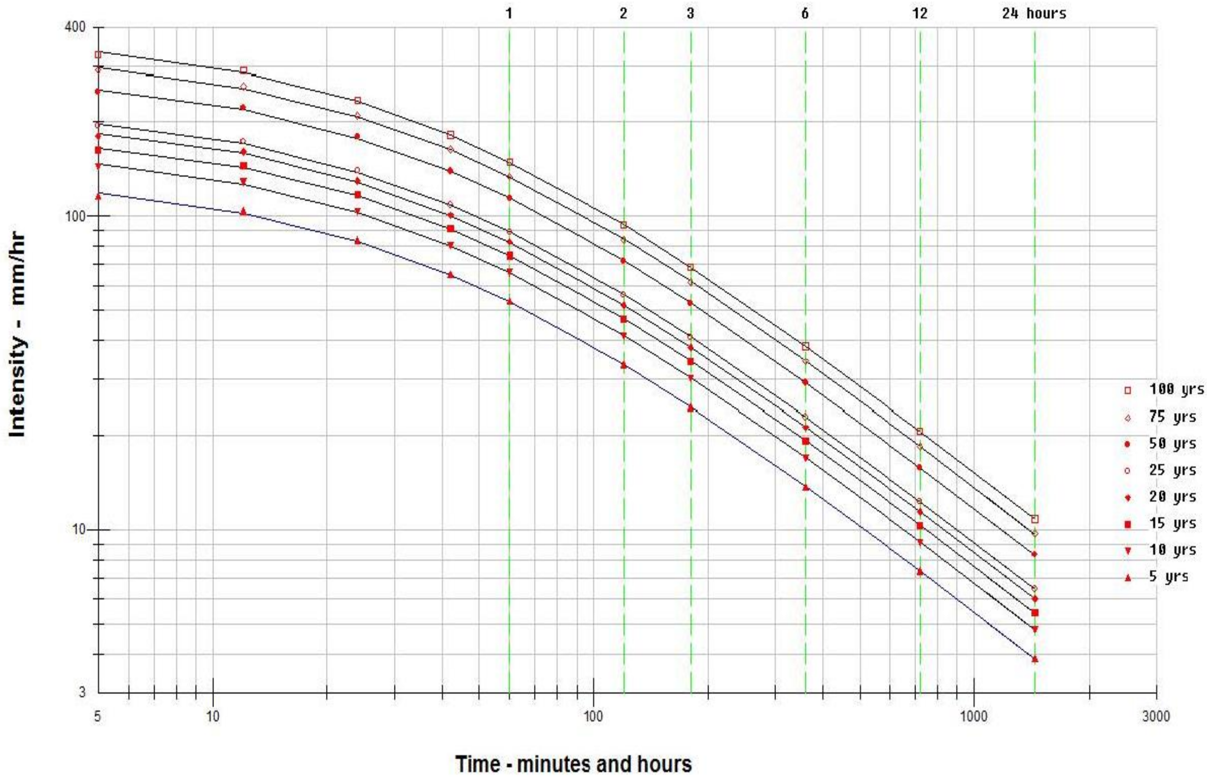
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Navrongo

Longitude: 1° 5'31.75"W

Latitude: 10°53'38.48"N

Duration (hrs)	Return period (yrs)								
	2	**5**	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	80.21	103.79	128.47	145.23	160.87	173.16	222.32	260.30	290.46
0.4	64.75	83.77	103.70	117.23	129.85	139.77	179.45	210.11	234.46
0.7	50.33	65.12	80.61	91.13	100.94	108.65	139.49	163.33	182.25
1	41.23	53.35	66.04	74.66	82.70	89.01	114.28	133.81	149.32
2	25.87	33.48	41.44	46.84	51.89	55.85	71.71	83.96	93.69
3	18.93	24.49	30.32	34.28	37.97	40.87	52.47	61.43	68.55
6	10.58	13.69	16.94	19.15	21.22	22.84	29.32	34.33	38.31
12	5.69	7.36	9.11	10.30	11.40	12.28	15.76	18.45	20.59
24	2.99	3.87	4.79	5.42	6.00	6.46	8.29	9.71	10.83



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

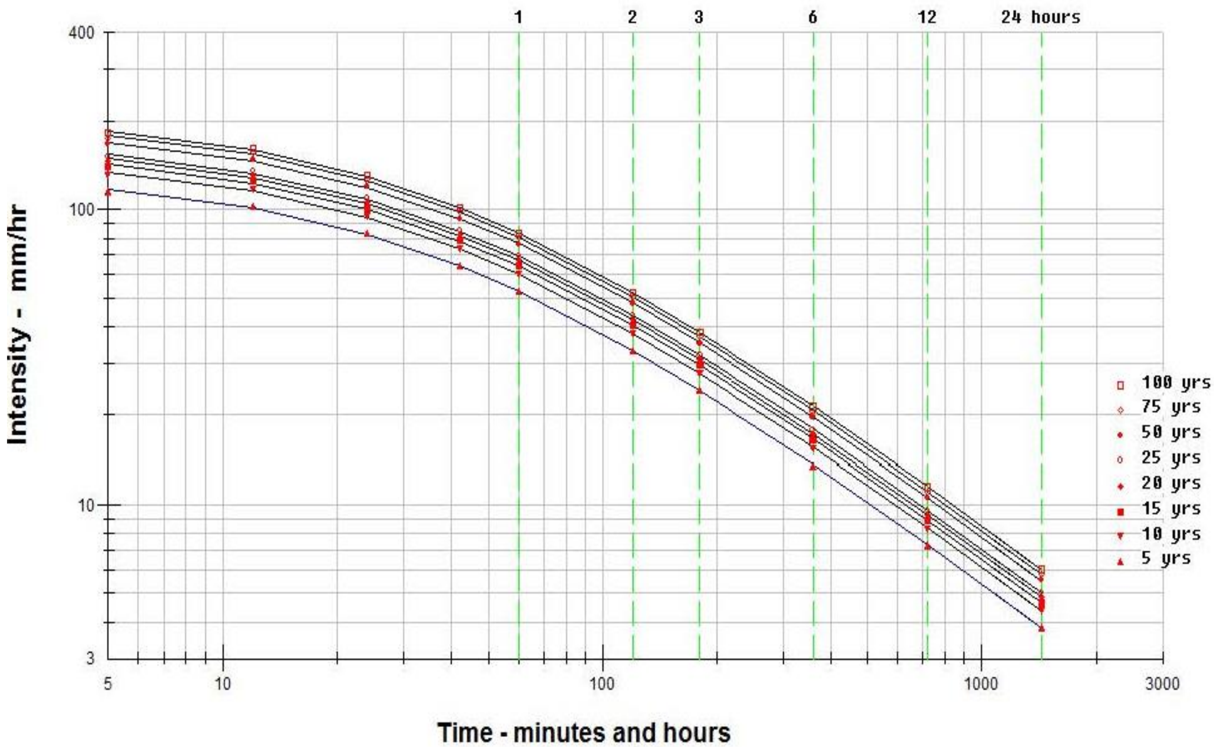
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Nkoranza

Longitude: 1°42'36.34"W

Latitude: 7°33'33.52"N

Duration (hrs)	Return period (yrs)								
	2	5	10	15	20	25	50	75	100
0.2	81.11	102.78	117.30	125.12	130.71	135.18	148.58	156.40	161.99
0.4	65.47	82.96	94.68	101.00	105.50	109.11	119.93	126.25	130.75
0.7	50.89	64.49	73.60	78.51	82.01	84.82	93.23	98.14	101.64
1	41.69	52.83	60.30	64.32	67.19	69.49	76.38	80.40	83.27
2	26.16	33.15	37.84	40.36	42.16	43.60	47.93	50.45	52.25
3	19.14	24.26	27.68	29.53	30.85	31.90	35.07	36.91	38.23
6	10.70	13.56	15.47	16.50	17.24	17.83	19.60	20.63	21.36
12	5.75	7.29	8.32	8.87	9.27	9.58	10.53	11.09	11.48
24	3.03	3.83	4.38	4.67	4.88	5.04	5.54	5.83	6.04



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

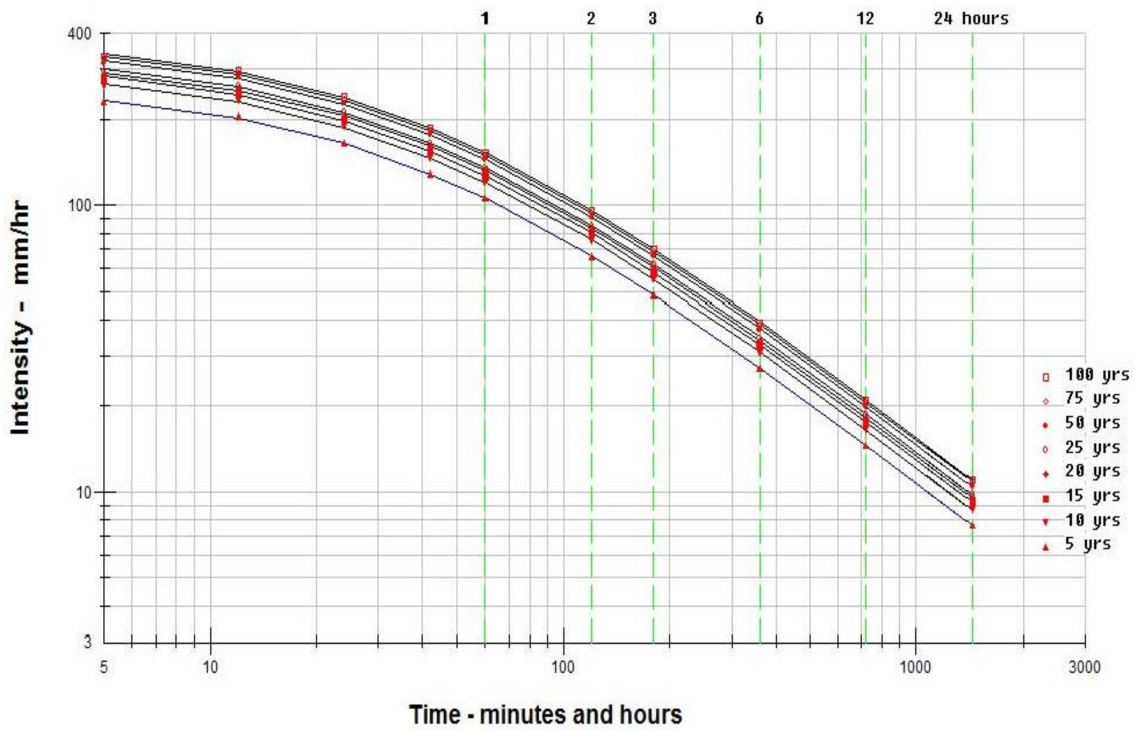
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Nkroful

Longitude: 2°19'24.22"W

Latitude: 4°57'57.93"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	205.56	233.49	248.01	256.95	263.65	281.53	291.58	297.17
0.4	165.92	188.47	200.19	207.40	212.81	227.24	235.36	239.87
0.7	128.98	146.50	155.62	161.22	165.43	176.65	182.95	186.46
1	105.67	120.03	127.49	132.09	135.53	144.72	149.89	152.76
2	66.30	75.31	80.00	82.88	85.04	90.81	94.05	95.85
3	48.51	55.10	58.53	60.64	62.22	66.44	68.81	70.13
6	27.11	30.79	32.71	33.89	34.77	37.13	38.46	39.19
12	14.57	16.55	17.58	18.22	18.69	19.96	20.67	21.07
24	7.67	8.71	9.25	9.58	9.83	10.50	10.88	11.08



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

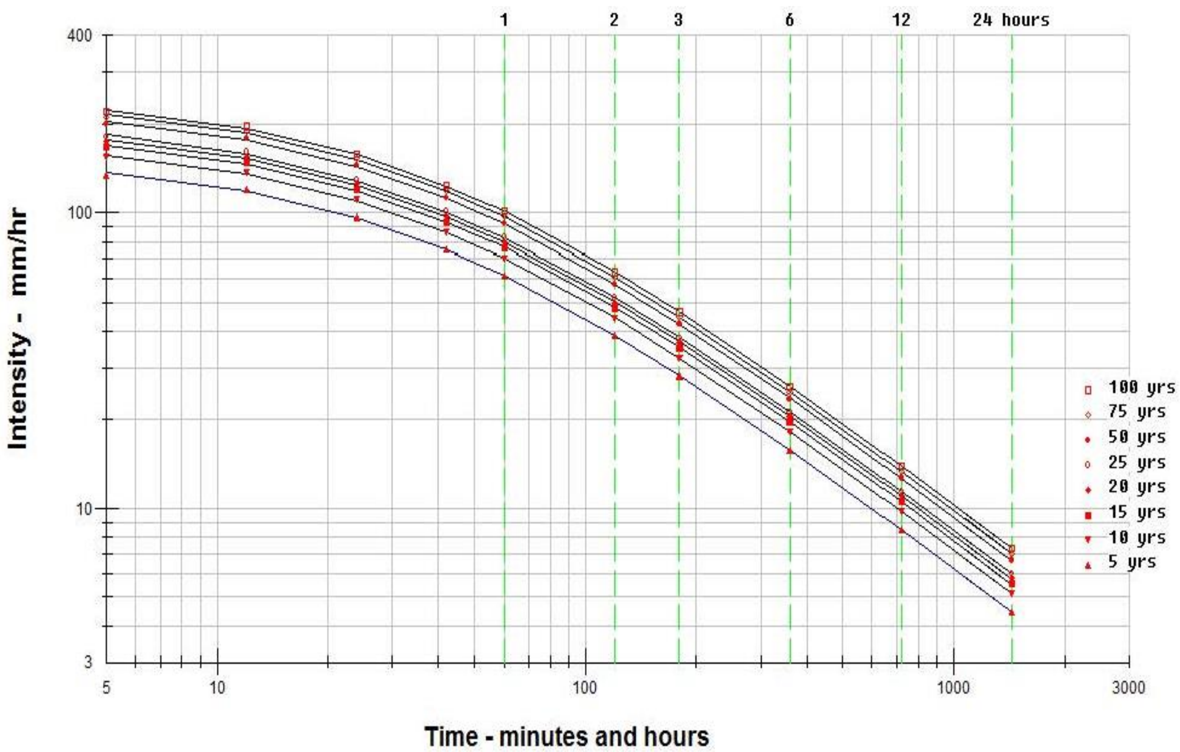
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Nsawam

Longitude: 0°21'4.69"W

Latitude: 5°49'10.03"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	119.54	137.41	148.58	155.29	160.87	178.75	188.80	196.62
0.4	96.49	110.92	119.93	125.34	129.85	144.28	152.40	158.71
0.7	75.00	86.22	93.23	97.44	100.94	112.16	118.46	123.37
1	61.45	70.64	76.38	79.83	82.70	91.89	97.06	101.08
2	38.56	44.32	47.93	50.09	51.89	57.65	60.90	63.42
3	28.21	32.43	35.07	36.65	37.97	42.18	44.56	46.40
6	15.77	18.12	19.60	20.48	21.22	23.57	24.90	25.93
12	8.47	9.74	10.53	11.01	11.40	12.67	13.38	13.94
24	4.46	5.13	5.54	5.79	6.00	6.67	7.04	7.33



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

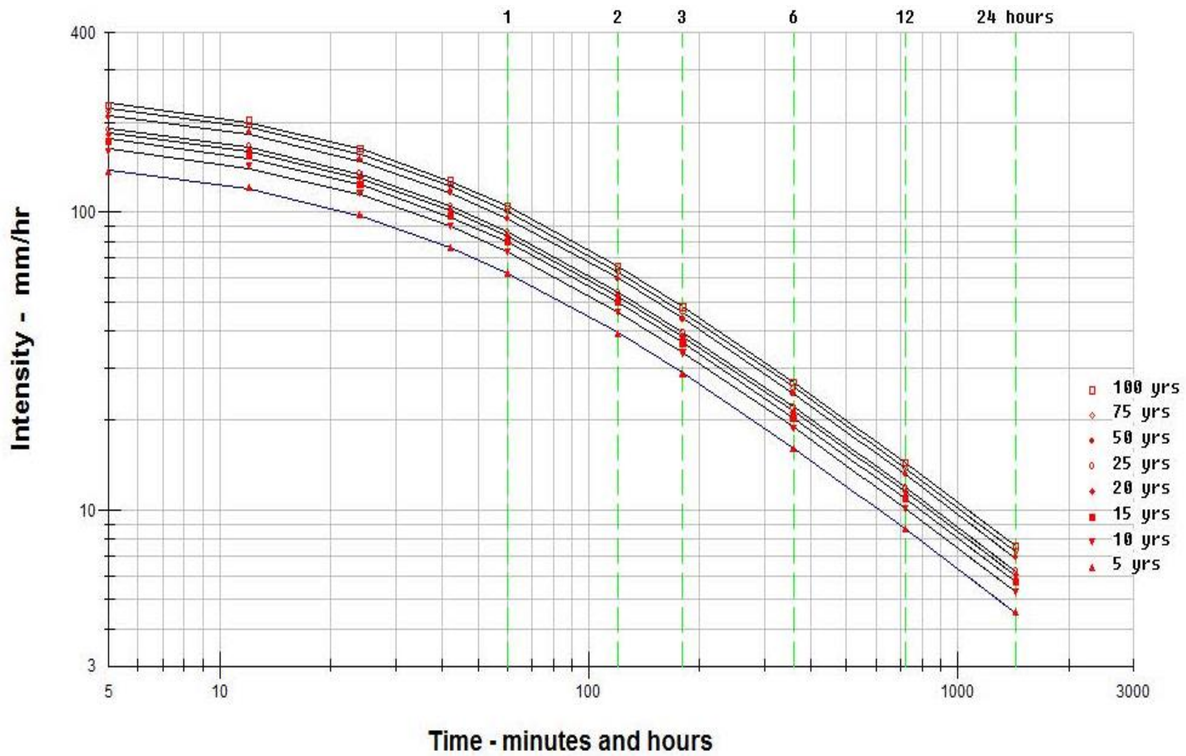
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Obuasi

Longitude: 1°41'28.50"W

Latitude: 6°12'4.35"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	121.77	143.00	154.17	161.99	167.58	185.45	195.51	203.33
0.4	98.29	115.42	124.44	130.75	135.26	149.69	157.81	164.12
0.7	76.41	89.72	96.73	101.64	105.15	116.36	122.67	127.58
1	62.60	73.51	79.25	83.27	86.14	95.33	100.50	104.52
2	39.28	46.12	49.73	52.25	54.05	59.82	63.06	65.58
3	28.74	33.75	36.38	38.23	39.55	43.77	46.14	47.99
6	16.06	18.86	20.33	21.36	22.10	24.46	25.78	26.82
12	8.63	10.14	10.93	11.48	11.88	13.15	13.86	14.41
24	4.54	5.33	5.75	6.04	6.25	6.92	7.29	7.58



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

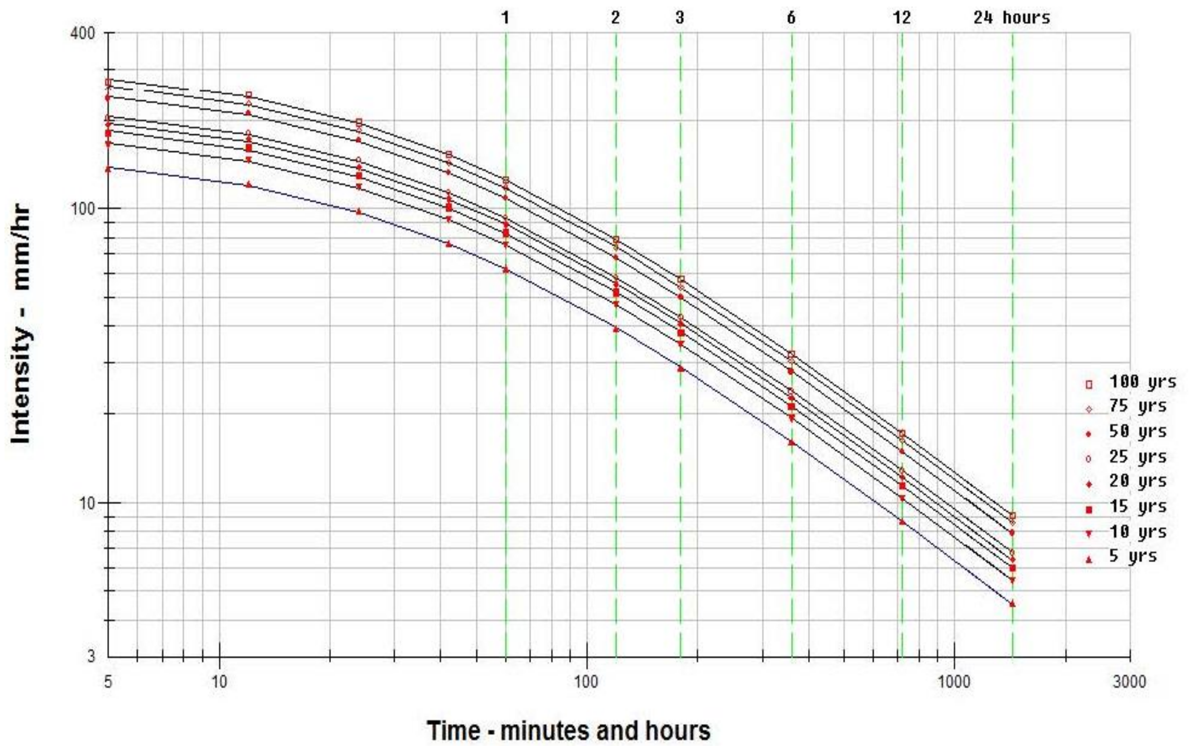
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Pomadze

Longitude: 0°39'4.40"W

Latitude: 5°20'38.03"N

Duration (hrs)	Return period (yrs)								
	2	5	10	15	20	25	50	75	100
0.2	89.60	121.77	146.35	160.87	172.04	180.98	211.15	229.02	243.54
0.4	72.32	98.29	118.13	129.85	138.87	146.08	170.43	184.86	196.58
0.7	56.22	76.41	91.83	100.94	107.95	113.56	132.48	143.70	152.81
1	46.06	62.60	75.23	82.70	88.44	93.03	108.54	117.73	125.20
2	28.90	39.28	47.20	51.89	55.49	58.38	68.10	73.87	78.55
3	21.15	28.74	34.54	37.97	40.60	42.71	49.83	54.05	57.48
6	11.82	16.06	19.30	21.22	22.69	23.87	27.85	30.20	32.12
12	6.35	8.63	10.38	11.40	12.20	12.83	14.97	16.24	17.27
24	3.34	4.54	5.46	6.00	6.42	6.75	7.88	8.54	9.08



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

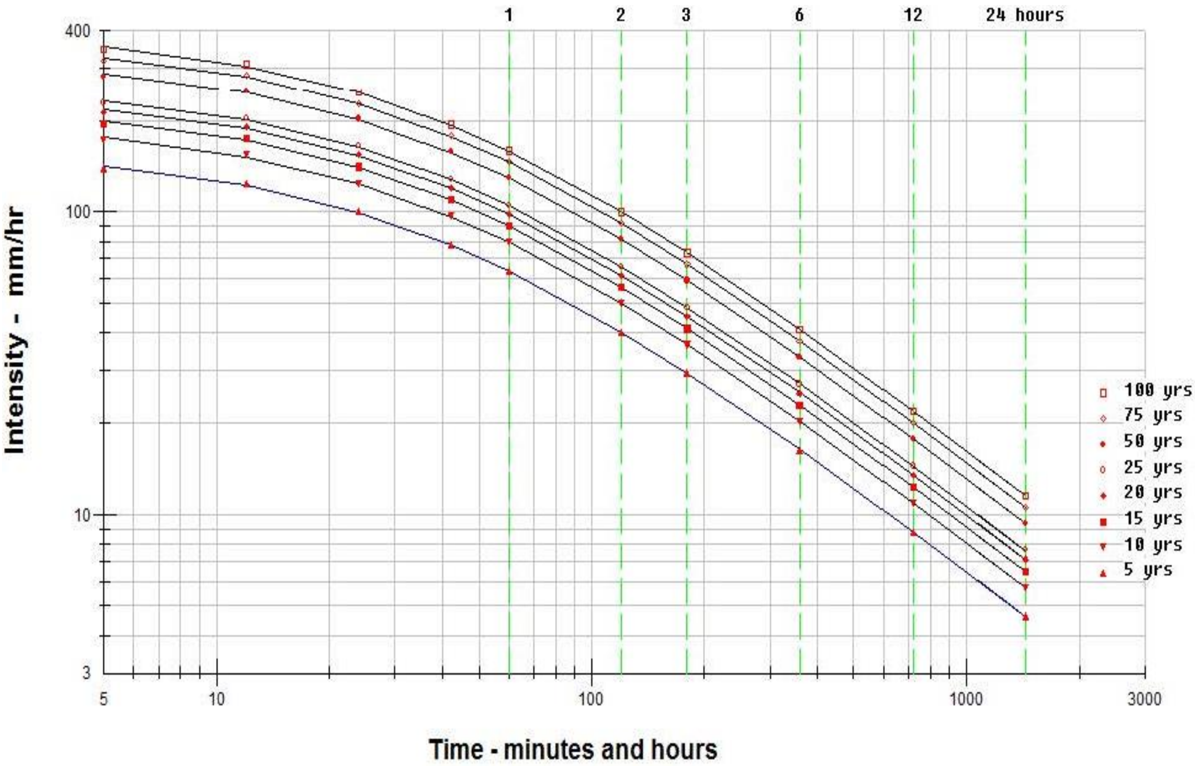
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Prang

Longitude: 0°53'3.12"W

Latitude: 7°59'37.58"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	124.01	154.17	174.28	191.04	204.44	251.36	283.76	309.46
0.4	100.09	124.44	140.67	154.20	165.02	202.89	229.04	249.78
0.7	77.81	96.73	109.35	119.87	128.28	157.72	178.05	194.17
1	63.75	79.25	89.59	98.20	105.10	129.22	145.87	159.08
2	40.00	49.73	56.21	61.62	65.94	81.08	91.53	99.81
3	29.27	36.38	41.13	45.09	48.25	59.32	66.97	73.03
6	16.35	20.33	22.99	25.20	26.96	33.15	37.42	40.81
12	8.79	10.93	12.36	13.54	14.49	17.82	20.12	21.94
24	4.63	5.75	6.50	7.13	7.63	9.38	10.58	11.54



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

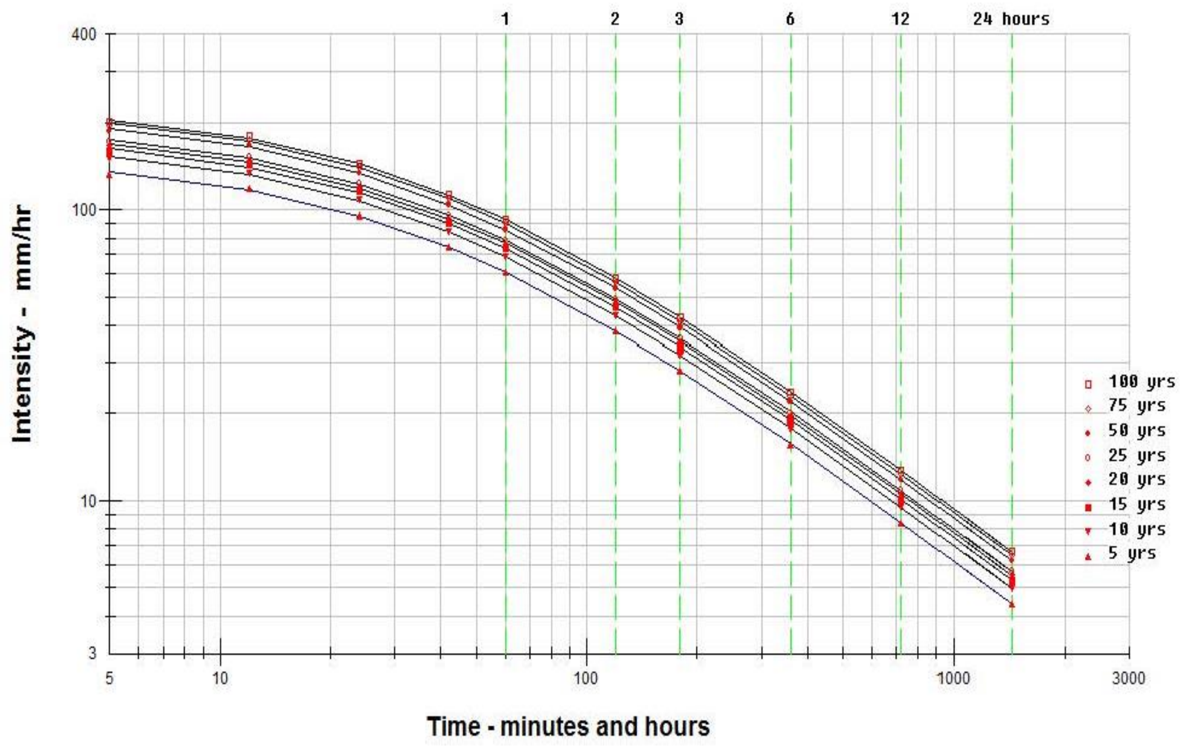
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Salaga

Longitude: 0°31'7.30"W

Latitude: 8°33'9.11"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	118.42	134.06	143.00	148.58	153.05	166.46	174.28	179.86
0.4	95.59	108.21	115.42	119.93	123.54	134.36	140.67	145.18
0.7	74.30	84.12	89.72	93.23	96.03	104.44	109.35	112.86
1	60.87	68.91	73.51	76.38	78.68	85.57	89.59	92.46
2	38.20	43.24	46.12	47.93	49.37	53.69	56.21	58.01
3	27.95	31.64	33.75	35.07	36.12	39.28	41.13	42.45
6	15.62	17.68	18.86	19.60	20.19	21.95	22.99	23.72
12	8.40	9.50	10.14	10.53	10.85	11.80	12.36	12.75
24	4.42	5.00	5.33	5.54	5.71	6.21	6.50	6.71



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

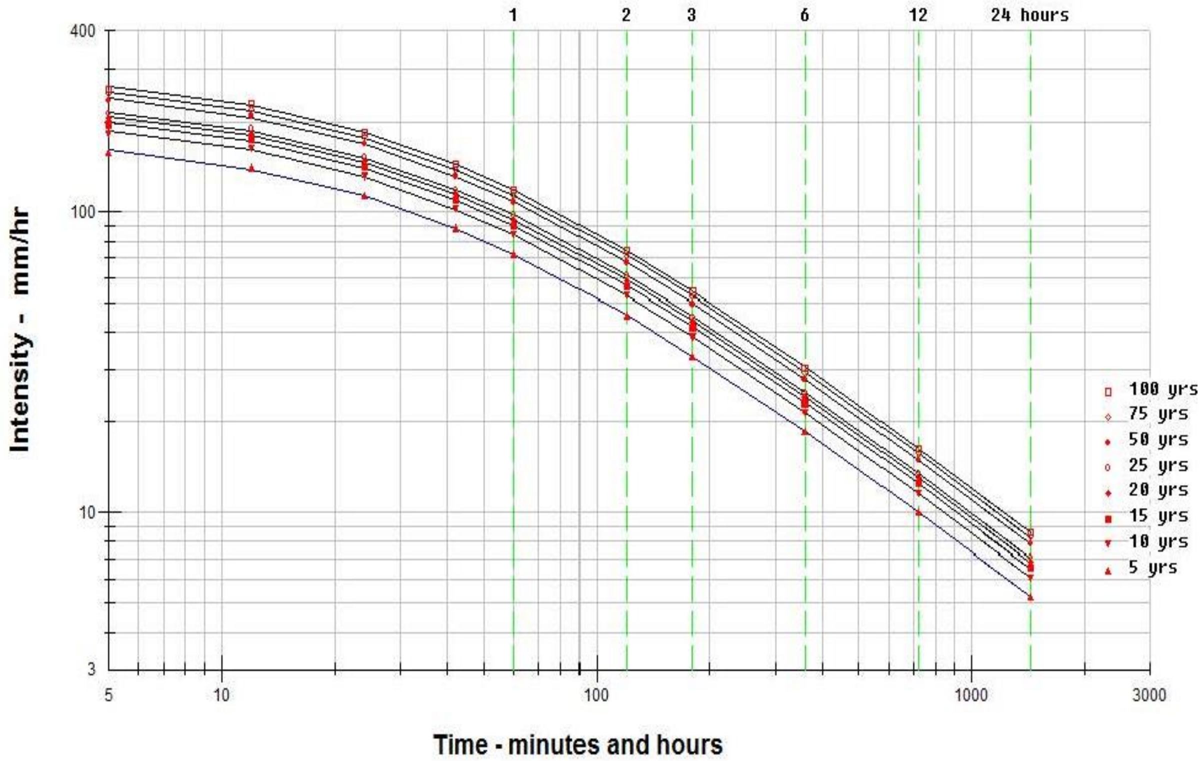
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Saltpond

Longitude: 1° 3'5.15"W

Latitude: 5°12'8.32"N

Duration (hrs)	**Return period (yrs)**							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	140.76	163.11	175.40	183.22	189.92	210.03	221.20	230.14
0.4	113.62	131.66	141.57	147.89	153.30	169.53	178.55	185.76
0.7	88.32	102.34	110.05	114.96	119.17	131.78	138.79	144.40
1	72.36	83.85	90.16	94.18	97.63	107.97	113.71	118.30
2	45.40	52.61	56.57	59.10	61.26	67.74	71.35	74.23
3	33.22	38.49	41.39	43.24	44.82	49.57	52.20	54.31
6	18.56	21.51	23.13	24.16	25.05	27.70	29.17	30.35
12	9.98	11.56	12.43	12.99	13.46	14.89	15.68	16.32
24	5.25	6.08	6.54	6.83	7.08	7.83	8.25	8.58



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

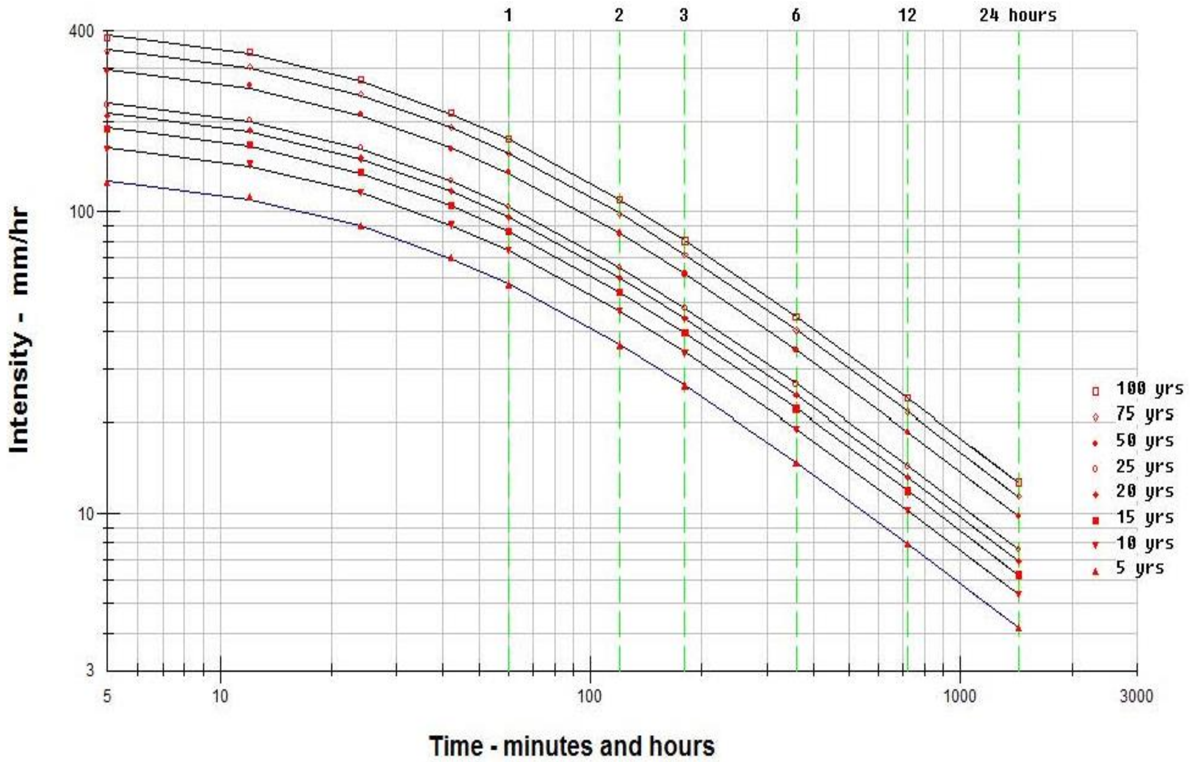
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Sandema

Longitude: 1°17'3.44"W

Latitude: 10°44'7.21"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	111.72	144.12	167.58	186.57	202.21	262.54	304.99	339.62
0.4	90.18	116.33	135.26	150.59	163.22	211.91	246.18	274.13
0.7	70.10	90.43	105.15	117.06	126.88	164.73	191.37	213.10
1	57.43	74.08	86.14	95.91	103.95	134.96	156.78	174.58
2	36.03	46.48	54.05	60.18	65.22	84.68	98.37	109.54
3	26.37	34.01	39.55	44.03	47.72	61.96	71.98	80.15
6	14.73	19.01	22.10	24.61	26.67	34.63	40.22	44.79
12	7.92	10.22	11.88	13.23	14.34	18.61	21.62	24.08
24	4.17	5.38	6.25	6.96	7.54	9.79	11.38	12.67



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

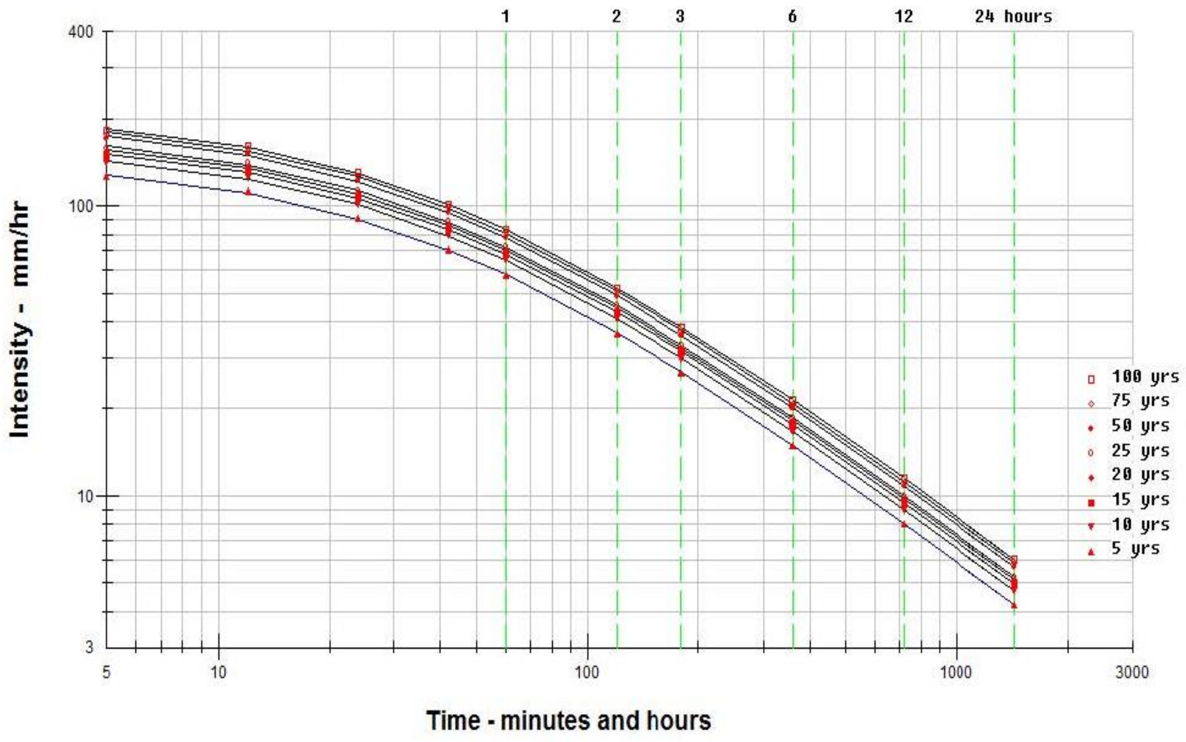
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Sefwi Bekwai

Longitude: 2°19'28.66"W

Latitude: 6°11'52.90"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	112.83	126.24	132.94	137.41	140.76	151.94	157.52	161.99
0.4	91.08	101.90	107.31	110.92	113.62	122.64	127.15	130.75
0.7	70.80	79.21	83.42	86.22	88.32	95.33	98.84	101.64
1	58.00	64.89	68.34	70.64	72.36	78.10	80.97	83.27
2	36.39	40.72	42.88	44.32	45.40	49.01	50.81	52.25
3	26.63	29.79	31.37	32.43	33.22	35.86	37.18	38.23
6	14.88	16.65	17.53	18.12	18.56	20.04	20.78	21.36
12	8.00	8.95	9.42	9.74	9.98	10.77	11.17	11.48
24	4.21	4.71	4.96	5.13	5.25	5.67	5.88	6.04



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

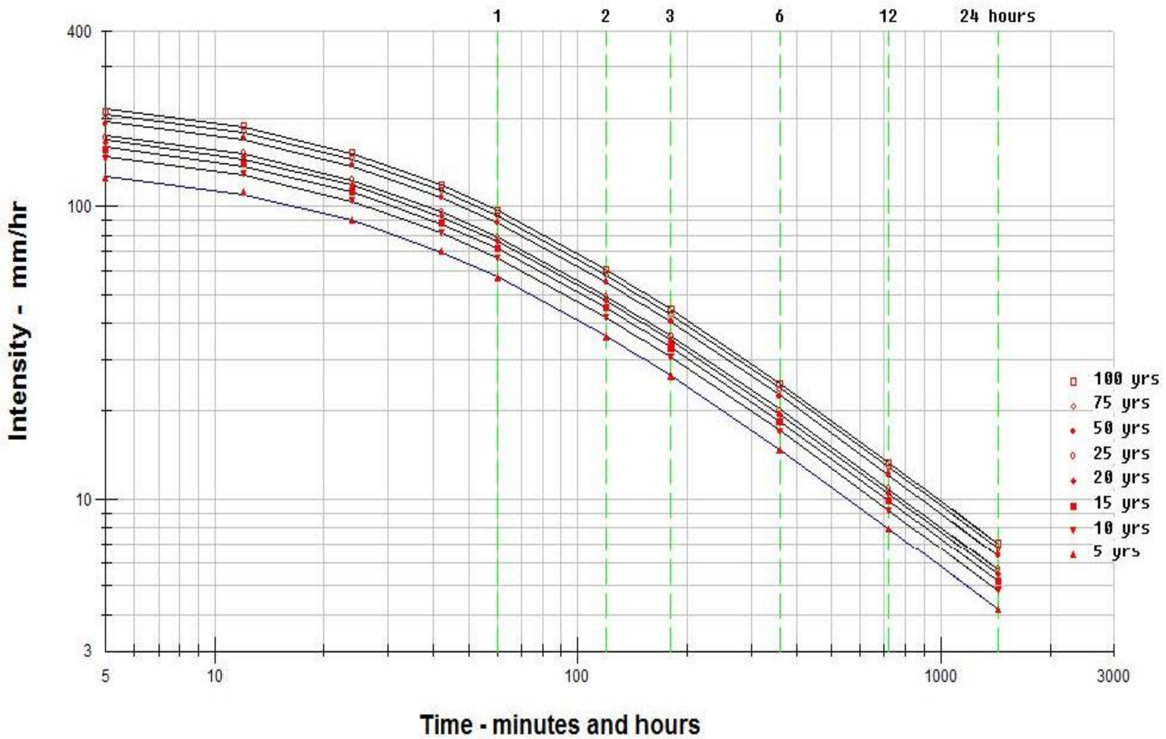
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Sunyani

Longitude: 2°18'44.29"W

Latitude: 7°20'5.79"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	111.72	129.59	139.65	147.47	153.05	170.93	180.98	188.80
0.4	90.18	104.60	112.72	119.03	123.54	137.97	146.08	152.40
0.7	70.10	81.31	87.62	92.53	96.03	107.25	113.56	118.46
1	57.43	66.62	71.79	75.81	78.68	87.87	93.03	97.06
2	36.03	41.80	45.04	47.56	49.37	55.13	58.38	60.90
3	26.37	30.58	32.96	34.80	36.12	40.34	42.71	44.56
6	14.73	17.09	18.42	19.45	20.19	22.54	23.87	24.90
12	7.92	9.19	9.90	10.45	10.85	12.12	12.83	13.38
24	4.17	4.83	5.21	5.50	5.71	6.38	6.75	7.04



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

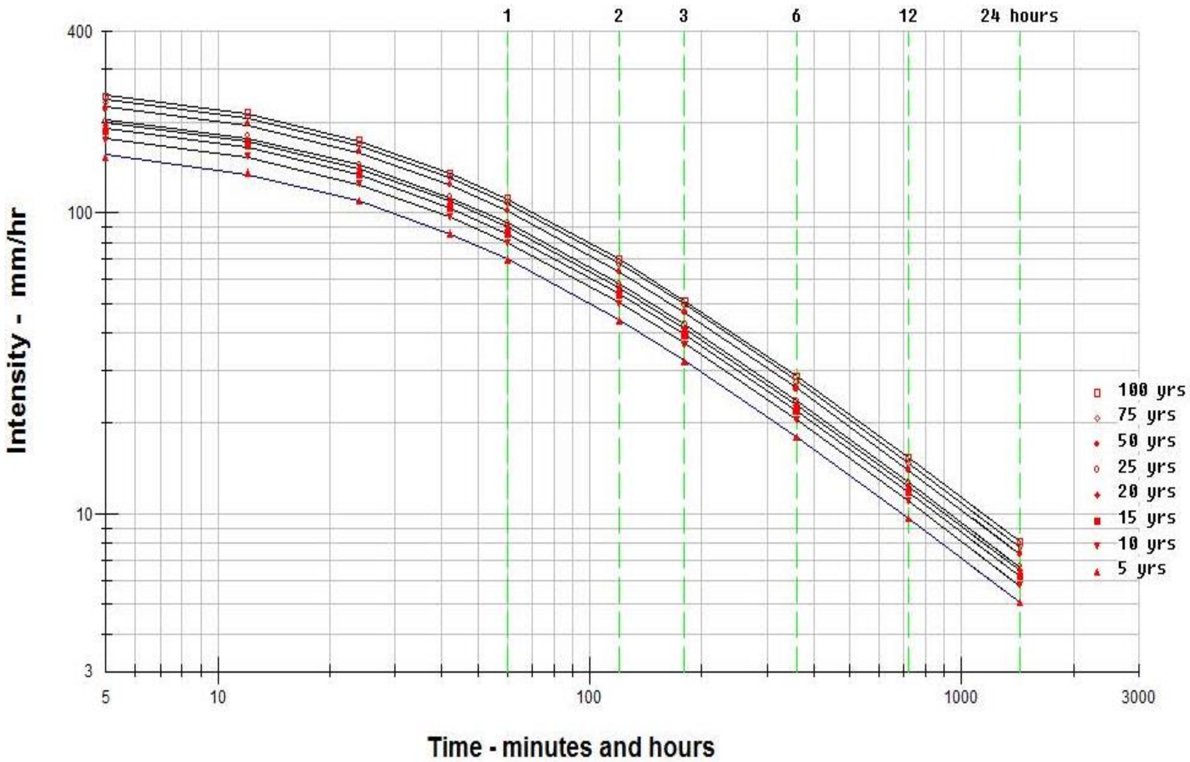
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Takoradi

Longitude: 1°46'59.15"W

Latitude: 4°54'5.69"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	136.29	155.29	166.46	174.28	179.86	197.74	208.91	216.73
0.4	110.01	125.34	134.36	140.67	145.18	159.61	168.63	174.94
0.7	85.52	97.44	104.44	109.35	112.86	124.07	131.08	135.99
1	70.06	79.83	85.57	89.59	92.46	101.65	107.39	111.41
2	43.96	50.09	53.69	56.21	58.01	63.78	67.38	69.91
3	32.17	36.65	39.28	41.13	42.45	46.67	49.30	51.15
6	17.98	20.48	21.95	22.99	23.72	26.08	27.55	28.58
12	9.66	11.01	11.80	12.36	12.75	14.02	14.81	15.36
24	5.08	5.79	6.21	6.50	6.71	7.38	7.79	8.08



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

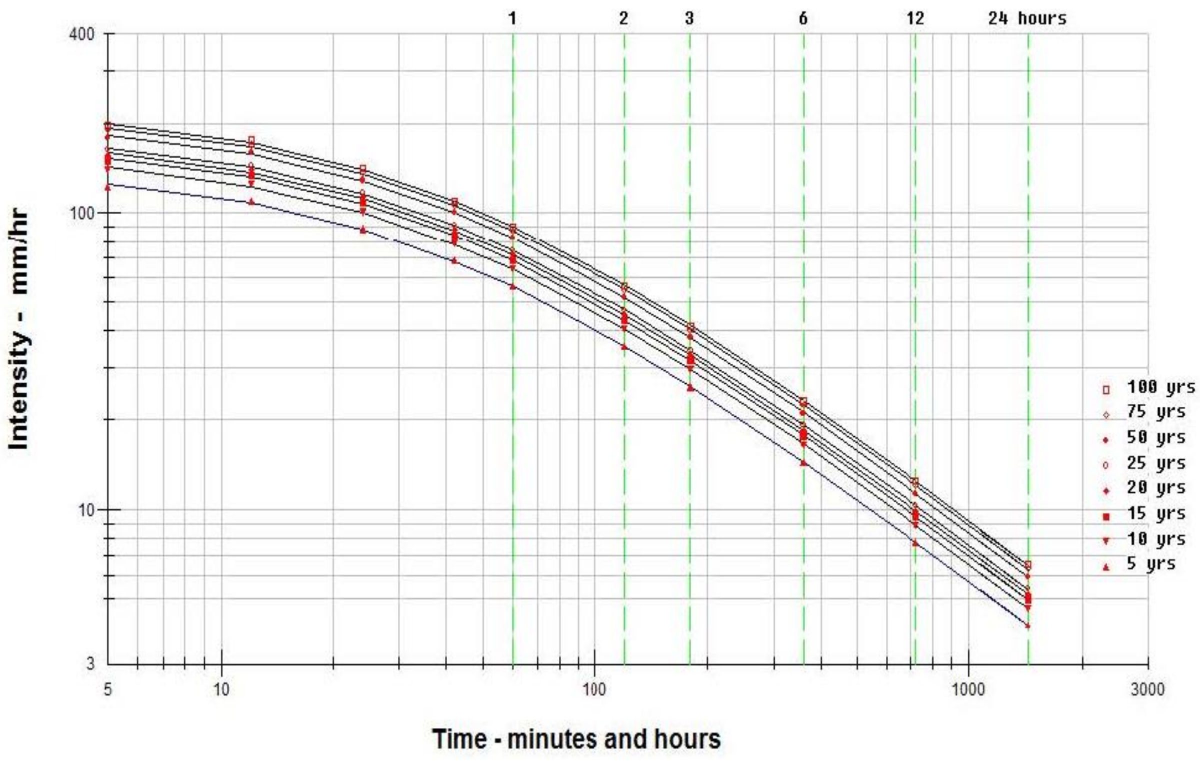
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Tamale

Longitude: 0°50'32.70"W

Latitude: 9°24'12.32"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	109.48	125.12	134.06	139.65	145.23	159.76	169.81	175.40
0.4	88.37	101.00	108.21	112.72	117.23	128.95	137.07	141.57
0.7	68.70	78.51	84.12	87.62	91.13	100.24	106.55	110.05
1	56.28	64.32	68.91	71.79	74.66	82.12	87.29	90.16
2	35.31	40.36	43.24	45.04	46.84	51.53	54.77	56.57
3	25.84	29.53	31.64	32.96	34.28	37.70	40.08	41.39
6	14.44	16.50	17.68	18.42	19.15	21.07	22.40	23.13
12	7.76	8.87	9.50	9.90	10.30	11.33	12.04	12.43
24	4.08	4.67	5.00	5.21	5.42	5.96	6.33	6.54



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

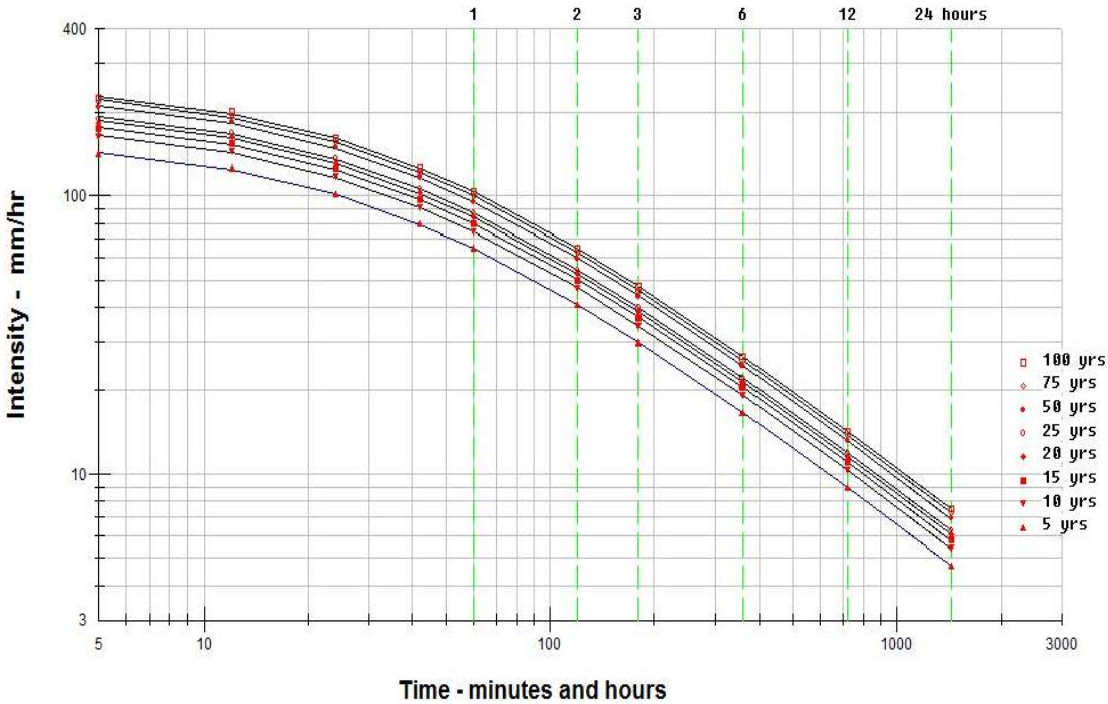
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Takwa

Longitude: 1°59'34.97"W

Latitude: 5°18'6.60"N

	Return period (yrs)							
Duration (hrs)	**5**	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	126.24	145.23	155.29	163.11	168.69	185.45	194.39	201.09
0.4	101.90	117.23	125.34	131.66	136.16	149.69	156.90	162.32
0.7	79.21	91.13	97.44	102.34	105.85	116.36	121.97	126.18
1	64.89	74.66	79.83	83.85	86.72	95.33	99.93	103.37
2	40.72	46.84	50.09	52.61	54.41	59.82	62.70	64.86
3	29.79	34.28	36.65	38.49	39.81	43.77	45.88	47.46
6	16.65	19.15	20.48	21.51	22.25	24.46	25.64	26.52
12	8.95	10.30	11.01	11.56	11.96	13.15	13.78	14.26
24	4.71	5.42	5.79	6.08	6.29	6.92	7.25	7.50



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

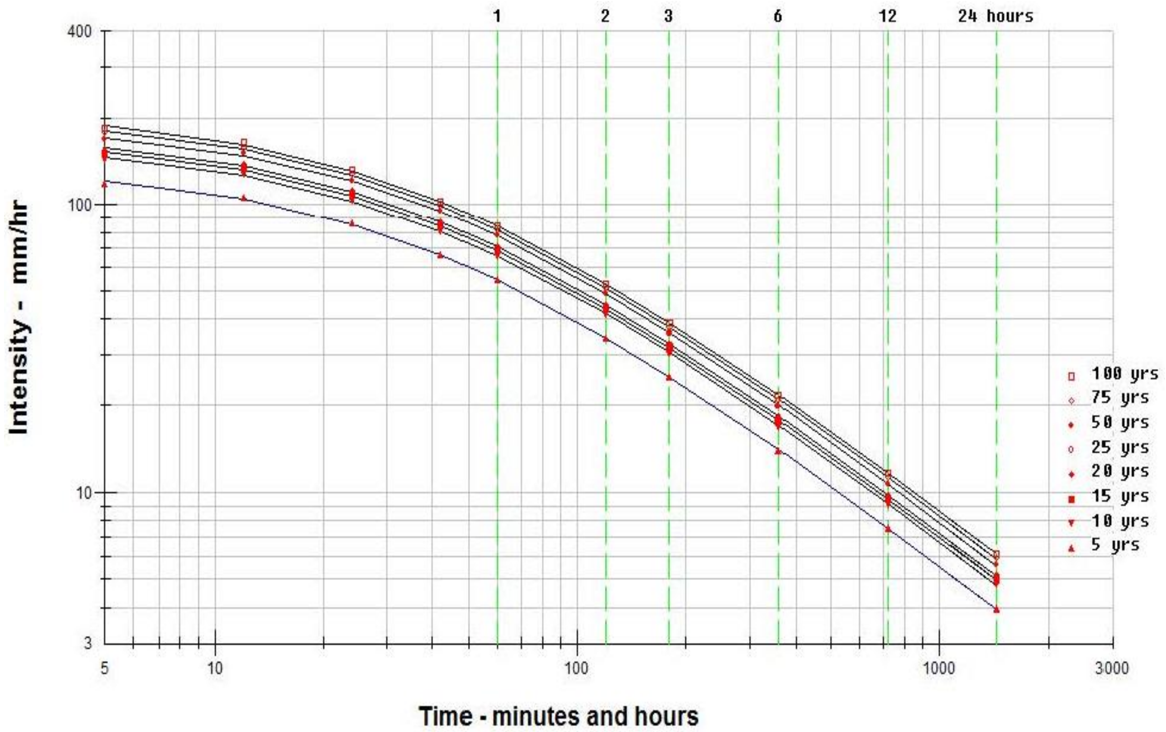
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Techiman

Longitude: 1°56'3.70"W

Latitude: 7°35'26.56"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	106.13	120.65	128.47	134.06	138.53	150.82	158.64	164.22
0.4	85.67	97.39	103.70	108.21	111.82	121.74	128.05	132.56
0.7	66.59	75.71	80.61	84.12	86.92	94.63	99.54	103.04
1	54.56	62.02	66.04	68.91	71.21	77.53	81.55	84.42
2	34.23	38.92	41.44	43.24	44.68	48.65	51.17	52.97
3	25.05	28.47	30.32	31.64	32.69	35.59	37.44	38.76
6	14.00	15.91	16.94	17.68	18.27	19.89	20.92	21.66
12	7.52	8.55	9.11	9.50	9.82	10.69	11.25	11.64
24	3.96	4.50	4.79	5.00	5.17	5.63	5.92	6.13



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

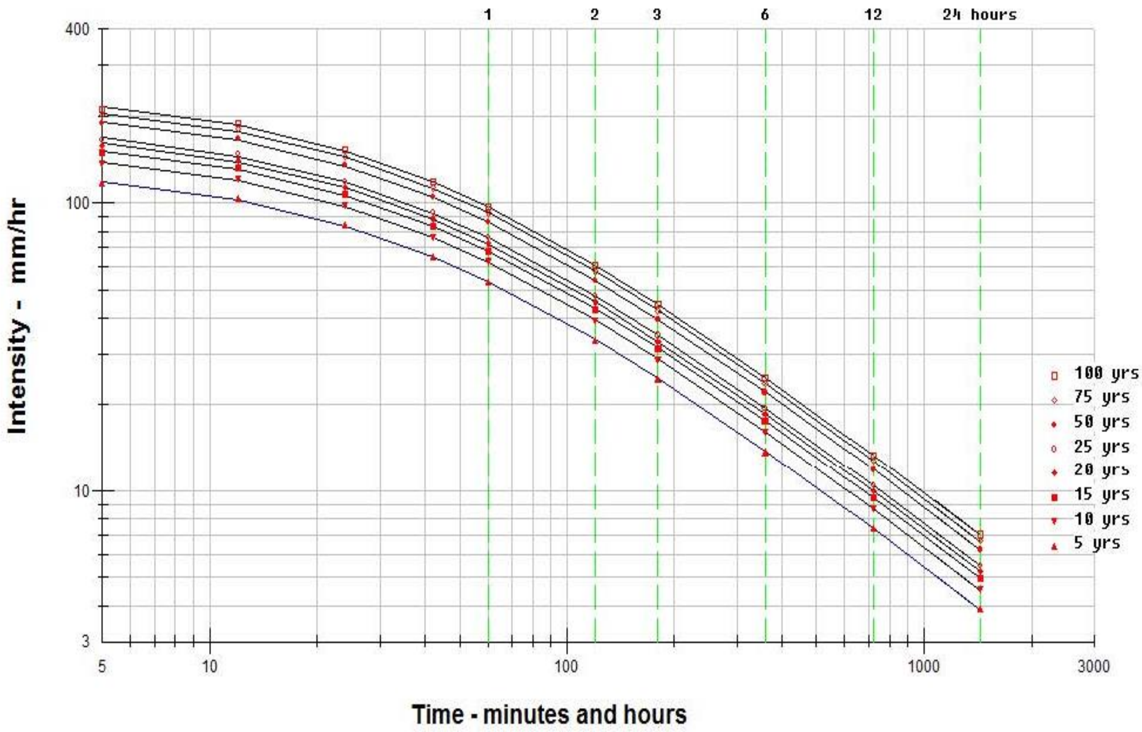
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Tema

Longitude: 0° 1'48.80"E

Latitude: 5°44'5.37"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	104.12	121.77	132.94	140.76	147.47	167.58	179.86	188.80
0.4	84.04	98.29	107.31	113.62	119.03	135.26	145.18	152.40
0.7	65.33	76.41	83.42	88.32	92.53	105.15	112.86	118.46
1	53.52	62.60	68.34	72.36	75.81	86.14	92.46	97.06
2	33.58	39.28	42.88	45.40	47.56	54.05	58.01	60.90
3	24.57	28.74	31.37	33.22	34.80	39.55	42.45	44.56
6	13.73	16.06	17.53	18.56	19.45	22.10	23.72	24.90
12	7.38	8.63	9.42	9.98	10.45	11.88	12.75	13.38
24	3.88	4.54	4.96	5.25	5.50	6.25	6.71	7.04



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

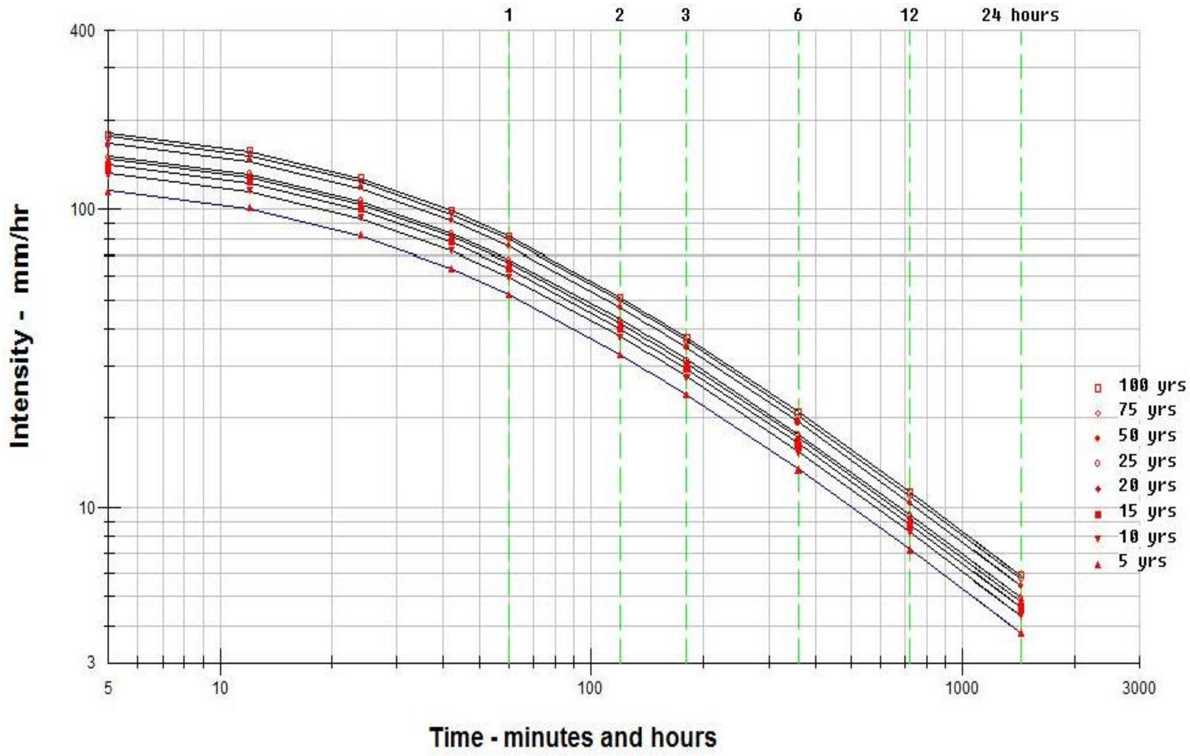
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Tumu

Longitude: 1°58'59.73"W

Latitude: 10°52'41.87"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	101.77	116.19	124.01	129.59	132.94	146.35	154.17	158.64
0.4	82.15	93.78	100.09	104.60	107.31	118.13	124.44	128.05
0.7	63.86	72.90	77.81	81.31	83.42	91.83	96.73	99.54
1	52.32	59.73	63.75	66.62	68.34	75.23	79.25	81.55
2	32.83	37.48	40.00	41.80	42.88	47.20	49.73	51.17
3	24.02	27.42	29.27	30.58	31.37	34.54	36.38	37.44
6	13.42	15.32	16.35	17.09	17.53	19.30	20.33	20.92
12	7.22	8.24	8.79	9.19	9.42	10.38	10.93	11.25
24	3.80	4.33	4.63	4.83	4.96	5.46	5.75	5.92



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

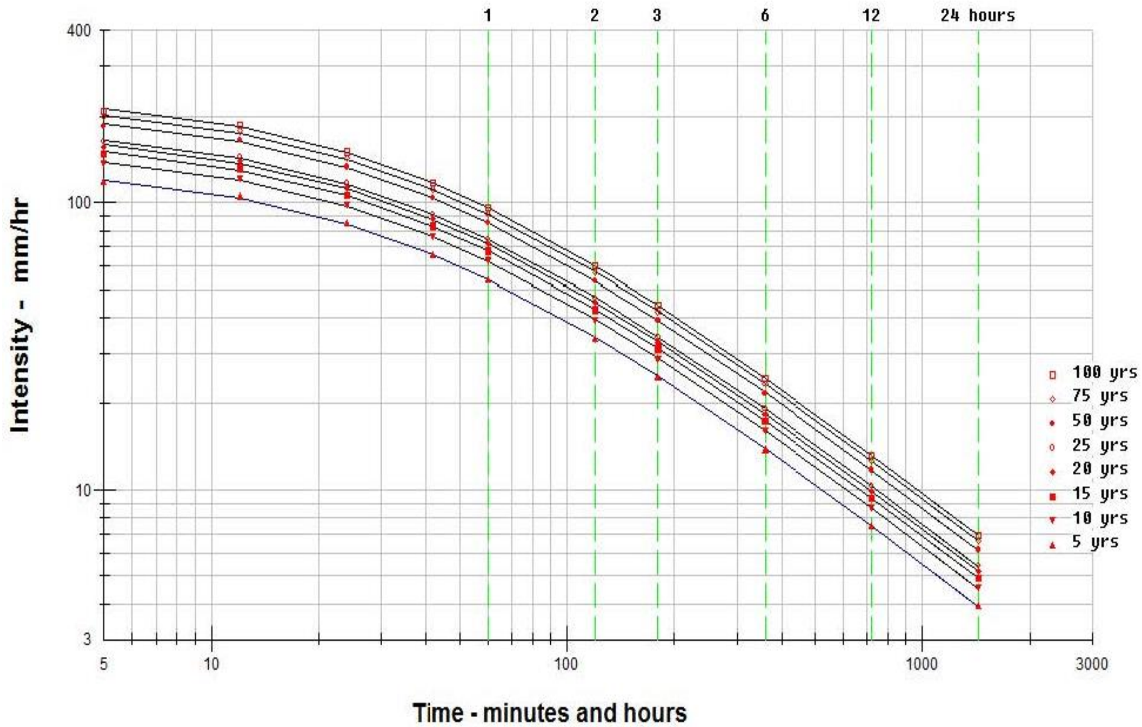
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Twifo Praso

Longitude: 1°32'42.28"W

Latitude: 5°36'25.15"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	105.35	121.77	131.83	139.65	145.23	165.34	177.63	186.57
0.4	85.04	98.29	106.41	112.72	117.23	133.46	143.38	150.59
0.7	66.10	76.41	82.71	87.62	91.13	103.74	111.45	117.06
1	54.16	62.60	67.77	71.79	74.66	84.99	91.31	95.91
2	33.98	39.28	42.52	45.04	46.84	53.33	57.29	60.18
3	24.86	28.74	31.11	32.96	34.28	39.02	41.92	44.03
6	13.89	16.06	17.39	18.42	19.15	21.81	23.43	24.61
12	7.47	8.63	9.35	9.90	10.30	11.72	12.59	13.23
24	3.93	4.54	4.92	5.21	5.42	6.17	6.63	6.96



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

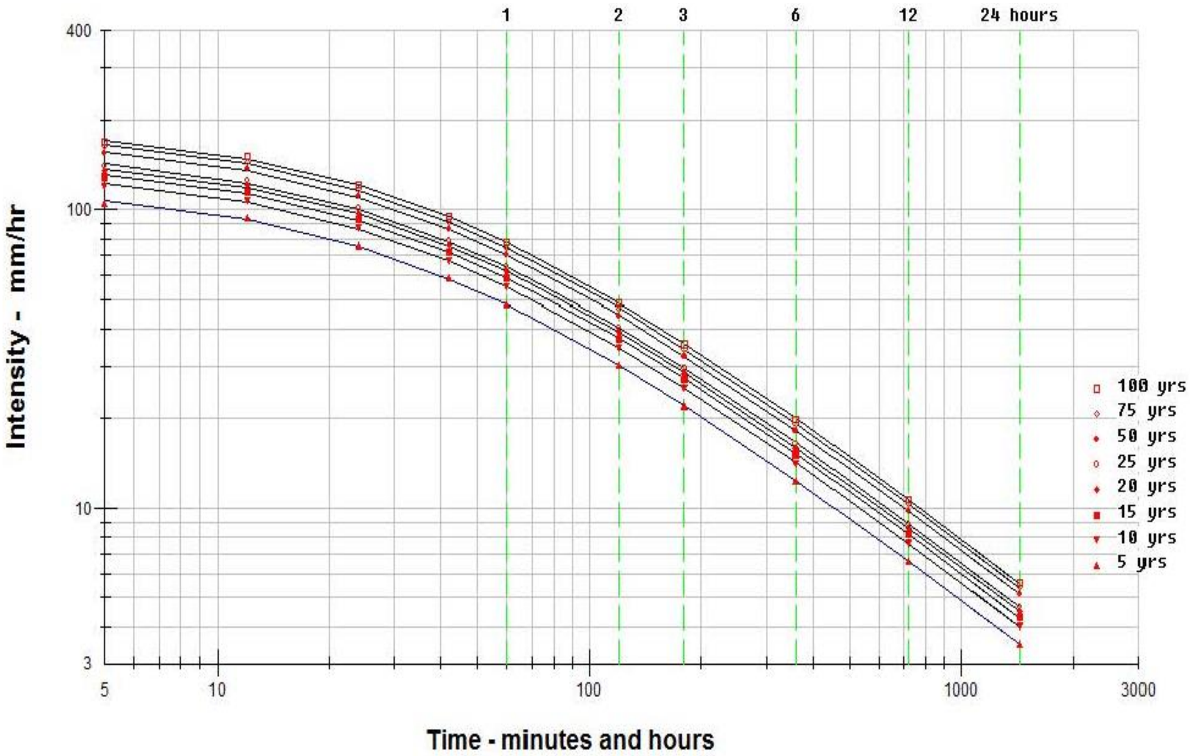
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Wa

Longitude: 2°30'35.51"W

Latitude: 10° 3'35.80"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	93.73	107.36	115.07	120.65	125.12	137.41	145.23	150.82
0.4	75.66	86.66	92.88	97.39	101.00	110.92	117.23	121.74
0.7	58.81	67.36	72.20	75.71	78.51	86.22	91.13	94.63
1	48.18	55.19	59.15	62.02	64.32	70.64	74.66	77.53
2	30.23	34.63	37.11	38.92	40.36	44.32	46.84	48.65
3	22.12	25.34	27.16	28.47	29.53	32.43	34.28	35.59
6	12.36	14.16	15.18	15.91	16.50	18.12	19.15	19.89
12	6.64	7.61	8.16	8.55	8.87	9.74	10.30	10.69
24	3.50	4.00	4.29	4.50	4.67	5.13	5.42	5.63



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

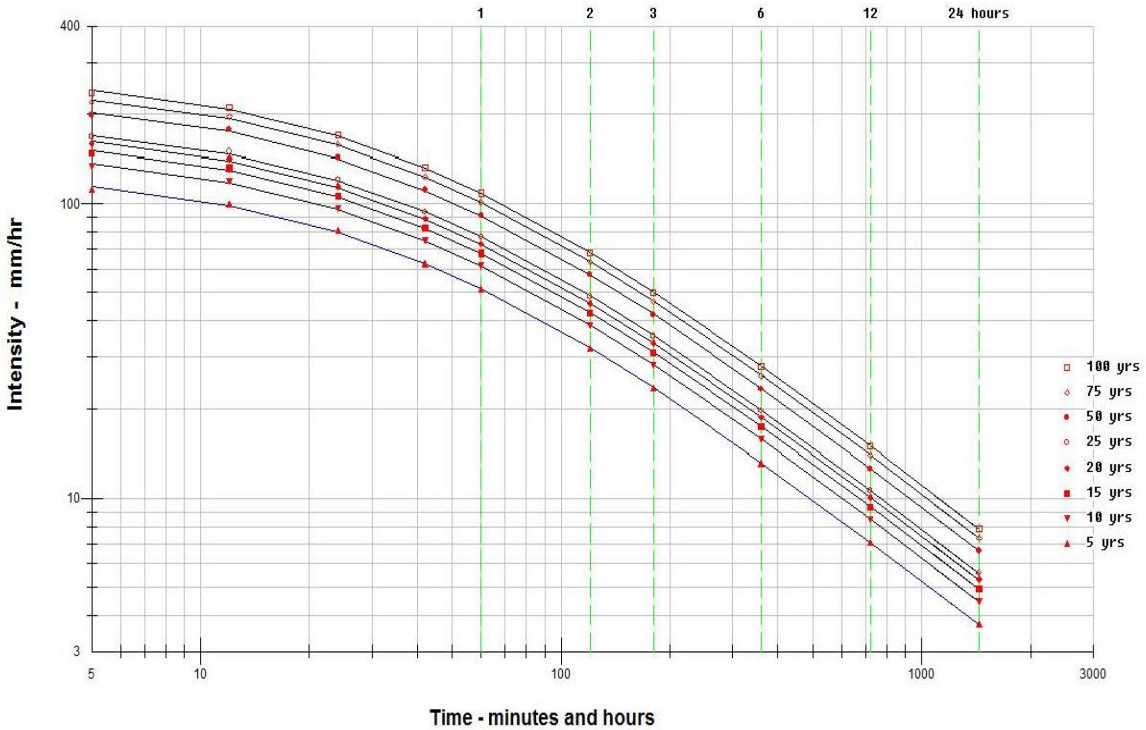
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Walewale

Longitude: 0°47'54.46"W

Latitude: 10°21'5.87"N

Duration (hrs)	Return period (yrs)							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	99.88	119.54	131.83	141.88	149.70	177.63	196.62	211.15
0.4	80.62	96.49	106.41	114.52	120.83	143.38	158.71	170.43
0.7	62.67	75.00	82.71	89.02	93.93	111.45	123.37	132.48
1	51.34	61.45	67.77	72.93	76.95	91.31	101.08	108.54
2	32.21	38.56	42.52	45.76	48.29	57.29	63.42	68.10
3	23.57	28.21	31.11	33.48	35.33	41.92	46.40	49.83
6	13.17	15.77	17.39	18.71	19.74	23.43	25.93	27.85
12	7.08	8.47	9.35	10.06	10.61	12.59	13.94	14.97
24	3.73	4.46	4.92	5.29	5.58	6.63	7.33	7.88



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

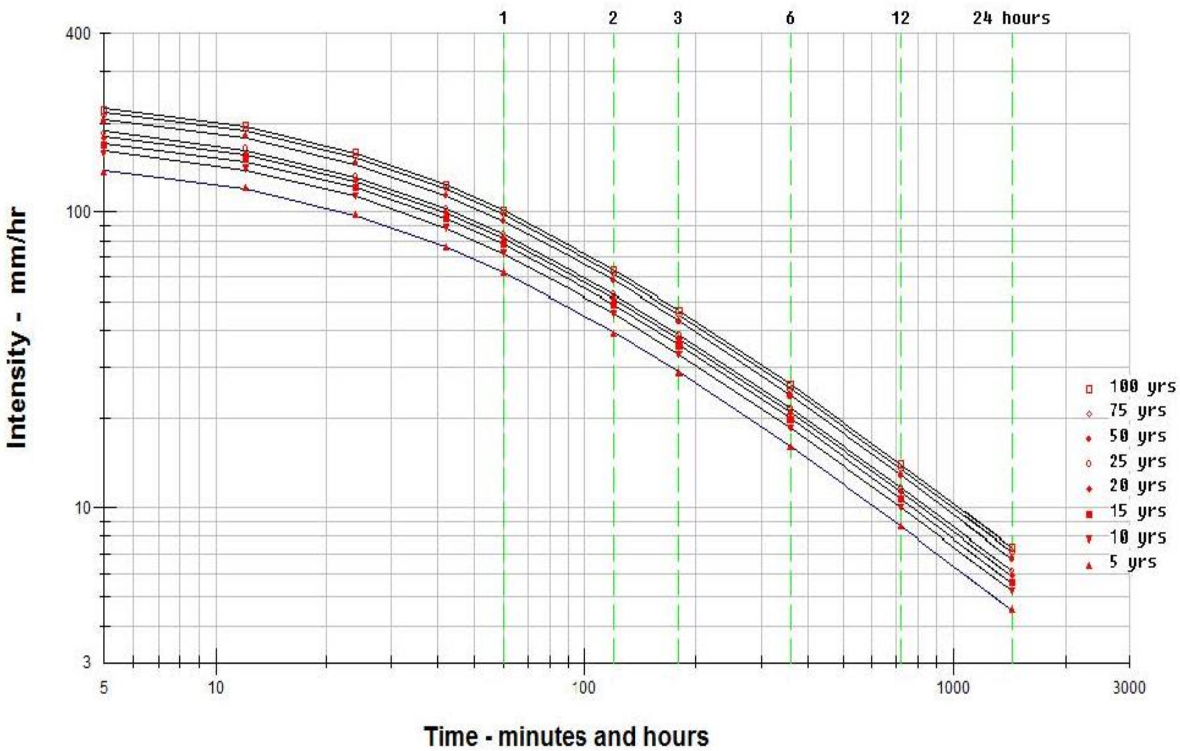
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Wassa Akropong

Longitude: 2° 5'18.02"W

Latitude: 5°46'59.12"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	142.33	164.53	176.28	185.42	191.95	211.54	223.29	231.13
0.4	121.77	140.76	150.82	158.64	164.22	180.98	191.04	197.74
0.7	98.29	113.62	121.74	128.05	132.56	146.08	154.20	159.61
1	76.41	88.32	94.63	99.54	103.04	113.56	119.87	124.07
2	62.60	72.36	77.53	81.55	84.42	93.03	98.20	101.65
3	39.28	45.40	48.65	51.17	52.97	58.38	61.62	63.78
6	28.74	33.22	35.59	37.44	38.76	42.71	45.09	46.67
12	16.06	18.56	19.89	20.92	21.66	23.87	25.20	26.08
24	8.63	9.98	10.69	11.25	11.64	12.83	13.54	14.02



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

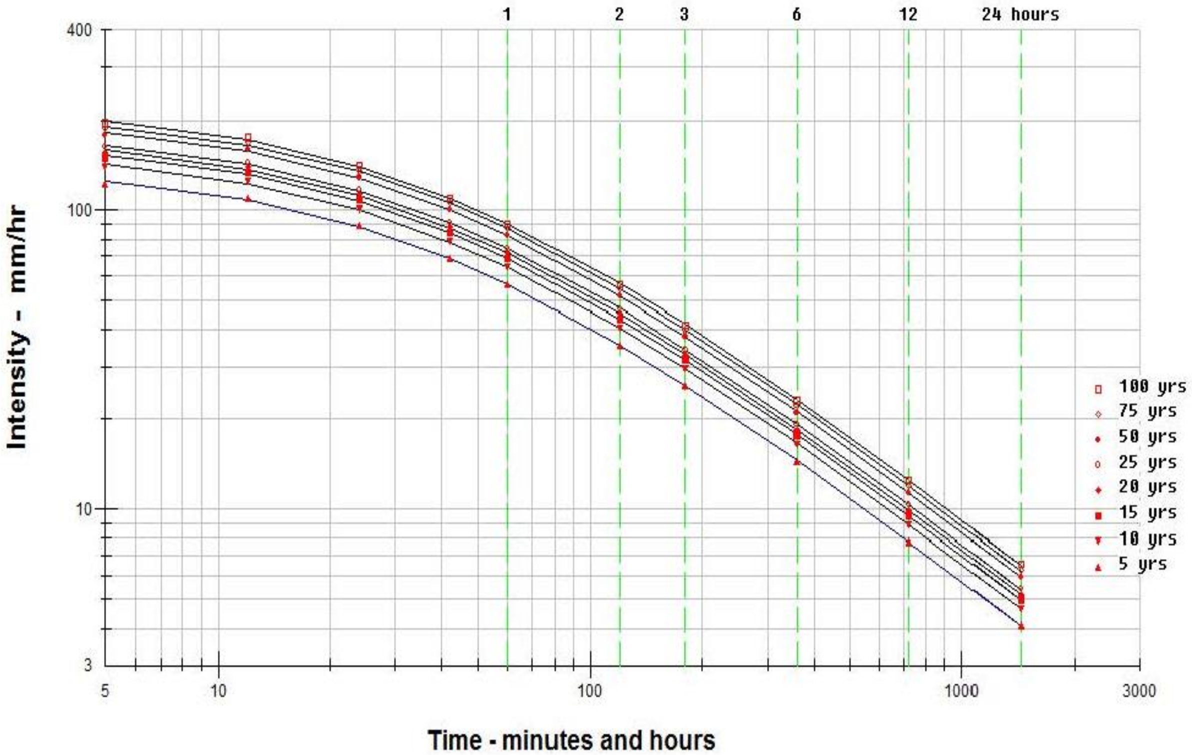
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Wenchi

Longitude: 2° 6'3.05"W

Latitude: 7°44'31.05"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	109.71	125.12	134.06	139.65	145.23	159.76	168.69	175.40
0.4	88.55	101.00	108.21	112.72	117.23	128.95	136.16	141.57
0.7	68.84	78.51	84.12	87.62	91.13	100.24	105.85	110.05
1	56.40	64.32	68.91	71.79	74.66	82.12	86.72	90.16
2	35.39	40.36	43.24	45.04	46.84	51.53	54.41	56.57
3	25.89	29.53	31.64	32.96	34.28	37.70	39.81	41.39
6	14.47	16.50	17.68	18.42	19.15	21.07	22.25	23.13
12	7.78	8.87	9.50	9.90	10.30	11.33	11.96	12.43
24	4.09	4.67	5.00	5.21	5.42	5.96	6.29	6.54



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

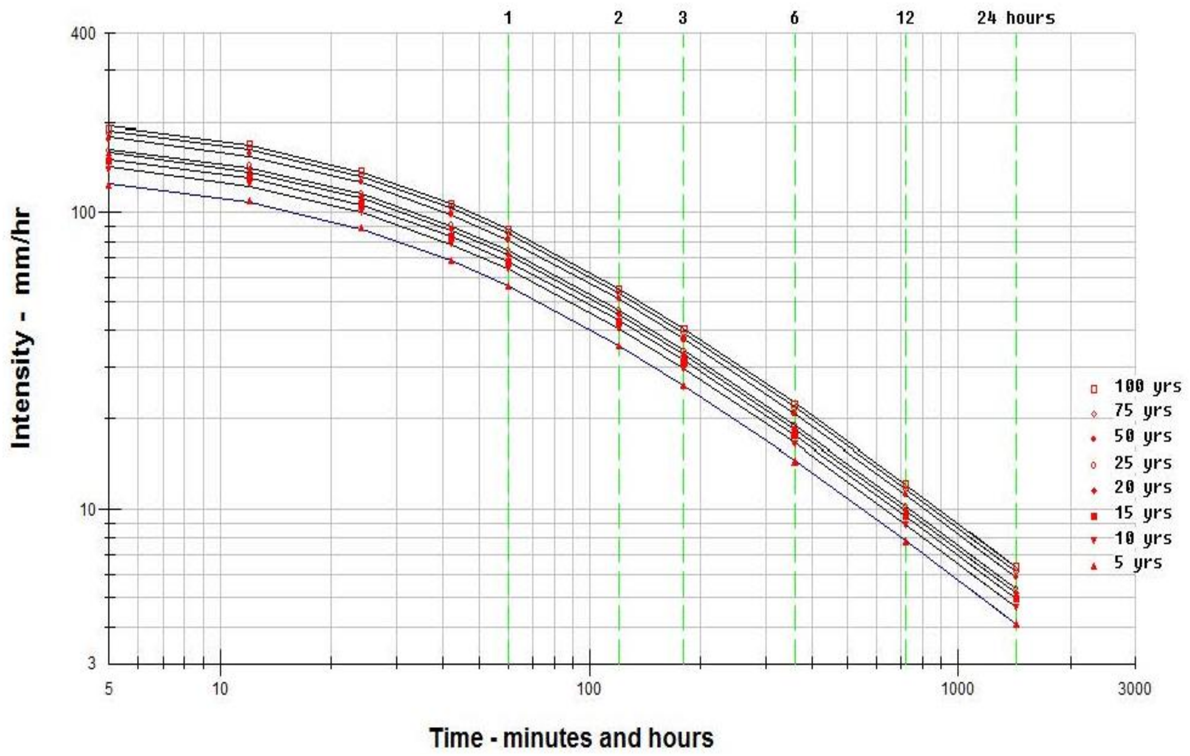
I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Yendi

Longitude: 0° 0'33.58"W

Latitude: 9°26'42.16"N

Duration (hrs)	Return period (yrs)							
	5	**10**	**15**	**20**	**25**	**50**	**75**	**100**
0.2	109.93	125.12	132.94	139.65	144.12	157.52	165.34	170.93
0.4	88.73	101.00	107.31	112.72	116.33	127.15	133.46	137.97
0.7	68.98	78.51	83.42	87.62	90.43	98.84	103.74	107.25
1	56.51	64.32	68.34	71.79	74.08	80.97	84.99	87.87
2	35.46	40.36	42.88	45.04	46.48	50.81	53.33	55.13
3	25.94	29.53	31.37	32.96	34.01	37.18	39.02	40.34
6	14.50	16.50	17.53	18.42	19.01	20.78	21.81	22.54
12	7.79	8.87	9.42	9.90	10.22	11.17	11.72	12.12
24	4.10	4.67	4.96	5.21	5.38	5.88	6.17	6.38



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

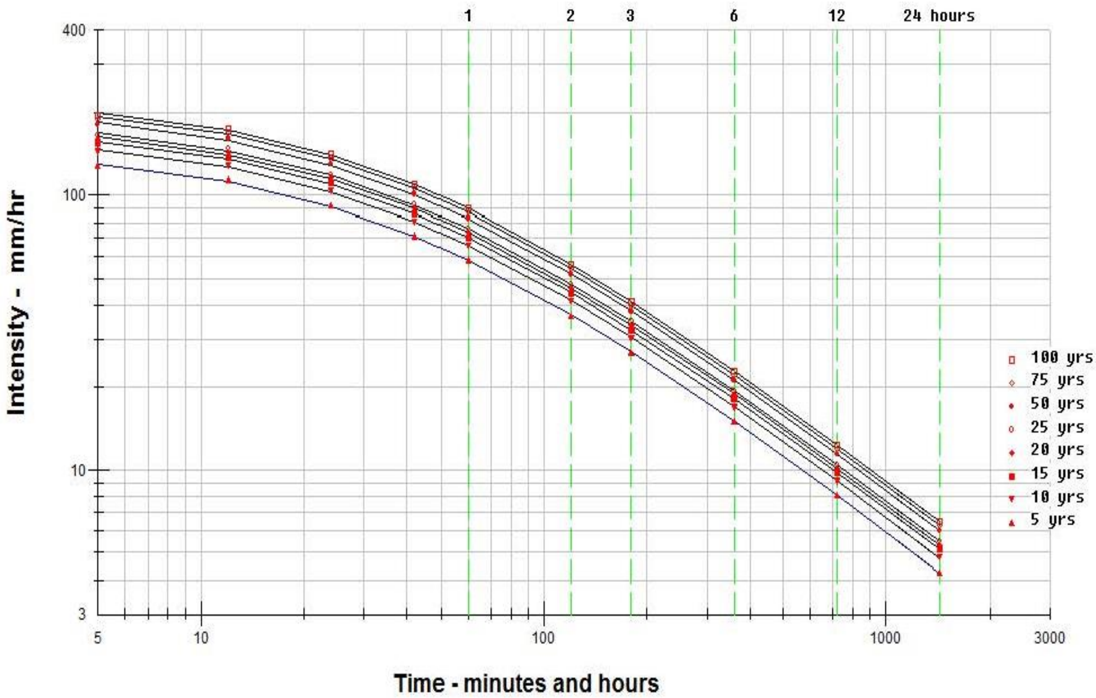
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Zabzugu

Longitude: 0°22'12.29"E

Latitude: 9°17'42.07"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	113.95	128.47	137.41	143.00	147.47	160.87	168.69	174.28
0.4	91.98	103.70	110.92	115.42	119.03	129.85	136.16	140.67
0.7	71.50	80.61	86.22	89.72	92.53	100.94	105.85	109.35
1	58.58	66.04	70.64	73.51	75.81	82.70	86.72	89.59
2	36.75	41.44	44.32	46.12	47.56	51.89	54.41	56.21
3	26.89	30.32	32.43	33.75	34.80	37.97	39.81	41.13
6	15.03	16.94	18.12	18.86	19.45	21.22	22.25	22.99
12	8.08	9.11	9.74	10.14	10.45	11.40	11.96	12.36
24	4.25	4.79	5.13	5.33	5.50	6.00	6.29	6.50



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)

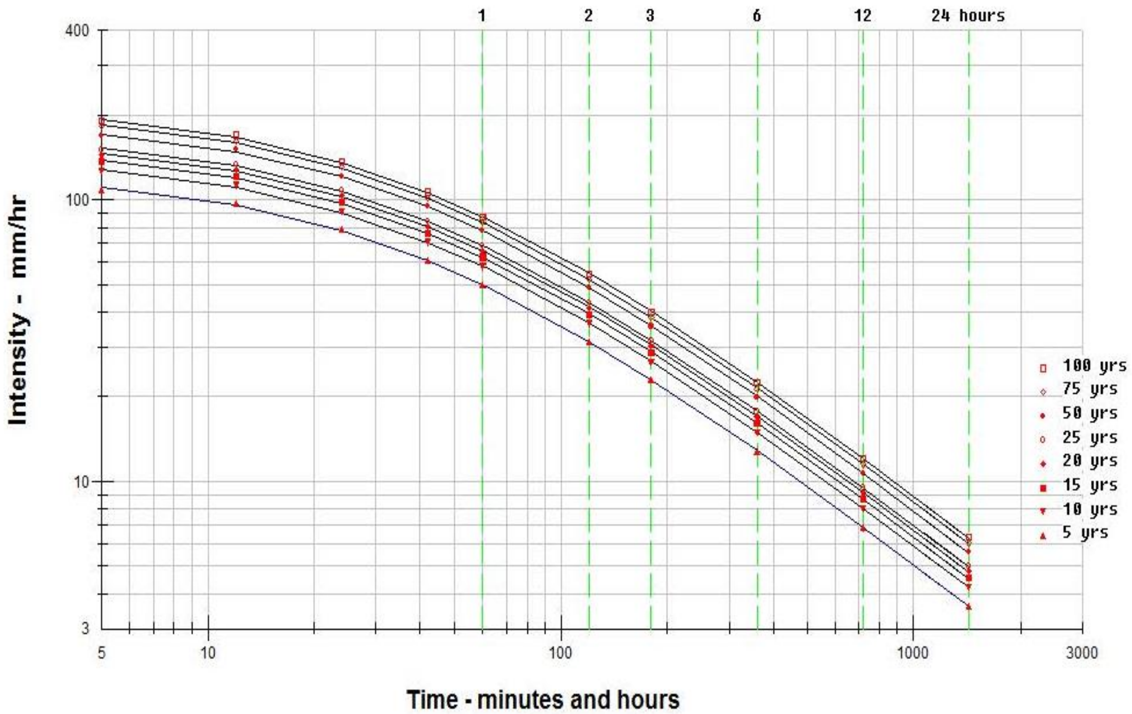
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

Weather Station: Zuarungu

Longitude: 0°48'28.72"W

Latitude: 10°47'46.09"N

Duration (hrs)	Return period (yrs)							
	5	10	15	20	25	50	75	100
0.2	96.97	112.83	121.77	128.47	134.06	150.82	161.99	169.81
0.4	78.27	91.08	98.29	103.70	108.21	121.74	130.75	137.07
0.7	60.84	70.80	76.41	80.61	84.12	94.63	101.64	106.55
1	49.85	58.00	62.60	66.04	68.91	77.53	83.27	87.29
2	31.28	36.39	39.28	41.44	43.24	48.65	52.25	54.77
3	22.89	26.63	28.74	30.32	31.64	35.59	38.23	40.08
6	12.79	14.88	16.06	16.94	17.68	19.89	21.36	22.40
12	6.87	8.00	8.63	9.11	9.50	10.69	11.48	12.04
24	3.62	4.21	4.54	4.79	5.00	5.63	6.04	6.33



$$I_{T_c,T_r} = \frac{I_{0.4,T_r}}{(T_c + 0.6)^{0.96}}$$

Where,

I_{T_c,T_r} = Intensity (mm) for time of concentration T_C (hr) at return period T_r(years)
$I_{0.4,T_r}$ = Intensity (mm) for time of concentration 0.4hr at return period T_r(years)

APPENDIX D TOLL GATES

A. Sample calculation for the determination of number of toll lanes

The number of toll gate lanes is calculated as described in **Section 11.7**. Sample calculation is discussed in this appendix.

1) Condition

The number of required lanes at toll gate can be computed considering:

- traffic volume
- required service time for toll collection and
- level of service (waiting time of the drivers).

If the traffic volume is large, required service time is long, more number of gates are required. In other words, more number of gates are required to improve the level of service (reducing waiting time of driver).
Queueing theory is used for estimation of the required number of lanes using the following parameters;

i. a: average vehicle entering time (s)
ii. b: average service time (s)
iii. s: number of lane (equals to gate number)
iv. Traffic intensity(ρ) is implied as ρ=b/a
v. Traffic intensity per gate is implied as u=b/sa
vi. Average waiting number of vehicles per lane q/s

The relationship between number of Toll lane(s), Average waiting vehicle number (q), and Traffic intensity (u) is computed as shown in **Table 2** and **Figure 1**.

Table 1 is computed under the following conditions for the design of number of gate lane (s).
Average service time (b): 6, 8, 10, 14, 18, 20 seconds
Average waiting number of vehicle per lane (q/s) : 1.0 and 3.0
Number of lane (s): 1 to 15

2) Traffic volume

Design traffic volume for toll gate lane calculation refers to the design hourly traffic volume. The design hourly traffic volume is obtained by multiplying the annual average daily traffic volume (AADT) by the K value and the D value. In calculation of gate numbers, careful attention requires for estimation of the design hourly traffic volume in order to avoid over-estimation.

3) Service time

The service time differs according to the method of toll collection and vehicle type, however, it is 8 to14 seconds in average for manual payment from study taken in the expressway in Japan. Based on study of the practices in other countries, the service time is 6 seconds at the entrance (slotting a magnetic card) and 14 seconds at the exit (toll payment by cash). In the case of a uniformed toll system , 8 seconds (toll payment by cash) is assumed.

4) Level of Service

The level of service is based on the average number of waiting vehicles per gate (q/s). The average waiting time is the value obtained by multiplying the average waiting number of vehicles by the average service time. If the average number of waiting vehicles per gate (q/s) is large, this imply that a long queue of waiting vehicle likely to occur at increases of traffic volume. Calculation is based on the assumption that vehicles are equally dispersed to all gates, however, in reality, vehicles may concentrate at gates near median even in heavily congested traffic. (thus gates at road edge side are often relatively vacant.) Therefore, the number of waiting vehicles near median becomes larger than the theoretical value. This is the reason why it is deemed appropriate to set 1.0 (q/s) as the design standard value. However, 3.0 may be adopted if terrain or other reasons that traffic is not hindered.

5) Calculation of the required number of lanes

The traffic intensity (ρ) is determined from the design traffic volume (DHV) and the service time (b) as follows:

$$\rho=\frac{b}{a}=\frac{DHV}{3600b} \qquad (B.1)$$

Since the single lane traffic intensity (u) is u=ρ/s, the designer shall look for the number of lanes (s) which ensures that the value (u) does not exceed the values shown in **Table 2**. **Table 2** shows the number of toll barrier lanes (s), average number of waiting vehicles (q/s), and traffic intensity per lane (u). When the traffic volume per lane (u), the service time (b), the number of lanes (s), and the level of service (q/s) are settled based on this calculation, the hourly number of vehicles that the toll barrier can process in an hour can be estimated by the **Equation (B.2)**:

$$\frac{3,600}{b}u\cdot s \qquad (B.2)$$

Table 1 Number of toll lane (s), Average number of waiting vehicles (q), and Traffic intensity (u)

s \ (q/s)	0.5	1.0	1.5	2.0	3.0	4.0	5.0	10.0
1	0.333	0.500	0.600	0.667	0.750	0.800	0.833	0.909
2	0.577	0.706	0.775	0.817	0.863	0.895	0.913	0.953
3	0.686	0.791	0.841	0.872	0.908	0.928	0.940	0.969
4	0.748	0.835	0.876	0.902	0.929	0.945	0.955	0.976
5	0.787	0.863	0.899	0.919	0.942	0.955	0.963	0.981
6	0.817	0.883	0.914	0.932	0.952	0.962	0.969	0.984
7	0.838	0.898	0.925	0.940	0.958	0.968	0.974	0.986
8	0.854	0.909	0.933	0.948	0.964	0.972	0.977	0.988
9	0.868	0.919	0.941	0.953	0.967	0.975	0.980	0.989
10	0.878	0.926	0.946	0.957	0.970	0.977	0.982	0.990
11	0.888	0.932	0.950	0.961	0.973	0.979	0.983	0.991
12	0.896	0.936	0.954	0.964	0.975	0.981	0.984	0.992
13	0.903	0.941	0.958	0.967	0.977	0.982	0.986	0.992
14	0.908	0.945	0.961	0.969	0.979	0.983	0.987	0.993
15	0.913	0.948	0.952	0.971	0.980	0.984	0.988	0.993
16	0.918	0.951	0.965	0.973	0.981	0.985	0.989	0.994
17	0.923	0.954	0.967	0.975	0.982	0.986	0.989	0.994
18	0.926	0.956	0.969	0.976	0.983	0.987	0.990	0.994
19	0.929	0.958	0.970	0.977	0.984	0.988	0.990	0.995
20	0.932	0.960	0.972	0.978	0.985	0.988	0.991	0.995

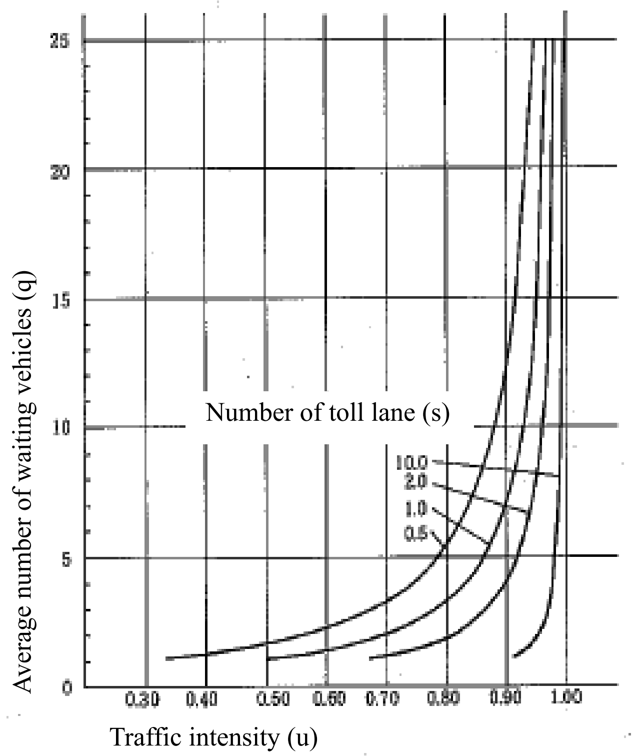


Figure 1 Number of toll lane (s), Average number of waiting vehicles (q), and Traffic intensity (u)

Sample calculation:

- AADT =17,500 vehicles/day (approximate value)
- K value (ratio of the 30^{th} hour traffic volume in respect to AADT) =0.09
- D value (ratio of traffic volume in the direction with the most traffic in the 30^{th} hour in respect to total traffic volume in both directions) =0.6
- DHV =AADT × K × D = 17,500 × 0.09 × 0.6 = 945 (vehicles/hour)
- Service time b = 8 seconds
- Average number of waiting vehicles (the level of service): 1 vehicle

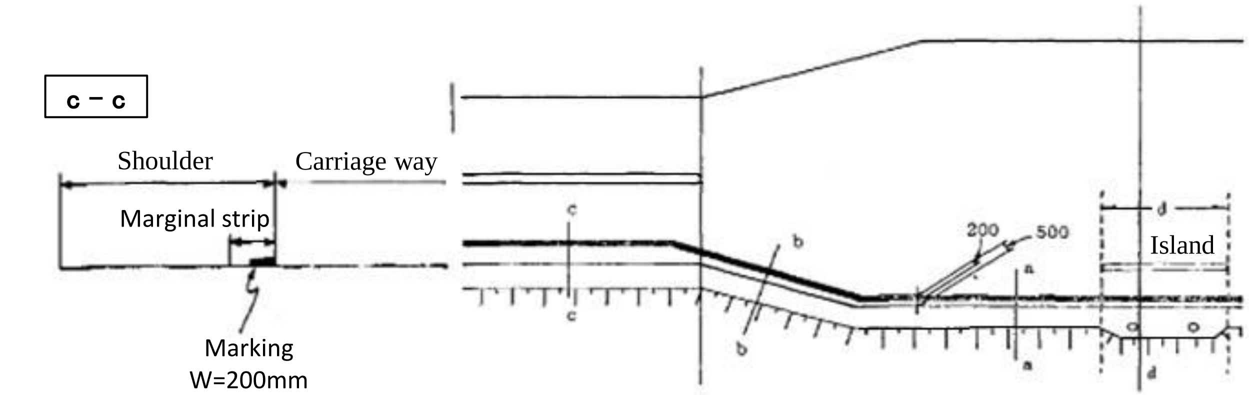
From **Table 1**, 3 lanes required to process the given DHV (945<1,070 vehicles).

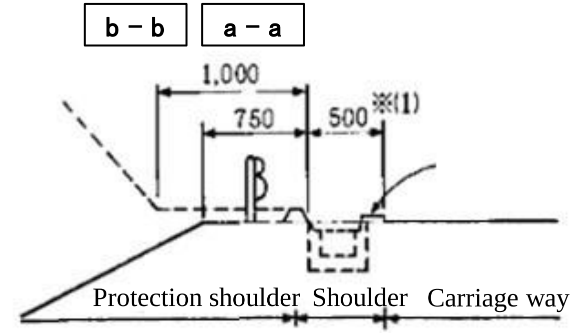
Table 2 Relationship between the Number of Lanes, Service Time and Average Number of Waiting Vehicles and Number of Vehicles that Can be Processed (vehicles/hour)

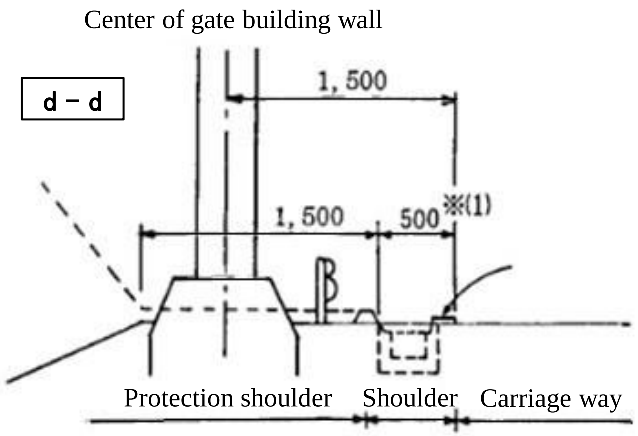
Service time (s)		6		8		10		14		18		20	
Average number of waiting vehicles (q/s)		1.0	3.0	1.0	3.0	1.0	3.0	1.0	3.0	1.0	3.0	1.0	3.0
Number of lanes (S)	1	300	450	230	340	180	270	130	190	100	150	90	140
	2	850	1040	640	780	510	620	360	440	280	350	250	310
	3	1420	1630	**1070**	1230	850	980	610	700	480	550	430	490
	4	2000	2230	1500	1670	1200	1340	860	960	670	740	600	670
	5	2590	2830	1940	2120	1550	1700	1110	1210	860	940	780	850
	6	3180	3430	2380	2570	1910	2060	1360	1470	1060	1140	950	1030
	7	3770	4020	2830	3020	2260	2410	1620	1720	1260	1340	1130	1210
	8	4360	4630	3270	3470	2620	2780	1870	1980	1450	1540	1310	1390
	9	4960	5220	3720	3920	2980	3130	2130	2240	1650	1740	1490	1570
	10	5560	5820	4170	4370	3330	3490	2380	2490	1850	1940	1670	1750
	11	6150	6420	4610	4820	3690	3850	2640	2750	2050	2140	1850	1930
	12	6740	7020	5050	5270	4040	4210	2890	3010	2250	2340	2020	2110
	13	7340	7620	5510	5720	4400	4570	3150	3270	2450	2540	2200	2290
	14	7940	8220	5954	6170	4760	4930	3400	3520	2650	2740	2380	2470
	15	8530	8820	6400	6620	5120	5290	3660	3780	2840	2940	2560	2650

B. B. Sample Plans of Toll Barrier Facilities (Chapter 11)

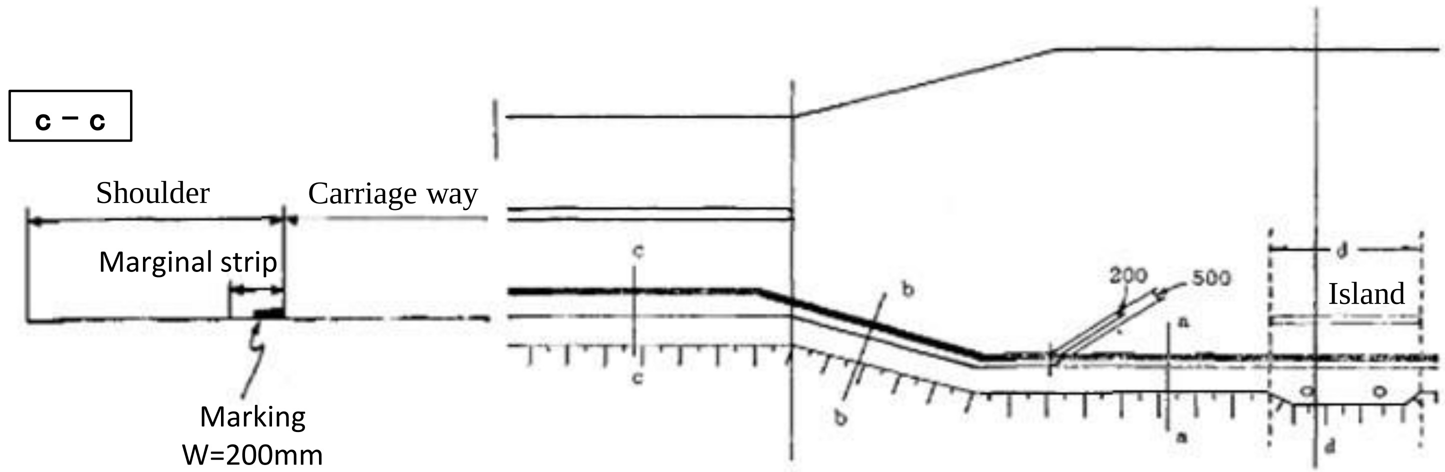
Markings around the Toll Gate (Standard Example)

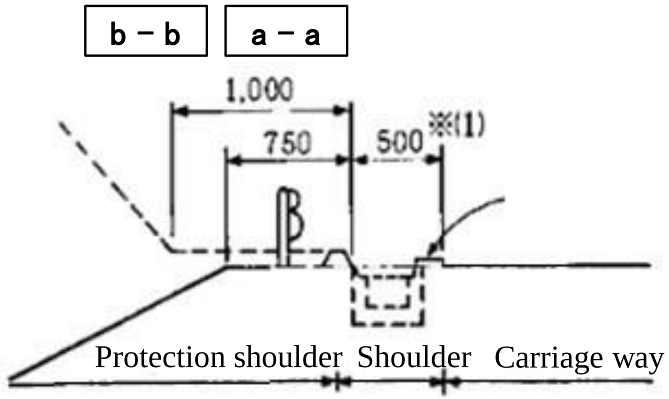




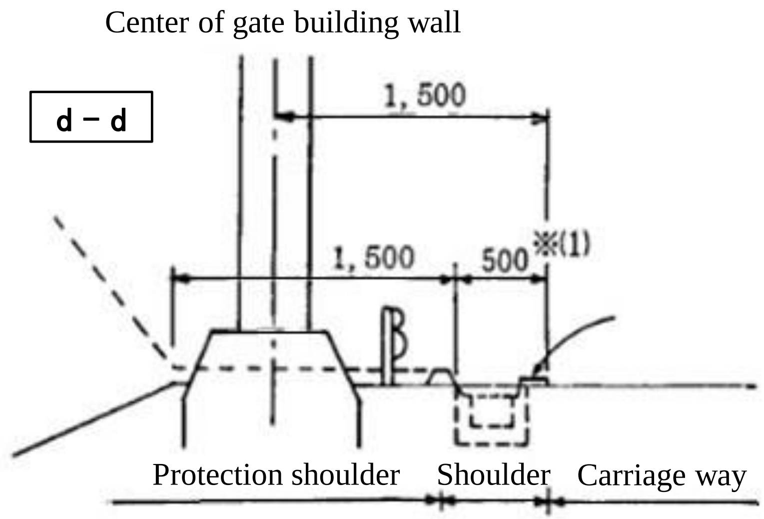


Marking width
(1) In case to set drainage
However, footing must be considered for the case of concrete walls for cut slope



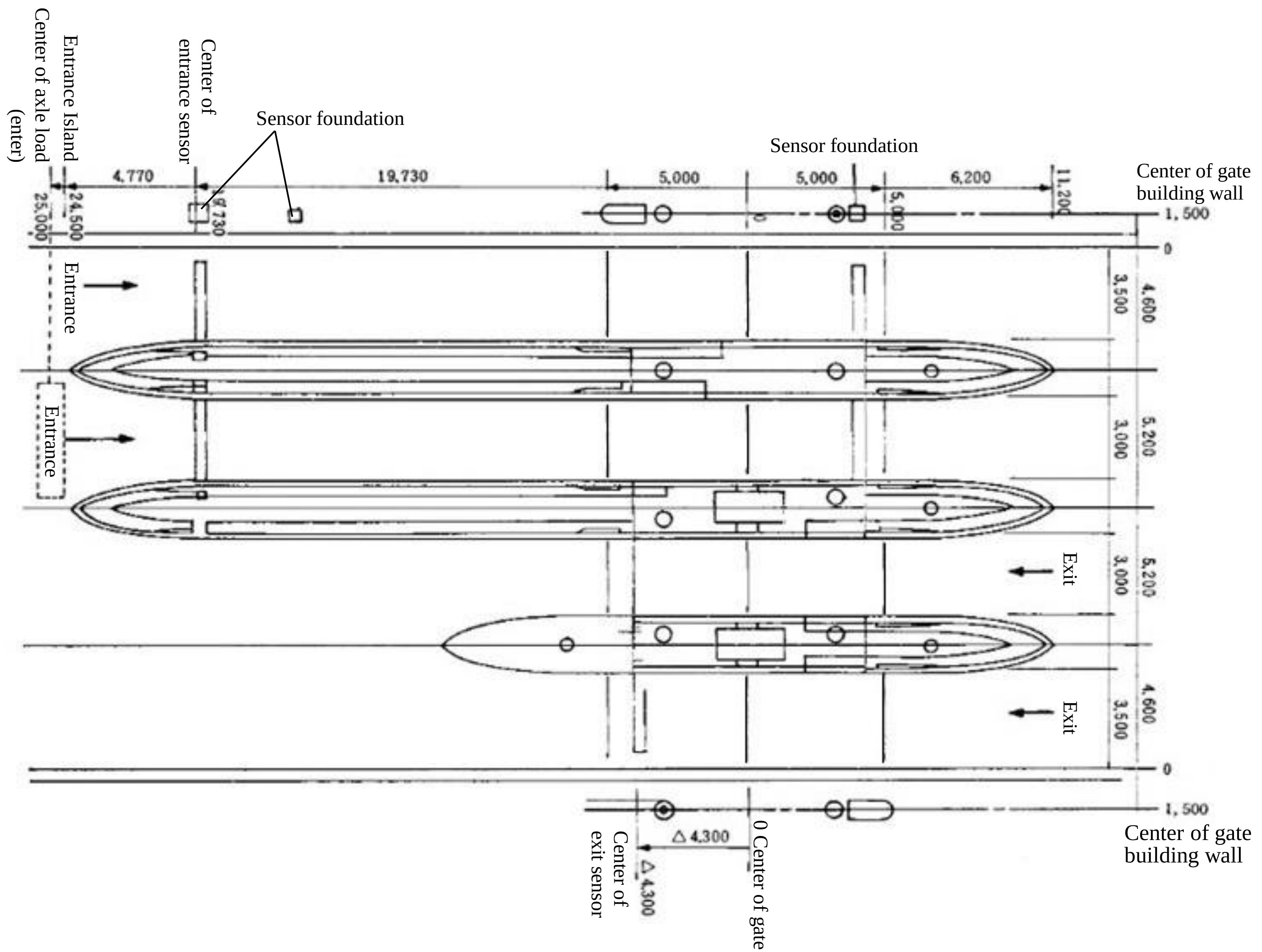


Marking width
※ (1) Case where drainage facilities are installed

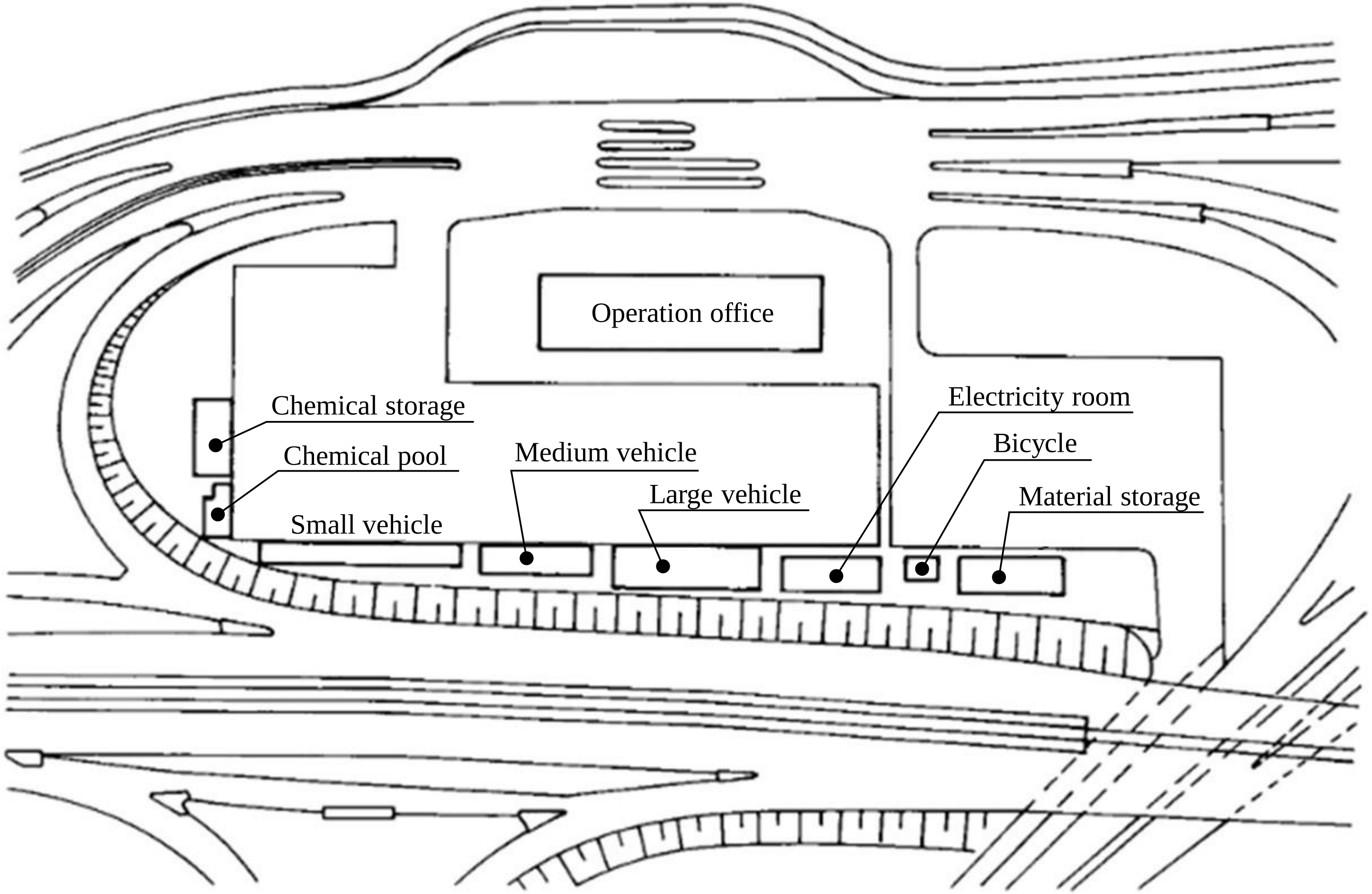


※ (1) Case where drainage facilities are installed
However, if slopes are a block masonry structure, etc., consider pier foundations.

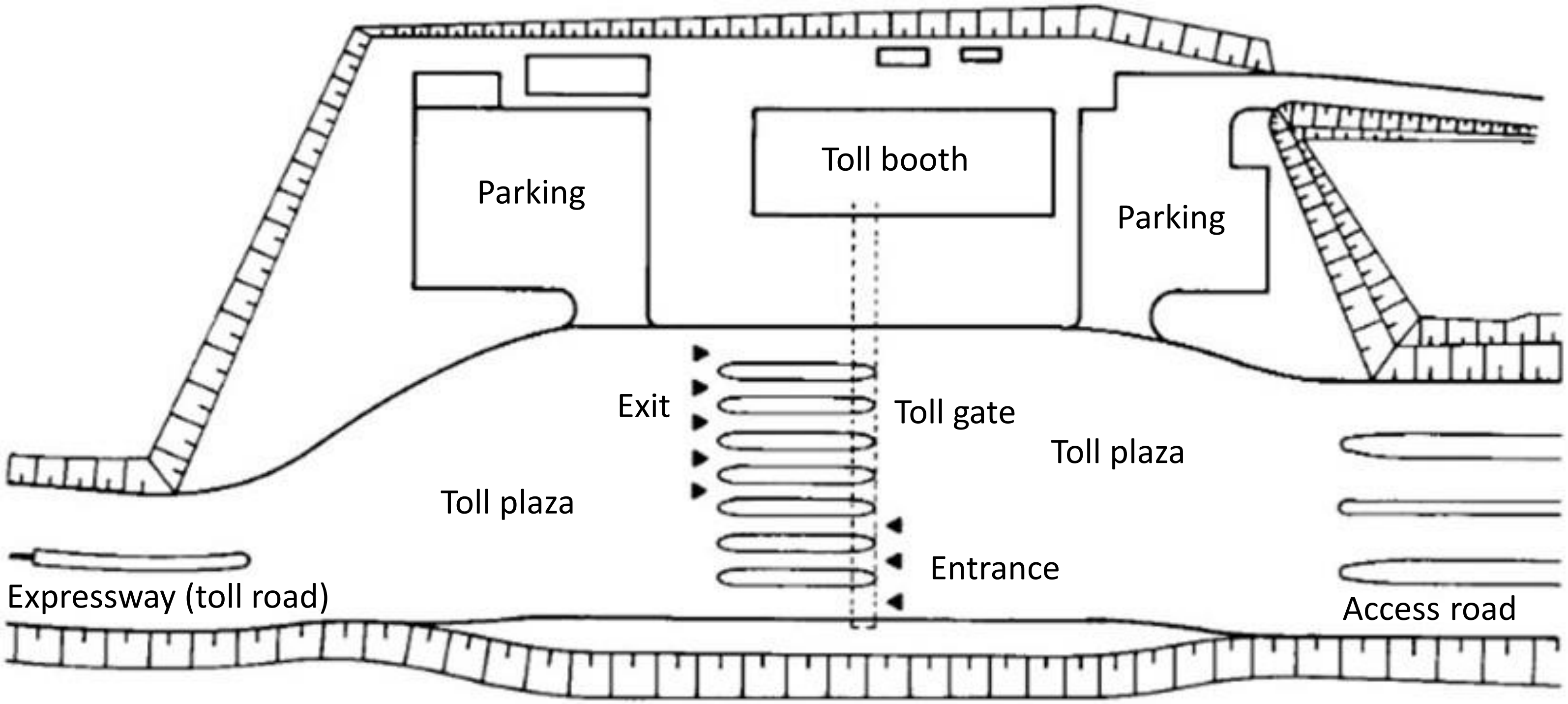
Plane View of Toll Island (Standard Example)



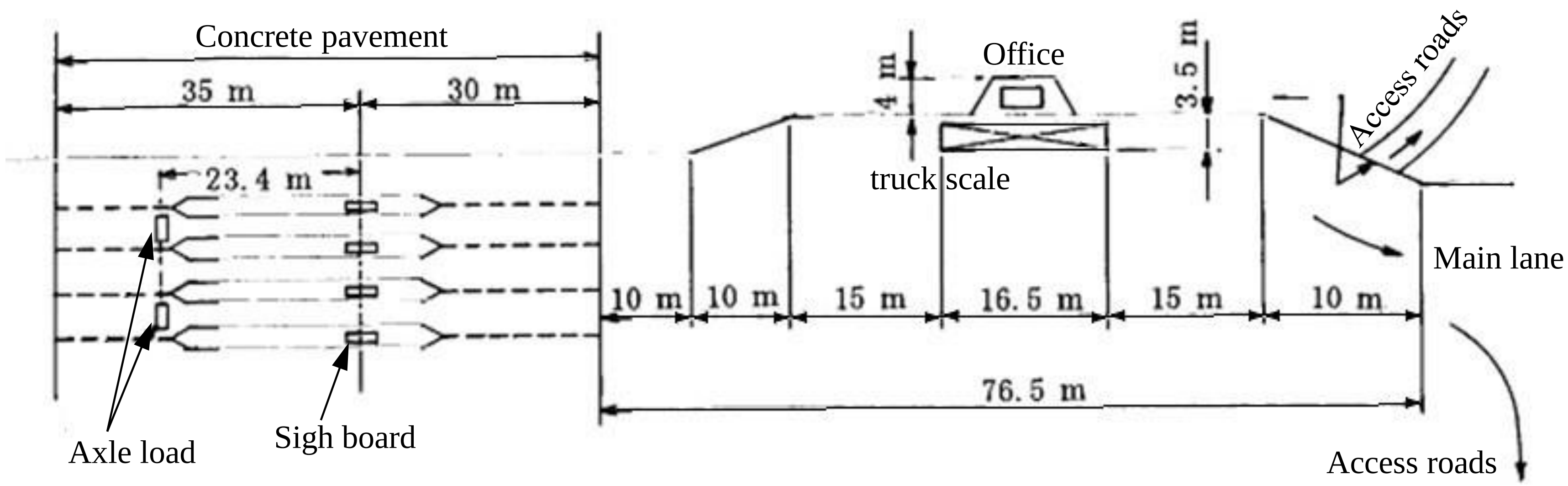
Example of Actual Toll Barrier 1 (Japan)



Example of Actual Toll Barrier 2 (Japan)



C. Sample Plans of Weight Bridge and other Control Equipment (Chapter 11)


Concrete pavement
35 m
30 m
23.4 m
Axle load
Sigh board
4 m
Office
truck scale
3.5 m
Access roads
Main lane
10 m
10 m
15 m
16.5 m
15 m
10 m
76.5 m
Access roads

APPENDIX E PRESENTATION OF DRAWINGS AND DESIGN REPORTS

A. Presentation of Drawings

It is necessary to set standards in the presentation of working drawings. This uniformity ensures proper management of design drawings and documentation for easy reference.

The aim of this appendix is to provide designers with a brief guidance on the contents and presentation of design drawings. The designer shall prepare drawings which are clear, accurate and with enough details to meet the intended purposes. Depending on their purpose the drawings can be classified as Preliminary Design Drawings for feasibility studies, Detailed Engineering Drawings for tendering and construction purposes and As-built Drawings for archive purposes. This appendix provides guidance on standards of preparing drawings for different purposes. The designer should adhere to the requirements of this guide, and the Terms of Reference (ToR) for the project.

Intermediate drawings submitted as Preliminary Drawings and Draft Final Drawings in the Detailed Engineering Design Projects shall be stamped to indicate that they are not yet ready for implementation.

For easy interpretation of working drawings, the size of the drawing sheet and the scale of the different drawings have been specified in this appendix under the various headings.

i. **Size of drawing sheet and margin**

The size of drawing paper is A1, miniature copy is A3 (297mm x 420mm). Refer to **Figure E.1**.

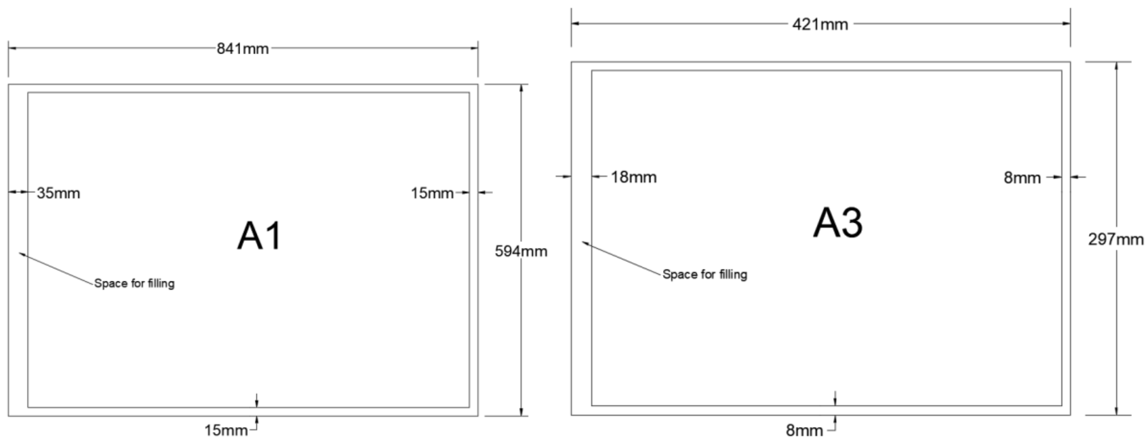


Figure E.1 Size and margins of A1 and A3 paper respectively

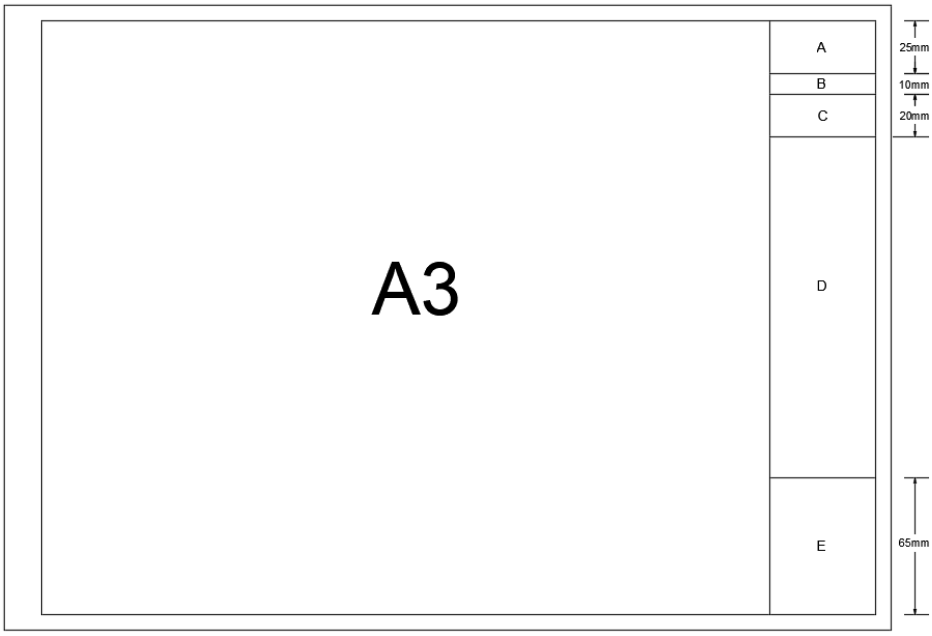
ii. **Title Column**

The title column type, horizontal or vertical style must be consistent throughout the drawing set. On any particular project, horizontal or vertical style title columns may be used, but not a combination of the two (refer to **Figure E.2**). They may be modified based on need, e.g., the inclusion of coat of arms, logo, etc.

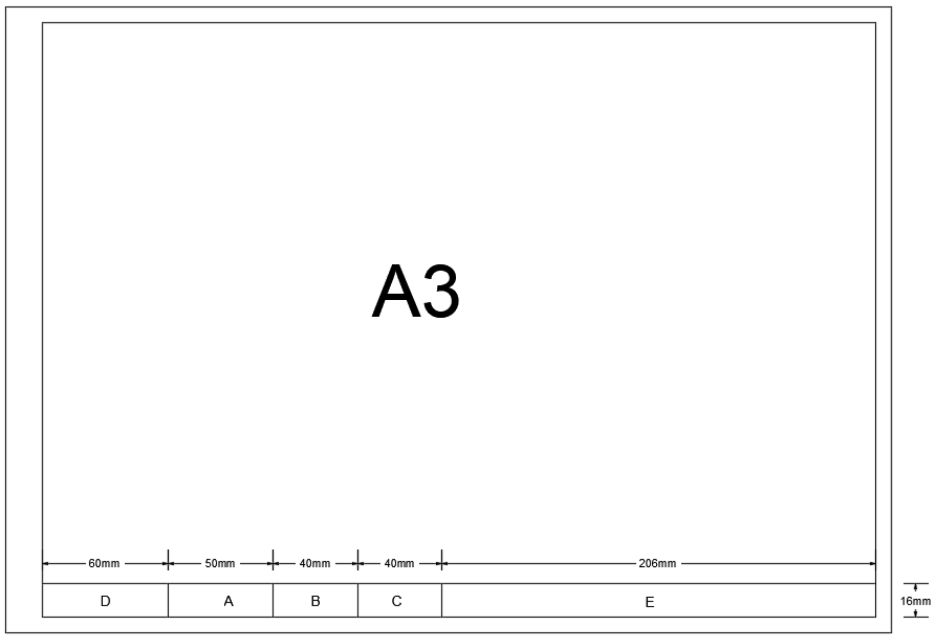
iii. Title Block.

The title block type, horizontal or vertical style must be consistent throughout the drawing set. On any particular project, horizontal or vertical style title blocks may be used, but not a combination of the two.

Vertical Title Block



Horizontal Title Block



Legend

A Government of the Republic of Ghana
  Ministry of Roads and Highways
B Name of Road Agency
C Name of Consultant
D Notes
E Title Block

Figure E.2 Types and dimensions of title column/block

All drawing title blocks shall clearly show:

- Project title
- Drawing description
- Scale (horizontal and vertical)
- Drawing number
- Sheet number
- Date
- Designed by, drawn by, drainage by, checked by, approved by, with the name and signature of the responsible engineer and the date clearly displayed.

The final drawings must be well detailed, clear, readable, concise, unambiguous and consistent to serve the intended purposes. The drawings shall include legends which shall define the lines and symbols used to represent different features to ensure uniform interpretation of the information depicted in the drawings. All the drawings such as drawings for landscaping, land acquisition, utilities etc. should be prepared in a scale sufficient to show all the important features for the intended purposes. For drawings prepared to show the location of related facilities, the plan can appear as a background and shown as faint lines while the intended information is drawn using more visible lines. This can be applicable to longitudinal drainage, utilities, road accessories, structures, traffic signs and traffic signals, landscaping, land acquisitions, etc. The following sections provide the drawing types to be included in projects drawings; however, what shall be included in projects depends on the size and complexity of the projects:

1. Cover sheet

The Document Cover Sheet provides an easily identifiable cover that helps protect the document contents. The details contained on the Document Cover Sheet should enable a reader to identify the job, without the need to open the document set (**Table E.1** and **Table E.2)**.

Cover Sheet must indicate the following:

Table E.1 Cover sheet for Road Agency

REPUBLIC OF GHANA <coat of arms> MINISTRY OF ROADS AND HIGHWAYS <NAME OF ROAD AGENCY> <PROJECT TITLE> <Year>

Table E.2 Cover sheet for Consultant

REPUBLIC OF GHANA
<coat of arms>
MINISTRY OF ROADS AND HIGHWAYS
<NAME OF ROAD AGENCY>

<PROJECT TITLE>

TENDER AND CONTRACT DOCUMENTS
VOLUME NO. X OF XX

<Head of Road Agency>		<Logo of Consultant>
<Name of Road Agency>		<Name of Consultant>
<Adress of Road Agency>	<Year>	<Adress of Consultant >

2. List of Drawings (Table of contents)

The List of Drawing is a summary index listing all relevant drawings included in a contract. The List of Drawings is used as an easy guide to reference a particular drawing of interest to a relevant sheet number.

The List of Drawings contains a listing of all drawings in sequential order of sheet number followed by drawing number and description can be divided into various drawing types.

3. Location map

The purpose of the location map is to show the site of the proposed road in relation to the surrounding area and geographical features. The location map should show an Index map of Ghana, project location together with project start and end chainages, route number, main town names etc.

The scale of the map is variable depending on the size and complexity of the project. The location map shall consist of maps/aerial photographs or ortho-photo maps.

4. Key plan

The key plan shows the stations, elevations of the survey reference pillars and the coordinates of the control points of the horizontal alignment including the survey reference pillars.

5. Typical cross sections.

The Typical Cross Section drawing illustrates the structural elements of the roadway, lateral distance (dimensions), crossfalls, batter slopes and subsurface drains. It provides the pavement structural thickness and materials, and include the location of kerb and channel, subsurface and surface drainage. It should clearly show how runoff from the road and adjoining ground is collected.

The typical cross section drawings are not usually drawn to any nominal scale but should be visually proportional with the drawing scale being specified as ‘Not to Scale’. Where there is the need to provide more than one typical cross section a constant nominal scale should be adopted for visual consistency between drawings.

Typical Cross Sections should be provided at locations where the road formation or cross sectional elements (dimensions and type) is consistent and applies over a reasonable length, e.g., town, rural, superelevated sections, etc. Specific Typical Sections whose application is restricted to a limited and specific area may be shown when the section is relevant.

Items shown include the existing ground surface and the finished surface in both cut and fill sections together with the composition of the cross sections, cross slopes, pavement structure, drainage facilities, road safety barriers (where applicable), road reserve boundary (where applicable) etc.

6. Plan and profiles (longitudinal) sections.

The plans and profiles constitute the basis for virtually all other drawings related to the project. The drawings consist of geometrical plans and vertical profiles.

Geometric plans are used to establish a baseline (datum) for the location and setting out of construction works. They are also used to establish the relationship between the design line and other design lines and/or traverse lines.

The longitudinal sections are used to obtain the vertical geometry of the roadway. Longitudinal details combined with the cross fall information are used by surveyors and contractors in various programs to obtain cut and fill values for both earthworks and pavement construction.

Labelling of the chainages of the plans and profiles shall be done at the end of every 25m and at the geometric points (e.g., start and end of curve) or any interval indicated in the Terms of reference. The drawings should have tables summarising all the important parameters required for setting out of curves including chainages and coordinates of beginning and end of circular curves and spirals, points of intersection for tangents, deflection angles for tangents, radii of the curves. Grids with Easting and Northing coordinates at convenient interval (normally 100m) should be provided as backgrounds to the plans to facilitate easy interpretation of the drawings. The orientation for each of the plan drawings should be in such a way that the chainages start from the left towards the right direction irrespective of the orientation of the North direction.

The plan should show the following:

- Existing survey (existing features such as road, buildings, drainage structures, major stream/river crossings, utilities, etc), topography or mapping background information
- Aerial photography/imagery (if available)
- Road reservation boundaries
- Existing contours
- Road reference pillars
- Labels for side roads, accesses, laybys etc
- Symbols and labels (existing size and proposed intervention) of cross drainage structures
- Design speed

- North arrow
- Proposed road centreline, carriageway, shoulder, walkway etc
- Carriageway and shoulder/walkway widths (where applicable)
- Road safety barriers (where applicable)

Profile drawings should provide existing ground level and finished road levels at intervals of 25m, or any interval indicated in the Terms of Reference. Important profile parameters such as the beginnings and ends of vertical curves, gradients and K-values (or radius) should be indicated in the drawings. Values of maximum superelevation and the superelevation diagram for each curve should be indicated in the profiles. Symbols and labels (existing size and proposed intervention) of cross drainage structures should be clearly shown on the profile.

Separate plan and profiles shall be produced for side roads, cross roads, service roads and interchange ramps/slips.

Designers shall use the standard symbols and parameters indicated in this guide to ensure uniform and standard interpretation of the drawings.

Chainages shall be presented using kilometres with a + sign in between to separate the kilometre and the metres. For example, 5+250 means chainage at 5 kilometres and 250metres from the start point.

The recommended scales are:

- Urban roads 1:500H/1:50V, 1:1000H/1:100V, 1:500H/1:100V, 1:1000H/1:200V
- Rural roads 1:2000H/1:200V, 1:2500H/1:250V, 1:2000H/1:400V, 1:2500H/1:500V

7. Cross Sections

Cross sections are sections at right angles to the centreline usually at every station along the road. Cross sections drawings are useful for the purpose of computation of volumes and for the purpose of setting out of the roads. For detailed design, the designer will be required to provide cross sections at an interval of 25m unless otherwise specified in the Terms of Reference. Cross sections shall also be provided at locations of culverts for the purpose of providing the invert levels. For preliminary design purposes cross sections may not be needed in some cases, however if there is a need the intervals shall be as stipulated in the terms of reference. Cross section drawings shall indicate all the necessary information for the road embankments and cuttings and drainage structures. The information to be provided includes:

- Proposed and existing sections
- Proposed offset and levels for road centreline, edge of carriageway, shoulder, inverts for ditches and culvers and edges and toes for embankments and cuttings. The existing levels for these offsets should also be provided
- String labels
- Existing road formation
- Reference line chainage

The recommended scales are 1:100, 1:200 or 1:400 Horizontal and Vertical with no vertical exaggeration.

8. Intersection Layout

The location of intersections, rest areas, lay byes, etc. shall be shown on the plan and profile drawings. However, the details of these facilities shall be shown on separate drawings, namely the intersection layout drawings. They will include details of different intersections found in a particular project and typical standardised solutions to accesses, laybys, etc.

They are to include:

- Existing survey, topography or mapping background information
- Aerial photography/imagery (if available)
- Proposed design
- Reference line and chainage
- Proposed land requirements
- Existing and proposed utilities
- Pavement, roadside and cross drainage structures
- Road safety barriers
- Typical dimensions – carriageway, lane and shoulder widths
- Design vehicle turn paths and reference to design vehicle used

Scale – 1:250 or 1:500

9. Drainage layout and profile

To include:

- Existing survey, topography or mapping background information
- Aerial photography / imagery (if available)
- Proposed design
- Reference line and chainage
- ROW boundaries
- Proposed land requirements
- Existing and proposed drainage networks (where applicable)
- Existing and proposed drainage sumps and basins
- Existing and proposed pavement, roadside and cross drainage structures
- Existing and proposed utilities
- Critical infrastructure (laybys, traffic islands, pedestrian crossings, driveways etc).
- Existing contours
- Catchment areas
- Mainstream channel (where applicable)
- Overland flow arrows (where applicable)
- Flow paths in all proposed drainage structures
- North arrow
- Road names
- Drainage longitudinal profile

Scale – 1:250, 1:500, 1:1000, 1:2000. A scale of 1:1000H/1:200V shall be adopted for the drainage longitudinal profile.

10. Drainage Schedule

It includes:

- Hydrological and hydraulic analyses of all existing and proposed drainage structures.
- Reference chainages (start and end in the case of longitudinal drains)
- Length of existing and proposed drains
- Size of existing and proposed drains
- Type of existing and proposed structure
- Existing condition and proposed intervention
- Inlet and outlet protection measures (for cross drainage structures)

11. Road furniture and accessories

These drawings shall include but not be limited to necessary information for the construction of road furniture and accessories, including road marking, kerbs, road signs and safety barriers.

Separate plan drawings (coloured) showing the location and type of road furniture shall be produced. In addition, a road furniture schedule shall be provided. It should indicate the reference chainage, type, size and colour of road furniture and any other information that will aid in their installation.

Signs shall be referred to by their type numbers, together with a small-scale illustration of the sign face. Road markings shall be designated by their respective numbers.

12. Structural drawings

The drawings for drainage structures and facilities such as bridges should be clear and sufficient to allow for their construction. Concrete class, invert levels for side drains, culverts and other drainage structures should be provided to allow for smooth construction works.

The structural drawings should include the followings:

i. Bridge

Final drawings for a bridge should consist of a site plan, a plan and elevation drawing, foundation plan, substructure drawings, superstructure drawings, deck elevation plan and tabulation, and boring logs. They should be assembled in that general order.

The deck elevation plan shall show finished deck elevations along the centrelines of longitudinal beams or girders, gutter lines, breaks in roadway cross slope, and on tops of parapets. The Bending schedules for all reinforcements to be used in the bridge shall be provided in the Bridge drawings. The bridge drawings must show any protection works proposed.

ii. Culvert

Final drawings for a culvert should consist of a site plan, a plan and elevation drawing, culvert cross-section, wing wall cross sections, detail drawings, roadway surface elevation plan and tabulation (if top of culvert is roadway surface), and boring logs. Bending schedules for all reinforcements to be used in the culvert shall be provided in the culvert drawings. They must also show any inlet and outlet protection measures proposed.

iii. Other structures

Final drawings for any other structure shall consist of a site plan, the necessary number of plan and sections, detail drawings, boring logs (where applicable), bending schedules for all reinforcements, substructure and superstructure drawings (where applicable), elevations (where applicable), concrete class and any other information that will be prescribed in the ToR.

13. Standard Drawings

These are standard drawings and information which are already available to the road agency and are required to be included in the design drawings e.g. Consultant's office and accommodation, culvert marker post, road reservation boundary pillars, typical junctions, etc.

14. Traffic signals, street lighting and electrical/electronic works

The drawings that provide all details required by a contractor to construct and erect traffic signals, street lighting and all other electrical works required of the project road. The drawings scale is normally variable depending on the amount of details required. However, the drawings should be visually proportional and legible.

15. Landscaping

Landscaping drawings are used to show planned roadside beatification to be implemented.

Such drawings will normally show:

- Existing trees or vegetation which shall be protected during the construction period
- Which areas shall undergo a special landscape beautification and a specification of the details of the beatification measures?

The scale of the drawings is normally variable depending to the amount of details required.

16. Pictorial or three dimensional presentations of road features

Sometimes it might be necessary to provide a visual impression of the road or other facilities to the stakeholders. The reasons could be to provide illustrations on how the road will fit the environment, where there is an environmental or road safety concern or to attract financiers/investors on the project.

Where a need arises for the drawing, it is important that the drawing is clear and visible with the scale indicated. If no scale has been used the designer shall show in the drawing that it is not to scale or abbreviation NTS.

17. Soils and Geological maps and details

Soil and Geological maps/drawings assist the contractor to identify designed homogenous sections of subgrade of the road project. They also enable contractors to identify the locations for good construction materials.

They shall include drawings that indicate positions and details obtained during geotechnical investigations activities at bridge locations, and at any other location that geotechnical investigations are necessary.

The drawings scale is normally variable depending to the amount of details required.

18. Quantities, mass haul diagrams etc.

These are drawings which incorporate the quantities of different materials for different sections of the road. These quantities include:

- The volume of cut and fill (normally to formation)
- Volume of each pavement structure
- Reusable volume

Mass haul diagrams maybe prepared in order to show the balance of cuts and fills and how the construction materials shall be utilised from different borrow pits. Provision of this information will depend on what is stipulated in the terms of reference or where the designer finds it to be important to facilitate smooth construction of the road.

19. Setting out information

These are information that help in the proper setting out of the road and all related structures. They include:

- Reference chainages (based on regular intervals of 25m, geometry points, or as specified in the Tor). Shorter intervals are recommended for smaller curves.
- Offsets
- Elevations
- Slopes
- Coordinates (Northings and Eastings)

B. Design Reports

The following reports are to be submitted by Consultants (or the Design Team as in the case of in-house design) as stated in the Employer's Requirements and or Terms of Reference.

- Pre-Feasibility/ Feasibility Study Report
- Preliminary Design Report
- Draft Detail Design Report
- Final Detail Design Report

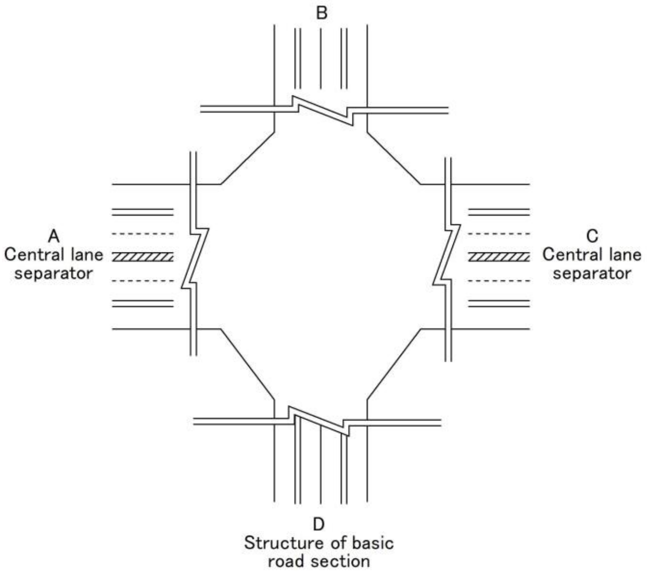
These are to include drawings (hard and editable AutoCAD copies). The Consultant may be asked to make a presentation of the reports where necessary. The report is to be on A4 sized paper with a similar cover page as the design drawings.

The Consultant shall only proceed to the next stage of the design after approval of reports and drawings and incorporation of review comments.

The number of copies, dates of submission and format of submission shall be consistent with the Standard Specification for Road and Bridge Works, 2007 unless otherwise specified in the Contract.

APPENDIX F PROCEDURE FOR DETERMINING GEOMETRIC STRUCTURES OF AT-GRADE INTERSECTIONS

F.1 Given conditions of design



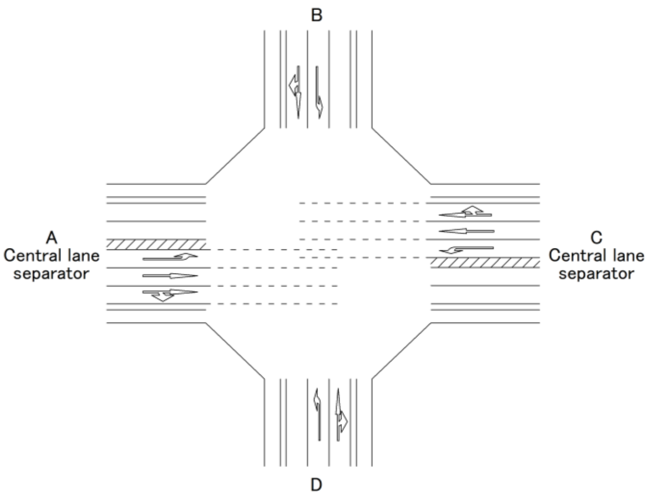
<Public–Private boundary line>

- Structure of basic road section
- The widths of the basic road section and the area of the at-grade intersection of roads A and C are the same.
- The widths of the area of the at-grade intersection of roads B and D are extended.

<Points to be checked>

- Design hourly volume
- Road classification
- Design speed

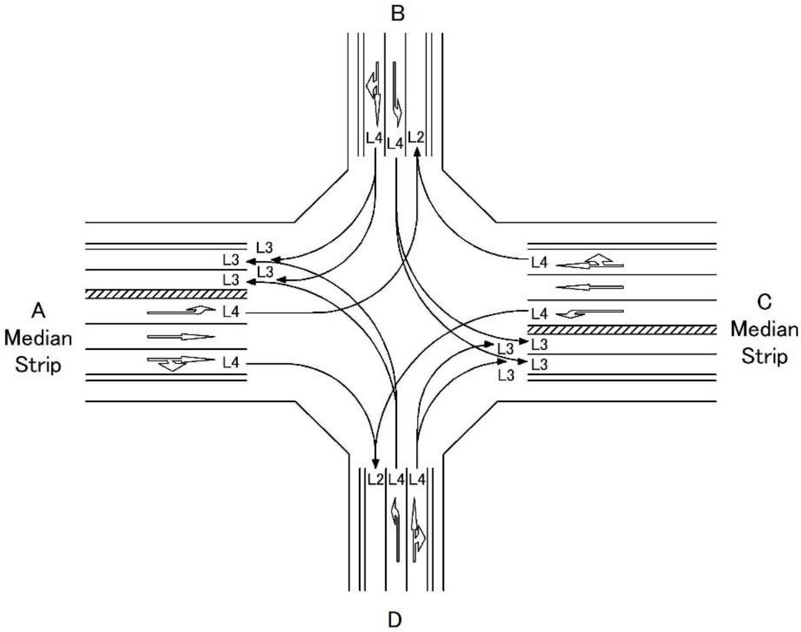
F.2 Determination of number of lanes and lane types



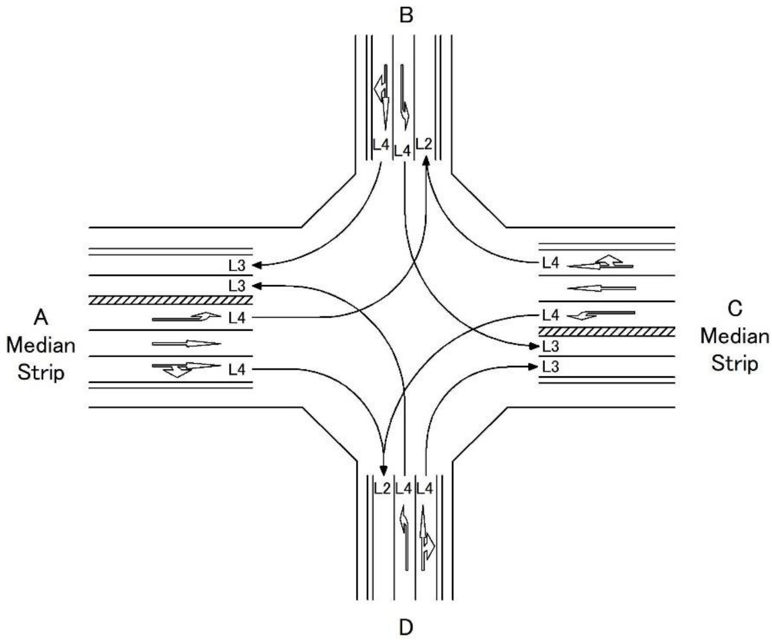
- Narrowing of central lane separators (example: 2.00 m → 1.00 m)
- Narrowing of width of shoulders (example: 1.0 m → 0.5 m)
- Narrowing of width of lanes (example: 3.25 m → 3.00 m)
- Narrowing of planting strips

Setting of basic road section lanes and Left-turn lanes

F.3 Determination of right-/left-turning design vehicles and their traversing type



Approach A or C → Exit B or D
L4 → L2
Approach B or D → Exit A or C
L4 → L3



When there are two lanes at the intersection exit and the signal has two phases, it is desirable that left-turn vehicles travel on the leftmost lane and right-turn vehicles travel on the rightmost lane

(a) Rural

Design Class			B1	C1	D1	E
Stop Sign Controlled	Approach		T4*	L4	L4	L1
	Exit	Main	T4*	L4	L3	L1
		Minor	-	L3	L3	L1
Signal Controlled	Approach		T4	L4	L4	L1
	Exit		T3	L3	L2	L1

* Indicates that if the design vehicle for the main road is different from that of the minor road, the design vehicle for the minor road is used.

(b) Urban

Design Class			B2	C2	D2	E
Stop sign controlled	**Approach**			L4	L4	L1
	Exit	**Main**	T4	L3	L2	L1
		Minor	-	L2	L2	L1
Signal Controlled	**Approach**			L4	L4	L1
	Exit			L2	L2	L1

* Indicates that if the design vehicle for the main road is different from that of the minor road, the design vehicle for the minor road is used.

Where:

T: Trailer
L: Large Vehicle

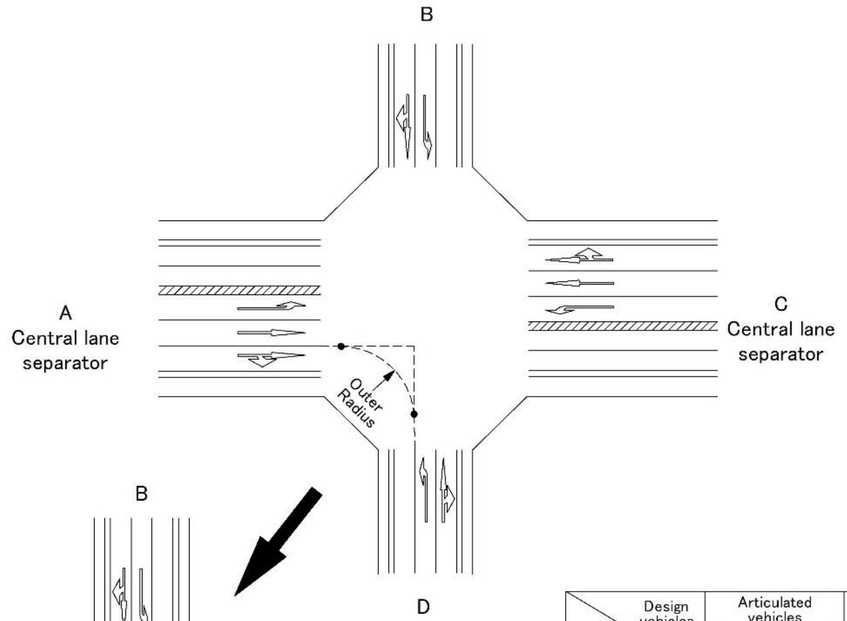
The numbers after the T and L describes the following turning paths.

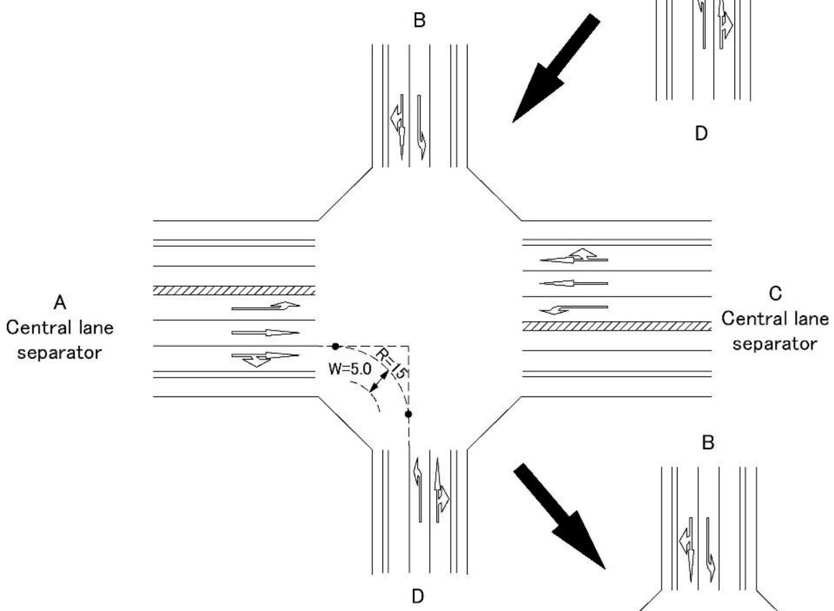
1. Use the full width of the carriageway
2. Use the right-hand side of the carriageway from the centre. Do not use the oncoming lane.
3. Use the left-turn lane or left-most lane (when turning left) or right-most lane (when turning right) and one other lane abutting it. However, the oncoming lane shall not be used.
4. Only use the right-turn lane or right-most lane (when turning right) or left-most lane (when turning left)

F.4 Design of right-/left-turn channels

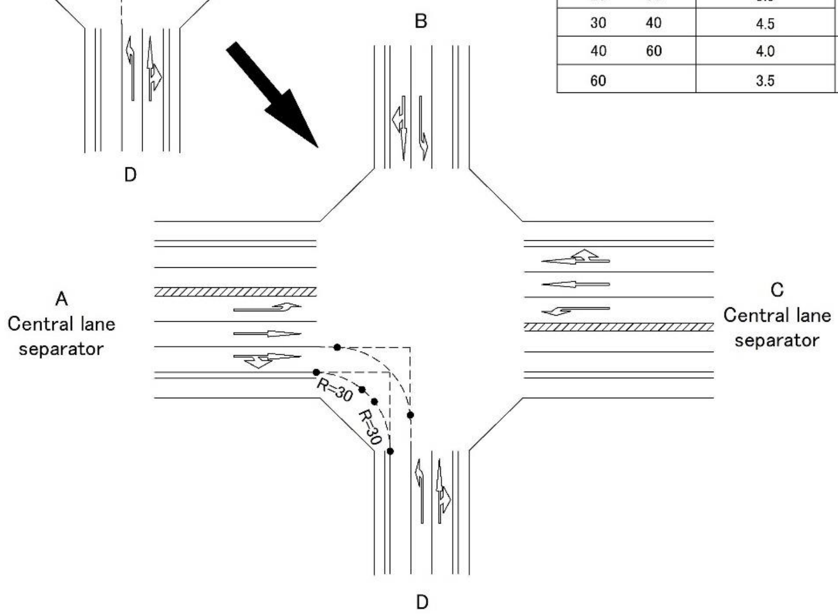
F.4.1 Design of left-turn channels

■ *The outer radius of the channel is designed to be in the range of 13–25 m.*
■ *This value is also applied to right-/left-turn channels of each intersection approach.*



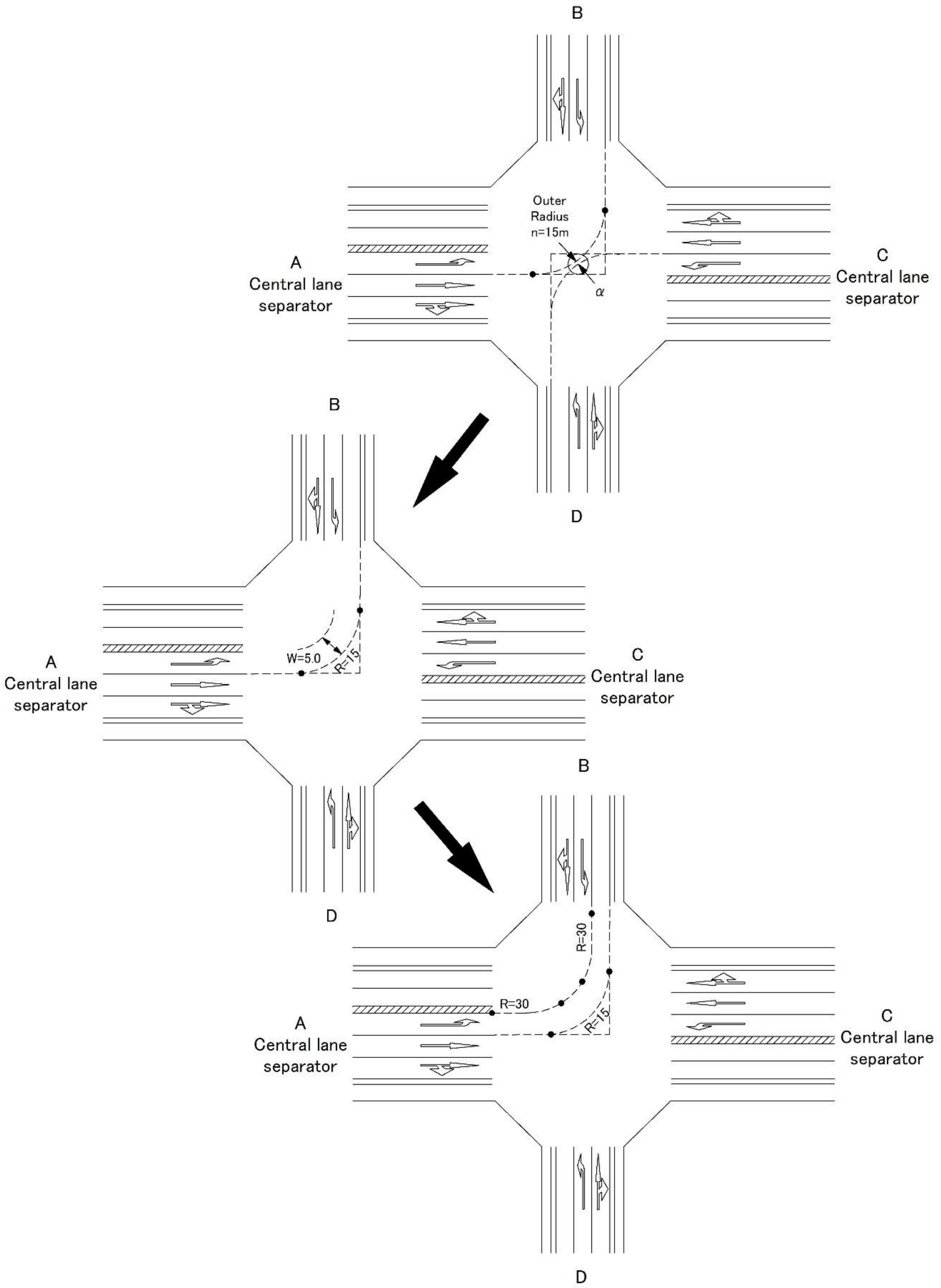


Outer radius(R) of channel \ Design vehicles		Articulated vehicles (Type 1, Type 2 Type 3 Class 1, Type 4 Class 1)	Regular vehicles (other roads)
13 ≤ R <14		8.5	5.5
14	15	8.0	
15	16	7.5	5.0
16	17	7.0	
17	19	6.5	
19	21	6.0	4.5
21	25	5.5	
25	30	5.0	4.0
30	40	4.5	
40	60	4.0	3.5
60		3.5	

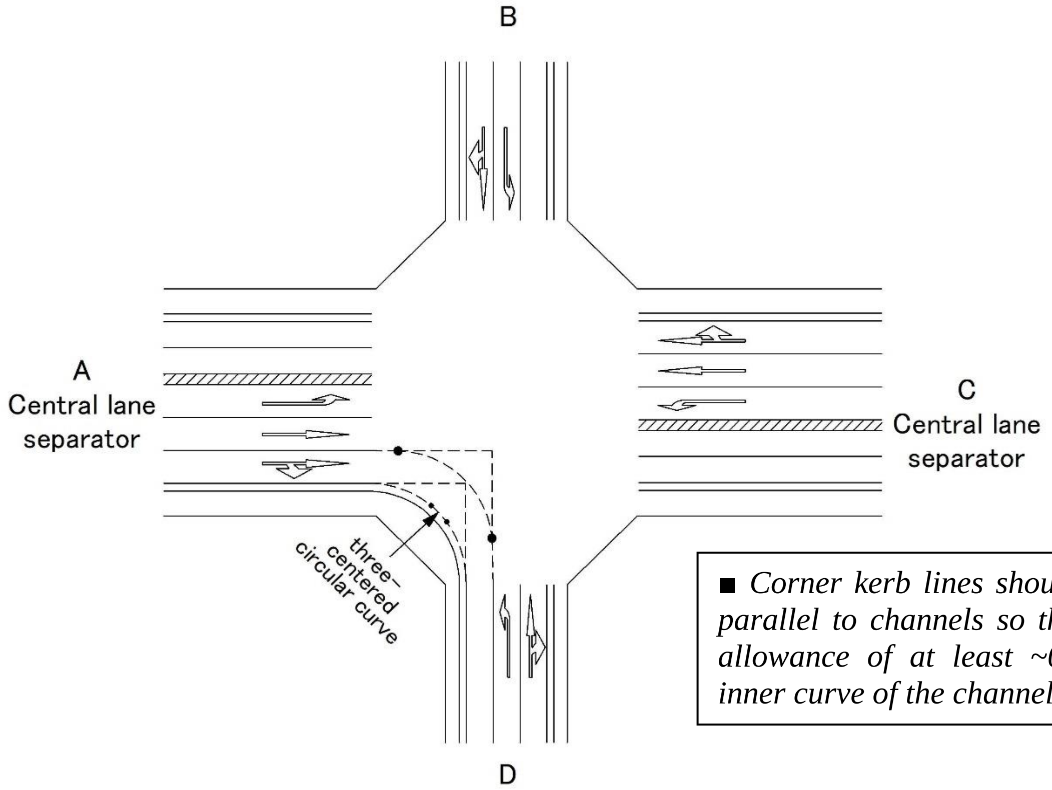


F.4.2 Design of right-turn channels

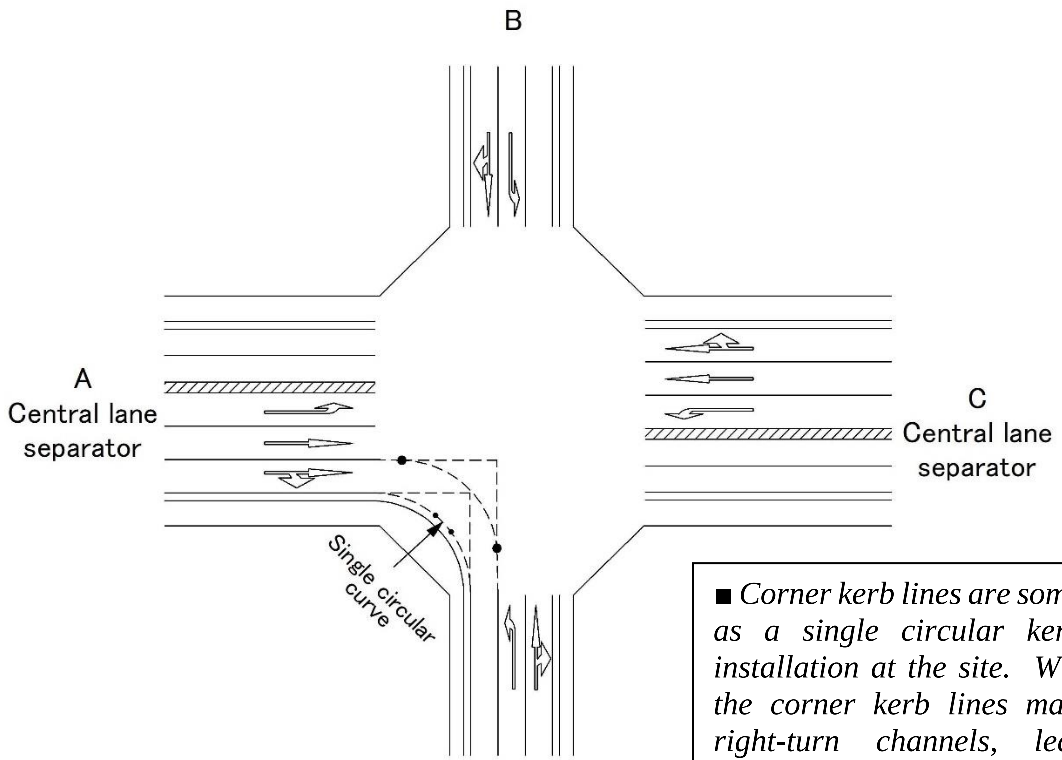
■ *The design method is basically the same as that for left-turn channels. Note that right-turn channels should not intersect with the right-turn channels on the oncoming lanes (point a in the figure)*



F.4.3 Corner kerb lines



■ *Corner kerb lines should be installed parallel to channels so that there is an allowance of at least ~0.5m from the inner curve of the channels.*



■ *Corner kerb lines are sometimes installed as a single circular kerb to facilitate installation at the site. With this method, the corner kerb lines may deviate from right-turn channels, leading to the dispersion of vehicles. Sufficient attention should be paid to safety issues in this case.*

F.4.4 Determination of placement of crosswalks and stop lines

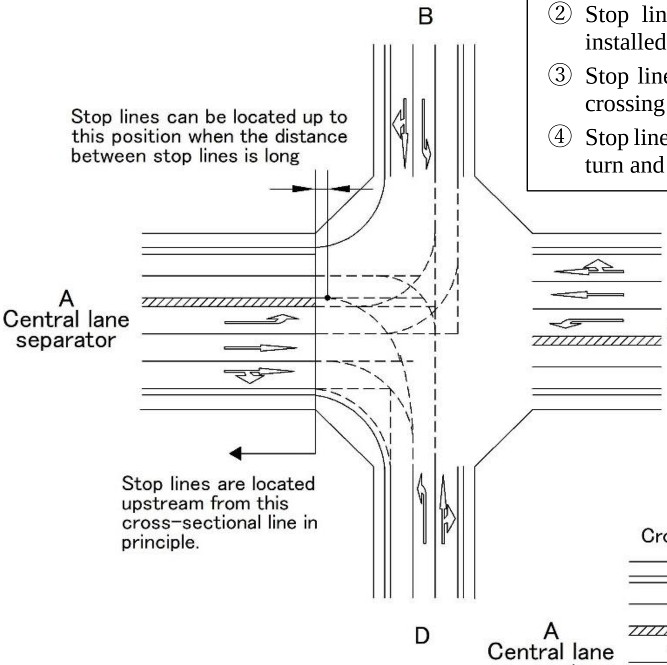
■ Points to note in determining the placement of stop lines

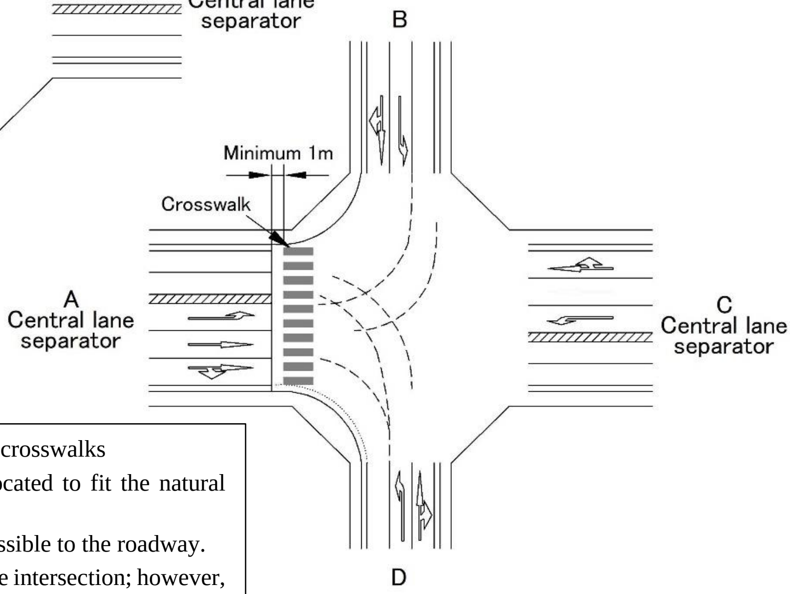
① Stop lines are perpendicular to the center line of the roadway in principle.

② Stop lines are at least 1m away from crosswalks when they are installed.

③ Stop lines are at positions where drivers can see the vehicles on the crossing road from a sufficient distance.

④ Stop lines are at positions that do not interfere with the passage of right-turn and left-turn vehicles on the crossing road.





■ Points to note in determining the placement of crosswalks

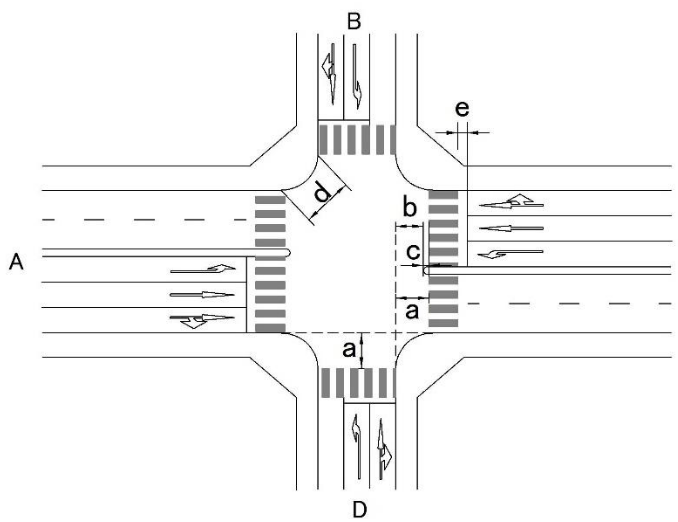
① Wherever possible, crosswalks should be located to fit the natural flow of pedestrians.

② Crosswalks should be as perpendicular as possible to the roadway.

③ Crosswalks should be close to the center of the intersection; however, they should not overlap with the crosswalks of the crossing road.

④ The desirable length of crosswalks is 15m or less.

⑤ The standard width of crosswalks is 4m in principle. This can be reduced to 3 m. When further reduction is required, the width can be reduced to less than 3 m.



a : At least 1m; 3–4 m when a space for vehicles stopping for pedestrians crossing the road is required.
b : Depends on the turning path of right-turn vehicles
c : 1–2 m
d : Space for left-turn vehicles stopping for pedestrians crossing the road
e : At least 1 m

F.4.5 Design of length of left-turn lanes

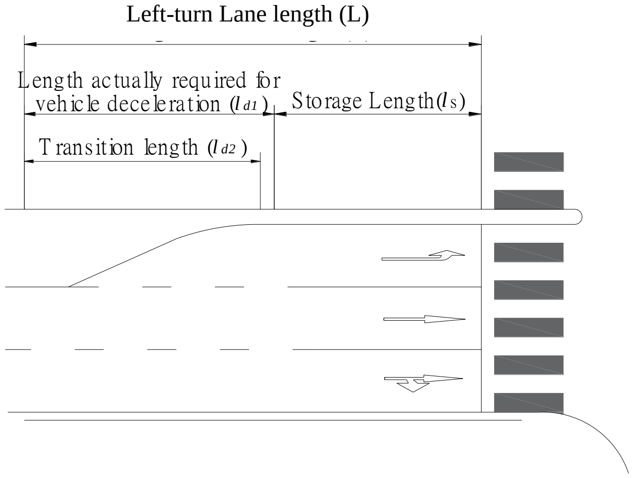


Figure F.1 Illustration of left turn lanes at intersection

i. Storage length (l_s)

$l_s = \xi \cdot N \cdot S$,

where

l_s : length required for storage [m],
ξ : additional lane length factor,
N : Average number of left-turn vehicles [vehicles/cycle], and
S : Average spacing of standing vehicles [m].

The additional lane length factor ξ is specified in **Table F.1**. It is desirable to ensure l_s of at least 30 m.

ii. Additional lane length factor ξ

Table F.1 Additional Lane length factor ξ

Average number of vehicles using additional lane (vehicle/cycle)	2 or less	3	5	8	10 or more
Additional lane factor	2.2	2.0	1.8	1.6	1.5

N and S are calculated by

$$N = \frac{n}{\frac{3,600}{C}}$$

Where

n: Number of left-turn vehicles per hour,
C: Cycle length, and
S : Average of the sum of headway and vehicles length (m)
5.0m(ratio of regular vehicles)/100
+12.0m(ratio of large vehicles/100)

The required storage length calculated using the above equation is ensured in the direction of road extension from the position of stop lines.

iii. Transition length (L_{d1})

$$l_{d1} = \frac{V \cdot \Delta W}{6}$$ -- Equation D-1

where

l_{d1} is the taper length [m],
V is the design speed [km/h], and
ΔW is the shift in the lateral direction [m] (corresponding to the width of the additional lane).

iv. Length actually required for vehicle deceleration (l_{d1})

Table F.2 Minimum length for deceleration (Ld)

Design speed (km/h)	Rural			Urban		
	Taper length (m) L_{d1}	Parallel length (m) L_{d2}	Deceleration length (m) L_d	Taper length (m) L_{d1}	Parallel length (m) L_{d2}	Deceleration length (m) L_d
120	100	180	280	70	90	160
100	90	100	190	60	30	90
80	70	50	120	50	20	60
60	60	10	70	35	-	35
50	50	-	50	30	-	30
40	50	-	50	25	-	25
30	20	-	20	20	-	20
20	20	-	20	10	-	10

v. Left-turn lane length (L)

$L = l_d + l_s$

vi. Length of section of main line shift (ls)

Table F.3 Length of section of main line shift (ls)

Design speed (km/h)	Rural		Urban	
	Equation	Minimum value (m)	Equation	Minimum value (m)
120	V×△W/2	-	V×△W/2	-
100		-		-
80		85		-
60		60	V×△W/3	40
50	V×△W/3	40		35
40		35		30
30		30		25
20		25		20

Note: *ΔW* is the shift in the lateral direction of main line [m]

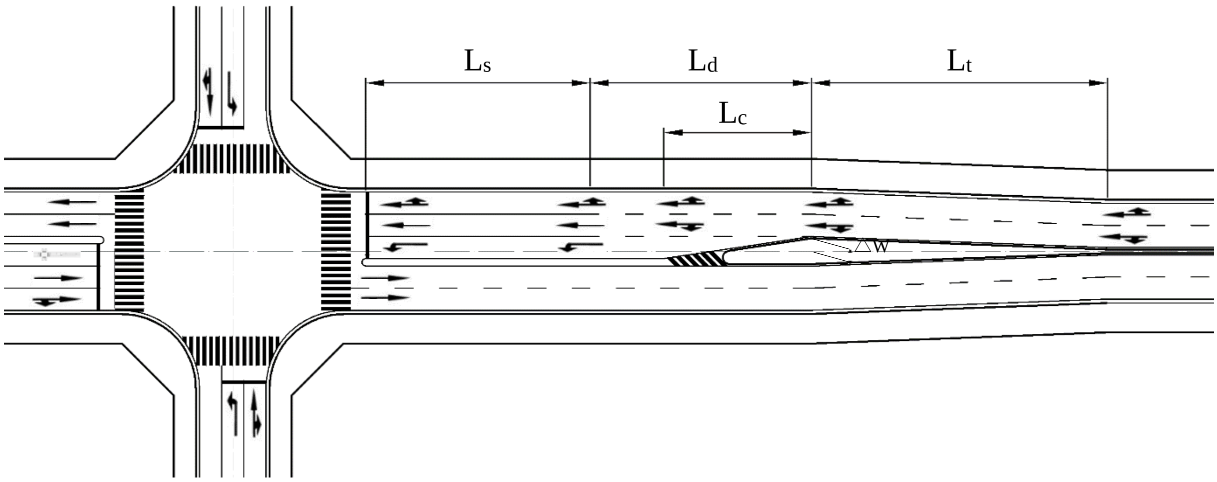


Figure F.2 Illustration of main line shift

F.4.6 Markings of lane lines, travel direction arrows, and channelization signs at intersection

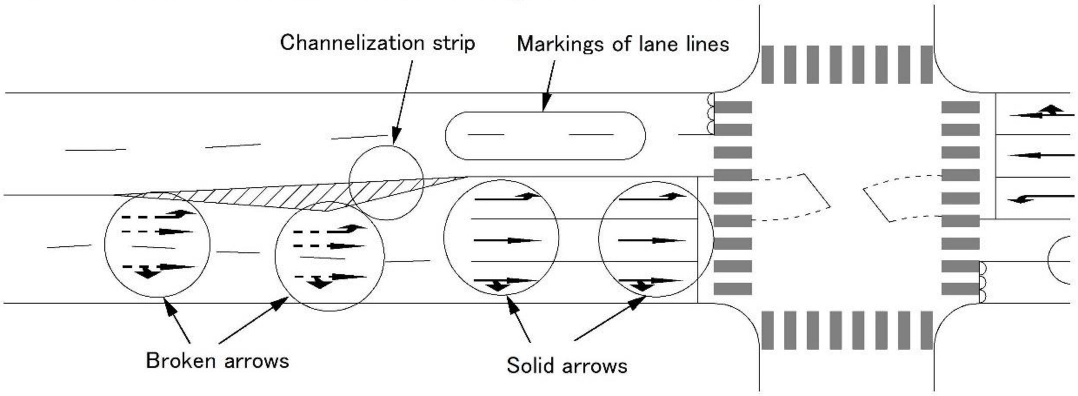
The positions of markings of lane lines are determined after the positions of stop lines and left-turn lanes have been set. Storage lanes are drawn as solid lines on the road. Other lines are drawn as solid lines with a length of 6.0m and an interval of 9.0m.

Two travel direction arrows are drawn on the each lane of the intersection approach within a 30m section from the stop line to the end of the solid marking of the lane line (5m before stop line and at end of solid marking of lane line). Broken arrows are drawn on the road further from the end of the solid marking of the lane line.

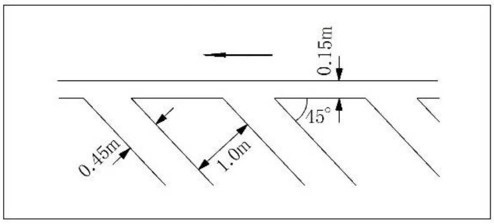
To stabilize the paths of left-turn and through vehicles in an intersection, channelization signs with a zebra pattern are drawn to indicate that drivers should not enter the zebra area. In addition, guidance lines are drawn to guide drivers of left-turn and through vehicles. The position and shape of the zebra pattern are determined depending on the channel of left-turn vehicles and the vehicle path of through vehicles.

Reference

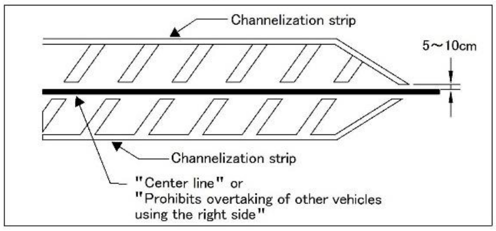
■Channelization strip, markings of lane lines, and travel direction arrows (including announcement)



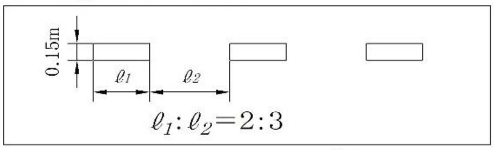
Channelization strip



Channelization strip

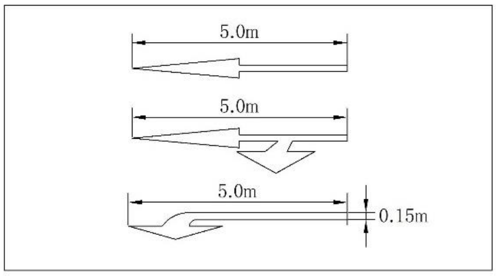


Markings of lane lines

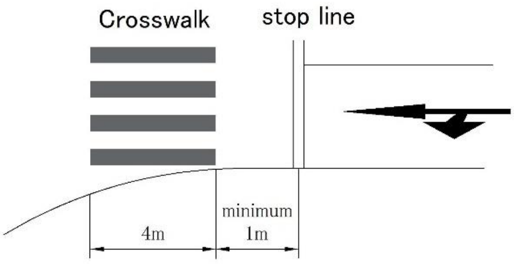


$^*\ell_1$=6m, ℓ_2=9m

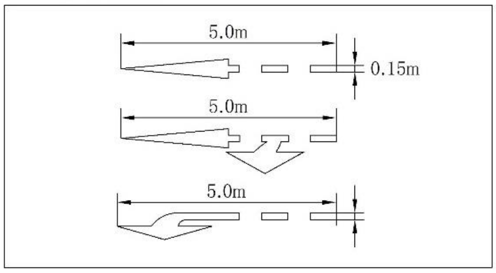
Solid arrows



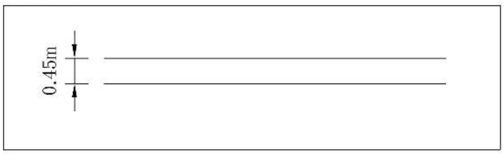
■Crosswalks and stop lines



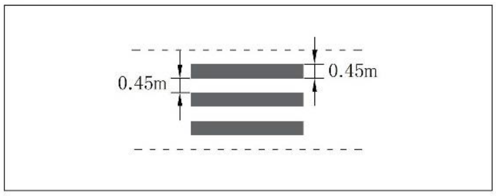
Broken arrows



Stop line



Crosswalk



APPENDIX G ROAD DESIGN CHECK SHEET

STAGE-1: PRELIMINARY DESIGN

PROJECT: ____

REVIEWED BY:____

DATE:____

CONDITIONS (Day/night-time, environmental, weather, traffic, others): ______________

____

ISSUES TO REVIEW	CHECK	COMMENTS
1. DESIGN CONCEPT, OBJECTIVES AND SCOPE		
(1) Have the design concept, objectives, and scope been designed adequately?		
2. SITE VISIT		
(1) Have the project design taken due cognizance of topography, geology, climate, irrigation channel, land use and connections of road drainage?		
(2) Have the design taken due cognizance of roadside conditions (including access way and rump), traffic conditions (including cyclists and pedestrians), road use conditions (including school routes, walkway structure, entrance section), and river conditions?		
(3) If there are houses or some facilities nearby the road, have the concerns about the impact of the subsidence by embankment checked?		
(4) Have the site conditions related to the construction plan been checked? (e.g., yards, construction water, turbid water treatment, power for construction, sites for constriction building, traffic conditions, access roads, and the progress of related peripheral construction)?		
3. BASIC DESIGN CONDITIONS		
(1) Have the technical standards, reference books and local regulations used in the design checked that they are the latest version?		
(2) Have the road structures been checked? (e.g., design class, traffic volume, design speed, typical cross section)		
(3) Have the geology, ground characteristics and groundwater conditions checked from previous survey reports?		
(4) Have the survey reports been checked? (e.g., coordinate system, height reference, topographic map, alignments)		
(5) Have the road earthworks checked that influencing effect, performance requirements and importance?		

CHECK: ✓=Yes, ×=No, NA=Not applicable

STAGE-1: PRELIMINARY DESIGN

PROJECT: ____

REVIEWED BY:____

DATE:____

CONDITIONS (Day/night-time, environmental, weather, traffic, others): _______________

____

ISSUES TO REVIEW	CHECK	COMMENTS
3-1. BASIC DESIGN FOR INTERSECTION		
(1) Have the structures and standards of the roads (e.g., main road, minor road, access road) been checked?		
(2) Have the design speeds for intersection and regulation speeds for existing roads been checked?		
(3) Have the intersection shapes been checked?		
(4) Have the design vehicles been checked?		
(5) Are flow lines for pedestrians and cyclists adequate?		
4. GEOMETRIC DESIGN		
(1) Are the adopted values for horizontal alignment and Vertical Alignment adequate? Are combinations of adopted values adequate? In the case of a revised design, has it been checked which parts of the design conditions have been changed?		
(2) Are values used for the geometric structure (e.g., walkway, lane width, superelevation, sight distance) appropriate?		
(3) Have connections with adjacent construction sections, earthworks, bridges, and tunnels (e.g., shoulder slope, wing walls, drainage, structural excavation) been considered?		
(4) Is the basis for determining the width clear and appropriate? (e.g., conformity with road standards, effective width in consideration of road attachment facilities).		
(5) Are drainage facilities, buried structures such as communication conduits, and guardrails provided with the necessary height of earth covering above the culvert for their installation?		
(6) Have the adopted values for the geometric structure of the grade separation and at-grade intersection section(s) been checked? In addition, have the offset (clearances) of the separation sections been checked?		

CHECK: ✓=Yes, ×=No, NA=Not applicable

STAGE-1: PRELIMINARY DESIGN

PROJECT: ____

REVIEWED BY:____

DATE:____

CONDITIONS (Day/night-time, environmental, weather, traffic, others): ______________

____

ISSUES TO REVIEW	CHECK	COMMENTS
4-1. GEOMETRIC DESIGN FOR INTERSECTION		
(1) Are the adopted values for horizontal alignment and Vertical Alignment adequate (main road and minor road)? Are combinations of adopted values adequate?		
(2) Is the length for the gentle gradient section appropriate?		
(3) Is the width configuration appropriate? In the width configuration, have bicycle spaces been considered?		
(4) Is the sight distance appropriate?		
(5) Are additional lanes installed appropriately?		
(6) Are the crossing angles adequate?		
(7) Is the shift method for the main line appropriate?		
5. EARTHWORKS AND SLOPES		
(1) Are the slope gradient, minor steps gradient, position, width and rounding shape for the cut appropriate?		
(2) Are the slope gradient, minor steps gradient, position, width and rounding shape for the embankment appropriate?		
6. DRAINAGE		
(1) Are the rainfall Intensity, design return period, drainage formula, runoff coefficient, and roughness coefficient set appropriately?		
(2) Is the calculation of the Discharge reasonable? (e.g., roughness coefficient)		
(3) Is the selection of drainage facilities such as shoulder ditches and crossing pipes appropriate? (economically, applicability, functionality, planning and maintenance)		
(4) Is the width margin set appropriately?		
(5) Is the drainage gradient (acceptable limits of flow velocity) appropriate?		

CHECK: ✓=Yes, ×=No, NA=Not applicable

STAGE-1: PRELIMINARY DESIGN

PROJECT: ____

REVIEWED BY:____

DATE:____

CONDITIONS (Day/night-time, environmental, weather, traffic, others): _______________

____

ISSUES TO REVIEW	CHECK	COMMENTS
７． WATER TREATMENT		
(1) Is the water supply system appropriate? Are relevant stakeholders coordinated regarding the consolidation of water channels?		
(2) Are the base drainage and slope drainage of the embankment structure adequate?		
８． PAVEMENT		
(1) Have the design conditions been checked? (e.g., traffic classification, design life, pavement type, Design CBR, application section）		
(2) Have the pavement design applicable standard and design methods been checked?		
９． SUBSTRUCTURE		
(1) Is the applied method of standard design appropriate?		
(2) If the case of providing gravity retaining walls or concrete block retaining walls are the reasons for providing retaining walls, are the height of retaining walls clear?		
(3) Have the application conditions for the concrete block retaining wall been checked?		
１０． ACCESSORIES		
(1) Have road accessories that require design been identified?		
(2) Are necessities of guardrails, installation manual, selection of type, installation conditions, and application of standard specifications appropriate?		
１１． RELATED ROAD (SIDE ROADS, ACCESS ROADS)		
(1) Are the widths, lengths, typical cross-sections and road geometries appropriate?		
(2) Have the heights of the roadside been considered?		

CHECK: ✓=Yes, ×=No, NA=Not applicable

STAGE-1: PRELIMINARY DESIGN

PROJECT: ____

REVIEWED BY:____

DATE:____

CONDITIONS (Day/night-time, environmental, weather, traffic, others): _______________

____

ISSUES TO REVIEW	CHECK	COMMENTS

CHECK: ✓=Yes, ×=No, NA=Not applicable

APPENDIX H GLOSSARY

Term	Definition
85^{th} percentile	Value of variable characteristic of individuals in a population, possessed by at or below 85 percent of that population. See annual average daily traffic.
AADT	See annual average daily traffic.
Absorption	The act or process of taking in water by inflow of atmospheric vapor, hydroscopic absorption, wetting, infiltration, influent seepage, and gravity flow of streams into sinkholes or other large openings.
Absorption capacity	The maximum rate at which a traffic stream can absorb additional vehicles.
Abstraction	That portion of rainfall which does not become runoff. It includes interception, infiltration, and storage in depression. It is affected by land use, land treatment and condition, and antecedent soil moisture.
Abutment	The support at either end of a bridge, usually classified as spill-through or vertical.
Acceleration lane	A speed-change lane used for increasing speed.
Access	The driveway by which vehicles and/or pedestrians enter and/or leave property adjacent to a road.
Access control	1. The prevention of vehicles and people crossing property lines by means of barriers or regulations. 2. Arranging matters so that vehicles and people have access at predetermined locations.
Access way	A private road or local street serving very low traffic volumes, whose design need not be dominated by traffic considerations. See also driveway.
Access-controlled Roads	A high-speed road restricted to motor vehicles with entry and exit movements at limited and designated grade-separated intersections
Accretion	1. The process of accumulation of silt, sand, or pebbles by flowing water; may be due to any cause and includes alluviation. 2. Gradual building up of a beach by wave action. 3. Gradual building of the channel bottom, bank, or bar due to silting or wave action.
ADT	The total traffic volume during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period.
Aggradation	General and progressive building up of the longitudinal profile of a channel by deposit of sediment.
Alignment	The geometric form of the centre line, or other reference line, of the carriageway with respect to the horizontal or vertical axes. See also property line.

Term	Definition
Alignment coordination	A road design technique which considers the relationship of the horizontal and vertical alignments and its influence on safety and the three-dimensional aspect of the finished carriageway.
Allowable Headwater	The depth or elevation of impounded water at the entrance to a hydraulic structure after which flooding, or some other unfavourable result could occur.
Alluvial Channel	A channel wholly in alluvium, no bedrock exposed in channel at low flow or likely to be exposed by erosion during major flow.
Alluvium	Unconsolidated clay, silt, sand, or gravel deposited by a stream in a channel, flood plain, fan, or delta.
Anabranched Stream	A stream whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars. The width of individual islands or bars is greater than three times the water width.
Annual Average Daily Traffic (AADT)	The total yearly traffic volume in both directions divided by the number of days in the year, expressed as vehicles per day.
Annual Flood	The highest peak discharge in a water year.
Annual Series	A frequency series in which only the largest value in each year is used, such as annual floods.
Antecedent Moisture Condition (AMC)	The degree of wetness of a watershed at the beginning of a storm.
Approach nose	The end of an island, median or separator, or area between diverging carriageways, which faces approaching traffic.
Approach speed	The representative vehicle speed (usually taken as the 85th percentile).
Aquifer	A porous, water-bearing geologic formation. Generally restricted to materials capable of yielding an appreciable supply of water.
Area Rainfall	The average rainfall over an area, usually as derived from or discussed in contrast with, point rainfall.
Armor	Artificial surfacing of channel beds, banks, or embankment slopes to resist scour and lateral erosion.
Armouring	The concentration of a layer of stones on the bed of the stream that are of a size larger than the transport capability of the recently experienced flow.
Arrestor bed	A bed of loose material (such as gravel) designed to slow a vehicle (such as a landing aircraft or a semitrailer) that is out of control
Arterial road	A road that predominantly carries through traffic from one region to another, forming principal avenues of communication for traffic movements. See also road.
Artesian	Pertains to groundwater that is under pressure and will rise to a higher elevation if given an opportunity to do so.

Term	Definition
At-grade crossing	Crossing at the same level, such as a railway crossing which is at the same level as a road, or a normal road intersection.
At-grade intersections	Intersections where at least one conflicting traffic stream has to stop or slow down considerably.
Auxiliary lane	The portion of carriageway adjoining the through traffic lanes, used for speed change, weaving, climbing, passing, or for other purposes supplementary to through traffic movement.
Auxiliary overtaking lanes	When there are slow-moving vehicles (slow-speed vehicles) and sections where it is difficult to overtake continue for a long time, there is a risk that the overall vehicle running speed will decline, leading to a reduction in the traffic processing capacity, and negatively influencing the safety and comfort of traffic. An auxiliary overtaking lane is installed for the purpose of enabling slow-speed vehicles and following vehicles (high-speed vehicles) to travel separately.
Average Daily Traffic (ADT)	The total traffic volume during a stated period, divided by the number of days in that period.
Average design speed	The weighted average of the design speeds within a road section, in which each subsection within the section is considered to have an individual design speed.
Average speed (running speed)	The speed over a specified section of road, being the distance divided by the travel time. The average for all traffic, or a component thereof, is the summation of distances divided by the summation of travel time.
Average weekday traffic (AWT)	The total traffic volume for all of the weekdays less public holidays in a stated period, divided by the number of days in that period.
Avulsion	A sudden change in the course of a channel, usually by breaching of the banks during a flood.
Axle group	A set of closely spaced axles acting as a unit, ie. axles spaced at less than 2.4 m.
Axle load	The load applied to a pavement by a single axle.
B	Barrel width, distance measured in meters.
Backfill (n)	Fill placed in an excavation.
Backwater	The increase in water-surface profile, relative to the elevation occurring under natural channel and flood-plain conditions, induced upstream from a structure, bridge, or culvert that obstructs or constricts a channel. It also applies to the water surface profile in a channel or conduit.
Baffle	A structure built on the bed of a stream to deflect or disturb the flow. Also, a device used in a culvert to facilitate fish passage.
Bank	1. An embankment or fill. 2. A fill in the line of a road. 3. Lateral boundaries of a channel or stream, as indicated by a scarp, or on the inside of bends, by the stream ward edge of permanent vegetal growth.

Term	Definition
Bar	An elongated deposit of alluvium, not permanently vegetated, within or along the side of a channel.
Barrier	An obstruction placed to prevent access to a particular area.
Barrier kerb (non-mountable kerb)	A kerb high enough to prevent or discourage driving off the carriageway. See also semi- barrier kerb.
Basecourse (base, road base)	One or more layers of material usually constituting the uppermost structural element of a pavement and on which the surfacing may be placed. It may be composed of fine crushed rock, natural gravel, broken stone, stabilised material, asphalt or portland cement concrete.
Base	See basecourse.
Base Flood	The 100-year flood.
Base Flow	Stream discharge derived from groundwater sources. Sometimes considered to include flows from regulated lakes or reservoirs. Fluctuates much less than storm runoff.
Basic capacity	Capacity of a road or area to accommodate moving and/or stationary vehicles regardless of the effect of delaying drivers, restricting their freedom to manoeuvre, or the need to maintain specified environmental standards. See also capacity.
Basic lanes	Those lanes forming the minimum number of lanes designated and maintained over a significant length of route, irrespective of changes in travel volume and the requirements of lane balance.
Basin Lag	The amount of time from the centroid of the rainfall hyetograph to the hydrograph peak.
Basin, Drainage	The area of land drained by a watercourse.
Batter	1. The uniform side slope of walls, banks, cuttings etc. 2. The amount of such slope or rake, usually expressed as a ratio of horizontal to vertical, distinct from grade. 3. To form a uniform side slope to a wall, bank, or cutting.
Bed (of A Channel or stream)	The part of a channel not permanently vegetated or bounded by banks, over which water normally flows.
Bed Load	Sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer.
Bed Material	Sediment consisting of particle sizes large enough to be found in appreciable quantities at the surface of a streambed.
Bed Shear (Tractive Force)	The force per unit area exerted by a fluid flowing past a stationary boundary
Bench	A ledge cut or formed in the batter of a cutting or natural slope to provide greater security against slips.

Term	Definition
Bend lanes	Intersections often become traffic capacity bottlenecks, and the most effective way to increase road capacity is to increase the capacity of intersections. Accordingly, for the sake of vehicles that turn right or left at intersections, lanes for right-turning vehicles and lanes for left-turning vehicles that are installed separately from the lanes for straight-moving vehicles are referred to as right-turn lanes and left-turn lanes respectively. Bend lanes are sometimes installed adjacent to lanes for straight-moving vehicles, and sometimes they are separated by traffic islands.
Benefit cost ratio	Ratio of the present value of economic benefits derived by the community from transport system improvements over the present value costs of those improvements. Costs and benefits are usually measured relative to a 'base case' which often implies doing nothing, i.e. the 'do nothing case'. It is often not practical to do nothing, and a certain level of expenditure is needed to maintain a minimum level of service. This expenditure is known as the 'do minimum case'.
Berm	1. A narrow shelf, path, or ledge formed typically at the top or bottom of an earth slope; also, a form of dike. 2. The shoulder of a road. 3. A grassed area between the kerb and footpath or between the footpath and property boundary. 4. A mound on the outer edge of a road above a fill batter to protect the batter from erosion.
Bicycle track/lane	The bicycle track is the part of a road that is intended solely for the passage of bicycles. Concerning width, as a rule, sufficient width is secured to enable bicycles to pass by each other or overtake. When a bicycle track is provided, it is necessary to physically separate it from the carriageway, pedestrian walkway and other parts by means of kerbstones, fences or other structures. Also, when installing bicycle track, a pedestrian walkway is also installed in tandem with it.
Borrow pits	A borrow pit is a hole, pit or excavation that has been dug for the purposes of removing gravel, clay and sand used in a construction project such as when building an overpass or embankment.
Boundary line	The boundary between a road reserve and the adjacent land.
Braided Stream	A stream whose surface is divided at normal stage by small mid-channel bars or small islands. The individual width of bars and islands is less than three times the water width. A single large channel that has subordinate channels.
Braking distance	The distance travelled by a vehicle in the period between the initial application of the brakes and coming to rest plus reaction time.

Term	Definition
Breakers	The surface discontinuities of waves as they break-up. They may take different shapes (spilling, plunging, surging). Zone of break-up is called surf zone.
Bridge	A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, having a tract or passageway for carrying traffic or moving loads, and having an opening measured along the centre of the roadway of more than six meters between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes. May also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening. Also, a structure designed hydraulically using the principles of open channel flow to operate with a free water surface but may be inundated under flood conditions.
Bridge Opening	The cross-sectional area beneath a bridge that is available for conveyance of water.
Bridge Waterway	The area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.
Broken-Back Culvert	A culvert comprising two or more longitudinal structure profiles. Such culverts are sometimes effective in reducing outflow velocities by the energy dissipation of a hydraulic jump.
Built-up Area	A developed area with buildings, structures and concentration of human activities. This may include a city, a town, a village or an industrial zone
Bulking	The increase in volume of a material resulting from disturbance or from changes in its condition, in particular from an increase in moisture content.
Bulking factor	The ratio of the final volume and the initial volume of a material after bulking has occurred.
Bus lane	A traffic lane reserved for, or primarily used by, buses either when transporting, loading or discharging passengers, or when standing.
By-pass	An alternative route which enables through traffic to avoid urban or congested areas, or other obstructions to movement. Usually, to divert heavy vehicles away from residential areas.
By-Pass	Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downstream. Also called carryover.
By-pass traffic	Traffic that uses a by-pass because drivers do not wish to stop on that road or in that area. See also traffic.
Camber	1. The transverse convexity given to the surface of a carriageway or footway. 2. The upward vertical curvature of a beam.
Capacity	A measure of the ability of a channel or conduit to convey water.

Term	Definition
Capacity (basic capacity, environmental capacity, parking capacity, road capacity)	The maximum rate of flow at which persons or vehicles can reasonably be expected to traverse a point or uniform segment of a lane or road during a given period of time under the prevailing road, traffic and control conditions.
Car park	Open area or structure designed for the storage of vehicles off the road network.
Carriageway	That part of the road normally used by vehicular traffic. Auxiliary traffic lanes, passing places, lay-bys and bus bays are included in this term but excluding shoulders.
Catch Basin	A structure with a sump for inletting drainage from a gutter or median and discharging the water through a conduit. In common usage it is a grated inlet with or without a sump.
Catch drain	A surface channel constructed along the high side of a road or embankment, outside the batter to intercept surface water.
Catchment	The watershed (implying all physical characteristics).
Catchment Area	A river catchment is an area of land where water collects when it rains, often bounded by hills. As the water flows over the landscape, it finds its way into streams and down into the soil, eventually feeding the river. Some of this water stays underground and continues to slowly feed the river in times of low rainfall. Every inch of land on the Earth forms part of a catchment.
Central business district (CBD)	Dominant centre of business and commercial activity within a given area. CBDs are characterised by high density office and retail development, large numbers of pedestrians and vehicles, and a heavy demand for parking.
Centreline	1. The line which defines the axis or alignment of the centre of a road or other work. 2. A marked line on the centre of the carriageway separating opposing traffic streams.
Chainage (station)	The distance of a point along a control line, measured from a datum point.
Channel	1. The bed and banks that confine the surface flow of a natural or artificial stream. Braided streams have multiple subordinate channels that are within the mainstream channel. Anabranched streams have more than one channel. 2. The course where a stream of water runs or the closed course or conduit through which water runs, such as a pipe.
Channel Lining	The material applied to the bottom and/or sides of a natural or manmade channel. Material may be concrete, sod, grass, rock, or any of several other types.
Channel Routing	The process whereby a peak flow and/or its associated stream flow hydrograph is mathematically transposed to another site downstream.

Term	Definition
Channelisation	A system of controlling traffic by the introduction of an island or islands, or markings on a carriageway to direct traffic into predetermined paths, usually at an intersection or junction.
Channelised intersection	An intersection provided with medians and islands for defining the trafficable area and to control specific movements.
Channelising island	A traffic island used at an intersection to confine specific movements of traffic to definite channels.
Check Dam	A low structure, dam, or weir across a channel for the control of water stage, velocity, or to control channel erosion.
Check Flow	A flow, larger or smaller than the design flow that is used to assess the performance of the facility.
Chute	Chutes are steep (greater than 15%) natural or man-made open channels used to convey water. They may be closed and usually require energy dissipation at their termini.
Circulating carriageway	The carriageway surrounding the central island of a roundabout.
Circulation	Pattern in which traffic moves in a given area.
Circumferential road	A roughly circumferential road about the centre of an urban area permitting traffic to avoid the centre of such areas. See also road.
Clear run-out area	A clear run-out area is the area at the toe of a non-recoverable slope available for safe use by an errant vehicle.
Clear zone	The area adjacent to the road that is clear of fixed objects and provides a recovery zone for vehicles that have left the carriageway.
Clearway	A portion of carriageway generally defined by signs, along which vehicles may not voluntarily stop or be left standing at prescribed times of the day.
Climbing lane	An auxiliary lane, usually on a long upgrade, primarily for the use of slow moving vehicles. See also passing lane.
Coastal Zone	The strip of land that extends inland to the first major change in terrain (lake shore features).
Coefficient of Discharge	The coefficient used for orifice flow processes.
Coefficient of side friction (sideways force coefficient)	The ratio of the resistance to sideways motion to the normal component of the force between the tyres of the vehicle and the pavement.
Collector road (distributor road)	A non-arterial road which collects an distributes traffic in an area, as well as serving abutting property. See also road.
Collector-distributor road	An auxiliary road, separated laterally from but generally parallel to a through road and joining it at a limited number of points. The road serves to collect from and distribute traffic to several local roads. See also service road.
Combination inlet	Drainage inlet usually composed of two or more inlet types, e.g., kerb opening and a grate inlet.

Term	Definition
Commercial vehicle (CV)	A vehicle having at least one axle with dual wheels and/or having more than two axles.
Compact Roundabout	Roundabout with a single traffic lane on the approach, exit and circulatory carriageway without flaring design at entries. If there are two traffic lanes, these are known as Compact 2-lane roundabouts. Three or more traffic lanes are not permitted for any part of the roundabouts.
Compound curve	A curve consisting of two or more arcs of different radii curving in the same direction and having a common tangent point or being joined by a transition curve.
Conduit	An artificial or natural channel, usually a closed structure such as a pipe.
Conflict point	Point of potential collision between vehicles involved in a manoeuvre.
Conjugate Depth	The alternate depth of flow involved with the hydraulic jump.
Continuity Equation	Discharge equals velocity times cross-sectional area (Q = V x A).
Continuity line	A longitudinal broken line of distinctive pattern which may be used to indicate the edge of that portion of the carriageway assigned to through traffic and which is intended to be crossed by traffic turning at an intersection or entering or leaving an auxiliary lane at its start or finish.
Contraction	The effects of a channel constriction on flow. The response of a river to the change in its bed load requirement as a result of a contraction of flow. The flow contraction is due to an encroachment of either the main channel or the flood plain by a natural constriction or the highway embankment.
Contraflow	Traffic flow in a direction opposite to the normal flow, e.g., a contraflow bus lane might be one that runs the 'wrong way' on a one-way street.
Control Point	An accurately surveyed coordinate location for a physical feature that can be identified on the ground. Control points are used in least-squares adjustments as the basis for improving the spatial accuracy of all other points to which they are connected.
Control Section	A cross section, such as a bridge crossing, reach of channel, or dam, with limited flow capacity, and where the discharge is related to the upstream water-surface elevation.
Controlled access road	A road for through traffic to which access from abutting properties or joining roads in controlled. See also road and limited access road.
Conveyance	A measure, K, of the ability of a stream, channel, or conduit to convey water. In Manning's formula $K = (1/n)AR^{2/3}$ (SI units).
Cordon survey	Survey of traffic crossing a cordon line. Used to obtain trip data on vehicle and/or persons travelling into, out of or through a study area.

Term	Definition
Corridor	Major area of travel between two points. (A corridor may include more than one major route and more than one form of transport).
Corrosion	The deterioration of pipe or structure by chemical action.
Cover	The extent of soil above the crown of a pipe or culvert. The vegetation or vegetational debris, such as mulch, that exists on the soil surface. In some classification schemes fallow or bare soil is taken as the minimum cover class.
Crash frequency	It is defined as the number of crashes occurring within a specific jurisdiction, on a roadway segment, or at an intersection.
Criterion	A standard, rule, or test on which a judgment is based.
Critical acceptance gap	The minimum gap in a traffic stream acceptable to drivers wishing to join or cross that stream, or to drivers in the opposing stream wishing to overtake. See also gap acceptance. The traffic density on a given road or carriageway when the traffic volume is at capacity.
Critical Depth	The depth at which water flows over a weir; this depth being attained automatically where no backwater forces are involved. It is the depth at which the energy content of flow is a minimum.
Critical movements	The traffic movements which determine the capacity and timing requirements of a signalised intersection.
Cross Drainage	The runoff from contributing drainage areas both inside and outside the highway right-of-way and the transmission thereof from the upstream side of the highway facility to the downstream side.
Crossfall	The slope, at right-angles to the alignment, of the surface of any part of the carriageway.
Crossing	A formal area set aside for other modes of transport to cross the road. Usually called cycle crossing, pedestrian crossing, railway crossing, etc., as appropriate.
Crossing sight distance	Sight distance required to enable traffic to start from rest and safely cross one or more traffic streams.
Cross-section	1. A vertical section, generally at right-angles to the centreline showing the ground. On drawings it commonly shows the road to be constructed, or as constructed. 2. The shape of a channel, stream, or valley viewed across its axis. In watershed investigations it is determined by a line approximately perpendicular to the main path of water flow, along which measurements of distance and elevation are taken to define the cross- sectional area.
Crown	1. The highest part of an arch. 2. The highest point on the cross-section of a carriageway with two-way crossfall.
Cul-de-sac	A street or road open for vehicular traffic at one end only.

Term	Definition
Culvert	A structure that is usually designed hydraulically to take advantage of submergence to increase hydraulic capacity. A structure used to convey surface runoff through embankments. A structure, as distinguished from bridges, that is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert. Also, a structure which is six meters or less in centreline length between extreme ends of openings for multiple boxes.
Cumulative Conveyance	A tabulation or graphical plot of the accumulated measures of conveyance; proceeding from one stream bank to the other.
Curves/bend	A horizontal curve unless otherwise specified as a vertical curve
Cut	The depth from natural surface of the ground down to the subgrade level. Also means to excavate.
Cut-off drain	An interceptor drain constructed along the top of a cutting or batter to prevent surface water running down the face.
Cut-off Wall	A wall that extends from the end of a structure to below the expected scour depth or scour-resistant material.
Cut-out	An opening which allows water to escape from the carriageway or channel to a drain or water course.
Cut-out drain	An open drain or ditch formed to drain water from the surface water channel to a side drain or natural water course.
Cutting	That portion of the road where the finished road surface is below the natural surface.
Cycle path	Separate carriageway devoted to the use of pedal cycles.
Cycle track	Separate carriageway or portion of road devoted to the use of pedal cycles.
Cycleway	Portion of road or footpath devoted to the use of pedal cycles.
D	Culvert diameter or barrel depth.
D_{15}	The particle diameter at the 15th percentile point on a size weight distribution curve.
D_{50}	Median size of rip rap. The particle diameter at the 50th percentile point on a size weight distribution curve.
D_{85}	The particle diameter at the 85th percentile point on a size weight distribution curve.
D_c	Critical depth of flow in meters.
Debris	Material transported by the stream, either floating or submerged, such as logs or brush.
Deceleration lane	A speed-change lane provided to allow vehicles to decrease speed.
Deflection	The vertical elastic (recoverable) deformation of a pavement surface between the tyres of a standard axle.

Term	Definition
Degradation	General and progressive lowering of the longitudinal profile of a channel by erosion.
Degree of saturation	The ratio, usually expressed as a percentage, of the number of vehicles entering an intersection in a specified period, to the number which could enter if all approaches were fully saturated during that period. May also be applied to an approach to an intersection.
Delay	The time lost while traffic is impeded by some element over which the driver has no control.
Demand	The traffic volume desiring to travel along a given route.
Demographic survey	Demographic surveys are surveys that wholly or primarily collect information on population characteristics and on the causes and consequences of population change.
Deposition	The settling of material from the stream flow onto the bottom.
Depression Storage	Rainfall that is temporarily stored in depressions within a watershed.
Depth-Area Curve	A graph showing the change in average rainfall depth as size of area changes.
Design Discharge or Flow	The rate of flow for which a facility is designed and thus expected to accommodate without exceeding the adopted design constraints.
Design Flood	A flood that does not overtop the roadway.
Design Flood Frequency	The recurrence interval that is expected to be accommodated without contravention of the adopted design constraints. The return interval (recurrence interval or reciprocal of probability) used as a basis for the design discharge.
Design Flow	See Design Discharge
Design Highwater Elevation	The maximum water level that a bridge opening is designed to accommodate without contravention of the adopted design constraints. The usual term used to describe the estimated water surface elevation in the stream at the project site for the design discharge.
Design life	The period during which the performance of a pavement, e.g. riding quality, is expected to remain acceptable.
Design period	A period considered appropriate to the function of the road. It is used to determine the total traffic for which the pavement is designed.

Term	Definition
Design process	Process for arriving at a final design for a transport project and normally divided into three distinct phases: 1. Functional Design: Preparation of the conceptual design with enough detail to ensure that the design will function as intended. 2. Preliminary Design: Finalisation of design in terms of calculations, specifications and estimates such that all aspects of the design are determined. See also preliminary engineering. 3. Documentation: Preparation of plans and documents describing the design sufficiently for it to be constructed
Design queue length	The length of a queue normally provide for when designing an intersection, usually calculated to provide adequate storage at a given probability level, normally 95 percent.
Design speed	A speed fixed for the design of minimum geometric features of a road.
Design Storm	A given rainfall amount, areal distribution, and time distribution used to estimate runoff. The rainfall amount is either a given frequency (25-year, 50-year, etc.) or a specific large value.
design traffic	The cumulative traffic, expressed in terms of Equivalent Standard Axles (ESAs), predicted to use a road over the structural design life of the pavement.
Design vehicle	A hypothetical road vehicle whose mass, dimensions and operating characteristics are used to establish design requirements.
Design volume	The number of vehicles expected to use the road and adopted for the purposes of geometric design, normally expressed as the number of vehicles per hour or per day.
Design year	The predicted year in which the design traffic would be reached.
Desire line	A straight line joining points or zones of trip origin and destination indicating the desired line of travel.
Desired speed	The speed over a section of road adopted by a driver or drivers when not as influenced by the road geometry or other environmental factors. See also speed environment.
Destination	Point or area in which a trip ends.
Detention Basin	A basin or reservoir incorporated into the watershed whereby runoff is temporarily stored, thus attenuating the peak of the runoff hydrograph.
Detour	An alternative route available to traffic during temporary closure of a road.
Deviation	An alternation to the alignment of a portion of a road, usually involving significant departure from the existing route.

Term	Definition
Digital Elevation Model (DEM)/ Digital Terrain Model (DTM)	A Digital Elevation Model (DEM) is a digital cartographic dataset that represents a continuous topographic elevation surface through a series of cells. Each cell represents the elevation (Z) of a feature at its location (X and Y). Digital Elevation Models are a “bare earth” representation because they only contain information about the elevation of geological (ground) features, such as valleys, mountains, and landslides, to name a few. They do not include any elevation data concerning non-ground features, such as vegetation or buildings.
Dike	An impermeable linear structure for the control or confinement of overbank flow. River training structure used for bank protection.
Direct Runoff	The water that enters the stream channels during a storm or soon after forming a runoff hydrograph. May consist of rainfall on the stream surface, surface runoff, and seepage of infiltrated water (rapid subsurface flow).
Directional interchange	An interchange, generally between two freeways or expressways, providing direct connections for some or all right-turning movements.
Directional split	The ratio of the one-way traffic volume on a road in the major direction to that in the minor direction during a specified period.
Directional/Guide Sign	Traffic signs providing directional guidance for road-users.
Discharge	The rate of the volume of flow of a stream per unit of time, usually expressed in m^3/s.
Dish channel	A channel with a 'u'-shaped cross-section. Normally found in urban areas.
Distributor road	See collector road.
Diverge Gores	The physical area immediately downstream of the paved area at diverges.
Diverging	Division of a single stream of traffic into separate streams.
Diverted traffic	Component of traffic which has changed its route but not its origin, destination or mode of travel. See also traffic.
Divided road (dual carriageway)	A road having a separate carriageway for each direction of travel.
Divided roads/Dual Carriageway Road	A road with two pavements physical separated by a verge, traffic island or safety barrier between opposing traffic
Downstream	The direction along a carriageway towards which the vehicle flow under consideration is moving.
Drain	A channel formed at the surface or a culvert, pipe or other similar construction for drainage.
Drainage	Natural or artificial means of intercepting and removing surface or sub-surface water (usually by gravity).

Term	Definition
Drainage Area	The area draining into a stream at a given point. The area may be of different sizes for surface runoff, subsurface flow, and base flow, but generally the surface flow area is used as the drainage area.
Drift	Debris that drifts on or near the water surface.
Driveway	A defined area used by vehicles travelling between a carriageway and a property adjacent or near to the road. See also access way.
Drop Inlet	Drainage inlet with a horizontal or nearly horizontal opening.
Dual carriageway	See divided road.
Economic analysis benefit-cost analysis (BCA) cost-benefit analysis (CBA)	Means of analysing investment or policy decisions by comparing the benefits and costs of such decisions as far as practicable in monetary terms. Future costs and benefits are discounted to represent present day values in a given or 'present' year
Edge break	A road failure where the edge of the seal breaks away.
EEdge line (pavement edge line)	A line used to differentiate the outer edge of the traffic lanes from the shoulder.
Effective Duration	The time in a storm during which the water supply for direct runoff is produced. Also used to mean the duration of excess rainfall.
Effective Particle Size	The diameter of particles, spherical in shape, equal in size and arranged in a given manner, of a hypothetical sample of granular material that would have the same transmission constant as the actual material under consideration.
Electronic Toll Collection System (ETC)	ETC is a system which allows drivers to automatically pay tolls on toll roads without stopping the car.
Embankment	A construction (usually of earth or stone) to raise the ground (or formation) level above the natural surface.
Emergency Services	All local services whether public or private or part of the road management which intervene in the event of an incident on the road, including the police, fire brigades and designated rescue or recovery teams.
Emergency Spillway	A rock or vegetated earth waterway around a dam, built with its crest above the normally used principal spillway. Used to supplement the principal spillway in conveying extreme amounts of runoff safely past the dam.
End Section	A concrete or metal structure attached to the end of a culvert for purposes of retaining the embankment from spilling into the waterway, appearance, anchorage, etc.
Energy Dissipation	The phenomenon whereby energy is dissipated or used up.
Energy Grade Line	A line joining the elevation of energy heads; a line drawn above the hydraulic grade line a distance equivalent to the velocity head of the flowing water at each section along a stream, channel, or conduit.

Term	Definition
Energy Gradient	Slope of the line joining the elevations of total energy along a conduit of flowing water.
Entry ramp	See on ramp.
Environmental capacity	Capacity of a road or area to accommodate moving and/or stationary vehicles with regard to the need to maintain specified environmental standards.
Ephemeral Stream	A stream or reach of a stream that does not flow continuously for most of the year.
Equalizer	A culvert or opening placed where it is desirable to equalize the water head on both sides of the embankment.
Equivalent car unit (ECU)	Measure involving the conversion of different types of vehicles into their equivalent passenger cars in terms of operating characteristics. See also passenger car units (PCU).
Equivalent car units	turning traffic and truck or bus traffic into its equivalent in terms of passenger cars, and the summation of these together with actual passenger cars for an approach.
Equivalent Cross- Slope	An imaginary straight cross-slope having conveyance capacity equal to that of the given compound cross-slope.
Erosion	The wearing away or scouring of material in a channel, opening, or outlet works caused by flowing water.
Escape Ramp/Lane	Facility with an arrester bed or net designed to slow down and stop a heavy vehicle failing to stop on a steep downhill grade
Evapotranspiration	Plant transpiration plus evaporation from the soil. Difficult to determine separately, therefore used as a unit for study.
Excess Rainfall	Direct runoff.
Exfiltration	The process where stormwater leaks or flows to the surrounding soil through openings in a conduit.
Exit ramp	See off ramp.
Expressway	A road mainly for through traffic, usually dual carriageway with full control of access. Intersections are generally grade separated. See also freeway and motorway.
Feather edge	The surface of the pavement layers between the shoulder hinge point and the subgrade surface.
Fetch	The distance the wind blows over water in generating waves.
Fill	1. The depth from the subgrade level to the natural surface. 2. That portion of road where the formation is above the natural surface. 3. The material placed in an embankment. See also embankment.

Term	Definition
Filter	A device or structure for removing solid or colloidal material from stormwater or preventing migration of fine-grained soil particles as water passes through soil. The water is passed through a filtering medium, usually a granular material or finely woven or non-woven cloth.
Filtering	1. The movement of vehicles in one stream of traffic across or into another stream of by gap acceptance. See also merging and weaving. 2. Permitted movement at a signalised intersection, whether specifically signalled or not, which conflicts with another traffic stream permitted to move at the same time. 3. Movement of through traffic along local roads, usually to avoid congested areas on the main arterial road system.
Filtration	The process of passing water through a filtering medium consisting of either granular material of filter cloth for the removal of suspended or colloidal matter.
First coat seal	An initial seal on a prepared basecourse which has not been primed.
Flanking Inlets	Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets is to intercept debris as the slope decreases and act as relief to the inlet at the low point.
Flared Inlet	A specially fabricated pipe appurtenance or a special feature of box culverts. This type of inlet is effective in reducing the calculated headwater.
Flared intersection	An intersection where the carriageway is widened to create passing, storage or speed-change lanes.
Flared Wingwalls	The part of a culvert headwall that serves as a retaining wall for the highway embankment. The walls form an angle to the centreline of the culvert.
Flat channel	A channel with a flat cross-section. Normally found in urban areas.
Flexible pavement	A pavement which obtains its load spreading properties mainly by intergranular pressure, mechanical interlock and cohesion between the particles of the pavement material. In the case of an asphalt pavement, this further depends on the adhesion between the bitumen binder and the aggregate, and the cohesion of that binder. Generally, any pavement in which the high strength Portland cement concrete is not used as a construction layer.
Floating car	Vehicle which is driven so that it travels at the average speed of traffic on a length of road. This is done by ensuring that the vehicle overtakes as many other vehicles as those which overtake it.

Term	Definition
Flood	In common usage, an event that overflows the normal banks. In technical usage, it refers to a given discharge based, typically, on a statistical analysis of an annual series of events.
Flood Frequency	The average time interval, in years, in which a given storm or amount of water in a stream will be exceeded.
Flood of Record	Reference to the maximum estimated or measured discharge that has occurred at a site.
Flood Pool	Floodwater storage elevation in a reservoir. In a floodwater retarding reservoir, the temporary storage between the crests of the principal and emergency spillways.
Flood Routing	Determining the changes in a flood hydrograph as it moves downstream through a channel or through a reservoir (called reservoir routing). Graphic or numerical methods are used.
Floodplain	The alluvial land bordering a stream, formed by stream processes, that is subject to inundation by floods.
Floodwater Retarding Structure	A dam, usually with an earthfill, having a flood pool where incoming floodwater is temporarily stored and slowly released downstream through a principal spillway. The reservoir contains a sediment pool and sometimes storage for irrigation or other purposes.
Flow Concentration	A preponderance of the streamflow.
Flow Distribution	he estimated or measured spatial distribution of the total streamflow.
Flow-Control Structure	A structure, either within or outside a channel, which acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.
Flume	An open or closed channel used to convey water.
following distance	The distance from the front of a vehicle to the rear of the one ahead in the same traffic lane.
footpath (pathway, walkway)	A public way reserved for the movement of pedestrians.
Ford	A location where a highway crosses a river or wash and allowing flow over the highway. Often with cut-off walls and markers.
Formation	The surface of the finished earthworks, excluding cut or fill batters.
Formation level	The general level of the surface of the ground proposed or obtained on completion of the earthworks.
Foundation	The soil or rock upon which a structure rests.
Free Outlet	Those outlets whose tailwater is equal to or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

Term	Definition
Free speed	The speed adopted by a driver when not influenced by the presence of other traffic.
Freeboard	The vertical distance between the level of the water surface, usually corresponding to design flow and a point of interest such as a low chord of a bridge beam or specific location on the roadway grade.
Freeway	A divided highway for through traffic with no access for traffic between interchanges and with grade separation at all intersections. See also expressway and motorway.
French drain	A drain formed of a trench typically 1 m deep by 0.6 m wide containing a porous or open- jointed pipe laid on, and backfilled with, a porous material. The drain is typically used for the collection of surface or ground water. Also known as a maori drain.
Frequency	In analysis of hydrologic data, the recurrence interval is simply called frequency.
Frontal Flow	The portion of flow which passes over the upstream side of a grate.
Froude Number	A dimensionless number that represents the ratio of inertial forces to gravitational forces. High Froude numbers are indicative of high flow velocity and high potential for scour.
Functional classification	Classification of roads into groups according to their function, ranging from, for example, principal routes for communication between major regions and capital cities, to those roads which provide almost exclusively for local residential traffic. Also known as road amenity classification.
Functional Values	Characteristics of surface water and wetlands. These include terrestrial and aquatic wildlife habitat, flood control, groundwater recharge, aesthetics, shore and bank line geometry, and water quality.
G	The acceleration of gravity, $9.81 m/s^2$.
Gabion	A rectangular basket made of steel wire fabric or mesh that is filled with rock of suitable size. Used to construct flow-control structures, bank protection, groins, and jetties.
Gap	The time interval between the departure at a point of one vehicle and the arrival at the same point of the next vehicle.
Gap acceptance	The acceptance of a gap in a traffic stream by a driver or pedestrian wishing to enter or cross that traffic stream or a driver in the opposing traffic stream wishing to overtake. See also critical acceptance gap.
General Scour	Scour involving the removal of material from the bed and banks across or most of the width of a channel and is not localized at an element such as a pier, abutment, or other obstruction to flow. Termed contraction scour.

Term	Definition
Generated traffic	1. Traffic created by a new or improved facility as a distinct from traffic which is diverted to a facility and normal traffic growth. 2. Traffic created by changes in land use. See also traffic.
Grade	1. A length of carriageway sloping longitudinally. 2. The rate of longitudinal rise or fall of a carriageway with respect to the horizontal, expressed as a ratio or as a percentage. Also termed gradient. 3. To design the longitudinal profile of a road. 4. To secure a predetermined level or inclination to a road or other surface. 5. To shape or smooth an earth, gravel, or other surface by means of a grader or similar implement. 6. To mix aggregates according to a particle size distribution.
Grade line (longitudinal section)	A vertical section, usually with an exaggerated vertical scale, showing the existing surface levels along a road centreline, or other specified line. It commonly also shows the levels to which the road is to be constructed or reconstructed. See also profile and longitudinal profile.
Grade separation	The separation of road, rail or other traffic so that crossing movements which would otherwise conflict are effected at different elevations. See also underpass and overpass.
Graded Filter	An aggregate filter that is proportioned by particle size to allow water to pass through at a specified rate while preventing migration of fine-grained soil particles without clogging.
Grade-separated intersections	Intersections where conflicting traffic streams are separated physically at different levels and meet via merges.
Grass verge	Grass area on side of road.
Grate	A grid of metal or other suitable material to prevent debris from entering a drain or pit or mud tank and to provide protection for pedestrians and vehicles.
Grate Inlet	Drainage inlet composed of a grate in the roadway section or at the roadside in a low point, swale, or ditch.
Groin	A structure in the form of a barrier, placed oblique to the primary motion of water, designed to control movement of bed load. Groins are usually solid but may be constructed with openings to control elevations of sediments.
ground level	The reduced level of any particular point on the surface of the ground.
Groundwater	Subsurface water occupying the saturation zone, that feeds wells and springs, or a source of base flow in streams. In a strict sense, the term applies only to water below the water table. Also called phreatic water.
Guardrail	See road safety barrier.

Term	Definition
Guide Banks	Embankments built upstream from one or both abutments of a bridge to guide the approaching flow through the waterway opening.
Gutter	That portion of the roadway section adjacent to the kerb that is used to convey storm runoff water.
H	Total energy head loss, measured in meters.
Half-diamond interchange	An interchange having single ramps in only two quadrants on the same side of the minor road. See also split diamond interchange.
Haul	The distance through which material is transported between points of loading and unloading.
Hc	The height of the hydraulic grade line above the outlet invert, in meters.
HCV	Heavy commercial vehicle.
H_E	Entrance head loss, measured in meters.
Head	The height of water above any datum.
Head Cutting	Channel degradation associated with abrupt changes in the bed elevation (head-cut) that migrates in an upstream direction.
Headloss	A loss of energy in a hydraulic system.
Headwall	The structural appurtenance usually applied to the end of a culvert to control an adjacent highway embankment and protect the culvert end.
Headwater, H_w	That depth of water impounded upstream of a culvert due to the influence of the culvert constriction, friction, and configuration.
Headway	The time interval between the passage of consecutive vehicles passing a given point, measured from front to front of the vehicles.
HHeavy vehicle	A truck having a tare weight in excess of 3 tonnes.
H_f	The friction headloss, measured in meters.
High shoulder	A road fault in which the shoulder is too high relative to the carriageway.
High Speed Roads	Roads with speed limit or operating speed of 70km/h or more
Highwater Elevation	The water surface elevation that results from the passage of flow. It may be "observed highwater elevation" as a result of an event, or "calculated highwater elevation" as part of a design process.
Highway	A principal road in a road system.
Historical flood	A past flood event of known or estimated magnitude.
holding line	See limit line.
Holding line	is to stop, normally used at intersections in conjunction a with 'STOP' and 'GIVE WAY' signs.
Horizontal alignment	Horizontal alignment is the positioning of a roadway, as shown in the plan view, using a series of straight lines called tangents connected by circular curves.

Term	Definition
Horizontal clearance	Lateral separation between an object the edge of carriageway
horizontal curve	A curve in the plane or horizontal alignment of a carriageway.
Hydraulic Grade Line	A profile of the piezometric level to which the water would rise in piezometer tubes along a pipe run. In open channel flow, it is the water surface.
Hydraulic Gradient	The slope of the hydraulic grade line.
Hydraulic Head	The height of the free surface of a body of water above a given point.
Hydraulic Jump	A hydraulic phenomenon, in open channel flow, where supercritical flow is converted to subcritical flow. This can result in an abrupt rise in the water surface.
Hydraulic Radius	A measure of the boundary resistance to flow, computed as the quotient of cross-sectional area of flow divided by the wetted perimeter. For wide shallow flow, the hydraulic radius can be approximated by the average depth.
Hydraulic Roughness	A composite of the physical characteristics that influence the flow of water across the earth's surface whether natural or channelized. It affects both the time response of a watershed and drainage channel, as well as the channel storage characteristics.
Hydraulics	The characteristics of fluid mechanics involved with the flow of water in or through drainage facilities.
Hydrograph	A graph showing, for a given point on a stream or for a given point in any drainage system, the discharge, stage, velocity, or other property
Hydrologic Soil Group	A group of soils having the same runoff potential under similar storm and cover conditions.
Hydrologic Soil-Cover Complex	A combination of a hydrologic soil group and a type of cover.
Hydrology	The study of the occurrence, circulation, distribution, and properties of the waters of the earth and its atmosphere.
Hyetograph	A graphical representation of average rainfall, rainfall-excess rates, or volumes over specified areas during successive units of time during a storm.
impedance	Measure of the difficulty of travelling through a network. Can be travel time, distance, cost or some combination of these.
Impermeable Strata	A stratum with a texture that water cannot move through perceptibly under pressure ordinarily found in subsurface water.
Impervious	Impermeable to the movement of water.
Improved Inlet	Flared, depressed, or tapered culvert inlets that decrease the amount of energy needed to pass the flow through the inlet and thus increase the capacity of culverts.

Term	Definition
Induced traffic	Additional traffic resulting from some improvement in a road or in traffic arrangements. See also traffic.
Infiltration	That part of rainfall that enters the soil. The passage of water through the soil surface into the ground. Used interchangeably herein with percolation.
Infiltration Rate	The rate at which water enters the soil under a given condition. The rate is usually expressed in millimetres per hour or day, or cubic meters per second.
Inflow	The rate of discharge arriving at a point (in a stream, structure, or reservoir).
Initial Abstraction (I_a)	When considering surface runoff, la is all the rainfall before runoff begins. When considering direct runoff, la consists of interception, evaporation, and the soil-water storage that must be exhausted before direct runoff may begin. Sometimes called 'initial loss."
Inlet	A structure for capturing concentrated surface flow. May be located along the roadway, in a gutter, in the highway median, or in a field.
Inlet Efficiency	The ratio of flow intercepted by an inlet to the total flow.
Inlet Time	The time required for stormwater to flow from the most distant point in a drainage area to the point at which it enters a storm drain.
Intensity	The rate of rainfall upon a watershed, usually expressed in millimetres per hour.
Interception	Precipitation retained on plant or plant residue surfaces and finally absorbed, evaporated, or sublimated. That which flows down the plant to the ground is called "streamflow" and not counted as true interception.
Interceptors drain	A type of side drain that prevents water from flowing towards the road and is normally sited away from the road.
Interchange	A grade separation of two or more roads with one or more interconnecting carriageways.
Interchange ramp	A carriageway within an interchange providing for travel between two arms (legs) of the intersecting roads.
Intersection	A place at which two or more roads cross at grade or with grade separation.
Intersection angle	1. The angle between two successive straights on the centreline of a carriageway. 2. The angle between the centrelines of two intersecting carriageways.
Intersection at grade	An intersection where carriageways cross at a common level.
Intersection leg (intersection arm)	Any one of the carriageways radiating from and forming part of an intersection.
Intersection point	The point where the two tangents to a curve or two grades meet.

Term	Definition
Intersectional friction	The retarding effect on traffic movement caused by potential and actual traffic conflicts at an intersection or the merger of two moving streams of traffic.
Invert	The lowest portion of the internal surface of a drain or culvert.
Inverted Siphon	A structure used to convey water under a road using pressure flow. The hydraulic grade line is above the crown of the structure.
Ìsland	See traffic island.
Isohyet	A line on a map, connecting points of equal rainfall amounts.
Jetty	An elongated obstruction projecting into a stream to control shoaling and scour by deflection of currents and waves. They may be permeable or impermeable.
Journey	Movement involving one or more trips, eg. a 'journey to work', which could involve a direct trip to work or an intermediate stop for some other but secondary, purpose; an 'origin to origin' journey, which could involve several trips, each for a particular purpose. Home-to- home journeys have also been termed tours. See also trip.
Journey time	See travel time.
Junction	A place where two or more roads meet.
Kerb	A raised border of rigid material formed at the edge of a carriageway or pavement.
Kerb and channel	Combined kerb and drainage channel.
Kerb Extension (Build-out, Bell-out)	Local physical extension of the kerbline which narrows down the carriageway
Kerb-Opening Inlet	Drainage inlet consisting of an opening in the roadway kerb.
Lag Time, T_L	The differences in time between the centroid of the excess rainfall (that rainfall producing runoff) and the peak of the runoff hydrograph. Often estimated as 60 percent of the time of concentration ($T_L = 0.6T_c$)
Land use	Use to which land is put, eg. Residential, commercial, open space. In transport analysis the term encompasses measures of social and economic activity that take place on the land, eg. size of population, number of employees.
Lane	1. A narrow road. 2. See traffic lane.
Lane drop	Diverge areas where one or more traffic lanes branches off, resulting in a decrease of mainline traffic lane
Lane gain	Merge areas where one or more traffic lanes join the road, resulting in an increase of mainline traffic lane
Lane line	A line (usually painted) other than the centre line which divides adjacent traffic lanes.

Term	Definition
Lane numbering	On a multilane roadway, the traffic lanes available for through traffic travelling in the same direction are numbered from left to right, when facing in the direction of traffic flow.
Lane separator	A separator provided between lanes carrying traffic in the same direction, to discourage or prevent lane changing, or to separate a portion of a speed change lane from through lanes.
Lanes	Since a single column of vehicles is defined as a line of vehicles moving in one direction, in cases of service roads where vehicles moving in both directions pass by each other, there is a carriageway but no lanes (sometimes referred to a single lane road for convenience). Moreover, to ensure safe and smooth traffic, it is necessary to have the required width corresponding to the sizes and speeds of running vehicles. Lanes comprise auxiliary lanes, speed change lanes and others that are endowed with special purposes. Inside the carriageway, lanes are indicated by lane markings.
Lay by	An area along roads or highways where vehicles may draw out of the through carriageway and park.
left turn lane	See turning lane.
Left-right staggered junction	A junction in which a driver turns to his left on entering an intersecting carriageway and then to his right in order to continue his route. The preferred configuration is right-left staggered junction intersecting.
Levee	A linear embankment outside a channel for containment of flow.
level crossing	Railway crossing at the same level as a road.
Level of service (LOS)	An index of the operational performance of traffic on a given traffic lane, carriageway or road when accommodating various traffic volumes under different combinations of operating conditions.
Light Detection and Ranging (LiDAR)	LiDAR is a method for determining ranges (variable distance) by targeting an object or a surface with a laser and measuring the time for the reflected light to return to the receiver.
Limit line	A transverse line, or lines, marked on a pavement to indicate the position at which a vehicle
Line of sight	The direct line or uninterrupted view between a driver and an object of specified height above the carriageway in his lane of travel.
Link	1. Road network: The portion of road between two intersections. Its basic characteristics are length, vehicle speeds, travel times and number of lanes. 2. Public transport network: The portion of a route between stations or bus stops or tram stops or ferry wharves. Its basic characteristics are length, the transport modes which use it, vehicle speeds, travel times and frequencies. 3. Traffic assignment: A connection between two nodes.

Term	Definition
Local area traffic management	Analysis of traffic characteristics, and the implementation of vehicle control measures within local area.
Local road (local street)	A road or street used primarily for access to abutting property. See also road.
Local Scour	Scour in a channel or on a flood plain that is localized at a pier, abutment, or other obstruction to flow. The scour is caused by the acceleration of the flow and the development of a vortex system induced by the obstruction to the flow.
Local street	See local road
longitudinal profile	The shape of a pavement surface measured as vertical distances from some datum parallel to the traffic. See also transverse profile.
longitudinal section (long section)	See grade line.
Luminous intensity	The quantity of visible light that is emitted in unit time per unit solid angle. The unit for the quantity of light flowing from a source in any one second (the luminous power, or luminous flux) is called the lumen.
Major / minor road system	System of control of a road network in which the priority at intersections or junctions is determined by the relative importance of the approach roads.
Manhole	A structure used to access a drainage system.
Manning's "n”	A coefficient of roughness, used in a formula for estimating the capacity of a channel to convey water. Generally, "n" values are determined by inspection of the channel.
manoeuvre	Any action on the part of a driver with regard to merging, weaving or overtaking.
Manual on Uniform Traffic Control Devices (MUTCD)	The Manual on Uniform Traffic Control Devices for Streets and Highways, or MUTCD defines the standards used by road managers nationwide to install and maintain traffic control devices on all public streets, highways, bikeways, and private roads open to public travel.
Marginal strip	Marginal strips have similar pavement thickness as the shoulder or median strip and are indicated by means of markings or different pavement colouring. The objective of a marginal strip is to secure marginal allowance for the running of vehicles and to induce the eyeline of drivers.
markings	Any lines painted on the road to control traffic movement or parking.
mass haul diagram	A graph on a base of distance showing cross-sectional area of cutting and fill on which the destination of the material from each cutting is indicated. It is often reduced to a diagram showing rectangles having areas proportional to cut or fill volumes.

Term	Definition
Mass Inflow Curve	A graph showing the total cumulative volume of stormwater runoff plotted against time for a given drainage area.
Maximum Probable Flood	The maximum probable flood is the greatest flood that may reasonably be expected, taking into collective account the most adverse flood related conditions based on geographic location, meteorology, and terrain.
Mean Daily Discharge	The average of mean discharge of a stream for one day, usually given in m^3/s.
Meanders	The changes in direction and winding of flow that are sinuous in character.
Median (Central Reserve)	The area between opposing traffic comprising the physical separation and shoulders on a divided road
Median barrier	A device used on multi-lane roads to keep opposing traffic in prescribed carriageways. See also guardrail.
Median island	A short length of median serving of localised purpose in an otherwise undivided road.
Median lane	A speed change lane within the median to accommodate right turning vehicles.
Median opening	A speed change lane within the median to accommodate right turning vehicles.
Merging	The converging of separate streams of traffic into a single stream. See also filtering and weaving.
Metered access	System allowing vehicles to enter a road only when traffic condition permits. See also access control.
Migration, Channel	Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.
Minimum turning path	The path of a designated point on a vehicle making its sharpest turn.
Minimum turning radius	The path of the minimum turning path of the outside of the outer front tyre of a vehicle. See also turning circle.
Mixed vehicle lane (Mixed traffic lane)	Mixed traffic lane is defined as a street lane for more than two kinds of vehicles to. drive on it.
Mode	Method of transport, e.g., motor vehicle travel (as driver or passenger), bus, light rail and walking.
Motorway	A defined class of road for which certain activities or uses are restricted or prohibited by legislative provision. See also expressway and freeway.
Mountable kerb	A kerb designed to define the edge of a carriageway, but which may be mounted or driven across, if the need arises, with little to risk of damage to a vehicle. See also semi-mountable kerb.
Multiple lanes	A carriageway with more than one traffic lane.
Natural Scour	Scour that occurs along a channel reach due to an unstable stream, no exterior causes.

Term	Definition
Nearside	1. The side of a vehicle closest to the kerb when the vehicle is travelling in the normal direction of travel. 2. The nearside of a road corresponds to the left-hand of the carriageway when looking in the direction of travel.
Nearside (Passenger side)	The right side in the direction of travel where driving is on the right side of a road, or the left side in the direction of travel where driving is on the left side of a road
No overtaking line	A continuous yellow painted line adjacent to the road centreline marking, which indicated that overtaking is not permitted.
Non-motorized Transportation	Description. Non-motorized Transportation (also known as Active Transportation and Human Powered Transportation) includes Walking and Bicycling, and variants such as Small-Wheeled Transport (skates, skateboards, push scooters and hand carts) and Wheelchair travel.
non-mountable kerb	See barrier kerb.
Non-recoverable slope	A non-recoverable slope is a slope which is considered traversable but on which an errant vehicle will continue to the bottom.
Normal Stage	The water stage prevailing during the greater part of the years.
Nth highest hour	The hourly traffic volume that is exceeded during (N-1) hours in the course of a year.
Number plate survey	Survey involving the recording of vehicle licence plate numbers at different locations for the purpose of developing origin-destination and/or travel time data.
Off ramp (exit ramp)	A carriageway to allow vehicles to leave a motorway or expressway.
Off tracking	The radial distance between the turning paths of the centre of the front axle and the centre of the rear axle.
Off-peak hour	A representative hourly flow indicative of the average flow outside the peak period.
Offset	Horizontal distance measured at right-angles to a datum or reference line
Offside	1. The side of a vehicle furthest away from the kerb when the vehicle is travelling in the normal direction of travel. It corresponds to the driver’s side of the vehicle. 2. The offside of a road corresponds to the right-hand of the carriageway when looking in the direction of travel.
On ramp (entry ramp)	A carriageway to allow vehicles to join an expressway or motorway.
One-Dimensional Water Surface Profile	An estimated water surface profile that accommodates flow only in the upstream-downstream direction
One-way road (one-way street)	A road or street on which all vehicular traffic travels in the same direction.
Operating speed	The highest overall speed, exclusive of stops, at which a driver can safely travel on a given section of road under the prevailing traffic conditions.

Term	Definition
Opposing traffic	The traffic stream travelling in the opposite direction to the vehicle or vehicles under consideration.
Optimum speed	The average speed at which traffic must move to attain the maximum traffic volume on a carriageway.
Ordinary High Water	The line on the shore established by the fluctuations of water and indicated by physical characteristics such as clear, natural line impressed on the bank, shelving, changes in the character of soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means that consider the characteristics of the surrounding areas.
Outfall	The point location or structure where drainage discharges from a channel, conduit, or drain.
Over dimension route	Route available for use by over dimension vehicles. See also truck route.
Over dimension vehicle	Vehicle which, due to its weight or dimensions, is precluded by legislation from using public roads without following a prescribed route or obtaining a permit from the relevant traffic authority.
overall travel speed	The total distance traversed by a vehicle divided by the total time required including all traffic delays.
Overland Flow	Runoff that makes its way to the watershed outlet without concentrating in gullies and streams (often in the form of sheet flow).
Overpass	A grade separation where the major road passes over an intersecting minor road or railway. See also grade separation and underpass.
Overtaking	The manoeuvre whereby a vehicle moves from a position behind to one in front of another vehicle travelling in the same direction.
Overtaking distance	The distance required for one vehicle to overtake another vehicle.
Overtaking lane	See passing lane.
Overtaking lanes	The part of a main road that is used for passing other vehicles and is nearest the centre of the road
Overtaking sight distance	The overtaking sight distance is the minimum distance open to the vision of the driver of a vehicle intending to overtake the slow vehicle ahead safely against the traffic in the opposite direction.
Painted Traffic Islands	Traffic islands demarcated by line markings and/or coloured surfacing which are traversable by a vehicle without difficulties.
Parapet	Safety barrier installed on a structure including bridges and retaining walls.
parking capacity	Total number of marked parking spaces provided within a parking facility. Also known as 'static capacity'. See also capacity.

Term	Definition
Parking lane	On urban roads, vehicles make frequent stops to use roadside facilities. Especially on 2-lane roads that have no spare allowance, if one vehicle parks, passing vehicles will be forced out into oncoming traffic, leading to extreme deterioration of traffic capacity and possibly causing accidents. Accordingly, in urban areas, parking lanes are installed in parts.
Parking space	An area intended for occupancy by a single parked vehicle.
Partial-Duration Series	A list of all events, such as floods, occurring above a selected base, without regard to the number, within a given period. In the case of floods, the selected base is usually equal to the smallest annual flood, in order to include at least one flood in each year.
Passenger car	For general traffic engineering use this term includes cars, taxis and station wagons, but does not include motor cycles and light commercial vehicles such as utilities and panel vans, unless otherwise specified.
Passenger car units (PCU)	A measure of traffic flow in terms of an equivalent number of passenger cars. See also equivalent car units.
Passing bay	A short widening of the carriageway provided to allow very slow vehicles to pull aside and be overtaken, usually in very steep terrain.
Passing lane	An auxiliary lane, including diverge and merge tapers, that is provided for slower vehicles to allow them to be overtaken.
Passing place	A widened length of a narrow carriageway at which vehicles can pass each other.
Passively Safe	Objects or areas with physical characteristics which are unlikely to incur serious injuries or fatalities when colliding by or ridden over by an errant vehicle travelling at the prevailing traffic speeds.
Paved shoulder	Part of the pavement outside the carriageway not designated for the normal use of traffic
Pavement	Paved area of the road which may be readily traversed by traffic. That portion of the road that is placed above the design subgrade level for the support of, and to form a running surface for, vehicular traffic.
Pavement markings	Surface markings, raised pavement markers, traffic domes and the like placed on the pavement for the control and guidance of traffic.
Peak Discharge	Maximum discharge rate on a runoff hydrograph.
Peak hour	The hour of the day having the highest traffic volume during the peak period.
Peak period	The period of the day having the highest volume of traffic.
Peak traffic flow	The traffic volume during a time period of specified length during which such volume is at its maximum.
Pedestrian crossing	A specially marked area giving legal rights to pedestrians crossing the road.

Term	Definition
Pedestrian walkway	The pedestrian walkway is the part of a road that is intended solely for the passage of pedestrians, and it is physically separated from the carriageway and other parts by means of kerbstones, fences or other structures. Accordingly, a part that is separated only by road markings or lane markings is not regarded as a pedestrian walkway.
Percentile speed	The spot speed at a particular location which is not exceeded by a specified percentage of all traffic passing.
Percolation	The movement or flow of water through the interstices or the pores of a soil or other porous medium. Used interchangeably herein with infiltration.
Perennial Stream	A stream or reach of a stream that flows continuously for all or most of the year.
Permeability	The property of a material that permits appreciable movement of water through it when it is saturated, and movement is actuated by hydrostatic pressure of the magnitude normally encountered in natural subsurface water.
Pervious Soil	Soil containing voids through which water will move under hydrostatic pressure.
pH	The reciprocal of the logarithm of the Hydrogen ion concentration. The concentration is the weight of hydrogen ions, in grams, per litter of solution. Neutral water has a pH value of 7.
Physical Traffic Islands	Traffic islands ed by line markings and/or coloured surfacing which in principle are traversable by a vehicle without difficulties.
Platooning	Platooning is a technology supervised by on-board drivers that allows multiple trucks to travel in a platoon on expressways, telecommunicate driving conditions to each other in real time, autonomously maintain distance between each other, and change or stay in lanes in coordination.
Point of Intersection (PI)	The point where two successive tangents intersect.
Point Rainfall	Rainfall at a single rain gauge.
Precipitation	The process by which water in liquid or solid state falls from the atmosphere.
preliminary engineering	Work of locating and designing, making surveys and maps, preparing specifications and estimates, and doing other engineering work before letting a contract for construction of a transport project. See also design process.
Principal Spillway	Conveys all ordinary discharges coming into a reservoir and all of an extreme discharge that does not pass through the emergency spillway.
Priority road	A road on which traffic has right-of-way over entering or crossing traffic at all intersections.

Term	Definition
Priority rule	Traffic regulation which assigns priority to one stream of traffic at an intersection.
Profile	1. A construction aid erected to assist in establishing a batter slope. 2. The shape of a pavement surface measured in a vertical plane, from a datum, parallel to the traffic flow. See also longitudinal section and longitudinal profile.
Project Affected Person (PAP)	(PAP) means the people directly affected by land acquisition for a community project through loss of part or all of their assets whether temporarily or permanently including land, houses, other structures, businesses, crops/trees, or other types of assets.
Property line (boundary, boundary line, frontage)	The legal boundary between a road reserve and the adjacent land.
Public road	A public place which as been provided for use by the public for traffic movement and has been declared, or proclaimed, notified or dedicated.
Public transport	Service by bus, rail, taxi or other means which provides transport to the public on a regular basis for payment of a prescribed fare.
Radial road	Road radiating from the centre of an urban area. See also road.
Rainfall Excess	The water available to runoff after interception, depression storage, and infiltration have been satisfied.
Rainfall Intensity	Amount of rainfall occurring in a unit of time, converted to its equivalent in millimetres per hour at the same rate.
Raised pavement marker	A device used to supplement or replace traffic lines on the road surface. It may be retroreflective.
Raised reflective pavement marker (RRPM)	Raised pavement marker with reflectors, fixed in the carriageway.
ramp	1. Carriageway within an interchange providing for travel between two arms (legs) of the intersecting roads. 2. Traffic assignment: a link between a freeway node and an arterial node. 3. Sloping section of road, such as connecting different levels in a car park.
Ramp terminal	The point on an interchange ramp at which it intersects with a surface road or street.
Rating Curve	A graphical plot relating stage to discharge.
Reach	A length of stream or valley, selected for purpose of study.
Reaction time	The time between the driver's reception of a stimulus and his taking the appropriate action, e.g., application of the brakes, response to signals.
Recession Curve	The receding portion of a hydrograph, occurring after excess rainfall has stopped.

Term	Definition
Recharge	Addition of water to the zone of saturation from precipitation or infiltration.
Recharge Basin	A basin excavated in the earth to receive the discharge from streams or storm drains for the purpose of replenishing groundwater supply.
Reconnaissance survey	The reconnaissance survey is an extensive study of an entire area that might be used for a road or airfield. Its purpose is to eliminate those routes or sites which are impractical or unfeasible and to identify the more promising routes or sites.
Recoverable slope	A recoverable slope is a slope on which a motorist may, to a greater or lesser extent, retain or regain control of a vehicle by slowing or stopping
Refuge area	An area, usually in the centre of a road, set aside for the exclusive of pedestrians.
Regional Analysis	A regional study of gauged watersheds that produce regression equations relating various watershed and climatological parameters to discharge. Use for design of ungauged watershed with similar characteristics.
Reseal	A seal applied to an existing sealed, asphalt, concrete or timber surface.
Reservoir Routing	Flood routing of a hydrograph through a reservoir.
Residential area	land largely occupied for residential purposes but which includes small shopping centres and ancillary features and primary schools.
Gretaining wall	A wall constructed to resist lateral pressure from the adjoining ground or to maintain in position a mass of earth.
Retard	A structure designed to decrease velocity and induce silting or accretion. Retard type structures are permeable structures customarily constructed at and parallel to the toe of slope.
Retention Basin	A basin or reservoir where water is stored for regulating a flood, that does not have an uncontrolled outlet. The stored water is disposed through infiltration, injection (or dry) wells, or by release to the downstream drainage system after the storm event. The release may be through a gate-controlled gravity system or by pumping.
Reverse curve	A curve consisting of two arcs of the same or different radii curving in opposite directions and having a common tangent or a transition curve at their junctions.
Reversible lane	A lane or carriageway assigned for the use of traffic in one direction at certain times, and in the opposite direction at other times.
Revetment	A rigid or flexible armour placed on a bank or embankment as protection against scour and lateral erosion.

Term	Definition
Ribbon development	Development, usually along a major road, that presents a continuous residential, shopping or business frontage having greater vehicular and pedestrian activity than land away from the road.
Right-left staggered intersection	An intersection junction in which a driver turns to his right on entering the intersecting carriageway and then to his left in order to continue his route. This is the preferred configuration to a left-right staggered intersection.
Right-of-Way (RoW)	The right of way is the total land area acquired for the construction of the roadway. RoW is the area of the road acquired for carriages way + other necessities + future extension, along its alignment.
Right-turn lane	See turning lane.
Ring road	A road which goes around, rather than through, an urban area. See also road and circumferential road.
Riprap	Medium to large size rock protection, against scour, applied (usually by dumping) to the face of an embankment.
Road	A route trafficable by motor vehicles.
Road base	See basecourse.
Road capacity	Maximum number of vehicles or pedestrians that can pass over a given section of a lane, road or footpath in one direction, or both directions for a two-lane or three-lane road, during a given time period under prevailing road and traffic conditions. It is the maximum flow rate that has a reasonable expectation of occurring. In the absence of a time modifier, capacity is an hourly volume. The capacity would not normally be exceeded without changing one or more of the conditions that prevail. In expressing capacity, it is essential the prevailing road and traffic conditions under which the capacity is available. See also capacity.
Road classification	Consistent terminology and designation of roads to provide a basis for planning and decision making by national and local government agencies responsible for various aspects of road administration.
Road furniture	A general term covering all signs and devices for the control, guidance and safety of traffic, and the convenience of road users.
Road geometry	Geometric design refers to the dimensions and arrangements of the visible features of a roadway. This includes pavement widths, horizontal and vertical alignment, slopes channelization, intersections and other features that can significantly affect the operations, safety and capacity of the roadway network.
Road hierarchy	The grading of roads according to increasing or decreasing importance of their traffic-carrying or other function.
Road hump (speed bump)	A vehicle speed control device in the form of a short, raised section of carriageway.

Term	Definition
Road inventory	Inventory of all road characteristics, ie. control devices, parking restrictions, road widths, number of traffic lanes, etc.
Road reserve	A legally described area within which facilities such as roads, footpaths and associated features may be constructed for public travel. Often called road.
Road Restraint System	A facility on one side of the road designed to prevent vehicles, pedestrians or slow vehicles from crossing to the other side.
Road Restraint System (RRS)	Restraint systems including VRS and fences or parapets for pedestrians
road safety barrier (guardrail)	A rail, or fence, erected to restrain vehicles which are out of control.
roadway	See carriageway.
Roadway Cross- Slopes	Transverse slopes and/or superelevation described by the roadway section geometry. Usually provided to facilitate drainage and/or resist centrifugal force.
Roadworks	A general term for any work on a road for construction, repair or maintenance.
Rotary	See roundabout.
Roughness	1. The consequence of irregularities in the longitudinal profile of a road with respect to the intended profile. It is measured by the unidirectional displacement of a standard (NAASRA) test vehicle relative to its axle, as the vehicle travels over the surface at a standard speed. 2. The estimated measure of texture at the perimeters of channels and conduits. Usually represented by the "n-value" coefficient used in Manning's channel flow equation.
Roundabout (rotary)	An intersection where all traffic travels in one direction around a central island.
Rounding	A curvature or curved section at the intersection of batters providing a transition between the two slopes.
Route	1. That combination of road sections connecting an origin and destination. 2. Traffic assignment: a continuous group of links connecting two centroids that normally requires the minimum time to traverse. 3. The path travelled by a public transport vehicle.
Rumble strip	Strips constructed across the carriageway for the purpose of reducing the speed of vehicles. Also used between opposing traffic lanes as a warning device
Running speed	See average speed.
Runoff	That part of the precipitation that runs off the surface of a drainage area after all abstractions are accounted for.
Runoff Coefficient (c)	A factor representing the portion of runoff resulting from a unit rainfall. Dependent on terrain and topography.

Term	Definition
Safety Barrier (Guardrail, Safety Fence)	Longitudinal VRS along the roadside or median provided for the containment of an errant vehicle in an impact
Safety ramp	A short trafficable spur road, usually with a steep upgrade, provided for emergency use by vehicles on steep downgrades.
Safety space	A space between a hazard or work site and safety barriers etc. to ensure traffic keeps a safe distance from the hazard or from workers on the work site.
Safety zone	An area of carriageway reserved for passengers to wait for trams or busses.
Sag curve	A concave vertical curve in the longitudinal profile of a road.
Satellite images	Satellite images are images of Earth collected by imaging satellites operated by governments and businesses around the world. Satellite imaging companies sell images by licensing them to governments and businesses such as Apple Maps and Google Maps
Saturated Soil	Soil that has its interstices or void spaces filled with water to the point at which runoff occurs.
Saturation flow	1. The flow of vehicles past a point on a carriageway which would be achieved if there is a continuous queue of vehicles upstream of that point, usually expressed as vehicles per hour. 2. The rate of flow of vehicles across a stop line at a signalized approach during the effective green interval if there is a continuous queue of vehicles waiting to move during that time, usually expressed as vehicles per hour of green.
Scour	The result of the erosive action of running water, excavating and carrying away material from the bed and banks of streams.
Scupper	A vertical hole through a bridge deck for the purpose of deck drainage, sometimes a horizontal opening in the kerb or barrier.
Seagull intersection	An intersection where a triangular island is used to separate turning traffic from through traffic in the same carriageway.
Seal	A thin layer of binder sprayed onto a pavement surface and having a layer of aggregate rolled in.
Sealed carriageway	That portion of the road pavement sealed to protect and waterproof the underlying pavement, (inclusive of sealed shoulders) and provide a suitable driving surface for vehicles.
Sealed shoulder	That portion of the sealed carriageway beyond the traffic lane, located between the traffic lane edge line and the edge of seal, generally flush and contiguous with the sealed carriageway.
Sediment Pool	Reservoir storage provided for sediment, prolonging the usefulness of floodwater or irrigation pools.
Sedimentation	The deposition of soil particles that have been carried by flood waters.

Term	Definition
Sedimentation Basin	A basin or tank in which stormwater containing settleable solids is retained for removal by gravity or filtration of a part of the suspended matter.
Seismic map (Seismic hazard map)	Seismic hazard is the hazard associated with potential earthquakes in a particular area, and a seismic hazard map shows the relative hazards in different areas. The maps are made by considering what we currently know about: Past faults and earthquakes.
Selected fill	Fill complying with specified requirements.
SSemi-barrier kerb	A kerb designed to deter vehicles from leaving the carriageway but less restrictive than a barrier kerb. See also barrier kerb.
Semi-mountable kerb	A kerb designed so that it can be driven across in an emergency or on special occasions without damage to the vehicle. See also mountable kerb.
separator (traffic separator)	An area separating adjacent carriageways, upon each of which traffic usually moves in the same direction.
Service area	An area with access to and from an adjacent highway or motorway, used to provide services and amenities to road users.
Service Roads	The service road (side road) is a road to secure access to/from road side where to/from the main road has a difficulty due to embankment or cutting, necessary section to install continues soundproof and others. Although the specifications for the service road shall apply as a carriageway, which are determined according to the design speed, it should be separately prescribed from the main road.
Service volume	The maximum traffic volume that can be accommodated on a carriageway or road at a specified level of service.
Services	Supply lines for water, electricity, gas, telephones, etc.
Shared zone	Residential road surfaced and marked such that motorists recognise it as an area shared by both vehicles and pedestrians. A reduced speed limit is necessary in such areas.
Shift	1. An alteration to the previously adopted position of the centreline. 2. The radial displacement of a circular curve from the tangent line resulting from the introduction of a transition curve.
Shoulder	Part of the pavement outside the carriageway not designated for the normal use of traffic
Shoulder drain	A drain through the shoulder to drain the subgrade.
Shoulder hinge point	In the cross-section of a road, the point at which the side slope would intersect with the unsealed shoulder, or in the absence of an unsealed shoulder, the sealed shoulder.
Shy line	The offset from the edge of a traffic lane beyond which a roadside feature does not cause drivers to slow unnecessarily, or steer away from, at their current travel speed.

Term	Definition
Side cut	That portion of a road on sloping ground where one edge only of the formation is in cut and the other edge is on the natural surface or on fill.
Side drain	A longitudinal surface drain or ditch usually U-shaped and generally located between the surface water channel and the legal road boundary. In some situations, the side drain may run immediately adjacent to the road pavement and collect surface water runoff from the road surface and adjacent land.
Side slope	The uniform side slope of walls, banks, cuttings or embankments, expressed as a ratio of 1 vertical on x horizontal as distinct from grade.
Sideways force coefficient	See coefficient of side friction.
Sight distance	The distance measured along the carriageway over which objects of defined height are visible to a driver.
SSight triangle	The area of land between two intersecting roadways over which vehicles on both roadways are visible to each driver.
SSite investigation	The examination of all those characteristics of a site which might affect the planning, design, construction and operation or performance of any engineering works on the site.
Skew	A measure of the angle of intersection between a line normal to the roadway centreline and the direction of the streamflow at flood stage on the lineal direction of the main channel.
Skewness	When data are plotted in a curve on log-normal paper, the curvature is skewness.
Skid resistance	The frictional resistance provided by the pavement surface to the vehicle tyres during braking or cornering manoeuvres. It is usually measured on wet surfaces.
Slip	A movement or fall of earth in a cut or bank.
Slip lane	A lane provided for left turning vehicles allowing them to avoid stopping at an intersection.
Slip road	A carriageway provided for vehicles to transfer between two adjacent carriageways having the same direction of travel.
Slope	1. The inclination of a surface with respect to the horizontal expressed as rise or fall in a certain longitudinal distance. 2. An inclined surface.
Slot drain	A line of slots in the pavement to allow water to drain -usually into a piped stormwater system.
Slotted Drain Inlets	Drainage inlets composed of a continuous slot built into the top of a pipe which serves to intercept, collect, and transport the flow
Slow Vehicle	Traffic comprising bicycles, electric bicycles, animal-drawn carts, animal herds etc.
Soffit	The inside top of the culvert or storm drain pipe.

Term	Definition
Soil erodibility	Soil erodibility. Erodibility describes or is a measure of the inherent resistance of geologic materials (soils and rocks) to erosion. Highly erodible geologic materials are readily displaced and transported by water.
Soil Porosity	The percentage of the soil (or rock) volume that is not occupied by solid particles, including all pore space filled with air and water.
Soil-Water-Storage	The amount of water the soils (including geologic formations) of a watershed will store at a given time. Amounts vary from watershed to watershed. The amount for a given watershed is continually varying as rainfall or evapotranspiration takes place
Space mean speed	The average of the speeds of vehicles within a given space or section of road at a given instant, or the average speed of a specified group of vehicles based on their average travel time over a section of road.
Span	1. The distance between the centres of adjacent supports of a bridge, beam or truss. 2. The superstructure of a bridge between two adjacent supports.
speed (85th percentile)	The speed at or below which 85 percent of the vehicles travel.
Speed change lane	An auxiliary lane, including tapered areas, primarily for the acceleration or deceleration of vehicles entering or leaving the through traffic lanes.
Speed environment	A basic design parameter for a section of road, representing the uniform desired speed of the 85th percentile driver. It can be measured on existing roads as the 85th percentile of the speed distribution on the longer straights or large radius curves over the section.
Speed hump	A vehicle speed control device in the form of a short, raised section of carriageway.
Speed survey	A traffic survey involving the measurement of the speed of vehicles.
Splash-Over	That portion of frontal flow at a grate that splashes over the grate and is not intercepted.
Splay	The triangular setting back of property lines adjacent to an intersection.
Split-diamond interchange	The combination of two half-diamond interchanges of opposite direction serving two closely- spaced surface roads, such that the movements available to turning traffic are substantially similar to those provided at a full-diamond interchange.
Splitter island	A short median island in the approach to an intersection.
Spot speed	The speed of a vehicle at a specified point.
Spread	The accumulated flow in and next to the roadway gutter. This water often represents an interruption to traffic flow during rainstorms. The lateral distance, in feet, of roadway ponding from the kerb.

Term	Definition
Spur	A structure, permeable or impermeable, projecting into a channel from the bank for the purpose of altering flow direction, inducing deposition or reducing flow velocity along the bank.
Spur Dike	A dike placed at an angle to the roadway for the purpose of shifting the erosion characteristics of stream flow away from a drainage structure. Often used at bridge abutments.
Stage	Height of water surface above a specified datum.
Stage construction	1. A construction sequence in which a road is initially constructed to an operational or structural standard lower than its ultimate standard and is subsequently upgraded to the ultimate standard. 2. Construction affecting an existing road in which work is carried out in a number of well-defined stages aimed at minimising disruption to traffic.
Stage-Discharge Relationship	A correlation between stream flow rates and corresponding water surface elevations. Sometimes referred to as the Rating Curve of a stream cross-section.
Staggered T intersection	An intersection in which the carriageway of one road is offset so as not to be continuous across the other road.
Station	1. 1. Location at which traffic survey data is collected. 2. Stopping place on a railway for trains to load and unload passengers and freight. 3. The distance of a point along a control line, measured from a datum point. See also chainage.
Stilling Basin	An energy dissipater placed at the outlet of a structure.
Stock underpass	A structure constructed to permit the passage of stock beneath a road.
Stop line	A transverse line or lines behind which vehicles must stand when stopped by police control, traffic signals or a regulatory sign.
Stopping distance	The distance travelled by a vehicle between the time when the driver receives a stimulus signifying a need to stop and the time the vehicle comes to rest. Usually, the minimum distance is implied.
Storage- Indication Method	A flood-routing method also often called the modified Puls method.
Storage lane	An auxiliary lane, usually at intersections, primarily for use by vehicles waiting to turn or cross.
Storm Drain	The water conveyance elements (laterals, trunks, pipes) of a storm drainage system, that extend from inlets to outlets.
Storm Duration	The period or length of storm.
Stream Contraction/Constriction	A narrowing of the natural stream waterway. Usually in reference to a drainage facility installed in the roadway embankment.

Term	Definition
Stream Reach	A length of stream channel selected for use in hydraulic or other computations.
Street	A road that has mainly continuous housing or buildings on one side or both. It provides access to houses, buildings, shops, etc. with frontages onto the street. A street, by definition, is therefore found only in an urban area.
Sub-arterial road	A road connecting arterial roads to areas of development and carrying traffic directly from one part of a region to another. See also road.
Sub-base	The material laid on the subgrade below the base either for the purpose of making up additional pavement thickness required, to prevent intrusion of the subgrade into the base, or to provide a working platform.
Subgrade	The trimmed or prepared portion of the formation on which the pavement is constructed. Generally taken to relate to the upper line of the formation.
Subgrade drain	A subsoil drain to drain water from the sub-grade.
Subgrade surface	The surface of the formation, excluding batter slopes, upon which the carriageway is constructed.
Submerged Inlets	Inlets of culverts having a headwater greater than about 1.2* D.
Submerged Outlets	Submerged outlets are those culvert outlets having a tailwater elevation greater than the soffit of the culvert.
Subsoil (subsurface) drain	A drain below the ground surface, the lower portion, or all, of which collects subsurface water throughout its length.
Substructure	In a bridge, the piers and abutments (including wing walls) which support the superstructure.
Subway	A structure constructed to permit the passage of pedestrians, cycles or stock beneath the road or railway or vehicles beneath the railway.
Summit curve (crest curve)	A convex vertical curve in a longitudinal profile of the road.
Sump	1. A concrete pit at the end of a water channel to settle out solids before the flow enters a pipe drain. Also known as catch pit and mud tank. 2. A hole or depression into which water is drained.
Superelevation	Superelevation is the transverse slope provided to counteract the effect of centrifugal force and reduce the tendency of vehicle to overturn and to skid laterally outwards by raising the pavement outer edge with respect to inner edge.
Superflood	Flood used to evaluate the effects of a rare flow event; a flow exceeding the 100-year flood. It is recommended that the superflood be on the order of the 500-year event or a flood 1.7 times the magnitude of the 100-year flood if the magnitude of the 500-year flood is not known.

Term	Definition
Suppressed traffic	Reduction in traffic volumes resulting from a change in traffic arrangements.
Surface Runoff	Total rainfall minus interception, evaporation, infiltration, and surface storage, and that moves across the ground surface to a stream or depression.
Surface Storage	Stormwater that is contained in surface depressions or basins.
Surface Water	Water appearing on the surface in a diffused state, with no permanent source of supply or regular course for a considerable time; as distinguished from water appearing in water courses, lakes, or ponds.
Surface water channel	An open drain or ditch formed for the collection and drainage of water runoff from the road's surface. The width of the channel shall be a minimum of 1.0 metre (0.5 metre either side of the invert).
Surfacing	The uppermost part of a pavement specifically designed to resist abrasion from traffic and to minimize the entry of water. It may be a sprayed seal, asphalt or other material.
Survey	See cordon survey, number plate survey and travel time survey.
Swale	An open vegetated drainage channel or shallow troughlike depression explicitly designed to carry, detain, partly treat and promote the filtration of stormwater runoff.
Swept path	The area which is traced by the extremities of the bodywork of a vehicle while turning.
Swept width	The radial distance between the innermost and outermost turning paths of a vehicle.
Synthetic Hydrograph	A graph developed for an ungauged drainage area, based on known physical characteristics of the watershed basin. A hydrograph determined from empirical rules.
T intersection (T junction)	A junction in which one road terminates approximately at right-angles to a through road, e.g., an intersection shaped like a 'T'.
Table drain	The side drain of a road adjacent to the shoulders, having its invert lower than the pavement base and being part of the formation.
Tailwater, TW	The depth of flow in the stream directly downstream of a drainage facility. Often calculated for the discharge flowing in the natural stream without the highway constriction. Term is usually used in culvert design and is the depth measured from the downstream flow line of the culvert to the water surface.
Tangent point	The point on the centreline where straight and curve meet tangentially.
Thalweg	The line connecting the lowest flow points along the bed of a channel. The line does not include local depressions.

Term	Definition
Theoretical capacity	The maximum number of vehicles that can pass a given point on a lane or carriageway during one hour under prevailing carriageway and traffic conditions, regardless of their effect in delaying drivers and restricting their freedom to manoeuvre.
Three-centred curve	A compound unidirectional curve consisting of three circular arcs of different radii.
Through car units	A measure used in traffic signal design calculation involving the conversion of each unit of
Through lane	A lane provided for the use of vehicles proceeding straight ahead.
TTidal flow	A means of increasing capacity under conditions of high traffic volume and marked directional split by means of reversible lanes or carriageways.
Time of Concentration, Tc	The time it takes water from the most distant point (hydraulically) to reach a watershed outlet. Tc varies but is often used as constant.
toe	1. The part of the base of a retaining wall which is on the side remote from the retained material. 2. The base of an earthen slope.
Toe drain	An interceptor drain constructed along the bottom of a batter to collect batter run-off.
Toe wall	A low retaining wall constructed at the foot of an earth slope.
Toll	Fee or charge for use of a road, bridge or tunnel.
Toll road	A road, bridge or tunnel available to traffic only upon payment of a fee. See also road.
Toll way	A motorway, for the use of which a toll must be paid.
Topographic map	A topographic map is a map that represents the locations of geographical features. Furthermore, these geographical features can be mountains, valleys, plain surfaces, water bodies and many more.
Townscape	The visual appearance of a town or urban area; an urban landscape.
Tractive Force	The drag on a stream bank caused by passing water, which tends to pull soil particles along with the streamflow, expressed as force per unit area.
Traffic	Any vehicles, persons or animals travelling on a road.
Traffic Calming	The promotion of favourable driving behaviour and control of vehicle speeds to be commensurate with the activities taking place along a road using specific measures
Traffic composition	The fraction (usually expressed as a percentage) of types of vehicles within the total traffic flow.
Traffic count	The process of determining the number of vehicles passing a given point or points during a specified period of time.
Traffic density	The number of vehicles, excluding parked vehicles, per unit length of

Term	Definition
Traffic divider (traffic separator)	A device used on multilane roads to keep traffic in prescribed lanes.
Traffic engineering	The measurement and study of traffic, the determination of its characteristics, and the application of the knowledge so gained to improving the safety, convenience and economy of road transport.
Traffic flow (traffic volume)	The number of vehicles passing a given point during a specified period of time.
Traffic generator	A development or area capable of generating traffic, eg. shopping complex, industrial area, car park.
Traffic island	A defined area, usually at a intersection, from which traffic is excluded and which is used for control of vehicular movements and for pedestrian refuge.
Traffic lane	A portion of the carriageway allotted for the use of a single line of vehicles.
Traffic lanes (travelled way)	That portion of a carriageway ordinarily assigned to moving traffic, and exclusive of shoulders and parking lanes.
Traffic management	The use of traffic engineering techniques to control the flow of traffic.
Traffic pattern	The variation and seasonal fluctuation in traffic flow.
Traffic platoon	A closely spaced group of vehicles on a carriageway, moving, or stopped and ready to move, with relatively large spaces ahead and behind.
Traffic regulations	Statutory rules in relation to driving and/or vehicular requirements, enforceable by law.
Traffic restraint	Procedure or quantitative term indicating an imposed limitation on the volume of motor vehicle traffic.
Traffic separator	See separator.
Traffic sign	A sign to regulate traffic and warn or guide drivers.
traffic stream	Traffic, usually vehicular, moving in one or more lines in the same direction.
traffic survey	The measurement and study of some aspect of traffic movement.
Traffic volume	Traffic movement expressed in vehicles per day or vehicles per hour, vehicles being Passenger Car Units
Traffic volume	See traffic flow.
Traffic volume count	See traffic count.
Transit lane	A traffic lane set aside for the use of buses, motorcycles, taxis and vehicles carrying a specified minimum number of occupants.
Transition curve	A curve of varying radius used for the purpose of easing a change of direction.

Term	Definition
Transition length	1. Horizontal alignment: The distance within which the alignment is changed in approach from straight to a horizontal curve of constant radius. 2. Crossfall: The distance within which the pavement crossfall is changed from normal to that appropriate to the curve. 3. Pavement widening: The distance within which the pavement width is changed from normal to that appropriate to the curve.
Transport planning	Planning of the operations and development of transport including the efficient and equitable allocation of resources.
Transport study	Analysis and synthesis of a specific transport problem. Usually involves data collection, analysis, forecasting, evaluation and recommendations.
Transport system	Sum of the interacting components which constitute a system for the purpose of transporting passengers and/or goods.
Transverse profile	The shape of a pavement surface measured as vertical distances from a datum perpendicular to traffic flow. See also longitudinal profile.
Trash Rack	A device used to capture debris, either floating, suspended, or rolling along the bed, before it enters a drainage facility.
Travel speed	The representative, usually 85th percentile, speed of traffic at a site.
Travel Time	The average time for water to flow through a reach or other stream or valley length.
Travel time (journey time)	Time required to travel between two points.
Travel time survey	Survey designed to obtain travel times over selected traffic routes.
Travelled way	See traffic lanes.
Traverse	A survey consisting of a continuous series of connected straight lines, the lengths and bearings of which are measured. When the lines form a complete circuit or lie between two known points it is termed a closed traverse; otherwise, it is termed an open traverse.
Tributaries	Branches of the watershed stream system.
Trip	1. One-way movement from one place to another for a particular purpose. See also journey. 2. Public vehicle operations: the movement by one vehicle or unit in one direction from the start of a route to the end of it.
Truck aprons	These allow large vehicles (trucks, buses, and recreational vehicles) to navigate the roundabout or turn without striking fixed objects or other motorists.
Truck route	Signposted route defined as being the most suitable for heavy transport and aiming to exclude commercial and residential areas.

Term	Definition
Trumpet interchange	An interchange at a T junction, generally providing interchange ramps for all movements.
Turning circle	The circle traced by the front outside wheel of the vehicle when it is turned to the full lock of its steering mechanism. See also minimum turning radius.
Turning lane (left-turn lane right-turn lane)	1. A lane reserved for turning traffic. 2. A storage and/or speed-change lane reserved for turning traffic.
Turning movement	The traffic volume making a specified turn at an intersection.
turning path	The path of a designated point on a vehicle making a specified turn.
Turning path diagram	A scale diagram showing the path of both the outside of the outermost wheel of a vehicle making a turn of specified angle and specified radius measured to the outermost wheel.
Turning roadway	A carriageway, usually one-way, at an intersection or interchange for turning vehicles.
Turning track width	The radial distance between the turning path of the outside of the outer front tyre and the outside of the rear tyre which is nearest the centre of the turn.
Typical cross-section	A cross-section of a carriageway showing standard dimensional details and features of construction.
U turn	A turn made on the carriageway usually without reversing the vehicle resulting in reversal of direction of travel.
Uncontrolled Spillway	A facility at a reservoir where floodwater discharge is governed only by the inflow and resulting head in the reservoir. Usually, the emergency spillway is uncontrolled.
Underpass	1. A grade separation where the major road passes under an intersecting minor road or railway. See also grade separation. 2. A tunnel constructed for the use of pedestrians, cyclists and/or stock.
Undivided Road (Single Carriageway Road)	A road on a single pavement without a physical separation between opposing traffic.
Ungauged Stream Sites	Locations where no systematic records are available regarding actual stream flows.
Uniform Flow	Flow of constant cross-section and average velocity through a reach of channel during an interval of time.
Uninterrupted flow	A condition in which a vehicle travelling in a traffic stream is not required to stop or slow down for reasons other than those caused by the presence of other vehicles in that stream.
Unit Hydrograph	A hydrograph of a direct runoff resulting from 1 centimetre of effective rainfall generated uniformly over the watershed area during a specified period of time or duration.

Term	Definition
Unsealed shoulder	That portion of the carriageway, located between the edge of seal and the shoulder hinge point, having a slope generally no steeper than 12:1, except on curves where the superelevation may increase the slope.
Unsteady Flow	Flow of variable cross-section and average velocity through a reach of channel during an interval of time.
Upstream	The direction along a carriageway from which the vehicle flow under consideration has come.
Urbanised Section	Section of the road traversing a built-up area with at-grade intersections or crossings and frontage activities
Utilities services	Services such as gas, water, electricity, telephone, sewer and stormwater.
Vehicle crossing	A formed area where vehicles can cross over channel and footpath.
Vehicle kilometres of travel (VKT)	Total vehicle kilometres of travel over a road segment or number of road segments for a certain period, usually a specified year.
Vehicle Restraint System (VRS)	Engineering system installed on a road to provide a level of containment for an errant vehicle.
Vehicle type	Classification of vehicles by type, eg. car, station wagon, utility, light commercial vehicle, etc, and/or by number of axles.
Vehicles per day (VPD)	The number of vehicles observed passing a point on a road in both directions for 24 hours.
Vehicles per lane per day (VLD)	The volume of traffic expressed as vehicles per lane per day.
verge	That area of road reserve located between the shoulder hinge point and the legal road boundary.
Verge (Unpaved Shoulder)	Unpaved area of the shoulder
Vertical alignment	The vertical alignment of a road is known as the grade line, the profile, the longitudinal section or more simply the long section.
Vertical curve	A curve in longitudinal profile of a carriageway to provide for gradual change of grade.
Viaduct	A long bridge composed of a series of spans, usually over land
Volume	Number of persons, vehicles or pedestrians passing a given point in a specified period of time.
Walk-over inspection	A search for road faults carried out by walking along the section of road.
Warrant	A criterion, usually numerical and related to usage levels, used to determine whether the installation of a traffic control device can be justified.
Water table	The natural level at which water stands in a borehole, well, or other depression, under conditions of equilibrium.
Water Table	The upper surface of the zone of saturation, except where that surface is formed by an impermeable body (perched water table).

Term	Definition
Watercourse	A channel where a flow of water occurs, either continuously or intermittently, with some degree of regularity.
Waterproofing	The process of rendering surfaces or materials impervious to water.
Watershed	The divide between catchment areas.
Waterway	1. A channel or stream. 2. The area available for water to pass through or under a structure.
Weaving	The movement in the same general direction of vehicles within two or more traffic streams intersecting at a small angle so that vehicles in one stream cross other streams gradually. See also filtering and merging.
Weaving area	The area of a carriageway in which weaving occurs.
Weaving distance	The length of a carriageway in which weaving occurs.
Weaving section	A length of one-way carriageway, designed to accommodate weaving, at one end of which two one-way carriageways merge and at the other of which they separate.
Weir Flow	Free surface flow over a control surface that has a defined discharge vs. depth relationship.
Wells	Shallow to deep vertical excavations, generally with perforated or slotted pipe backfilled with selected aggregate. The bottom of the excavation terminates in pervious strata above the water table.
Wetted Perimeter	The boundary over which water flows in a channel or culvert taken normal to flow.
Wide Centreline Marking	Centreline marking which has an overall width larger than normal and is generally in the order of 0.6m to 1.0m but may be even wider e.g., 1.5m to 2m.
Y intersection (Y junction)	A junction in which a road joins a through road at an oblique angle.
Yield	Yield means let other road users go first. It's not just other cars. Don't forget about bicycles and pedestrians. Unlike with stop signs, drivers aren't required to come to a complete stop at a yield sign and may proceed without stopping -- provided that it is safe to do so.
Zone	Portion of a study area, designated as such for particular land use and traffic analysis purposes.
Zoning	Partitioning of a city or town by ordinance into sections reserved for different purposes of land usage such as residences, business or manufacturing. Commonly used zoning terms are residential, commercial, industrial, public purposes, recreational, special uses, etc.