ROAD AND BRIDGE WORKS

THE REPUBLIC OF UGANDA

MINISTRY OF WORKS AND TRANSPORT

ROAD DESIGN MANUAL

Volume 1: Geometric Design

January 2010

PREAMBLE

This Road Design Manual Volume I: Geometric Design is one of a series of Engineering Specifications, Standards, Manuals and Guidelines issued by the Ministry of Works and Transport. The Manual is part of the revised Road Design Manual, November 1994.

The four Volumes of the Road Design Manual include:
a) Road Design Manual: Vol. I Geometric Design;
b) Road Design Manual: Vol. II Drainage Design;
c) Road Design Manual: Vol. III Pavement Design; and
d) Road Design Manual: Vol. IV Bridge Design.

The Manual gives guidance and recommendations to the Engineers responsible for the design of roads in Uganda. It complements the Ministry's efforts in providing guidance to the construction industry by setting uniform standards to be used in the construction of infrastructure facilities that meet the needs of the users.

Geometric design is an essential component in the design of roads. The procedures for the geometric design of roads presented in this Manual are applicable to all classified roads as defined herein. The contents of the Manual are partly guidelines and recommendations to be considered and partly standards which as a general rule should be adhered to. However, in some instances special conditions may demand modifications to the standards set herein. In such cases special consideration should be given in consultation with the Engineer-In-Chief, Ministry of Works and transport.

Further, this Manual is a technical document, which, by its very nature, requires periodic updating from time to time arising from the dynamic technological developments and changes. The Ministry, therefore, welcomes proposals on areas for further development and revision stemming from the actual field experience and practice. It is hoped that the comments will contribute to future revisions of the Manual expected to lead to better and more economical designs.

Ministry of Works and Transport
P.O. Box 10
Entebbe

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SECTION 1: GENERAL

1.1 Introduction

1.1.1 General

The Geometric Design Manual sets forth the policy and standards to be adopted for the design of rural roads in Uganda. Geometric Design is an essential component in the design development of roads. This Geometric Design Manual is one part of the revised and developed version of the Road Design Manual, November 1994. The revised and developed Road Design Manual is prepared in four parts.

Part I: Geometric Design Manual
Part II: Hydrology and Hydraulics Design
Part III: Pavement Design Manual
Part IV: Bridge Design Manual

This Road Design Manual supersedes the Road Design Manual of November 1994 and it is intended for use in the design of all rural roads in Uganda.

1.1.2 Purpose

The purpose of this design manual is to give guidance and recommendations to the engineers responsible for the geometric design of rural roads in Uganda.

1.1.3 Scope

The procedures for the geometric design of roads presented in this manual are applicable to all classified roads as defined in this Geometric Design Manual.

The contents of the manual are partly guidelines and recommendations to be considered, and partly standards which as a general 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 Engineer-In-Chief.

The use of the procedures described in this manual should help in achieving reasonable uniformity in geometric design for a given set of conditions. It is designed to assist the road design engineer in Uganda and it is hoped that the road design engineer will contribute by putting forward any proposals for further development and revision stemming from the actual field experience and practice and also which he considers will result in a better and more economical design.

1.1.4 Organization of the Manual

The manual is divided into 12 sections as follows:

Section 1 is dealing with general such as introduction, units of measurement, definition and abbreviations.

Section 2 deals with preliminary design considerations. Specifically, it lists procedures for identification of potential alignments in the route corridor selection process.

Section 3 is dedicated to survey requirements.

Section 4 is the road system, which discusses on the road categorization and classification in Uganda. It deals with division of roads into classes and design classes and also gives summary of geometric design parameters for the different design classes of road.

Section 5 is dealing with design controls and criteria affecting the selection of the geometric design values. These include design vehicles, driver performance, traffic characteristics, capacity and level of service.

Section 6 is elements of design, which deals with the various sight distances, horizontal alignment and vertical alignment.

Section 7 is dealing with cross section elements, which discusses mainly on lane widths, shoulders, medians, clear zones, right of way, side and back slopes and gives typical cross sections of the different design classes of roads.

Section 8 discusses at-grade junctions, including design requirements, selection of junction type, t-junctions, cross junctions and roundabouts; sight distances; and junction elements including turning lanes and traffic islands.

Section 9 is Interchanges or grade-separated junctions

Section 10 is Speed Management, Section 11 is Other Road Facilities and Section 12 is Road Furniture, Road Marking and Miscellaneous items.

1.2. Units of Measurement and Language

The language of the manual is English.

The standard units of measurement to be used are based on the International system (SI) units. However, the units applicable to road design also include some units which are not strictly part of SI.

Multiples and sub-multiples of SI units are formed either by the use of indices or prefixes. Definitions of applicable prefixes are given below.

The basic units and the derived and supplementary units which will normally be required for road design are 1isted in the table below.

Table 1-1: Definitions of Prefixes

Prefix	Symbol	Factor by which the unit is multiplied
mega	M	10^{6}
kilo	K	10^{3}
hecto	H	10^{2}
deca	da	10
deci	d	10^{-1}
centi	c	10^{-2}
milli	m	10^{-3}
Micro	µ	10^{-6}

Table 1-2: Basic units, multiples and sub-multiples

Item	Unit	Symbol	Recommended Multiples and sub-multiples
Length	Meter	m	km, mm
Mass	Kilogram	kg	Mg, g, mg
Time	Second	s	Day (d), hour (h), minute (min)
Area	Square meter	m^2	km2, hectare (1ha = 10,000m2), mm^2
Volume (solids)	Cubic meter	m^3	cm^2, mm^3
Volume (liquid)	Litre	l	ml ($1ml=10^{-3}$ l, $1ml=1cm^3$)
Density	Kilogram per cubic meter	kg/m^3	$1Mg/m^3=1kg/10^3l$, g/ml
Force	Newton	N	MN, kN ($1N=1\ kgm/s^2$, 1kgf=9.81N)
Pressure and Stress	Newton per square meter	N/m^2	kN/m^2, N/mm^2
Velocity (Speed)	Meter per second	m/s	km/h (1 km/h=1/3.6(m/s))
Angle	Degree or grade	^{o} ^{g}	Minute (‘), second (“”) (360^o circle) (400^g circle)
Temperature	Degree Celsius	^oC	

1.3. Document Preparation

Recommended Drawing Scales

The following scales are recommended when preparing detailed design drawings, plans and charts.

Location Map (whole of Uganda)		1 in 1,000,000
Location Plan	Variable	1 in 1,500,000
		1 in 1,000,000
		1 in 200,000
Key Plan	Generally	1 in 50,000
Plan and Longitudinal profile		
Section Plan		1 in 2,000
Longitudinal Section	Horizontal	1 in 2,000
	Vertical	1 in 200

Cross-Sections

Individual cross-sections 1 in 200 / 1/400

Typical Cross Section 1 in 50 (natural) Pavement Detail (Show half cross section) 1 in 25 (natural)

Details of Junctions and Lay Byes	1 in 250/50/10 1 in 500/100
Details of Fencing and Gates	1 in 20
Drainage Details	1 in 200/100/50/20/10/5
Guardrail Details	1 in 100/25/10

Culverts	1 in 200/100/50/25	
Borrow Pit Key Plan	1 in 50,000	
Materials Utilization chart		
Horizontal	1 in 20,000	
Vertical	to suit	

Recommended Document Sizes

The following sizes are recommended when preparing documents.

Drawing Size	Originals	A1 Preferred
	Tender Documents	A3 (A1 reduced)
	Other Documents	A4

1.4 Departures from Standards

It is anticipated that there may be situations where the designer will be compelled to deviate from these standards. An example of a Departure from Standard could be the use of a gradient greater than the absolute maximum value and also the use of radius less than the allowable minimum for any specified class of road. Where the designer departs from a standard, he must obtain written approval and authorization from the Engineer-In-Chief of MoWT. The Designer shall submit the following information to the Engineer-In-Chief:

- The number, name, and description of the road
- The facet of design for which a Departure from Standards is desired;
- A description of the standard, including normal value, and the value of the Departure from Standards
- The reason for the Departure from Standards, and
- Any mitigation to be applied in the interests of safety.

The Designer must submit all major and minor Departures from the Standards and his proposal for approval. If the proposed departures from the Standards are acceptable, the departures from the Standards will be given approval by the Engineer-In-Chief.

1.5 Definitions and Abbreviations

List of abbreviations and definitions, relevant to geometric design as well as terminologies used for cross – section, horizontal curve, superelevation, are included here.

ABBREVIATIONS

A

AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
ADT	Average Daily Traffic

B

BM	Bench Mark
BVC	Beginning of Vertical Curve

C

CAD	Computer Aided Design
CADD	Computer-Aided Design and Drafting
CL	Center line
CMP	Corrugated metal pipe
CS	Curve to spiral
CWW	Carriageway width

D

DHV	Daily High Volume
DS	Design standard
DTM	Digital Terrain Model
DV	Design vehicle

E

E	External distance
EVC	End of Vertical Curve

F

FSE	Full superelevation

G

GPS	Global Positioning System

H

HAL	Horizontal Alignment Listing
HCM	Highway Capacity Manual

L

LC	Long chord
LOS	Level of service

LW	Lane width

M

MOFED	Ministry of Finance and Economic Development
MoWT	Ministry of Works and Transport
MUTCD	Manual on Uniform Traffic Control Devices

N

NC	Normal Crown

P

PC	Point of Curvature
PCN	Project Concept Note
PI	Point of Intersection
PIP	Public Investment Plan
POT	Point of Tangent
PSD	Passing Sight Distance
PT	Point of Tangency
PVI	Point of Vertical Intersection

R

RCP	Reinforced concrete pipe
RFCS	Road Functional Classification System
ROW	Right-of-way
RP	Reference Point
RPSD	Reduced Passing Sight Distance
RRW	Road reserve width

S

S	Shoulder
SSD	Stopping Sight Distance

T

TBM	Temporary Bench Mark
TRL	Transport Research Laboratory
TRRL	Transport and Road Research Laboratory

V

VPI	Vertical Point of Intersection

DEFINITIONS

A

Access

Way whereby the owner or occupier of any land has access to a public road, whether directly or across land lying between his sand and such public road.

Access control

The condition whereby the road agency either partially or fully controls the right of abutting owners or occupiers to direct access to and from a public highway or road.

Acceleration Lane

An auxiliary lane to enable a vehicle to increase its speed so that it can more safely merge with through traffic.

At-Grade Junction

Junction where all roadways join or cross at the same level.

Auxiliary Lane

Part of the roadway adjoining the carriageway for parking, speed change, turning, storage for turning, weaving, truck climbing, and for other purposes supplementary to through traffic movement.

Average Annual Daily Traffic (AADT)

Total yearly traffic volume in both directions divided by the number of days in the year.

Average Daily Traffic (ADT)

Total number of vehicles (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.

Average running speed

The distance summation for all vehicles divided by the running time summation for all vehicles. Also referred to as space mean speed whereas time speed is simply the average of all recorded speeds.

Axis of rotation

The line about which the pavement is rotated to superelevate the roadway.

B

Back Slope

Area proceeding from ditch bottom to the limit of the earthworks.

Barricade

A portable or fixed barrier used to close all or a part of a road to vehicular traffic.

Borrow

Material not obtained from roadway excavation but secured by widening cuts, flattening cut back slopes, excavating from sources adjacent to the road within the right-of-way, or from selected borrow pits as may be noted on the plans.

Bollard

A device placed on a street refuge or traffic island to provide a measure of protection for pedestrians and to warn drivers of these obstructions. It also usually indicates by means of a traffic sign the direction to be taken by vehicles. The device is generally illuminated at night.

Bridge

A structure erected with a deck for carrying traffic over or under an obstruction and with a clear span of six meters or more. Where the clear span is less than six meters, reference is to a culvert.

Bus Lay-Byes

Lay-by reserved for public service vehicles.

Bypass

A road on the fringe of a town or village to enable through traffic to avoid congested areas or other obstructions to movement.

C

Camber

The slope from a high point (typically at the center line of a road) across the lanes of a highway. It is also called Crossfall.

Capacity

The maximum number of vehicles that can pass a point on a road or in a designated lane in one hour without the density being so great as to cause unreasonable delay or restrict the driver's freedom to maneuver under prevailing roadway and traffic conditions.

Capping Layer

A layer of selected fill material placed on the topmost embankment layer or the bottom of excavation.

Carriageway

Part of the roadway including the various traffic lanes and auxiliary lanes but excluding shoulders.

Center Lane

On a dual three-lane road, the middle lane of the three lanes in one direction.

Centerline

Axis along the middle of the road.

Central Reserve

An area separating the carriageways of a dual carriageway road.

Circular Curve

Usual curve configuration used for horizontal curves.

Channelising Island

A traffic island located in the carriageway area to control and direct specific traffic movements to definite channels.

Channelisation

The separation or regulation of conflicting traffic movements into definite paths of travel by the use of pavement markings, raised islands, or other suitable means to facilitate the safe and orderly movement of traffic, both vehicular and pedestrian.

Channelised Junction

An at-grade junction in which traffic is directed into definite paths by island and road marking

Clear Zone

Unencumbered roadside recovery area.

Climbing Lane

An auxiliary lane in the upgrade direction for use by slow moving vehicles and to facilitate overtaking, thereby maintaining capacity and freedom of operation on the carriageway.

Cloverleaf

A four-way interchange in which inner loops are provided for movements and direct outer connections for right-turn-movements. A cloverleaf has ramps for turning movements

Coefficient of Friction

Ratio of the frictional force on the vehicle and the component of the weight of the vehicle perpendicular to the frictional force.

Collector Roads

Secondary Roads linking locally important centers to each other, to more important centers or to higher class roads.

Compound Curve

Curve consisting of two or more arcs of different radii curving in the same direction and having a common tangent or transition curve where they meet.

Control of Access

Conditions where the right of owners or occupants of adjoining land or other persons to access, light, air or view in connection with a road is fully or partially controlled by public authority.

Crest

Peak formed by the junction of two gradients.

Crest Curve

Convex vertical curve with the intersection point of the tangents above the road level.

Criterion

A yardstick according to which some or other quality of the road can be measured. Guideline values are specific numerical values of the criterion.

Critical Slope

Side slope on which a vehicle is likely to overturn.

Critical length of grade

The maximum length of a specific upgrade on which a loaded truck can operate without an unreasonable reduction in speed. A speed reduction of 15 km/h or more is considered “unreasonable”.

Crossfall

The tilt or transverse inclination of the cross-section of a carriageway which is not cambered, expressed as a percentage.

Cross-Roads

Four-leg junction formed by the intersection of two roads at approximately right angles.

Cross-Section

Vertical section showing the elevation of the existing ground, ground data and recommended works, usually at right angles to the centerline.

Crown

Highest portion of the cross-section of a cambered roadway.

Curb

Border of stone, concrete or other rigid material formed at the edge of the roadway or footway.

Cycle Way

Way or part of a road for use only by pedal cycles.

D

Deceleration Lane

An auxiliary lane to enable a vehicle leaving the through traffic stream to reduce speed without interfering with other traffic.

Deflection Angle

Successive angles from a tangent subtending a chord and used in setting out curves.

Departure from Standards

Deviation from values given in the reference, requiring prior approval of the Ministry (MoWT).

Design Capacity

Maximum number of vehicles that can pass over a lane or a roadway during a given time period without operating conditions falling below a pre-selected design level.

Design Speed

A speed selected and used for design and which links road function, traffic flow and terrain to the design parameters of sight distances, curvature, and superelevation to ensure that a driver is presented with a reasonably safe and consistent speed environment. It is in practice, most roads will only be constrained to minimum parameter values over short sections or on specific geometric elements.

Design Traffic Volume

Number of vehicles or persons that pass over a given section of a lane or roadway during a time period of one hour or more.

Design Vehicle

Vehicle whose physical characteristics and proportions are used in setting geometric design.

Design Volume

Volume determined for use in design, representing traffic expected to use the road.

Diamond Junction

A four-way interchange with a single one-way ramp in each quadrant. All right-turns are made at grade on the minor road.

Diverging

Movement of a vehicle out of a traffic stream.

Diversion

An alternative route for traffic to avoid congestion, obstruction or other hazard.

Divided Road

Road in which there are two physically separated roadways reserved for traveling in opposite directions.

Dual Carriageway Road

A road in which there are two physically separated carriageways reserved for travelling in opposite directions

E

Economical Limit of Haul

Distance through which it is more economical to haul excavated material than to waste and borrow.

Embankment

That portion of the road prism composed of approved fill material, which lies above the original ground and is bounded by the side slopes, extending downwards and outwards from the outer shoulder breakpoints and on which the pavement is constructed.

Escarpment (Terrain)

Steep country inclusive of switchback sections and side hill traverses; transverse terrain slope > 70 percent.

Eye Height

Assumed height of drivers' eyes above the surface of the roadway used for the purpose of determining sight distances.

F

Feeder Road

Lowest level of road in the network hierarchy with the function of linking traffic to and from rural areas, either directly to adjacent urban centers, or to the Collector road network.

Fill

Material which is used for the construction of embankments.

Filtering

The permitted movement of one or more lines of traffic streams while the remaining lines are stopped.

Flat (Terrain)

Flat terrain with largely unrestricted horizontal and vertical alignment; transverse terrain slope up to 5 percent.

Flush Curb

A concrete structure, usually continuous at the edges of the carriageway and/or paved shoulder, providing them with lateral support. It is usually flush with their surfaces.

Free Haul

Maximum distance through which excavated material may be transported without added cost above the unit bid price.

Footway

Portion of a road reserved exclusively for pedestrians.

Fork-junction

A Y-junction in which one the arm of the Y does not deviate from the stem.

G

Geometric (Design) Standards

Guidelines for limiting values of road alignment and cross-section design.

Grade Separated Junction

Junction where two roads cross at different levels and are connected by ramps.

Grade Separation

Crossing of two roads, or a road and a railway at different levels.

Gradient

A rate of rise or fall on any length of road with respect to the horizontal. It is usually expressed as a percentage of vertical rise or fall in meters / 100 meters of horizontal distance.

Guardrail

Continuous barrier erected alongside a road to prevent traffic from accidentally leaving the roadway or from crossing the median.

Gyratory Traffic

Vehicular traffic flowing round a system of one-way streets or a roundabout.

Gyratory System

A system of one-way carriageways which together allow a continuous passage of traffic around a central area which may or may not contain buildings.

H

Half-cloverleaf

A four-way interchange in which loops and outer connections are provided in two quadrants to given grade separation to the major road, but on the minor road the right-turning movements take place at grade.

Horizontal Alignment

Direction and course of the road centerline in plan.

Horizontal Clearance

Lateral clearance between the edge of shoulder and obstructions.

Horizontal Curve

Curve in plan.

I

Interchange

Network of roads at the approaches to a junction at different levels that permits traffic movement from one to the other one or more roadways or roads.

J

Junction (Intersection)

a) Common zone of two or more roads allowing vehicles to pass from one to the other;

b) Meeting of one road with another.

K

K-value

Ratio of the minimum length of vertical crest curve in meters to the algebraic difference in percentage gradients adjoining the curve.

L

Lane

Strip of roadway intended to accommodate a single line of moving vehicles.

Lay-by

Part of the road set aside for vehicles to draw out of the traffic lanes for short periods.

Left -Hand Lane

On a dual roadway, the traffic lane nearest to the verge or shoulder.

Left Turn Lane

An auxiliary lane to accommodate deceleration and storage of left-turning vehicles at junctions.

Left-right Stagger

A cross-roads at which a driver intending to cross a major road, turns to his left on entering the intersecting road, and then to his right in order to continue on his route.

Level of Service

Qualitative rating of the effectiveness of a road in serving traffic, measured in terms of operating conditions.

Limited Access Road

Road with right of access only at a limited number of places.

Link Road

National Road linking nationally important centers.

Local Road

Road (or street) primarily for access to adjoining property. It may or may not be a classified road.

Longitudinal Profile

Outline of a vertical section of the ground, ground data and recommended works along the centerline.

M

Main Access Road

Primary Road linking provincially important centers to each other or to higher class roads.

Marker Post

Post, generally fitted with reflective material or small reflecting studs, but not usually lighted, erected off the roadway to give warning or guidance to traffic.

Meeting Sight Distance

Distance required to enable the drivers of two vehicles traveling in opposite directions on a two-way road with insufficient width for passing to bring their vehicles to a safe stop after becoming visible to each other. It is the sum of the stopping sight distances for the two vehicles plus a short safety distance.

Median

Area between the two carriageways of a dual carriageway road. It excludes the inside shoulders.

Merging

Movement of a vehicle or vehicles into a traffic stream.

Mountainous (terrain)

Terrain that is rugged and very hilly with substantial restrictions in both horizontal and vertical alignment; transverse terrain slope 25-75 percent.

Motorway

A road having dual carriageways and shoulders, with complete grade separation. It is for the exclusive use of prescribed classes of motor vehicles.

Multi-leg Junction

A junction with five or more legs.

N

Network (Hierarchy)

Classification of roads according to International Trunk, National Trunk, Primary, Secondary, and Minor.

Nonrecoverable Slope

Transversible side slope, where the motorist is generally unable to stop or return to the roadway.

Normal Crossfall

Difference in level measured traversely across the surface of the roadway.

O

Object Height

Assumed height of a notional object on the surface of the roadway used for the purpose of determining sight distance.

Operating Speed

Highest overall speed at which a driver can travel on a given road under favorable weather conditions and under prevailing traffic conditions without at any time exceeding the safe speed as determined by the design speed on a section-by-section basis.

Optimum Speed

The speed at which the maximum possible traffic flow (traffic capacity) can be attained.

Overpass

Grade separation where the subject road passes over an intersecting road or railway.

P

Parking Bay

Area provided for taxis and other vehicles to stop outside of the roadway.

Passenger Car Unit (PCU)

A unit of road traffic, equivalent for capacity purposes to one normal private car. The private car is thus the unit and other vehicles are converted to the same unit by a factor depending on their type and circumstances.

Passing Bay

Widened section of an otherwise single lane road where a vehicle may move over to enable another vehicle to pass.

Passing Sight Distance

Minimum sight distance on two-way single roadway roads that must be available to enable the driver of one vehicle to pass another vehicle safely and comfortably without interfering with the speed of an oncoming vehicle traveling at the design speed, should it come into view after the overtaking maneuver is started.

Pavement

A multi-layered horizontal structure which is constructed for the purpose of carrying traffic.

Pavement Layers

The layers of different materials, which comprise the pavement structure.

Peak Hour Traffic

The highest number of vehicles found to be passing over a section of lane or carriageway during 60 consecutive minutes.

Pedestrian Crossing

Transverse strip of roadway intended for the use of pedestrians crossing the road. The crossing may be uncontrolled or controlled.

Pedestrian Guard Rail

A protective fence between two carriageways to discourage pedestrians from crossing the road.

Pedestrian Refuge

Raised platform or a guarded area so sited in the roadway as to divide the streams of traffic and to provide a safety area for pedestrians.

Point of Intersection (PI)

The internal angle formed by two successive straights.

R

Ramp

a) Inclined section of roadway over which traffic passes for the primary purpose of ascending or descending so as to make connections with other roadways;

b) Interconnecting length of road of a traffic interchange or any connection between roads of different levels, on which vehicles may enter or leave a designated road.

Ramp Terminal

The general area where a ramp connects with a through carriageway.

Recoverable Slope

Side slope of limited grade such that a motorist can generally return to the roadway.

Reverse Curve

Composite curve consisting of two arcs or transitions curving in opposite directions.

Right Hand Lane

On a dual roadway, the traffic lane nearest to the median or near to the central reserve.

Right-left Stagger

A cross-roads at which a driver, intending to cross a major road turns to his right on entering the intersecting road, and then to his left in order to continue on his route.

Right-Turn Lane

Auxiliary lane to accommodate deceleration and storage of right- turning vehicles at junctions.

Right-of-Way

Strip of land, in which the road is or will be situated and where no other work or construction may take place without permission from the Roads Agency (MoWT). The width of the road reserve is measured at right angles to the centerline.

Ring Road

A road around an urban area enabling traffic to avoid it.

Road

Way for vehicles and for other types of traffic which may or may not be lawfully usable by all traffic.

Road Bed

The natural in-situ material on which the embankment or capping layers are to be constructed.

Road Functional Classification

Classification of roads according to service provided in terms of the road hierarchy.

Road Prism

The cross sectional area bounded by the original ground level and the sides of slopes in cuttings and embankments excluding the pavement.

Roadway

Part of the road comprising the carriageway, shoulders and median.

Roadway Width

Measurement at right angle to the centerline incorporating carriageway, shoulders and, when applicable, median.

Roadside

General term denoting the areas adjoining the outer edges of the shoulders.

Road Width

A measurement at right angle to the centreline incorporating travelled way, shoulders and, when applicable, central reserve.

Road Reserve

A strip of land, in which the road is or will be situated and where no other work or construction may take place without permission from the Road Agency (MoWT). The width of the road reserve is measured at right angles to the centerline.

Rolling (Terrain)

Terrain with low hills introducing moderate levels of rise and fall with some restrictions on vertical alignment; traverse terrain slope 5-20 percent.

Roundabout

Road junction designed for movement of traffic in one direction around a central island.

S

Safety Rest Area

Roadside area with parking facilities for the motorist to stop and rest.

Sag Curve

Concave vertical curve with the intersection point of the tangents below the road level.

Scenic Overlook

Safety rest area primarily for viewing scenery.

Service Area

Land with access to and from a road allocated for the provision of certain amenities and services.

Service Road

A subsidiary road connecting a principal road with adjacent buildings or properties facing thereon, and connected with the principal road only at selected points.

Shift

The lateral displacement of a circular curve, measured along the radius, consequent upon the introduction of a transition curve.

Shoulder

Part of the road outside the carriageway, but at substantially the same level, for accommodation of stopped vehicles for emergency use, and for lateral support of the carriageway .

Shoulder Breakpoint

The point on a cross section at which the extended flat planes of the surface of the shoulder and the outside slope of the fill and pavement intersect.

Side Drain

A longitudinal drain offset from, and parallel to, the carriageway.

Side Slope

Area between the outer edge of shoulder or hinge point and the ditch bottom.

Sight Distance

Distance visible to the driver of a passenger car measured along the normal travel path of a roadway to the roadway surface or to a specified height above the roadway surface, when the view is unobstructed by traffic.

Single Lane Road

Road consisting of a single traffic lane serving both directions, with passing bays.

Scissors Junction

A four-leg junction formed by the oblique intersection of two roads.

Speed

Rate of movement of vehicular traffic or of specified components of traffic, expressed in kilometers per hour (km/h).

Speed Bump

Device for controlling the speed of vehicles, consisting of a bar or recess on the roadway.

Stopping Sight Distance

Distance required by a driver of a vehicle traveling at a given speed, to bring his vehicle to a stop after an object on the roadway becomes visible. It includes the distance traveled during the perception and reaction times and the vehicle braking distance.

Street

A road which has become partly or wholly defined by buildings established along one or both frontages.

Superelevation

Inward tilt or transverse inclination given to the cross section of a roadway throughout the length of a horizontal curve to reduce the effects of centrifugal force on a moving vehicle; expressed as a percentage.

Superelevation Run-off

Length of road over which superelevation is reduced from its maximum value to zero.

Switchbacks

Sequence of sharp curves at or near minimum radius employed to traverse a mountainous or escarpment terrain section.

T

T-Junction

Three-leg junction in the general form of a T.

Tangent

Portion of a horizontal alignment of straight geometrics.

Taper

Transition length between a passing place, auxiliary lane or climbing lane and the standard roadway.

Tenth, Twentieth, Thirtieth, etc. Highest Annual Hourly Volume

The hourly traffic volume on a given section of a road that is exceeded by 9, 19, 29, etc., respectively, hourly volumes during a designated year.

Through Road

Road primarily for through traffic in relation to the area considered, on which vehicular traffic is usually given priority over the traffic on intersecting roads. It may or may not be a classified road.

Traffic

Vehicles, pedestrians and animals traveling along a route.

Traffic Capacity (Capacity)

Maximum number of vehicles which has a reasonable expectation of passing over a given section of a lane or a roadway in one direction or in both directions for a two-lane single roadway road, during a given time period under prevailing road and traffic conditions

Traffic Flow

Number of vehicles or persons that pass a specific point in a stated time, in both directions unless otherwise stated.

Traffic Lane

Part of a carriageway intended for a single stream of traffic in one direction, which has normally been demarcated as such by road markings.

Traffic Island

Central or subsidiary area raised or marked on the roadway, generally at a road junction, shaped and placed so as to direct traffic movement.

Traffic Volume

The number of vehicles or persons that pass over a given section of a lane or a roadway during a time period of one hour or more. Volume is usually expressed in one of the following terms:

Transition Curve

Curve whose radius changes continuously along its length, used for the purpose of connecting a tangent with a circular arc or two circular areas of different radii.

Transition Length

Length of the transition curve.

Travelled Way

That part of the carriageway used for the movement of vehicles, exclusive of auxiliary lanes, bus-bays, etc.

Trumpet Junction

A grade separated T-junction.

Turning Lanes

Lanes which separate turning vehicles from the through traffic lanes.

Typical Cross-Section

Cross-section of a road showing standard dimensional details and features of construction.

U

Underpass

A grade separation where the subject road passes under an inter¬secting road or railway.

Uncontrolled Pedestrian Crossing (Zebra-crossing)

A pedestrian crossing marked by a series of white longitudinal strips extended transversely across the width of the carriage¬way and accompanied by traffic sign (“Pedestrian Crossing”), where a pedestrian has priority over all vehicles by which he would be impeded.

V

Verge

That part of the road outside the carriageway and generally at substantially the same level. It may contain footpaths, cycle tracks or ditches.

Vertical Alignment

Direction of the centerline of a road in profile.

Vertical Curve

Curve on the longitudinal profile of a road, normally parabolic.

Visibility Splay

A triangular area bordered by intersecting roads and kept free of obstructions (except essential traffic signs) to enable a driver who is required to give way to have unobstruc¬ted visibility along the major road.

W

Waste

Material excavated from roadway cuts but not required for making the embankments. It must be pointed out that this material is not necessarily wasted as the word implies, but can be used in widening embankments, flattening slopes, or filling ditches or depressions for erosion control.

Weaving

Movement in the same general direction of vehicles within two or more traffic streams intersecting at a shallow angle so that the vehicles in one stream cross other streams gradually.

Weaving Length

The length of carriageway in which weaving may take place.

Weaving Section

The area of carriageway in which weaving may take place.

Y-junction

A three-leg junction in the general form of a Y.

TERMINOLOGIES

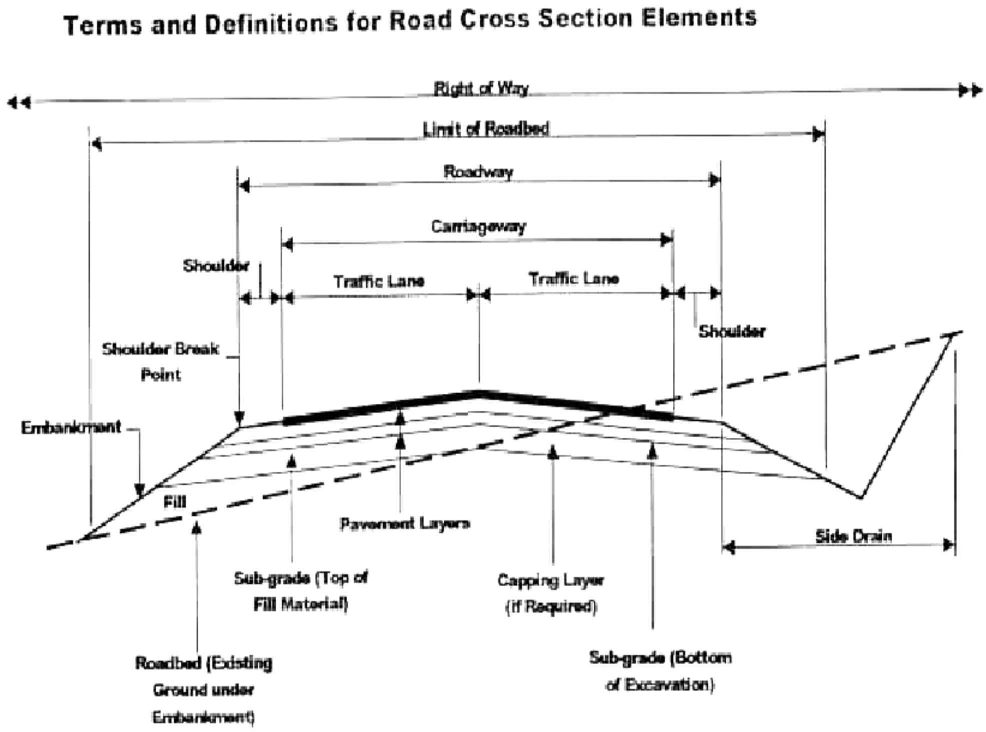
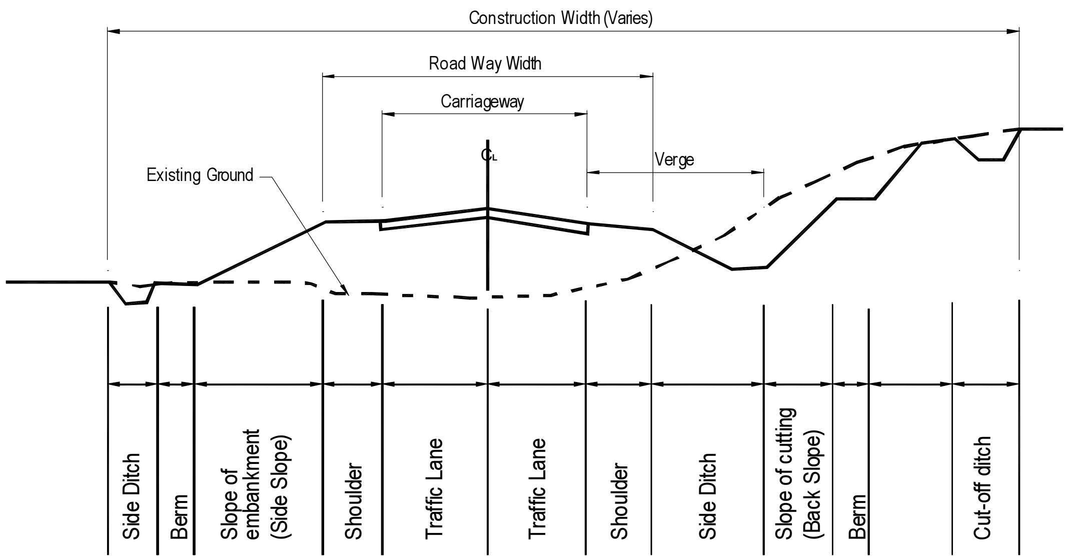
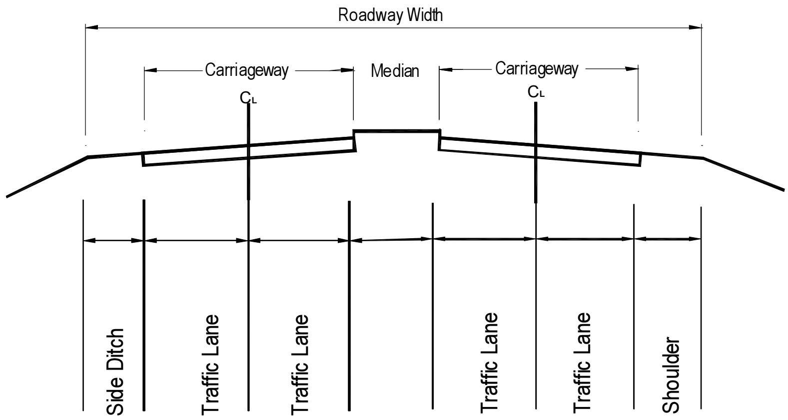


Figure 1 - 1: Road Cross Section Elements

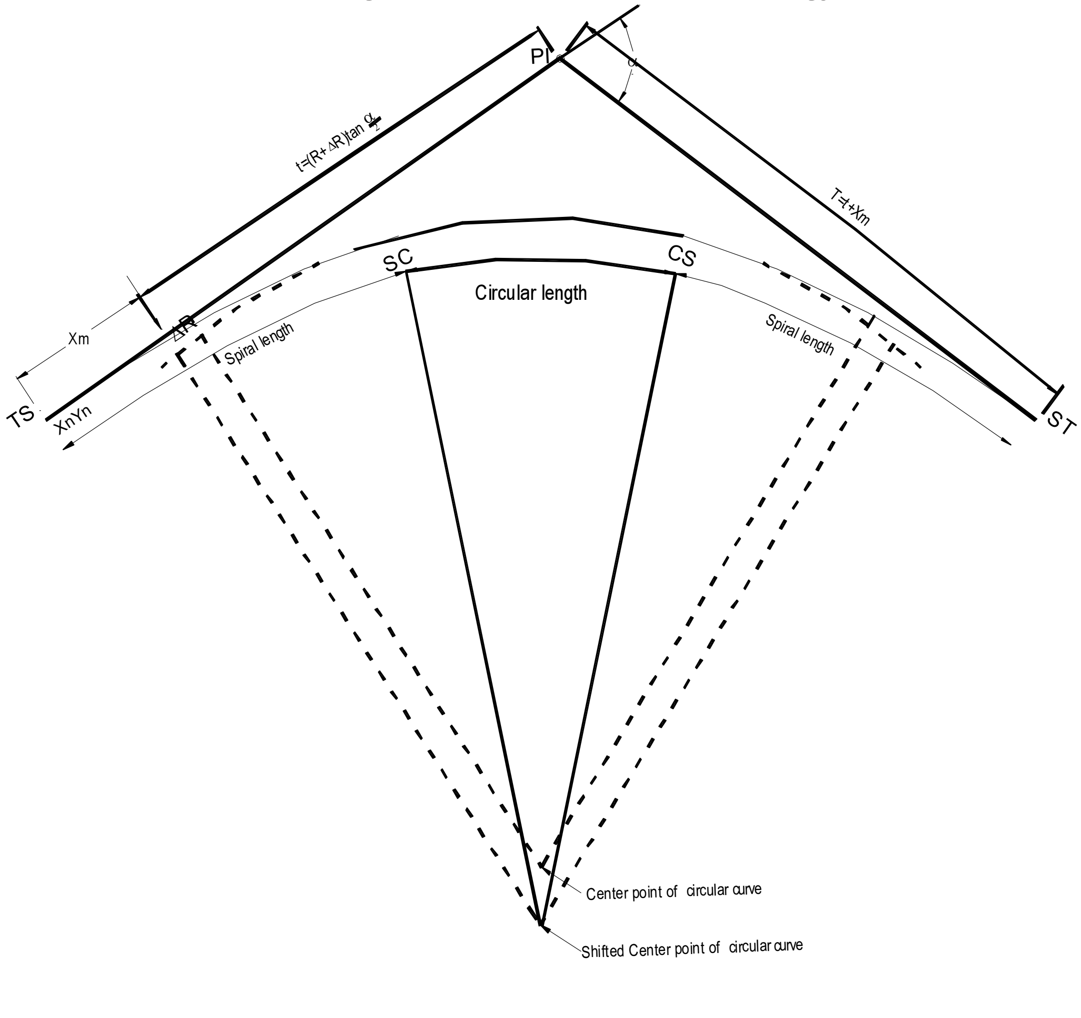


a) Single Carriageway roads (Rural Section)

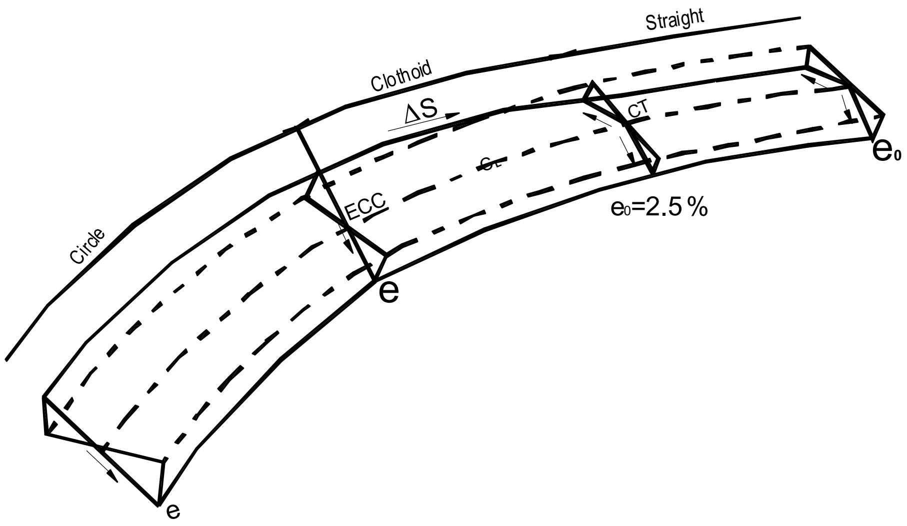


b) Dual Carriageway roads (Rural Section)

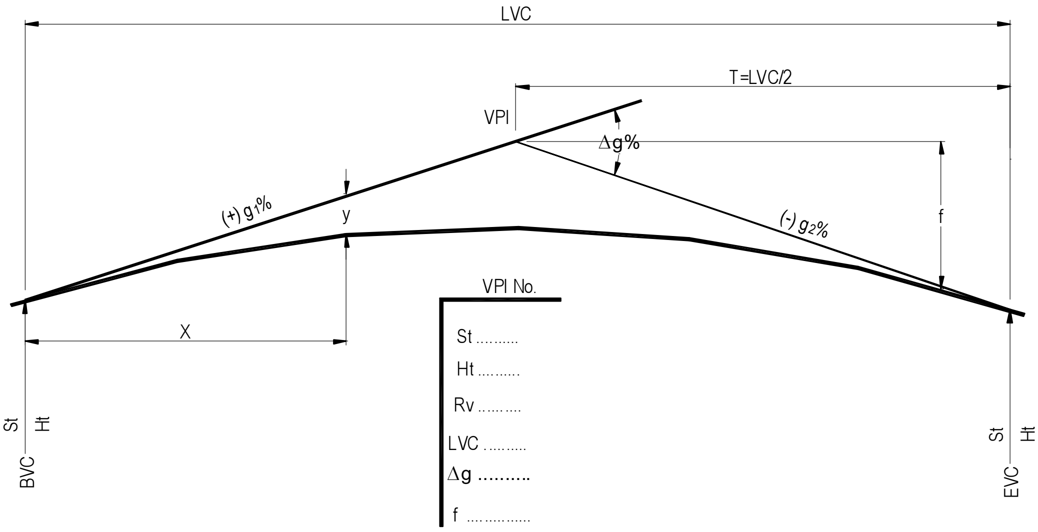
Figure: 1-2: Cross Section Terminology



SYMBOL	DESCRIPTION
R	Radius of circular curve
TS	Tangent to spiral
SC	spiral to curve
CS	Curve to spiral
ST	Spiral to tangent
PI	point of Intersection
LS	length of spiral
LC	Length of circular curve
ΔR	Shift
X_m	Absisa of center point
X_n Y_n	Coordinate at station No.
α	Deflection Angle
T	Tangent Length
L_{CC}	Total curve length (circle + spiral)



SYMBOL	DESCRIPTION
e_O	Normal crossfall
e	Super elevation
ΔS	Rate of change of super elevation
CT	Clothoid to Tangent
ECC	End of circle to clothoid
CL	Centerline of road



Symbol	Description	Symbol	Description
g	Gradient (%)	Bvc	Beginning of vertical curve
PVI	Point of intersection of vertical curve	Evc	End of vertical curve
St	Station chainage	T	Tangent length of vertical curve
Ht	Height above sea level (m)	y	Tangent offset (vertical)
Rv	Equivalent radius of vertical curve (m)	x	Horizontal length in plan
Lvc	Length of vertical curve (m)	f	Center correction
Δg	Algebraic difference in gradients (%)	Ls	Stopping sight distance
		Lm	Meeting sight distance
		Lp	Passing sight distance

SECTION 2: ROUTE CORRIDOR SELECTION

2.1 The Project Cycle

2.1.1 General

Projects are planned and carried out using a sequence of activities commonly referred to as the project cycle. There are many ways of defining the steps in the sequence but the following terminology in road projects is used by MOWT: - identification, feasibility and preliminary engineering study, detailed design, procurement and funding, construction supervision and management, operation and project evaluation.

2.1.2 Identification

The first stage of the cycle is to find potential projects. This is sometimes known as the pre-feasibility stage. Projects are generally speaking identified by any of the following methods.

i. Suggestions from MOWT technical staff and political leaders

ii. Proposals by MOWT to extend existing programmes or projects.

Projects that are identified to support approved strategies and are financially, economically and environmentally sound will be selected for further development by the MOFED and included into PIP. This process usually involves consulting of donor agencies. Once a project has been accepted for further development a Project Concept Note (PCN) will be prepared, often in co-operation with the potential donor agency. The Project Concept Note outlines the basic elements of the project, its proposed objective, likely risks (environmental including), alternative scenarios to conducting the project and likely timeframe for the project approval process.

Identification of projects is done very carefully to distinguish promising projects from dubious ones and also avoid halting of projects at a later stage after arousing the expectations of interested groups.

2.1.3 Feasibility and Preliminary Engineering Study

Projects that have been identified and for which Project Concept Note has been prepared will have go through the next phase, the feasibility and preliminary engineering study as well as environmental and social impact assessment.

The study and environmental/social assessment provides enough information for deciding whether to/not to proceed to a more advanced stage of planning. The level of detail of this study depends on the complexity of the project and how much is known already about the proposal. This study defines the objective of the project. It also considers alternative ways of achieving these objectives and eliminating poor alternatives. The study provides the opportunity to mould the project to fit its physical and social environment in such a way as to maximise the return on the investment. Once this study has indicated which project alternative is likely to be most worthwhile, detailed planning and analysis then begins.

Project appraisal, performed by borrowing banks and/or donor agencies concludes this phase of the project implementation.

Note: The study may also indicate that the project is not viable and thus, has to be abandoned.

2.1.4 Detailed Design

Detailed design of the project usually is the responsibility of the MOWT and follows government's provisional commitment to the project as a result of the outcome of the feasibility and preliminary engineering study. MOWT usually employs a Consultant to conduct the detailed engineering study and to prepare consequent Tender Documentation. Financiers and/or donor agencies (if any involved) will supervise this process.

Several decisions which affect economic performance are taken throughout the design; and economic appraisal often results in redesign.

2.1.5 Procurement / Tendering

This stage involves negotiations with potential financiers, invitations to tender and negotiations with Consultants, contractors and suppliers.

Once financing arrangements have been finalised, the Employer, often in co-operation with the financiers, will prepare the tenders for consulting services, for construction of works and for supplies.

Chosen Consultants will usually assist to review the tender documentation, prepare invitations for contractors to pre-qualify, invite tenders for construction and analyse the tenders received from contractors and prepare Contracts to be signed by the Employer.

2.1.6 Construction Supervision and Management

At the beginning of this stage the Employer will enter into contracts for the construction and supervision of the works. These contracts usually will have to be reviewed by the financier.

Consultants are recruited to assist the Employer to ensure that procurement guidelines, contract drawings, specifications, schedules and other contract documents are followed in the execution of works.

Contractors are employed to execute the works in accordance with Contract Documents.

2.1.7 Operation

Once a certificate of completion has been issued and works have been taken over by the Employer, the constructed road or bridge may be put into use. It is during this phase that benefits of the infrastructure are realised and its maintenance undertaken.

2.1.8 Project Evaluation

The final phase of the project cycle is evaluation. This consists of looking back systematically at the successful and unsuccessful elements of the project experienced during implementation and the lessons acquired can help to learn how planning can be improved in the future.

For evaluation to be successful, it is important that data about the project is collected and recorded in a systematic way throughout all the stage of the project cycle. Project evaluation may be carried out using different methods. In the same cases when project evaluation has been carried out by MOWT, but if each stage of the project, that need to be brought to the attention of the project management.

Project evaluation should lead to specific recommendations about improving aspects of the project design which is then used to improve on-going and future planning.

2.2 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, earth works, 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 Section describes the methodology for analyzing possible corridors and selecting the optimum route from technical, economic, social and environmental considerations.

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

• What are the constraints in regard to the beginning and ending points of the road? Must these be at existing junctions in villages or towns? Do economic considerations such as amount of earthworks limit the alternatives?

• Through which villages must the route pass? Must the route pass directly through these villages, or can linking roads connect the villages? If so, what are the implications to the villages in terms of lost trade?

• 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?

• 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, and horizontal and vertical curves?

The desk study comprises a review of published and unpublished information concerning the physical, economic and environmental characteristics of the study area. Some of the data that may be required for the desk studies are the following sources:

• Published literature covering a range of topics including road construction and maintenance case histories and geological, economic and environmental reviews;

• Topographical maps;

• Geological maps, agricultural soil maps and other natural resource maps; and,

• Aerial photographs.

For studying and selecting suitable alignment corridors, a detailed analysis based on topographic maps, aerial photographs, geological maps, hydrological maps, land use and land cover maps and the like are required.

2.3 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, maintenance cost and operational cost.

Using the 1:50,000 scale maps and with knowledge of the controlling requirements/constraints as listed in Section 2.2, it is possible to trace out some possible 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. Assume also that the contour interval on the 1:50,000 maps are 20 meters. A preliminary alignment needs to be selected in such a way that a distance of no less than 200 meters (0.4 cm on the map) is used to achieve the 20-meter interval, giving a 10% grade.

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

- Topographic, geologic, and physical characteristics;
- Number, type and characteristics of water courses;
- Potential risk of slides, slope instability or floods;
- Human settlements affected by the road; and,
- Environmental impact of the selected route.

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

The evaluation criteria could be the following:

- What are the relative lengths of the alternatives? Normally the shortest distance is preferable.
- What are the average and mean gradients of the alternatives? Normally the least severe grade alternative is preferred. However, the relation of minimum grade may be the inverse to the shortest length route.
- Which alternative more closely follows an existing road or track? This makes survey and construction easier and may indicate the route of least earthworks.
- 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 curves than a route through mountainous terrain.
- Which route remains for a longer period on the crest of the terrain? Such an alignment minimizes the need for drainage structures.
- Which alignment minimizes the need for land acquisition? The amount of farm land to be taken by the road. Which alignment minimizes the need to demolish buildings and houses less resettlement?

• 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?

• Which route results in the least environmental disturbance to the surrounding area?

• Which route has the least overall project cost, including both design and construction?

2.4 Site Visit And Survey

After the preliminary office work, a site visit must be made to the road. Where terrain constraints made 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 make a site inspection visit:

• Highway Engineer;

• Soils and Materials Engineer;

• Hydrologist;

• Surveyor;

• Bridge/Structures Engineer;

• Environmentalist/Sociologist, and,

• Local Administrative Personnel.

In most cases, the information obtained from the reconnaissance survey will require to significantly modify 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:

• Topographic and geomorphologic characteristics;

• The location of topographical constrains, such as cliffs, gorges, ravines, rock out crops, and any other features not identified by the desk study;

• Slope steepness and limiting slope angles identified from natural and artificial slopes (cutting for paths, agricultural terraces and existing roads in the region);

• Slope stability and the location of pre-existing land slides;

• Geology, tectonics, rock types, geological structures, rock outcrops, dip orientations, rock strength and rip-ability;

• Approximate percentage of rock in excavations;

• Availability of construction materials sources and their distribution;

• Soil types and depth (a simple classification between residual soil and colluvium is useful at this stage);

- Soil erosion and soil erodibility;
- Slope drainage and groundwater conditions;
- Hydrology, drainage stability and the location of shifting channels and bank erosion;
- Land use, land cover and their likely effect on drainage;
- Likely foundation conditions for major structures;
- Approximate bridge spans and the sizing and frequency of culverts;
- Flood levels and river training/protection requirements;
- Environmental considerations, including forest resources, land use impacts and socio-economic considerations;
- Verify the accuracy of the information collected during the desk study;
- The possibility of using any existing road alignments including local re-alignment improvements; and
- 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.

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.

2.5 Recommendations

The route corridor selection process will be ended at the selection and recommendation of the best and most viable route for detailed study and design taking due account of construction cost, benefits to the local population, and length of travel for each alternative. It also gives details as to why a certain alignment alternative was selected and why others were rejected or not considered. This will be concluded with a preparation and submission of route corridor selection report, which selects and gives recommendation on the best route alignment for detailed study and design.

SECTION 3: ROAD SURVEY PROCEDURED AND REQUIREMENTS

3.1 General

3.1.1 Introduction

This Section presents the survey requirements associated with the geometric design process. Survey data for design purposes consists of mapping of sufficient detail for the level of design being undertaken. In most cases a digital terrain model (DTM) for use with computer design software may be required.

The survey data requirement is dependent on project type and can be collected by aerial photography, field topographical survey, or a combination of the two.

The project designer is responsible for identifying the appropriate survey data requirements (type of data, accuracy, and area of coverage). The project designer is also responsible for obtaining the survey data and for selecting the method of data collection.

The following factors should be considered when determining and deciding the survey data collection method:

1. Size and scope of the project;

2. Time requirements to move from data collection to the start of design;

3. Estimated data collection costs; and;

4. Level of accuracy and detail needed.

3.1.2 Units of Survey Measurement

All distances and heights shall be in meters.

Angular measurement shall be in degrees, minutes and seconds based on the 3600 circle.

3.1.3 Datum and Distance Measurement

Co-ordinates shall be based on the National Trigonometric System unless otherwise authorised.

Levels shall be referred to mean sea level and related to a Local National Bench Mark unless otherwise authorised.

All staked distance, must be horizontal.

3.1.4 Bench Marks

Standard bench mark establishment, leveling procedure and standard accuracy are gito be followed with the following limitations to be adhered to:

- A bench mark is to be established at an interval of max. 700 meters, depending on the terrain, along the line close to the right of way, and at all major structures (bridges and box culverts);
- Bench marks must be inter-visible;

- Standard bench bark is shown in Figure 3-1;
- Every bench mark is to be checked leveled by a forward run and a subsequent backward run forming a closed "loop"; and,
- The following standard of accuracy is to be maintained:

Where:
C = maximum permissible error of closure in centimeters,
K = distance between bench marks in kilometers

This gives Table 3-1 for comparison of accuracy

Table 3-1: Required Level of Accuracy for Surveys

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

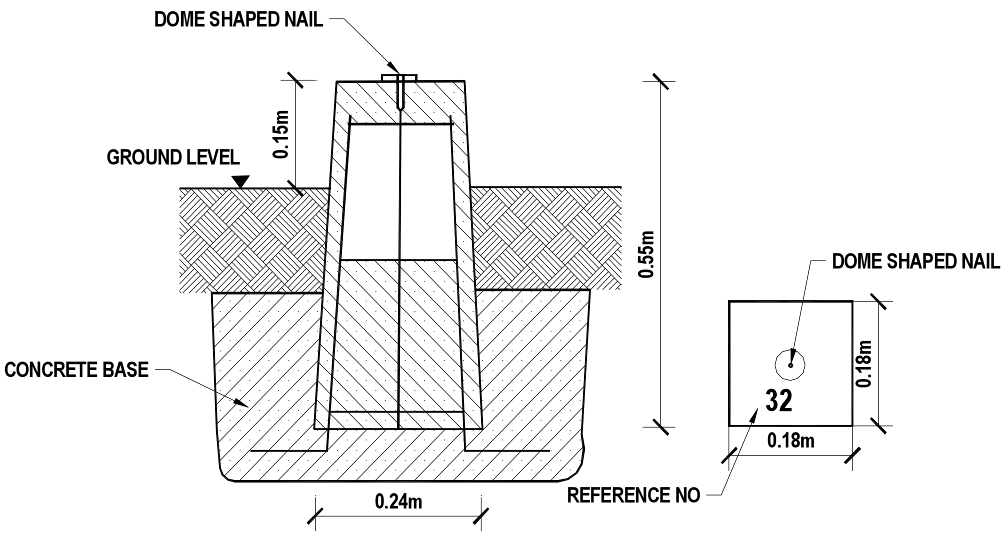


Figure 3-1: Typical Bench Mark (Concrete Beacon)

3.1.5 Survey Data

Survey data for road design purposes should have the required amount of data and mapping of enough detail for the level of design to be undertaken. For this purpose the use of computer design software is required and the surveying instruments should be appropriate for the job. The GPS (Global Positioning System) and the Total Station, are used whose out put should be compatible with the computer aided design (CAD) system.

As mentioned earlier survey data acquisition and collection method varies depending on the size and scope of the project, the design method, the type of design required, the time requirements to move from data collection to the start of design, the budget allocated for the data collection and the level of accuracy and detail needed.

Based on aforementioned the data collection method should either be by photogrammetry, ground survey or combination of the two.

3.2 Methods of Data Collection

3.2.1 Photogrammetry

The use of photogrammetry will require the establishment of a base line traverse and the commissioning of air photography at a scale of between 1:5,000 and 1:30,000. This method of data collection is preferable for mapping and DTM (Digital Terrain Model). 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. Photogrammetry from aerial photographs of 1:25,000 scale can yield uncontrolled contour mapping at a maximum scale of 1:5,000, with contours at 5-meter intervals. However, if a project is small, has dense foliage, or requires only mapping of limited features, a field survey is the logical choice. Some fieldwork like fixing wing points, primary control points, and GPS surveying will be required in order to accomplish air photography.

Elevations of photogrammetric DTM points on hard surfaces are accurate to within ±60 millimetres. 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 or data should come from ground surveys.

The scale of photography is an important factor to consider in the reliability and ground resolution of the interpretation. Table 3-2 gives guidance and indicates the optimum scales of photography required to perform various desk study and design tasks.

Table 3-2: Air 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	1: 10,000 - 1: 15,000
Ground (contour) model for preliminary alignment design and quantities	1: 10,000 - 1: 15,000
Detailed Design	
Ground (contour) model for detailed alignment design and quantities	1: 5,000 - 1: 10,000

The photogrammetric method of data collection follows the steps explained here below.

3.2.1.1 Aerial Photographing

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.

First the center line, left and right wing points of the selected route corridor will be marked on available small scale topographic map. The next step will be navigate using a topographic map, on which the selected route corridor is clearly marked, and establish on the ground the center and wing points with white painted 1.0m by 1.0m square concrete monument with their serial number clearly denoted.

Establish primary control points (bench marks) at an approximate interval of 5 km with two secondary control points close to each of the primary control points and pre-marking of same for low flight. Along an existing road, control points and bench marks are also to be established on head walls of structures (bridge or culvert). This is done by chiseling a cross and marking a square or a circle around the cross on the headwall of structures with a white paint.

These control points and the bench marks should be pre-marked in such a way that they can be easily 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 GPS on the already established center points, wing points, primary control points, secondary control points, and bench marks (by static method) and determine the X, Y, Z co-ordinates, thereby relating the coordinates with the National Grid System. Wherever difficulties arise for observations by a GPS, the survey crew can use the Total Station. Due to the required accuracy, the vertical coordinate “Z” is to be determined by a leveling crew using level. Differential leveling by run and check back will be conducted between consecutive bench marks.

3.2.1.2 Advantages of Aerial Mapping (Photogrammetry)

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

- It covers a wider area allowing for comparison and selection of the best route;
- Helps to avoid unnecessary disturbance of private or public properties when acquiring right of way for the final location of the route;
- 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; and,
- 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.

3.2.1.3 Limitations

The photogrammetric method of data collection has some limitations, which restrict the data acquisition and force to use the ground survey method. These conditions happen during smoke and mist occurrences over the surveyed areas during aerial photographing, hidden features by dense growth of ever green forests, right of way features, measurement of critical elevations at existing structures and similar features. In such cases the use of the ground survey is required. In addition to this the method is relatively expensive for small scale work and also the level accuracy is not high as compared to ground survey.

3.2.2 Ground Survey

An approximate alignment should first be drawn onto plotted contour maps and enlarged prints of aerial photographs in the office prior to embarking on detailed fieldwork. Having the approved route corridor, the Location Engineer, with a survey team, will flag the approximate centerline.

Topographical ground surveys should use appropriate surveying equipment such as Total Stations or GPS to collect data in respect of road alignment and cross sections and all bridge sites and culvert sites 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.

In order to handle the ground surveying work for detailed design of road properly the following survey crews should be formed and organized.

1. The location crew.
2. The GPS crew.
3. The leveling crew.
4. The Total Station crew.

The location crew: - headed by the Location Engineer will do the flagging, monumenting and staking out works.

First, the line will be flagged using the ranging poles and the abney 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.

The GPS crew: - this crew establishes monuments for the traverse control points, along the selected and flagged line. A control traverse should be established using GPS or coordinated and tied into the National Grid System. These control points 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 centerline, since they should stay undisturbed at least until the construction time to be used as references. They should be established either from concrete, or on rock out crops or simply buried large boulders clearly marked or on head walls of existing structures, over which Total Station instrument could stand. The primary control points will be established at approximately every 5kms.

Two inter-visible secondary control points should be established for each primary control point, for back sighting and as reserve points in case any one of them is lost or damaged.

The primary control points should be coordinated in X, Y, Z and should be tied to the National Grid System.

The leveling crew: - The leveling crew establishes all the bench marks on the left or right hand side of the center line and runs the differential leveling between consecutive bench marks in a closed loop system that is by running the forward run, and by running the check back between the same two consecutive bench marks. The leveling work, specially the differential leveling is separately done because in the Z co-ordinate the GPS is not reliable. It is only the engineer's level that is the most accurate instrument as far as the vertical control is concerned. And if available the digital level is recommended otherwise an optical level.

The Total Station crew: - The Total Station crew will handle the profile leveling, the cross section leveling, and all other topographical survey works including details of bridge crossings, culverts, town sections and other areas which need detail survey.

The Total Station will stand on the primary control point and will take orientation on the inter-visible secondary control points and will continue its work on topography and detail and close it on the next primary control point, there by checking the error of closure. If it's within the allowable the routine work will continue, if not the former activity will be repeated. The error from the proceeding successive control points, 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 and 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 can be checked and verified by the surveyors shortly after the fieldwork.

3.3 Road Survey Procedures

3.3.1 Location

Location Survey may be undertaken in two alternative ways: the first one is location by flagging, monumenting and centerline marking by using the transit and the second one is without the use of the transit.

3.3.1.1 Flagging, Monumenting and Centerline Marking

a) <u>Along New Road Alignment</u>

This procedure will be followed for new locations where the centerline has to be fixed at field.

The flagging crew flags the line using the abney level and ranging poles, then the transit crew follows and makes the monumenting, of PIs, POT's and RP's there by establishing the PI's, POT's and the RP'S (Reference points for the PI's). Two reference points will be established for every PI, whose distances from the PI and the angle from the back tangent are observed using transit and recorded in the field book.

Then the marking of centerline of road or staking will continue. Centerline marking or stations are normally staked at every approximately 20 meters, both at straights (tangents) and at curves. If curves are sharp, then they will be staked at every approximately 10 meters.

b) <u>Along Existing Alignment</u>

For existing roads and even for some new locations whose centerline need not be fixed at the field (at the option of the Engineer in charge of the design), one has to follow the location method without transit. In such cases mounmenting and staking will be omitted. Based on the desktop studies of the aerial photographs and topographic maps the selected corridor will be located and the traversing will be shown by use of only ranging poles and paints.

Then the Total Station crew will follow and take details along the selected traverse line for a certain band width of road corridor. In this case PI's are located on the ground using ranging pole and the transit party may not be required. The leveling crew will of course run the differential leveling.

3.3.1.2 Transferring PI's on Aerial photos

After location of PI's on the ground in both methods discussed above, the Location Engineer or the photogrammetrist transfers the PI's and traces the route alignment itself at almost their exact positions on the aerial photographs of the area through which the route passes. Then the

PIs are pinpointed on the photographs. Simultaneously the location of the crossing points of major and minor drainages will be fixed. After fixing the crossing of the drainage the relevant catchment areas of the respective major and minor drainages will be delineated. Catchment areas for major drainages will be delineated on topographic maps scale 1:50,000 or smaller. Catchment areas of minor drainages, which can not be delineated on topographic maps will be delineated on aerial photographs. Finally, the areas of delineated catchment areas will be calculated either by a planimeter or by a computer after being digitized in order to be used for further hydrological and hydraulic studies.

3.3.2 GPS Survey

A control traverse should be established using GPS or coordinated and tied into the National Grid System. These points shall be referenced in the field in permanent concrete posts and shall be shown on the plan and profile drawings. GPS survey is commenced by monumenting and establishing of primary control traverse points at approximately 5kms interval and also secondary control points and taking reading of them using GPS. Concrete beacons made from pre-cast concrete will be used as monument for control points and buried firmly and permanently into the ground. These control points should be tied to the National Grid System. It is from these control points taking as a basis that every detail survey will be carried out.

3.3.3 Leveling

The leveling work will start after bench marks are established at appropriately every 300-500meters and lower in rough terrains and 500 – 700 meters in flat terrains. Additionally temporary bench marks have to be established on either side of bridge crossings. Bench Marks will be located outside the road limit and where one is visible from the other along the road.

Following the establishment of benchmarks, differential Leveling will be carried out. This is a closed loop leveling run between two bench marks 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 disclosure on each previously established Bench mark shall be within the prescribed tolerance. Where the difference is outside this limit the run must be repeated.

The profile and cross section leveling can be done simultaneously by the Total Station crew. This is done by taking the reading of intermediate sights of the full stations and odd stations (breaks) along the centerline. At the same time the cross sections of the respective full and odd stations whose profile was taken will also be taken simultaneously. This is done by taking readings left and right of the centerline for at least 50-75 meters offset on either side. The offset can be more depending on the terrain and the type of work done, especially along stream beds and river channels.

Profile leveling will be run between each pair of consecutive benchmarks, previously established, and the leveler must close on each successive bench marks 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 bench marks in the case of failing light, a sudden storm, etc., a Temporary bench mark (TBM) shall be established from which the work may be resumed the next time.

3.3.4 Detail Survey

Detailed topographic survey of bridge crossings, stream lines, towns, villages etc. will be carried out. Road edges, cuts, ditch edges, culverts, hilltops, water crossings and embankments will be taken. All physical features adjacent to the line whether natural or artificial are to be recorded within the required band width on either side of the centerline in open country and in small villages (market centers) and towns. Each cross section will comprise 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 will be used to generate a Digital Terrain Model (DTM) for the whole road. All pertinent features including buildings, drainage structures details, built up areas, etc. will be recorded for inclusion on the design drawings. Detailed and extensive site investigation and surveys shall 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.

3.3.5 Bridge Sites

On bridge sites detailed topographic survey will be done at 10 to 20 meters interval for a length of 100-400meters on both upstream and downstream. This activity will also be done on both approaches of the bridge site for a length of 100 to 200 meters depending on the terrain.

The collection of data should be conducted in such a way as to acquire a complete and comprehensive field data. On an existing structure the data shall be collected at the top deck surface, and corresponding invert (bed level on both sides) and also at existing and high water mark including existing water level, with the date of data collection properly recorded, including all particular features of the bridge, flow channel and the surroundings. Additionally data should be taken at several points top and bottom on the wing walls and all feature points of approach roads and also high water mark and existing water level with its date of data collection. For new crossing data should be collected top of banks, bottom, center, high water mark, existing water level and other natural features of the flow channel should be taken. Figure 3-2: is a sketch for guidance of surveyors in topographic survey of water crossing or drainage structures sites.

3.3.6 Town Sections

Whenever the line passes through villages or towns detailed survey works will be done to show the natural and man made objects within the right of way.

3.4 Collecting and Transferring Of Data

3.4.1 Collecting of Data

Survey data requests will typically originate from the unit responsible for the design, and these data should also serve the requirements of construction. The project designer has the responsibility to ensure that survey data obtained by design meets construction needs, eliminating the need for additional pre-construction ground data.

Data can be collected and recorded by Total Station either by inbuilt memory, memory card or by an electronic data collector electronic (field book) in the following formats.

a) The SVH Format – Slope distance, vertical angle and horizontal angle format; and,

b) The NEZ (ENZ) format- this is the co-ordinate format whose the northing, easting and the Z coordinate (elevation) format.

Data can also be collected and recorded using suitable survey equipment other than Total Station, capable of collecting the required survey data and mapping data for design.

3.4.2 Data transferring (output)

The collected and recorded data should be transferred to a computer. This can be done in the following ways.

1. If the data was collected by a memory card then the data will be transferred through a card reader to the computer by using cables.

2. If the data was collected by in-built memory of Total Station, then the data could be downloaded by directly connecting the Total Station with the computer with a cable.

3. If the data was collected and recorded by a data collector (electronic filed book) then the data will be transferred directly from the data collector to the computer by using the appropriate cables.

The transferred or downloaded data will be sent to the design office by a diskette or a CD to generate the digital terrain model and for further design works.

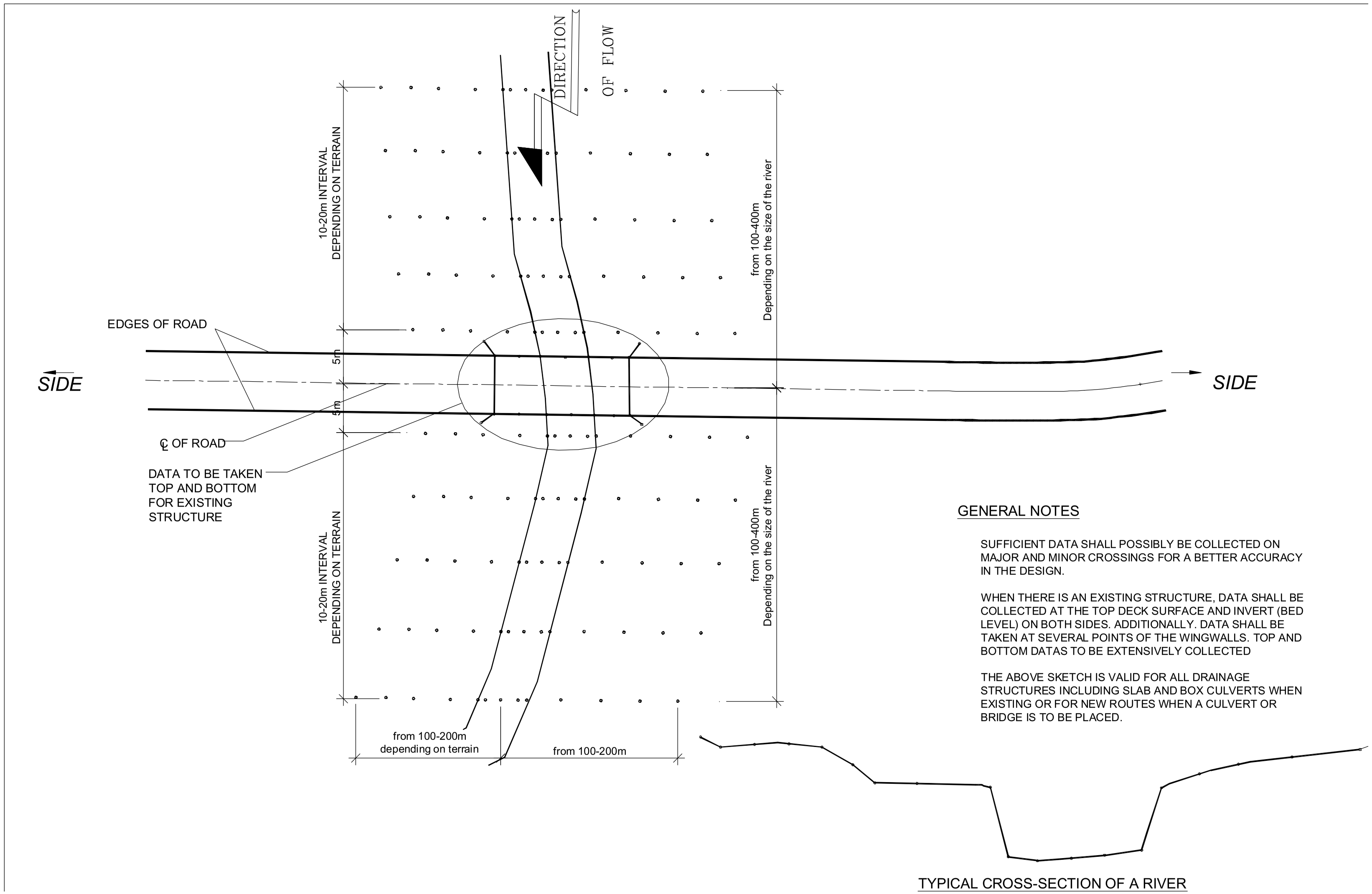


Figure 3-2: Guide for topographic survey of water crossing or drainage structures sites.

SECTION 4: THE ROAD SYSTEM

4.1 General

Roads have two basic traffic service functions which, from a design standpoint, are incompatible. These functions are:

a) to provide traffic mobility between centers and areas; and,

b) to provide access to land and properties adjoining the roads.

For roads whose major function is to provide mobility, i.e. to cater for through and long-distance traffic, high and uniform speeds and uninterrupted traffic, high flows are desirable. For roads whose major function is to provide land access, high speeds are unnecessary and, for safety reason, undesirable. Thus, the function of a particular road in the national, regional and local road network has a significant impact on the design criteria to be chosen, and the design engineer has to give careful consideration to this aspect in the early stages of the design process. The following steps are required:

1. Classification of the road in accordance with its major function.
2. Determination of the level of access control compatible with the function of the road.
3. Selection of geometric design standards compatible with function and level of access control.

When the functional classification and level of access control has been selected, design standards can be applied which will encourage the use of the road as intended. Design features that can convey the level of functional classification to the driver include carriageway width, continuity of alignment, spacing of functions, frequency of access, standards of alignment and grades, traffic controls and road reserve widths.

4.2 Concept of Classification

The Road Functional Classification establishes a hierarchy of roads according to the importance of each road in the road network, the socio-economic function they serve and intend to serve. The classification enables the agency that is responsible for the development and maintenance of the roads to plan and programme road maintenance and upgrading works. Design standards and level of maintenance are directed by the road functional class, together with other indicators such as existing and predicted traffic on the road.

The functional classification of a road is based on its proposed function and roads grouped under a particular category or class will be characterised by the level of service they provide.

Functional classification of road is one of the aspects that a road design class should follow or depend on.

4.3 Division into Functional Class

The rural roads in Uganda are divided into the following 5 classes according to their major function in the road networks.

Class A: International Trunk Roads

Roads that link International Important Centers. Connection between the national road system and those of neighbouring countries. Major function is to provide mobility

Class B: National Trunk Roads

Roads that linking provincial capitals, main centers of population and nationally important centers. Major function is to provide mobility

Class C: Primary Roads

Roads linking provincially important centers to each other or to a higher class roads (urban/rural centers). Linkage between districts local centers of population and development areas with higher class road. Major function is to provide both mobility and access

Class D: Secondary Roads

Roads linking locally important centers to each other, to a more important centers, or to a higher class roads (rural/market centers) and linkage between locally important traffic generators and their rural hinterland. Major function is to provide both mobility and access.

Class E: Minor Roads

Any road link to minor center (market/local center) and all other motorable roads. Major function is to provide access to land adjacent to the secondary road system.

Roads of the highest classes, A and B, have as their major function to provide mobility and have longer trip lengths. They are required to provide a high level of service with a high design speed. The roads of Classes C and D serve a dual function in accommodating shorter trips and feeding the higher classes or road. For these roads an intermediate design speed and level of service is required. Road Class E have short trip length and their primary function is to provide access. Design speeds and level of service for these roads may be low.

4.4 Control Of Access

Uncontrolled access to road side development along whose major function is to provide mobility will result in an increased accident hazard, reduced capacity and early obsolescence of the roads. In order to preserve major roads as high standard traffic facilities it is necessary to exercise access control, whereby the right of owners or occupants of land to access is controlled by the Road Authority.

Although control of access is one of the most important means for preserving the efficiency and road safety of major roads, roads with out access control are equally essential as land service facilities. The following three levels of access control are applicable:

(1) Full access control:- means that the authority to control access is exercised to give preference to through traffic by providing access connections with selected public roads only and by prohibiting direct access connections.

(2) Partial access control:- means that the authority to control access is exercised to give preference to through traffic to a degree in that, in addition to access connections with selected public roads, there may be (some) private access connections.

(3) Unrestricted access:- means that preference is given to local traffic, with the road serving the adjoining areas through direct access connection. However, the detailed location and layout of the accesses should be subject to approval by the Road Authority in order to ensure adequate standards of visibility, surfacing, drainage, etc.

Road function determines the level of access control needed. Roads of higher classes have their major function to provide mobility, while the function of lower classes is to provide access. Motorways should always have full control of access. For all purpose roads the following general guidelines are given for the level of access control in relation to the functional road classification:

Functional Class	Level of Access Control	
	Desirable	Reduced
A	Full	Partial
B	Full or Partial	Partial
C	Partial or Unrestricted	Partial
D	Partial	Unrestricted
E	Partial or Unrestricted	Unrestricted

The reduced levels of access control may have to be applied for some road projects because of practical and financial constraints.

Control of access is accomplished either by the careful location of accesses, by grouping accesses to reduce the number of separate connections to the through traffic lanes or by constructing service roads which intercept the individual accesses and join the through lanes at a limited number of properly located and designed junctions. In every case the location and layout of all accesses, service roads and junctions should be carefully considered at the design stage and include in the final design for the project.

4.5 Division Into Road Design Class

There are six Design Classes of road. Design Class road I, II, & III are bitumen surfaced. Design Class A, B, & C are gravel surfaced. Design class I is further divided into two. Ia is four lane and Ib two lane. The division into Road Design Class is governed by the design speed and design traffic. There are many factors that affect the capacity of a road. These include design speed, width, lateral clearance, grade, alignment, weaving sections, ramp Terminals, traffic composition, type of surface, and level of service. It is therefore not possible to stipulate precise design volumes for each class of road. The values given in the table should be used as a guide. Each road must be assessed individually during the feasibility and preliminary design stage.

Table 4-2a: Road Design Classes

Design Class	Capacity [pcu x 1,000/day]	Road-way width[m]	Maximum Design speed Kph			Functional Classification				
			Level	Rolling	Mountainous	A	B	C	D	E
Ia Paved	12 - 20	20.80-24.60	120	100	80	√				
Ib Paved	6 – 10	11.0	110	100	80	√	√			
II Paved	4 – 8	10.0	90	70	60	√	√	√		
III Paved	2 – 6	8.6	80	70	50	√	√	√		
A Gravel	4 – 8	10.0	90	80	70		√	√	√	
B Gravel	2 – 6	8.6	80	60	50				√	√
C Gravel		6.4	60	50	40					√

Table 4-2: Road Design Classes (continued)

Design class	Right of Way width [m]	Road way width [m]	Carriage way			Shoulder width [m]	Median width [m]
			Width [m]	Lane width [m]	No. of lane		
Ia Paved	60	20.80-24.60	14.6	3.65	4	2 x 2.5	1.2 – 5.0
Ib Paved	60	11.0	7.0	3.5	2	2 x 2.0	-
II Paved	50	10.0	6.0	3.0	2	2 x 2.0	-
III Paved	50	8.6	5.6	2.8	2	2 x 1.5	-
A Gravel	40	10.0	6.0	3.0	2	2 x 2.0	-
B Gravel	30	8.6	5.6	2.8	2	2 x 1.5	-
C Gravel	30	6.4	4.0	4.0	1	2 x 1.2	-

For the four lane road (Bitumen Ia), from a minimum median width of 1.2 in order to accommodate barrier for the median up to a median width of 4.8 m, which will provide a 3.6 m wide lane for right-turning movement and 1.2 m median separator will be required.

Table 4-3: Geometric Design Parameters for Design Standard Ia Paved (dual carriageway)

Design Element	Unit	Flat	Rolling	Mountainous	Urban/Peri-Urban
Design Speed	km/h	120	100	80	50
Min. Stopping Sight Distance	m	205	160	115	60
Min. Passing Sight Distance	m	795	670	545	345
Min. Horizontal Curve Radius	m	710	415	240	100
Max. Gradient (desirable)	%	3	4.5	6	6
Max. Gradient (absolute)	%	4	6.5	8	8
Minimum Gradient in cut	%	0.5	0.5	0.5	0.5
Maximum Superelevation	%	7	7	7	4
Crest Vertical Curve stopping	K_{min}	103	80	32	9
Crest Vertical Curve passing	K_{min}	664	475	310	126
Sag Vertical Curve stopping	K_{min}	50	37	25	11
Normal Cross fall	%	2.5	2.5	2.5	2.5
Shoulder Cross fall	%	4	4	4	4
Right of Way	m	60	60	60	40

Table 4-4: Geometric Design Parameters for Design Standard Ib Paved

Design Element	Unit	Flat	Rolling	Mountainous	Urban/Peri-Urban
Design Speed	km/h	120	100	80	50
Min. Stopping Sight Distance	m	205	160	115	60
Min. Passing Sight Distance	m	795	670	545	345
Min. Horizontal Curve Radius	m	710	415	240	100
Max. Gradient (desirable)	%	3	4.5	6	6
Max. Gradient (absolute)	%	4	6.5	8	8
Minimum Gradient in cut	%	0.5	0.5	0.5	0.5
Maximum Superelevation	%	7	7	7	4
Crest Vertical Curve stopping	K_{min}	103	80	32	9
Crest Vertical Curve passing	K_{min}	664	475	310	126
Sag Vertical Curve stopping	K_{min}	50	37	25	11
Normal Cross fall	%	2.5	2.5	2.5	2.5
Shoulder Cross fall	%	4	4	4	4
Right of Way	m	60	60	60	40

Table 4-5: Geometric Design Parameters for Design Standard II Paved

Design Element	Unit	Flat	Rolling	Mountainous	Urban/Peri-Urban
Design Speed	km/h	90	70	60	50
Min. Stopping Sight Distance	m	135	95	75	58
Min. Passing Sight Distance	m	605	485	410	345
Min. Horizontal Curve Radius	m	320	185	130	100
Max. Gradient (desirable)	%	3.5	5.5	6	6
Max. Gradient (absolute)	%	5.5	7.5	8	8
Minimum Gradient in cut	%	0.5	0.5	0.5	0.5
Maximum Superelevation	%	7	7	7	4
Crest Vertical Curve stopping	K_{min}	43	22	14	9
Crest Vertical Curve passing	K_{min}	307	246	176	126
Sag Vertical Curve stopping	K_{min}	30	20	15	11
Normal Cross fall	%	2.5	2.5	2.5	2.5
Shoulder Cross fall	%	4	4	4	4
Right of Way	m	50	50	50	30

Table 4-6: Geometric Design Parameters for Design Standard III Paved

Design Element	Unit	Flat	Rolling	Mountainous	Urban/Peri-Urban
Design Speed	km/h	80	70	50	50
Min. Stopping Sight Distance	m	115	95	60	60
Min. Passing Sight Distance	m	545	485	345	345
Min. Horizontal Curve Radius	m	240	185	85	100
Max. Gradient (desirable)	%	4	5.5	9	9
Max. Gradient (absolute)	%	6	7.5	11	11
Minimum Gradient in cut	%	0.5	0.5	0.5	0.5
Maximum Superelevation	%	7	7	7	4
Crest Vertical Curve stopping	K_{min}	32	22	9	9
Crest Vertical Curve passing	K_{min}	310	246	126	126
Sag Vertical Curve stopping	K_{min}	25	20	11	11
Normal Cross fall	%	2.5	2.5	2.5	2.5
Shoulder Cross fall	%	4	4	4	4
Right of Way	m	50	50	50	30

Table 4-7: Geometric Design Parameters for Design Standard A Gravel

Design Element	Unit	Flat	Rolling	Mountainous	Urban/Peri-Urban
Design Speed	km/h	90	80	70	50
Min. Stopping Sight Distance	m	135	115	95	60
Min. Passing Sight Distance	m	605	545	485	345
Min. Horizontal Curve Radius	m	320	240	185	100
Max. Gradient (desirable)	%	3.5	5	7	7
Max. Gradient (absolute)	%	5.5	7	9	9
Minimum Gradient in cut	%	0.5	0.5	0.5	0.5
Maximum Superelevation	%	7	7	7	4
Crest Vertical Curve stopping	K_{min}	43	32	22	9
Crest Vertical Curve passing	K_{min}	307	310	246	126
Sag Vertical Curve stopping	K_{min}	30	25	20	11
Normal Cross fall	%	4	4	4	4
Shoulder Cross fall	%	4	4	4	4
Right of Way	m	40	40	40	30

Table 4-8: Geometric Design Parameters for Design Standard B Gravel

Design Element	Unit	Flat	Rolling	Mountainous	Urban/Peri-Urban
Design Speed	km/h	80	60	50	50
Min. Stopping Sight Distance	m	115	75	60	60
Min. Passing Sight Distance	m	545	410	345	345
Min. Horizontal Curve Radius	m	240	130	85	100
Max. Gradient (desirable)	%	4	6	9	9
Max. Gradient (absolute)	%	6	8	11	11
Minimum Gradient in cut	%	0.5	0.5	0.5	0.5
Maximum Superelevation	%	7	7	7	4
Crest Vertical Curve stopping	K_{min}	32	14	9	9
Crest Vertical Curve passing	K_{min}	310	176	126	126
Sag Vertical Curve stopping	K_{min}	25	15	11	11
Normal Cross fall	%	4	4	4	4
Shoulder Cross fall	%	4	4	4	4
Right of Way	m	30	30	30	30

Table 4-9: Geometric Design Parameters for Design Standard C Gravel

Design Element	Unit	Flat	Rolling	Mountainous	Urban/Peri-Urban
Design Speed	km/h	60	50	40	50
Min. Stopping Sight Distance	m	75	60	45	60
Min. Passing Sight Distance	m	410	345	285	345
Min. Horizontal Curve Radius	m	130	85	55	100
Max. Gradient (desirable)	%	5	7	10	7
Max. Gradient (absolute)	%	7	9	12	9
Minimum Gradient in cut	%	0.5	0.5	0.5	0.5
Maximum Superelevation	%	7	7	7	4
Crest Vertical Curve stopping	K_{min}	22	14	5	9
Crest Vertical Curve passing	K_{min}	246	176	86	126
Sag Vertical Curve stopping	K_{min}	15	11	8	11
Normal Cross fall	%	4	4	4	4
Shoulder Cross fall	%	4	4	4	4
Right of Way	m	30	30	30	30

Tables 4-3 – 4-9 Combined: Geometric Design Parameters for all the Design Classes of roads

Design Element	Unit	Paved Ia				Paved Ib				Paved II				Paved III				Gravel A				Gravel B				Gravel C			
		Flat	Rolling	Moun-tainous	Urban / Peri-Urban	Flat	Rolling	Moun-tainous	Urban / Peri-Urban	Flat	Rolling	Moun-tainous	Urban / Peri-Urban	Flat	Rolling	Moun-tainous	Urban / Peri-Urban	Flat	Rolling	Moun-tainous	Urban / Peri-Urban	Flat	Rolling	Moun-tainous	Urban / Peri-Urban	Flat	Rolling	Moun-tainous	Urban / Peri-Urban
Design Speed	km/h	120	100	80	50	110	100	80	50	90	70	60	50	80	70	50	50	90	80	70	50	80	60	50	50	60	50	40	50
Min. Stopping Sight Distance	m	205	160	115	60	180	160	115	58	135	95	75	58	115	95	60	60	135	115	95	60	115	75	60	60	75	60	45	60
Min. Passing Sight Distance	m	795	670	545	345	730	670	545	345	605	485	410	345	545	485	345	345	605	545	485	345	545	410	345	345	410	345	285	345
Min. Horizontal Curve Radius	m	710	415	240	100	530	415	240	100	320	185	130	100	240	185	85	100	320	240	185	100	240	130	85	100	130	85	55	100
Max. Gradient (desirable)	%	3	4.5	6	6	3	4.5	6	6	3.5	5.5	6	6	4	5.5	9	9	3.5	5	7	7	4	6	9	9	5	7	10	7
Max. Gradient (absolute)	%	4	6.5	8	8	4	6.5	8	8	5.5	7.5	8	8	6	7.5	11	11	5.5	7	9	9	6	8	11	11	7	9	12	9
Minimum Gradient in cut	%	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Maximum Superelevation	%	7	7	7	4	7	7	7	4	7	7	7	4	7	7	7	4	7	7	7	4	7	7	7	4	7	7	7	4
Crest Vertical Curve stopping	K_{min}	103	80	32	9	103	62	32	9	43	22	14	9	32	22	9	9	43	32	22	9	32	14	9	9	22	14	5	9
Crest Vertical Curve passing	K_{min}	664	475	310	126	664	475	310	126	307	246	176	126	310	246	126	126	307	310	246	126	310	176	126	126	246	176	86	126
Sag Vertical Curve stopping	K_{min}	50	37	25	11	50	37	25	11	30	20	15	11	25	20	11	11	30	25	20	11	25	15	11	11	15	11	8	11
Normal Cross fall	%	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	4	4	4	4	4	4	4	4	4	4	4	4
Shoulder Cross fall	%	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4
Right of Way	m	60	60	60	40	60	60	60	60	50	50	50	30	50	50	50	30	40	40	40	30	30	30	30	30	30	30	30	30

SECTION 5: DESIGN CONTROL AND CRITERIA

5.1 Design Vehicles

Both the physical characteristics including turning capabilities of vehicles and the proportions of variously sized vehicles using the road are positive controls in geometric design. Therefore, it is necessary to examine all vehicle types, select general class groupings, and establish representatively sized vehicles within each class for design use. Vehicle characteristics affecting design include power to weight ratio, minimum turning radius, travel path during a turn, vehicle height and width. The main road elements affected are gradient, road widening in horizontal curves and junction design. In the design of road facility the largest design vehicle likely to use that facility with considerable frequency or a design vehicle with special characteristics that must be taken into account in dimensioning the facility is used to determine the design of such critical features as radii at intersections and radii of horizontal curves of roads.

The present vehicle fleet in Uganda includes a high number of four-wheel drive passenger/ utility vehicles, buses and overloaded trucks. Accordingly the five design vehicles indicated in Table 5.1 will be used in the control of geometric design until a major change in the vehicle fleet is observed and detailed information on the different vehicle types using the roads in Uganda becomes available.

Table 5-1: Dimensions of Design Vehicle

Design Vehicle type	Symbol	Overall (m)			Overhang (m)		Wheel base (m)	Minimum design turning radius (m)	Minimum inside radius (m)
		Height	width	Length	Front	Rear			
4 x 4 passenger car	DV-1	1.3	2.1	5.8	0.9	1.5	3.4	7.3	4.2
Single unit truck	DV-2	4.1	2.6	9.1	1.2	1.8	6.1	12.8	8.5
Single unit bus	DV-3	4.1	2.6	12.1	2.1	2.4	7.6	12.8	7.4
Semitrailer combination large	DV-4	4.1	2.6	16.7	0.9	0.6	6.1 & 9.1	13.7	5.8
Interstate Semitrailer	DV-5	4.1	2.6	21.0	1.2	0.9	6.1 & 12.8	13.7	2.9

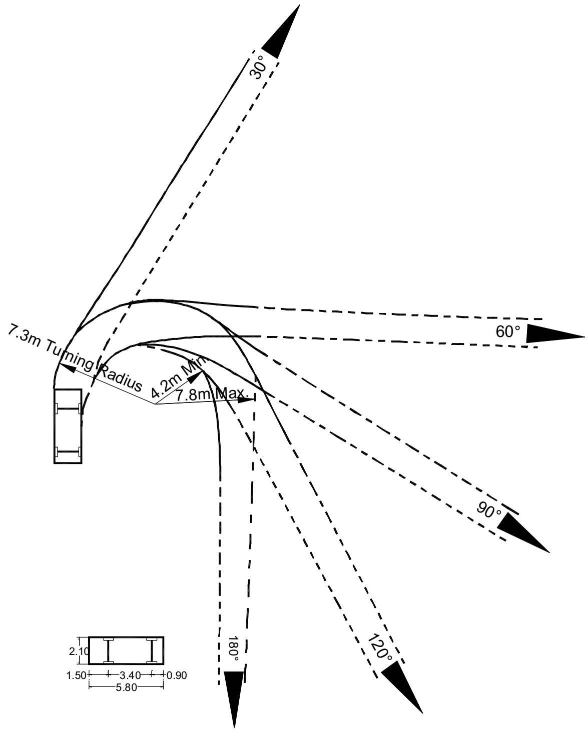


Figure 5-1: Turning path for design Vehicle no. 1 (DV-1)

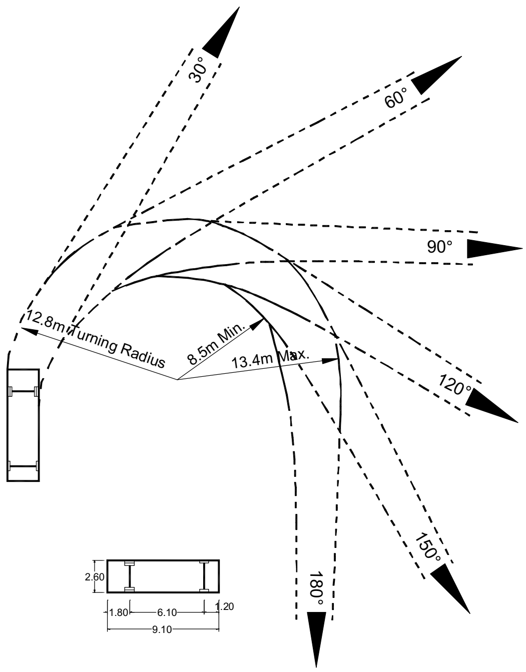


Figure 5-2: Turning path for design Vehicle no. 2 (DV-2)

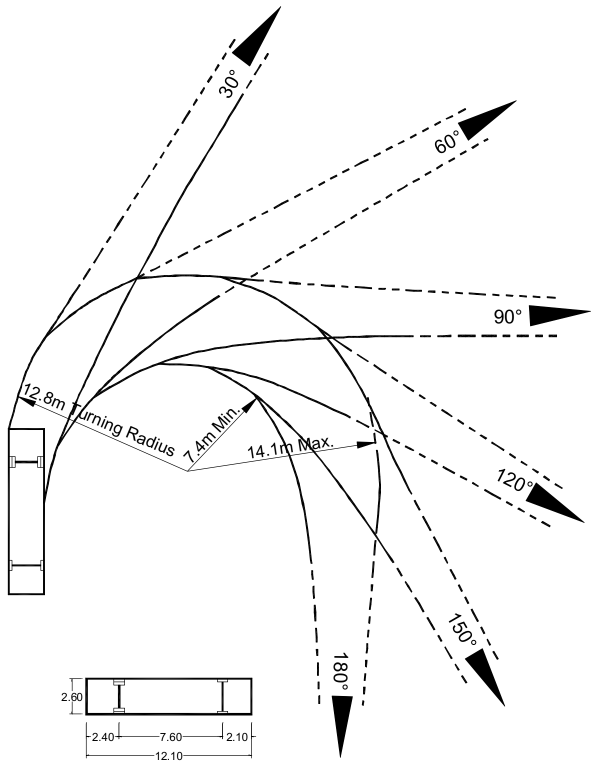


Figure 5-3: Turning path for design Vehicle no. 3 (DV-3)

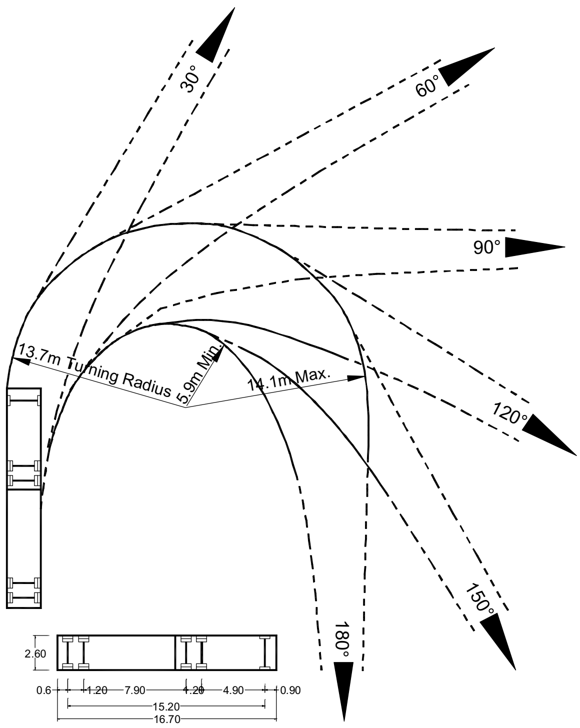


Figure 5-4: Turning path for design Vehicle no. 4 (DV-4)

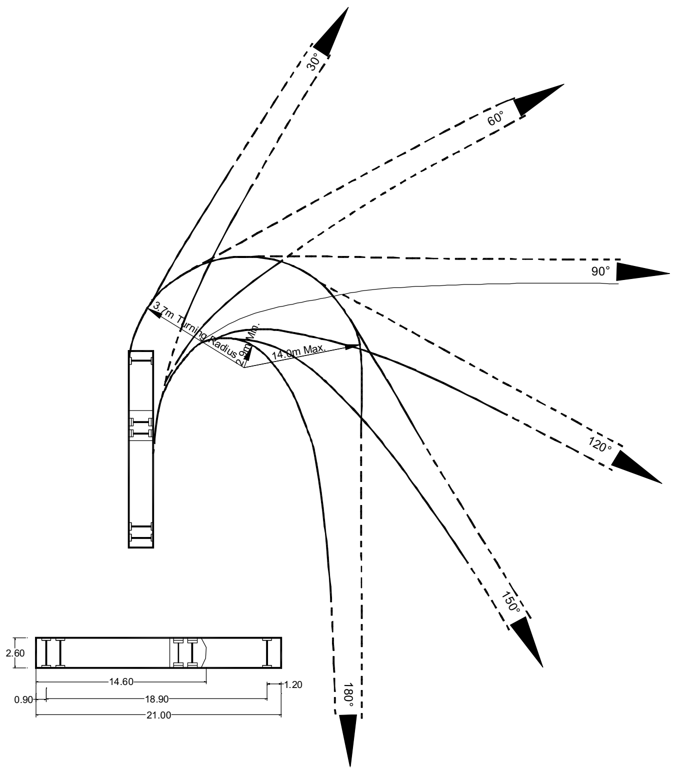


Figure 5-5: Turning path for design Vehicle no. 5 (DV-5)

5.2 Terrain

The geometric design elements of a road depend on the transverse terrain of land through which the road passes. Transverse terrains are categorized into four classes as follows:

Flat: Level or gently rolling country which offers few obstacles to the construction of a road having continuously unrestricted horizontal and vertical alignment (transverse terrain slope around 5%).

Rolling: Rolling, hilly or foothill country where the slopes generally rise and fall moderately gently and where occasional steep slopes may be encountered. It will offer some restrictions in horizontal and vertical alignment. (20% ≥ transverse terrain slope > 5%).

Mountainous: Rugged, hilly and mountainous country and river gorges. This class of terrain imposes definite restrictions on the standard of alignment obtainable and often involves long steep grades and limited sight distances (70% ≥ transverse terrain slope > 20%)

Escarpment: In addition to the terrain classes given above, a fourth class is added to cater those situations whereby the standards associated with each of the above terrain types cannot be met. Escarpment situations are those places where it is required switchback road alignments or side hill traverse sections where earthwork quantities are huge (transverse terrain slope >70%).

5.3 Driver Performance

Appreciation and consideration of driver performance is essential to proper highway design and operation. When a design is incompatible with human capabilities (both the driver and any other road user) the opportunities for error and accident increases. Knowledge of human performance, capabilities and behavioral characteristics is thus a vital input into the design task.

All road users do not behave in the same way and designs should cater for substantial differences in the range of human characteristics and a wide range of responses. Elderly drivers and pedestrians with a variety of age-related sensory-motor impairments have the potential to adversely affect the highway system’s safety and efficiency. Therefore:

- A road should conform what drivers expect based on previous experience; and,
- Drivers should be presented with clear clues about what is expected of them.

The driving task is a complex task of performing several activities simultaneously based on the information received in transit and information already possessed and making decisions and control. The driver work load comprises of:

- Navigation: trip planning and route following;
- Guidance: following the road and maintaining a safe path in response to traffic conditions; and,
- Control: steering and speed control

These tasks require the driver to receive and process inputs, consider the outcome of alternative actions, decide on the most appropriate, execute the action and observe its effects through the reception and processing of new information. There are numerous problems inherent in this sequence and tasks, arising from both the capabilities of the human driver, and the interfaces between the human driver, and the interfaces between the human and other components of the road traffic system (the road and the vehicle). These include adequate or insufficient input available for the task at hand (e.g. during night time driving, as a result of poor sight distance, or because of complex intersection layouts). When they become overloaded, drivers shed part of the input to deal with that judged to be more important. Most importantly, drivers are imperfect decision-makers and may make errors, including in the selection of what input to shed.

The designer must provide all the information the driver needs to make a correct decision timeously, simultaneously ensuring that the information is provided at a tempo that does not exceed the driver's ability to absorb it. A common characteristic of many high accident locations is that they place large or unusual demands on the information-processing capabilities of drivers. Insufficient operation and accident usually occur where the chance for information-handling errors is high. At locations where the design is deficient, the possibility of error and inappropriate driver performance increases.

Prior experience develops into a set of expectancies that allows for anticipation and forward planning, and these enable the driver to respond to common situations in predictable and successful ways. If these expectancies are violated, problems are likely to occur, either as a result of a wrong decision or of an inordinately long reaction time. There are three types of driver expectancy:

Continuation expectancy:- This is the expectation that the events of the immediate past will continue. It results, for example, in small headways, as drivers expect that the leading vehicle will not suddenly change speed. One particularly perverse aspect of continuation expectance is that of subliminal delineation, e.g. a line of poles or trees or lights at night which suggests to the driver that the road continues straight ahead when, in fact, it veers left or right. These indications are subtle, but should always be locked out for during design.

Event expectancy:- This is the expectation that events that have not happened will not happen. It results, for example, in disregard for "at grade" railway crossings and perhaps for minor intersections as well, because drivers expect that no hazard will present itself where none has been seen before. A response to this situation is more positive control, such as an active warning device at railway crossings that requires that the driver respond to the device and not to the presence of a hazard.

Temporal expectancy:- This is the expectation that, where events are cyclic (e.g. traffic signals), the longer a given state prevails, the greater is the likelihood that change will occur. This, of course, is a perfectly reasonable expectation, but it can result in inconsistent responses. For example, some drivers may accelerate towards a green signal, because it is increasingly likely that it will change, whereas others may decelerate. A response to this is to ensure to the extent possible, that there is consistency throughout the road traffic system to encourage predictable and consistent driver behavior.

The combined effect of these expectancies is that:

- drivers tend to anticipate upcoming situations and events that are common to the road they are traveling;
- the more predictable the roadway feature, the less likely will be the chance for errors;
- drivers experience problems when they are surprised;
- in the absence of evidence to the contrary, drivers assume that they will only have to react to standard situations;
- the roadway and its environment upstream create an expectation of downstream conditions; drivers experience problems in transition areas and locations with inconsistent design or operation, and

- expectancies are associated with all levels of driving performance and all aspects of the driving situation and include expectancies relative to speed, path, direction, the roadway, the environment, geometric design, traffic operations and traffic control devices.

Drives Reaction

It takes time to process information. After a person's eyes detect and recognize a given situation, a period of time elapses before muscular reaction occurs. Reaction time is appreciable and differs between persons. It also varies for the same individual, being increased by fatigue, drinking, or other causes. The AASHTO brake reaction time for stopping has been set at 2,5 s to recognize all these factors. This value has been adopted for Uganda case as well.

Often drivers face situations much more complex than those requiring a simple response such as steering adjustments or applying the brakes. Recognition that complex decisions are time-consuming leads to the axiom in highway design that drivers should be confronted with only one decision at a time, with that decision being binary, e.g. "Yes" or "No" rather than complex, e.g. multiple choice. Anything up to 10 seconds of reaction time may be appropriate in complex situations.

Design Response

Designers should strive to satisfy the following criteria:

- Driver's expectations are recognized, and unexpected, unusual or inconsistent design or operational situations avoided or minimized.
- Predictable behavior is encouraged through familiarity and habit (e.g. there should be a limited range of intersection and interchange design formats, each appropriate to a given situation, and similar designs should be used in similar situations).
- Consistency of design and driver behavior is maintained from element to element (e.g. avoid significant changes in design and operating speeds along a roadway).
- The information that is provided should decrease the driver's uncertainty, not increase it (e.g. avoid presenting several alternatives to the driver at the same time).
- Clear sight lines and adequate sight distances are provided to allow time for decision-making and, wherever possible, margins are allowed for error and recovery.

With the major response to divers' requirements being related to consistency of design, it is worthwhile considering what constitutes consistency. Consistency has three elements that are the criteria offered for the evaluation of a road design:

Criterion I — Design consistency-which corresponds to relating the design speed to actual driving behavior which is expressed by the 85th percentile speed of passenger cars under free-flow conditions;

Criterion II — Operating speed consistency which seeks uniformity of 85th percentile speeds through successive elements of the road and; and,

Criterion III — Consistency in driving dynamics – which relates side friction assumed with respect to the design speed to that demanded at the 85th percentile speed.

In the case of criterion 1, if the difference between design speed and 85th percentile speed on an element such as a horizontal curve is less than 10 km/h, the design can be considered good. A difference of between 10 km/h and 20 km/h results in a tolerable design and differences greater than 20 km/h are not acceptable.

In the case of Criterion 2, the focus is on differences in operating speed in moving from one element, e,g, a tangent, to another, e.g. the following curve. A difference in operating speed between them of less than 10 km/h is considered to be good design and a difference of between 10 and 20 km/h is tolerable. Differences greater than 20 km/h result in what is considered to be poor design.

For the third Criterion the side friction assumed for the design should exceed the side friction demanded by 0,01 or more. A difference between -0,04 and 0,01 results in a fair design. A value of less than -0,04 is not acceptable. A negative value for the difference between side friction assumed for design and the side friction demanded means that drivers are demanding more side friction than is assumed to be available –a potentially dangerous situation.

Other Road Users

Pedestrians

The interaction of pedestrians and vehicles should be carefully considered in road design, principally because 50 percent of all road fatalities are pedestrians.

Pedestrian actions are less predictable than those motorists. Pedestrians tend to select paths that are the shortest distance between two points. They also have a basic resistance to changes in gradient or elevation when crossing roadways and tend to avoid using underpasses or overpasses that are not convenient.

Walking speeds vary from a 15th percentile speed of 1,2 m/s to an 85th percentile of 1,8 m/s, with an average of 1,4 m/s. The 15th percentile speed is recommended for design purposes.

Pedestrians' age is an important factor that may explain behaviour that leads to collisions. It is recommended that older pedestrians be accommodated by using simple designs that minimize crossing widths and assume lower walking speeds. Where complex elements such as channelisation and separate turning lanes are featured, the designer should assess alternatives that will assist older pedestrians.

Pedestrian safety is enhanced by the provision of:

- median refuge islands of sufficient width at wide intersections, and
- lighting at locations that demand multiple information gathering and processing.

Cyclists

Bicycle use is increasing and should be considered in the road design process. Improvements such as:

- paved shoulders;
- wider outside traffic lanes in case of separate from carriage way cycle lane is to be provided;
- bicycle-safe drainage grates;
- adjusting manhole covers (if exists) to the grade; and,
- maintaining a smooth and clean riding surface.

can considerably enhance the safety of a street or highway and provide for bicycle traffic.

At certain locations it may be appropriate to supplement the existing road system by providing specially designated cycle paths.

5.4 Traffic Characteristics

The design of road, or any part thereof, should be based upon factual information including factors related to traffic. It should be based on the traffic volumes, which the road will have to accommodate. Traffic directly affects the geometric features of design, such as widths, horizontal and vertical alignments and indicates the need for improvement.

5.4.1 Volume

Traffic data for road design include volumes for days of the year and times of the day as well as the distribution of vehicles by types and by weights. The data also include information on trends from which the designer may estimate the traffic to be expected in the future.

For low volume roads the design control is AADT in the “design year”. For routes with large seasonal variations the design control is ADT during the peak months of the “design year”. The “design year” is usually selected as year 10 after the year of opening to traffic. It is the last year of the design life of the road or any other facility.

5.4.1.1 Design Volume

The volume of traffic estimated or expected to use a certain facility during the design year, which is 10 – 20 years in the future.

5.4.1.2 Average Annual Daily Traffic (AADT)

The total traffic volume for the year divided by 365. For two-lane rural road the total traffic in both direction is taken.

5.4.1.3 Average Daily Traffic (ADT)

The total volume of traffic during the 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. For two-lane rural road the total traffic volume for both directions of flow is taken.

Knowledge of ADT volume for a road is important for many purposes such as determining annual usage as justification for proposed expenditure and for design of structural elements of a road. However, its direct use in the geometric design of road is not appropriate because it does not indicate the variation in the traffic occurring during the various months of the year, days of the week, and hours of the day.

The most appropriate design control for low-volume road is AADT in the design year. On major road carrying heavy traffic volumes throughout the year (current AADT > 1,000), hourly traffic has to be used for determination of the Design Volume.

5.4.1.4 Peak-Hour Traffic and Design Hourly Volume

Peak-Hour Traffic is the Traffic volumes for an interval of time usually one hour. It would be wasteful to base the design on the maximum peak-hour traffic of the year. This is because of the reason that this traffic volume would occur only during one of a very few hours of the year and the traffic in most of the year would rarely be great enough to make full use of the resulting facility. On the other hand the use of the average hourly traffic would result in an inadequate design. The hourly traffic volume used in design should not be exceeded very often or by very much.

Therefore it is necessary to determine best suited traffic volume that would not occur rarely or that is not exceeded very often or by very much. In determining best suited hourly traffic for use in design is a curve showing variation in hourly traffic volume during the year. It shall be prepared by arranging all of the hourly volume of one year, expressed as a percentage of ADT, in a descending order of magnitude. A relationship between the number of hours with hourly volume and hourly traffic as percentage of ADT is plotted. A review of the curve leads to the conclusion that the hourly traffic used in design should be the 30th highest hourly volume of the year, abbreviated as 30 HV. Hence the design hourly volume (DHV) should be the 30 HV of the design (future) year chosen for design. Exceptions may be made on roads with high seasonal fluctuation, where a different volume may need to be used.

DHV is then expressed as DHV = AADT x K or ADT x K where K is estimated from the ratio of the 30th HV to the AADT from a similar site. The 30th HV is the 30th highest hourly volume during the year.

The 30th HV expressed as a fraction of ADT can vary as indicated in the following table.

Table 5-2: Traffic Condition and 30th HV as a fraction of ADT

Traffic Condition	30th HV as a fraction of ADT
Rural Arterial (average value)	0.15
Rural Arterial (maximum value)	0.25
Heavily trafficked road under congested urban conditions	0.08 – 0.12
Normal urban conditions	0.10 – 0.15
Road catering for recreational or Other traffic of seasonal nature	0.20 – 0.30

Higher percentages in the table refer to roads with relatively high concentration of traffic during rush-hours or large seasonal changes.

For rural roads the 30th HV is often used as the Design hourly volume, DHV.

The general approach to operational analysis is to compute the service volumes, SV, for a given road section for each level of service and compare these values with the existing or forecast design hourly volume, DHV.

The general analysis based on average terrain, geometric and traffic conditions and is usually applied to road sections of at least 5km length. Analysis of specific grades on a two-¬lane road is more complex, involving operation of upgrade vehicles, and the procedures outlined in HCM should be referred to.

On rural roads with average fluctuation in traffic volume, 30 HV approximates 15% of the ADT.

5.4.1.5 Composition of Traffic

For design purposes, the composition of traffic meaning the composition of each of the different types of vehicles and also percentage of truck traffic during peak hours should be known. Besides being heavier, trucks generally are slower and occupy more roadway space and consequently impose a greater traffic effect on roads than passenger vehicles do. Vehicle in the truck class are normally those having 4000kg or greater gross vehicle mass (GVM) rating of the manufacturer and vehicles having dual tires on rear axle.

5.4.2 Speed

Speed is a design control and criteria and is one of the most important factors to the traveler in selecting alternate routes or transportation modes. The attractiveness of a public transportation system and a new road are each weighed by the traveler in terms of time, convenience, and money saved and this is directly related to speed.

5.4.2.1 Design Speed

Design speed is a measure of the quality of a road. Geometric design elements such as vertical and horizontal alignment, sight distances and superelevation, which are directly related to design speed. It may be defined as the maximum safe speed that can be maintained over a given section of the road where conditions are so favorable that the design features of the road govern. It must be emphasized that the design speed adopted for a particular stretch of road is intended to provide an appropriate consistency between geometric elements rather than being an indicator of actual vehicle speeds at any particular location on the road system. It depends on topography and should be logical with respect to the adjacent land use, and functional classification of road.

For a balanced road design all permanent features of the road are related to the selected design speed. The cross-sectional elements are not directly related to the design speed, but they affect the vehicle speed, and higher standards should be accorded these features for higher design speed.

5.4.2.2 Operating speed

Operating speed is the highest overall speed at which a driver can travel on a given road under favorable weather conditions and under prevailing traffic conditions without at any time exceeding the safe speed as determined by the design speed on a section-by-section basis.

5.4.2.3 Running speed

Running speed is the speed of a vehicle over a specified section of highway, being the distance traveled divided by running time (the time the vehicle is in motion).

5.5 Capacity

The ability to accommodate vehicular traffic is a primary consideration in the planning, design and operation of highways.

Capacity can be defined as the maximum number of vehicles per unit of time that can be handled by a particular roadway component or section under the prevailing conditions. Road capacity information are useful for (a) transportation planning studies to assess the adequacy or sufficiency of existing road network to service current traffic and to estimate the time in the future when traffic growth may overtake capacity; (b) it is important in design of road dimensions, number of lanes and minimum length of weaving length; (c) in traffic operation analysis in improvement of traffic operation

The traffic flow at capacity level is unstable and minor disturbances in the traffic streams may cause stop-go operations. Consequently a Design Capacity is instigated which is less than the maximum capacity and is related to a “Level of Service”. Level of Service expresses the effectiveness of the road in terms of operating conditions. It is a qualitative measure of the effect of traffic flow factors, such as speed and travel time, interruptions, freedom of maneuver, driver comfort and convenience, and indirectly, safety and operation costs.

The choice of level of service shall generally be based on economic considerations.

Six levels of service are defined. These vary from level A which is the free flow condition, where drivers can maintain their desired speed (low volume and high speed); to level E where the traffic is approaching saturation with drivers traveling at low speed due to high volume of traffic. The traffic volume at level of service E is the capacity of the facility. Level of service F is the forced flow condition where the traffic density is the maximum with drivers subjected to frequent stop-go and queues. Volumes here vary from 0 to capacity, but usually are approaching zero.

The maximum volume that can be carried at any selected level of service is referred to as the service volume for that 1evel. The traffic flow rates that can be served at each level of service are termed as service flow rate. Once a level of service has been identified as applicable for design, the accompanying service flow rate logically become the design service flow rate, implying that if the traffic flow rate using the facility exceeds that value, operating conditions will fall below the level of service for which the facility was designed.

Table 5-3: Level of Service Characteristics by road type

Level of Service	Two lane rural	Multi lane rural without access control
A	Average travel speed of ≥ 93km/hr.Most passing maneuvers can be made with little or no delay.Service flow rate a total of 420 pcu/hr for both direction and about 15% of capacity can be achieved.	Average travel speed ≥ 96 km/h.Under ideal conditions, flow rate is limited to 720 pcu/lane/hr or 33% of capacity.
B	Average travel speed of ≥ 88km/hr.Flow rates may reach 27% of capacity with continuous passing sight distance. Flow rate of 750 pcu/hr total for both direction.	Reasonably free flow. Volume at which actions of preceding vehicle will have some influence on following vehicles. Flow rates will not exceed 55% of capacity or 1,200 pcu/lane/h at 96 km/hr average travel speed under ideal condition.
C	Flow still stable. Average travel speed of ≥ 84km/hr.Flow rates under ideal condition equal to 43% of capacity with continuous passing sight distance or 1,200 pcu/hr total for both direction.	Stable flow to a flow rate not exceeding 75% of capacity or 1,650 pcu/lane/h, under ideal conditions maintaining at least a 95 km/hr average travel speed.
D	Approaching unstable flow. Average travel speed of ≥ 80km/hr.Flow rates, two directions, at 64% of capacity with continuous passing opportunity, or a total of 1,800 pcu/hr for both direction.	Approaching unstable flow at flow rates up to 89% of capacity or 1,940 pcu/lane/h at an average travel speed of about 92 km/h under ideal condition.
E	Average travel speeds in neighborhood of 72 km/hr. Flow rates under ideal conditions, total two way, equal to 2,800 pcu/hr. Level E may never be attained. Operation may go directly from Level D directly to Level F.	Flow at 100% of capacity or 2,200 pcu/lane/hr under ideal conditions. Average travel speeds about 88 km/h.
F	Forced congested flow with unpredictable characteristics. Operating speeds less than 72 km/hr.	Forced flow congested condition with widely varying volume characteristics. Average travel speed of less than 50 km/h.

The capacity values in the above table are expressed in passenger car units in order to take into account the influence of capacity of different vehicle mixes on different gradients.

The following guide values are given for the conversion factors applicable to different vehicle types in different terrain:

Table 5-4: Conversion factor of vehicle into equivalent passenger car

Vehicle Type	Terrain		
	Level	Rolling	Mountainous
	p.c.u.		
Passenger cars	1.0	1.0	1.5
Light goods vehicle	1.0	1.5	3.0
Medium goods vehicle*	2.5	5.0	10.0
Heavy goods vehicle	3.5	8.0	20.0
Buses	2.0	4.0	6.0
Motor cycles, Scooters	1.0	1.0	1.5
Pedal cycles	0.5	0.5	NA

* also representative for combined group of medium and heavy goods vehicles and buses.

The following definitions apply to the different vehicle types mentioned in the above table.

Passenger cars:	Passenger vehicles, with less than nine seats.
Light goods vehicle:	Land rovers, minibuses and good vehicles of less than 1,500kg unladen weight with payload capacities less than 760 kg.
Medium goods vehicle:	Maximum gross vehicle weight 8,500 kg.
Heavy goods vehicle:	Gross vehicle weight greater than 8,500 kg.
Buses:	All passenger vehicles larger than minibus.

Example: An hourly traffic volume of 5 passenger cars, 10 light goods vehicle, 10 medium goods vehicle, 40 heavy goods vehicles, and 20 buses totaling 85 vehicles in rolling terrain represents [(5 x 1.0) + (10 x 1.5) + (10 x 5.0) +(40 x 8.0)+(20 x 4.0)] = 470 passenger car units per hour.

The traffic flow rates that can be served at each level of service are termed as service flow rates. Once a level of service has been identified as applicable for design, the accompanying service flow rate logically becomes the design service flow rate, implying that if the traffic flow rate using the facility exceeds that value, operating conditions will fall below the level of service for which that facility was designed. Once a level of service has been selected, it is desirable that all elements of the roadway are consistently designed to this level. Design service flow rate is the maximum hourly flow rate of traffic that a certain road of designed dimensions would be able to serve without the degree of congestion falling below a prescribed level of service.

Design service flow rate is the maximum hourly flow rate of traffic that a projected road of designed dimensions would be able to serve without the degree of congestion falling below a pre-selected level of service.

The objective in road design is to create a facility with dimensional values and alignment characteristics such that the resulting design service flow rate (design capacity) is at least as great as to the traffic flow rate during the peak 15-minute period of the design hour, but not enough greater as to represent extravagance or waste. Where this objective is accomplished, a well-balanced, economical road system will result.

Because flow is not uniform throughout an hour, there are certain periods within an hour during which congestion is worse than at other times. The HCM considers operating conditions prevailing during the most congested 15-minute period of the hour to establish the service level for the hour as a whole. Accordingly, the total hourly volume that can be served without exceeding a specified degree of congestion is equal to or less than 4 times the maximum 15-minute count. The factor used to convert the rate of flow during the highest 15-minute period to the total hourly volume is the peak hour factor (PHF). The PHF may be described as the ratio of the total hourly volume to the number of vehicles during the highest 15-minute period multiplied by 4. It is never greater than 1.00 and is normally within the range of 0.75 to 0.95. For example, if the maximum flow of rate (Design Capacity) that can be served by a certain two lane road without excessive congestion is 1,500 vehicles per hour during the peak 15-minute period, and further if PHF is 0.85, the total hourly volume that can be accommodated at that level of service is 1,500 x 0.85 = 1,275 vehicles or 85 percent of the traffic flow rate (capacity) during the worst congested 15-minute period.

Table 5-5: Level of Service Criteria for two-lane rural roads

Level of Service	Percent Time Delay	Percent no Passing Zones (1)	Volume Level Terrain	Capacity Ratio Rolling Terrain	(V/C) Mountainous Terrain (2)
A	< 30	0	0.15	0.15	0.14
		20	0.12	0.12	0.09
		40	0.09	0.07	0.07
B	< 45	0	0.27	0.26	0.25
		20	0.24	0.23	0.20
		40	0.21	0.19	0.16
		60	0.19	0.17	0.13
C	<60	0	0.43	0.42	0.39
		20	0.39	0.39	0.33
		40	0.36	0.35	0.28
		60	0.34	0.32	0.23
		80	0.33	0.30	0.20
D	<75	0	0.64	0.62	0.58
		20	0.62	0.57	0.50
		40	0.60	0.52	0.45
		60	0.59	0.48	0.40
		80	0.58	0.46	0.37

Note 1. The percentage of road length along which sight distance is less than 450m may be used as percent no passing zones.

Note 2. Ratio of hourly volume to an ideal capacity of 2,800 passenger cars in both directions.

A volume-capacity ratio higher than 0.65 should normally not be applied to road design.

Guidelines for selection of design levels of service for the different classes of rural roads are given in the following table.

Table 5-6: Selection of Level of Service for Rural Roads

Road Functional Class		Level of Service			
		Level terrain	Rolling terrain	Mountainous terrain	Road Design Class
A	International Trunk Roads	B	B	C	Ia Paved Ib, II and III Paved
B	National Trunk Roads	B	C	D	Ib, II and III Paved, A Gravel
C	Primary Roads	C	D	D	II and III Paved, A Gravel
D	Secondary Roads	D	D	E	A and B Gravel
E	Minor Roads	E	E	E	B and C Gravel

5.6 Determination of Service Volume (SV)

The traffic data required for determining the service volume include the two-way hourly volume, the directional distribution of traffic flow and the proportion of trucks and buses in the traffic stream.

SV, for paved two-lane rural road, expressed as vehicles per hour, can be determined from the general relationship:

$$SV_i = 2800 \times (V/C)_i \times fW \times fT \times fD$$

Where:

SV_i = Total service volume for level of service i in vehicles per hour.

$(V/C)_i$ = Volume-capacity ratio for level of service i

fW = Lane width factor.

fT = Truck factor.

fD = Directional factor

a. <u>Lane Width Factor</u>

For road sections with lane width less than 3.5m, the reduction factor, fW, applicable to the road types selected for the present geometric standard, is given in the following table.

Table 5-7: Lane Width Factor (fW)

Lane Width	Factor
m	fW
3.5	1.00
3.35	0.89
3.00	0.73
2.75	0.61

b. Truck Factor

The truck factor, fT is dependent on the level of service, the type of vehicle and the type of terrain.

The truck factor is determined from the relationship:

$$fT = \frac{1}{1 + PT\,(ET - 1) + PB\,(EB - 1)}$$

Where:

PT = proportion of trucks in the traffic stream, expressed as a decimal

PB = proportion of buses in the traffic stream, expressed as a decimal

ET = passenger car equivalent for trucks.

EB = passenger car equivalent for buses.

The usual traffic stream is composed of a mixture of vehicles: Passenger cars, buses, trucks and cycles. Factors for converting other types of vehicles to equivalent passenger car traffic in different terrain type are given in the following table:

Table 5-8: Average Passenger Car Equivalents For Trucks and Buses on Two-lane Road

Vehicle Type	Level of Service	Type of terrain		
		Level	Rolling	Mountainous
Truck, ET	A	2.0	4.0	7.0
	B and C	2.2	5.0	10.0
	D and E	2.0	5.0	12.0
Buses, EB	A	1.8	3.0	5.7
	B and C	2.0	3.4	6.0
	D and E	1.6	2.9	6.4

c. Directional Factor

All of the V/C value given in the table 5-4 for selection of Level of Service for Rural Roads are for 50/50, directional distribution of traffic on a two-lane road. For other directional distributions, the factors shown in the table for directional factors shall be applied.

Table 5-9: Directional Factor

Directional distribution	80/20	70/30	60/40	50/50
Directional factor, f_D	0.83	0.89	0.94	1.00

5.7 Factors Other Than Traffic Volume That Affect Operating Conditions

Among the other factors other than traffic volume that affect operating condition are weaving sections, Ramp Terminals, and Traffic compositions.

Weaving Sections are road segments where the pattern of traffic entering and leaving at contiguous points of access results in vehicle paths crossing each other.

Ramps and ramp terminals are features that can adversely influence operating conditions on roads if their use is excessive or if their design is deficient.

5.8 Planning

The planning procedure shall enable the transport authorities to perform a general planning of a two-lane road system.

The traffic demand will be expressed in terms of AADT of some forecast year and the geometric and terrain data will only be generally classified. For planning procedures the estimated maximum AADT's for two-lane rural roads presented in the table for Maximum AADT may be used as a guide-line. The table gives the maximum AADT's applicable to different levels of service and different types of terrain.

The planning criteria used in the table for maximum AADT assume the following:

- Traffic mix 25% of trucks;
- Directional split 60/40;
- No passing zone
    - i. Level terrain 20%
    - ii. Rolling terrain 40%
    - iii. Mountainous terrain 60%
    - iv. Ratio of 30th HV to AADT 0.15

Table 5-10: Maximum AADT's for Two Lane Rural Roads

Level of service	Maximum AADT		
	Level terrain	Rolling terrain	Mountainous terrain
A	1600	700	300
B	3200	1650	700
C	5200	3000	1250
D	8700	4500	1900

5.9 Application

For design volumes close to the maximum capacity of the road, operational analysis should be performed for alternative designs to document the impact on traffic operations from horizontal and vertical alignment.

Where computations indicate that a two-lane road is not adequate for existing or projected demands, various multi-lane options may be considered and analyzed, using the methodology described in "Highway Capacity Manual" (HCM). It should, however, be born in mind that the procedures in HCM reflect North American operating experience, and should always be evaluated with respect to its applicability to the local prevailing conditions.

5.10 Safety Considerations

5.10.1 Background

The view on traffic safety and traffic safety work has changed with the development of traffic and the role of road traffic in the society. In the early years of motorization, cars were looked upon as horse drawn carriages. Safety measures were mainly focused on vehicle requirements. The development of technology, especially increased power and speed of motor vehicles, made the comparison with horse-drawn carriages out-of-date. Safety measures were focused on adapting people to this new traffic situation.

Today, the whole transportation system, of which the road traffic system is one part, is contemplated. Safety measures are focused on reducing the exposure of risks, eliminating risk factors and reducing the consequences of accidents. Typical measures are speed limits and separation of motorized traffic from other types of traffic. With this approach, the purpose of traffic safety work in road design is mainly to eliminate the risk factors and mitigate the consequences of accidents. The long-term objective is that no one should be seriously injured or killed when using the road traffic system.

5.10.2 Injury Risks

The risk of being injured or killed in an accident increases considerably with the increase in speed. Many studies have shown that:

- The number of injury accidents increases with the square of the average vehicle speed; and,
- The number of fatal accidents increases with the fourth power of the average vehicle speed.

Figure 5-6 shows how the risk of being killed in a crash varies with collision speed. The graph for pedestrians is well supported by research results, while the graphs for vehicle collisions are partly based on expert assessments.

The graph shows that the risk of being killed increases rather slowly from 0 – 10% of risk of being killed. Beyond 10% the risk increases rapidly. From this it can be concluded that a road transport system should be designed to avoid conflicts at speeds where the risk to be killed is higher than around 10 percent. This means that speeds should not exceed:

- 30 km/h in a pedestrian/vehicle collision;
- 50 km/h in a side-on vehicle/vehicle or vehicle/object collision; and,
- 70 km/h in a head-on vehicle/vehicle or vehicle/object collision.

From this, some basic planning and design rules can be derived, for example:

- Vulnerable road users should be separated from motor vehicle traffic;
- At points of conflict between vulnerable road users and motor vehicles, speeds should be low (preferably 30 km/h or lower);
- Intersections should be designed to reduce collision speeds, especially for side-on collisions (preferably to 50 km/h or lower); and,
- The risk for head on collisions and collisions with rigid objects must be reduced to the greatest possible extent.

However, measures to lower the speed, for example in intersections, are not sufficient. Measures to reduce the risk of conflicts and the consequences of collisions must also be taken. Examples of such measures are: the use of standard type intersections and reducing the number of potential conflict points and the sizes of conflict areas.

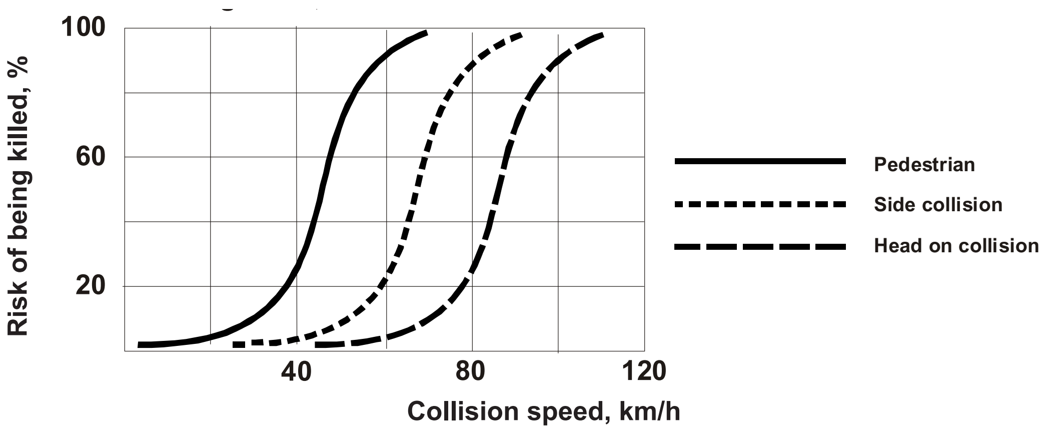


Figure 5-6: The risk of being killed in traffic accidents depending on collision speed

5.10.3 Safety Responsibility

Research has shown that the human being is an unreliable operator in the road traffic system. The most typical mistakes made by drivers and other road users are common to almost all drivers and not limited to only a few. Consequently, all road users can be expected to make mistakes which can lead to accidents. A road traffic system in which such common human mistakes leads to fatal and serious injuries cannot be accepted. Common human mistakes should not lead to catastrophes.

The responsibility for road safety must be shared between the road users and the road transport system providers (mainly road authorities and vehicle manufacturers as well as legislative, surveillance and enforcement bodies). The road users' responsibility is to follow the system requirements – i.e., to obey the traffic laws and regulations, use available protection equipment and behave with good judgment and responsibility. The responsibility of the system provider is to provide a road system designed to minimize the risk of accidents and to only allow accidents imposing forces to the human body that can be resisted without serious injuries. This responsibility lies to a great extent with the road designer.

5.10.4 Safety Considerations in Design

Designing safety into roads is one of the main objectives of geometric design. It is important that safety features are built into the road from the very start of the design. Safety considerations in roads have the two objectives to provide design features to:

- Prevent accidents, and
- Reduce the seriousness of the accidents that occur.

5.10.5 Accident Prevention

For the prevention of accidents the following points are especially important:

- Provision of physical separation between motor vehicles in opposing directions and also with other road users (especially pedestrians and cyclists);
- Provision of a balanced design, i.e. compatibility between the various design elements;
- Avoidance of surprise elements for the drivers, for example abrupt changes in standard, insufficient visibility or poor phasing of horizontal and vertical alignment;
- Avoidance of situations where drivers must make more than one decision at the time;
- Provision of design features that reduce speed differentials between vehicles, for

example flat grades and speed change lanes;

- Proper location and design of intersections;
- Proper design, application and location of traffic signs, road markings and other traffic control devices;
- Provision of design elements compatible with traffic volumes and type of traffic; and,
- Provision of proper drainage of the road surface.

5.10.6 Reducing the severity of accidents

A lot can be done to reduce the severity of accidents that we fail to prevent. The basic principles are:

- There should be a clear zone (safety zone) along each side of the road that is clear of hazards such as lighting columns, other utility poles, rocks, drainage structures, etc.;
- Roadside slopes should be as flat as feasible (1:4 or flatter);
- Large diameter sign posts and other supports which must be located within the clear zone should be of a breakaway type; and,
- Safety barriers should be provided to protect vehicles from hitting dangerous obstacles that cannot be removed or made breakaway and also to protect vehicles from falling of the road down embankments.

5.10.7 Speed and traffic safety

Actual vehicle speeds on a road section is of utmost importance for level-of-service as well as for traffic safety. The speed at which a driver will choose to travel a section of road is generally determined by what he considers safe based on the visual concept of the cross-section and road alignment with terrain adaptation, surface quality and other conditions. Individual driver decisions will vary significantly due to personality, experience and vehicle capacity. The standard deviation of speeds is normally some 10 % in European countries.

Road and traffic history reveals that drivers tend to overestimate safe speeds. Speed limits should be imposed where speeding is a problem, but compliance with signs is low, and enforcement is difficult. Consequently it is crucially important to reinforce the speed limits with physical speed control measures, especially on through roads in trading centres and towns.

Modern traffic safety research indicates that the number of fatal accidents is proportional to the average speed in power four, fatal and serious injury accidents in power three and fatal and all injury accidents in power two as illustrated in Figure 5-7.

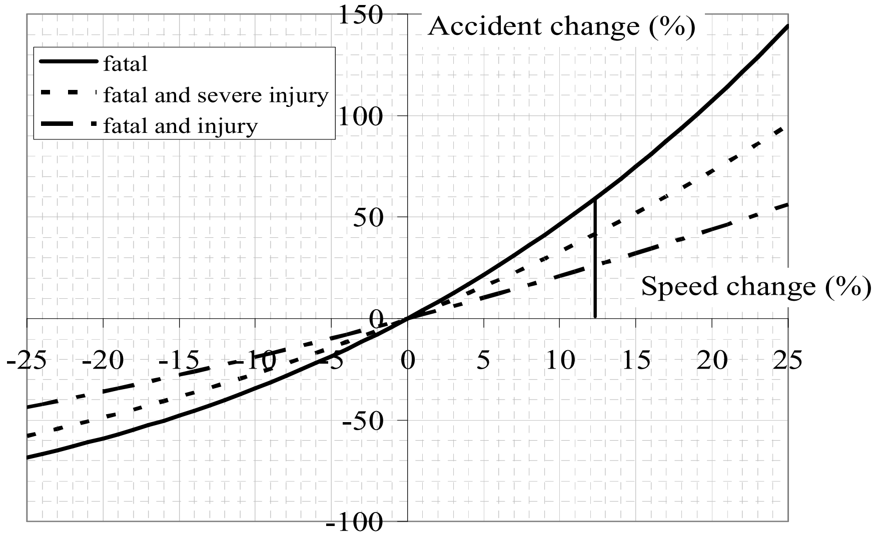


Figure 5-7: Speed power law

This speed-power law gives as an example for an increase of average speed from 80 km/h to 90 km/h, some 12 %, i.e. without compensation of improved drivers, vehicles or road design an increase of:

- the number of fatal accidents with some 60 %;
- the number of fatal and serious accidents with some 40 %; and,
- the total number of fatal and injury accidents with some 25 %.

Separating vulnerable road users from motorised traffic has great safety benefits. Whether and how to do this requires careful assessment. The main ways of achieving separation in rural areas are:

- unpaved shoulder;
- paved shoulder separated from the traffic lane with an edge line marking; and,
- separate footway – in some circumstances it may also be used by cyclists.

For roads through trading centres and towns the options are:

- paved shoulders;
- a footway (or combined footway/cycleway) physically separated from the traffic lane by a barrier kerb or similar;
- a raised, kerbed footway; and,
- a raised, kerbed footway with a service road beyond it.

The general recommendations on separation are as follows: (see also section 7)

- A separate footway must be provided on Class A and B rural roads if they are to have a speed limit of 100 km/h and over (Road Design Classes Ia and Ib Paved);
- Rural roads with a speed limit of 80 km/h shall be provided with paved shoulders for use by pedestrians and cyclists – on sections where the volume of pedestrians is moderate or high a separate footway shall be provided; and,
- A separate footway shall normally be provided alongside through roads in built-up areas.

5.11 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 traffic from environmental vulnerable areas. The negative effects are related to both the road as a physical feature and to the traffic on the road.

5.11.1 Effects Related to the Road as a Physical Feature

Effects related to the road as a physical feature are generally difficult to quantify. In many cases it is necessary to seek the advice and services from other professions to reach a proper evaluation of the problems and to find adequate mitigation measures. The following factors should be considered in the location and design of roads:

- The preservation of the natural beauty and scale of the countryside and the adaptation to the conditions and architecture of the city;
- 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
- 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; and,
- The avoidance or reduction of visual intrusion.

5.11.2 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:

- Noise pollution;
- Air pollution;
- Ground water pollution;
- Vibrations; and,
- Severance of areas (barrier effect).

Among the solution to avoid the problems is to locate the road outside trading centers 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 respective professionals to properly evaluate the impacts and establish proper and adequate mitigation measures.

5.12 Economic Considerations

5.12.1 Introduction

The total costs for the society for roads and road traffic are to a large extent depending on the design of each individual road project. 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, travel time costs and accident costs etc may offset the extra construction costs for a road with higher design standard. The economic outcome of the design will be decided both in the route selection and in the geometric design of the chosen route.

5.12.2 Route Selection

The steps to be followed in route corridor selection is discussed in Section 2. To find the optimum alternative for each project several different routes should be tried. Some of these should be chosen and compared in a comprehensive study including the following types of effects:

- Land acquisition and intrusion;
- Construction and operation costs;
- Road user costs including travel time, accidents, vehicle operating costs; and,
- Environmental effects.

Some effects, for example, road construction and maintenance costs and also travel time and accidents, can be described in monetary values. Other effects, like traffic noise and air pollution, can be quantified but are difficult to value in money (though valued in most European countries), while other effects, for example impacts on wild life or other valuable resources, generally only can be described verbally.

A cost-benefit and objective analysis including as many effects as possible should be made for all studied alternatives.

5.12.3 Detailed Design

The geometric standards of individual elements of the road will usually vary with the terrain. Elements for which the chosen geometric standards are difficult to obtain should be identified. The economic consequences of different standards for these elements should be considered and a cost-benefit analysis including construction and road user costs should be made. In general, the higher the class of road, and hence volume of traffic, the more likely will benefits from road user savings lead to the justification of a higher road standard.

The standards described in this manual are intended to provide guidance for designers rather than to be considered as rigid minima. For some projects or elements relaxations of standards may be essential in order to achieve an acceptable rate of return of investment.

SECTION 6: ELEMENTS OF DESIGN

6.1 Alignment

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.

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.

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 mishap. This requires all the alignment 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 alignment and sags.

These basic assumptions give minimum parameters for vertical and horizontal alignment elements. They also give a number of recommendations 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 maximise 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.

6.1.1 Alignment Choice and Terrain Adaptation

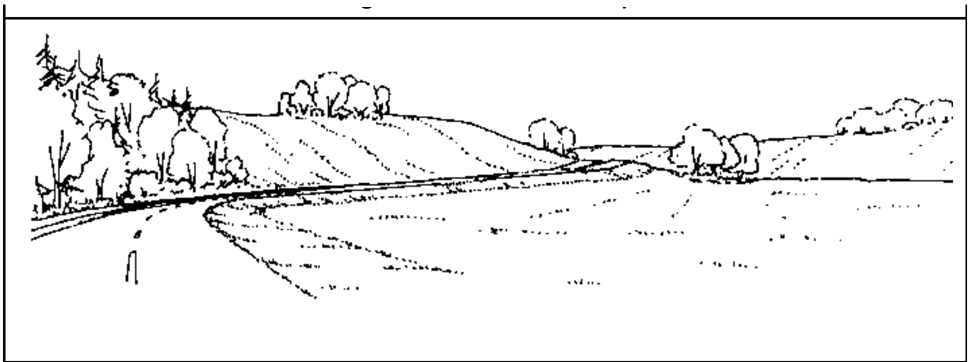
To locate a road in a landscape is a challenge with constraints and possibilities. It is a technical and also an architectural process.

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 not to adorn or to emphasize. Three basic concepts unite and constitute the technical and the architectural process:

- Scale;
- Space; and,
- Rhythm.

The scale is the size of the landscape to locate the road in. Some typical landscape sizes are illustrated in Figure 6.1.

Large-scaled landscape



Small-scaled landscape

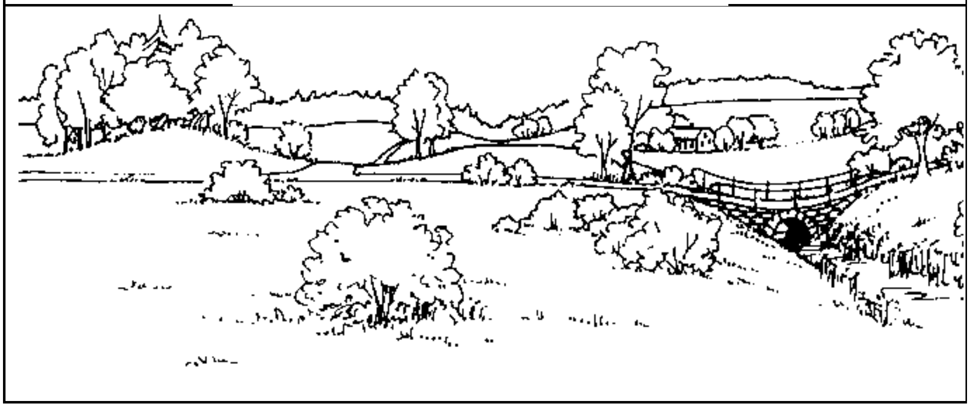
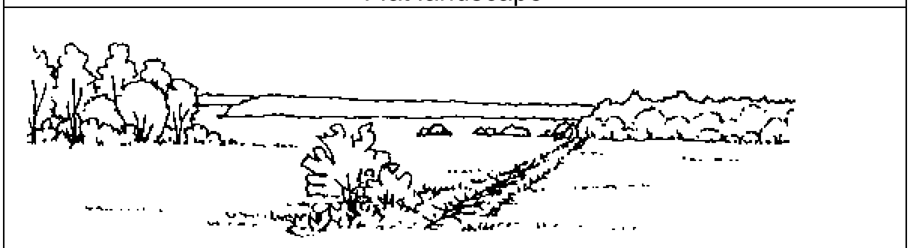


Figure 6-1: Description of the scale concept

Landscapes can be differentiated in types such as:

Flat landscape



Slightly hilly landscape

Hilly landscape

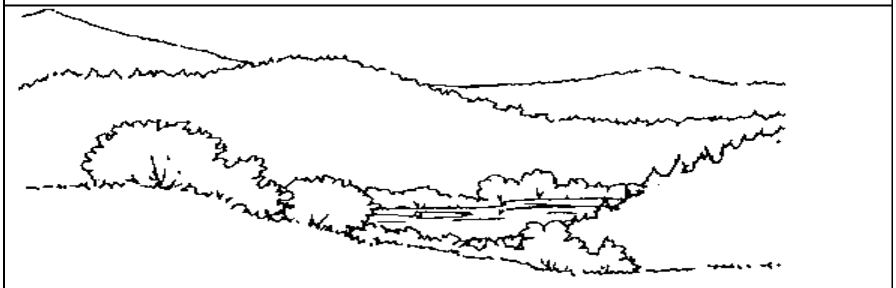
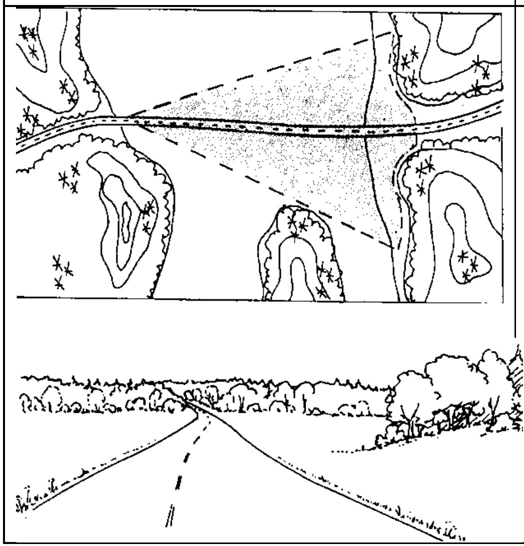


Figure 6-2: Examples of landscape types

Space limited by ridges and trees	The eye view angle decreases with speed



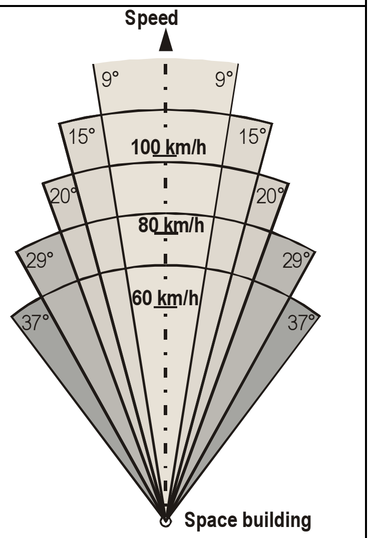


Figure 6-3: Driver's space or room concept

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:

- Terrain (mainly topography), vegetation, buildings;
- Road design, i.e. cross-section, horizontal and vertical alignment; and,
- Crossing bridges and road embankments.

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 is to design and have constructed a road

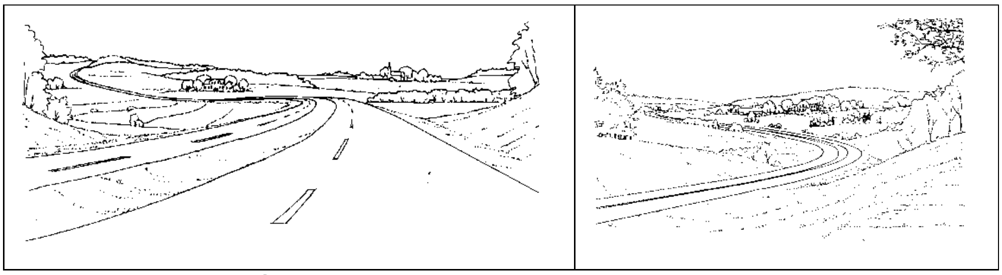


Figure 6-4: Some examples on rhythmical landscape adaptation

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.

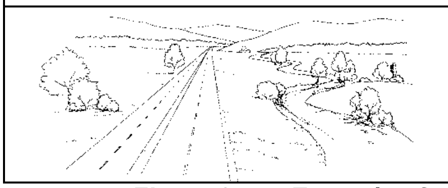
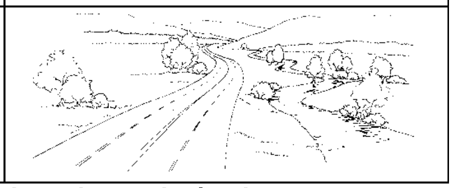
No harmony	Harmony
	

Figure 6-5: Example of adaptation to the landscape

The approach and methodology in respect of the practical work with alignment choice and terrain adaptation can be divided into three steps:

- Inventory of constraints and opportunities;
- Route planning; and,
- Detailed design of the alignment.

<u>Step 1: Inventory</u>

The workshop for the road and the road designer is the terrain area. It is important to analyse the terrain to understand its constraints and possibilities.

Constraints could be areas not allowed to be used or only to be used as constraints, e.g. existing or planned buildings, rivers, roads, geotechnically difficult areas etc. Possibilities to create the desired right scale and rhythm are the talent of the designer to combine the technical requirements with the freedom given by the terrain area. It is important in the initial phase to get good knowledge and a visual concept about the terrain. The terrain should be "walked". Good maps, terrestrial and aerial photos are essential but it should be stressed again that the solution of the road design is found in the terrain.

Step 2: Route planning

There are two alternative methods for identifying the route: the tangent method and the arcs method.

Tangent method (see Figure 6.6)

The straight strategy gives the road line iteratively by defining straights and then to combine these by arcs. An advantage could be effective use of sight distances.

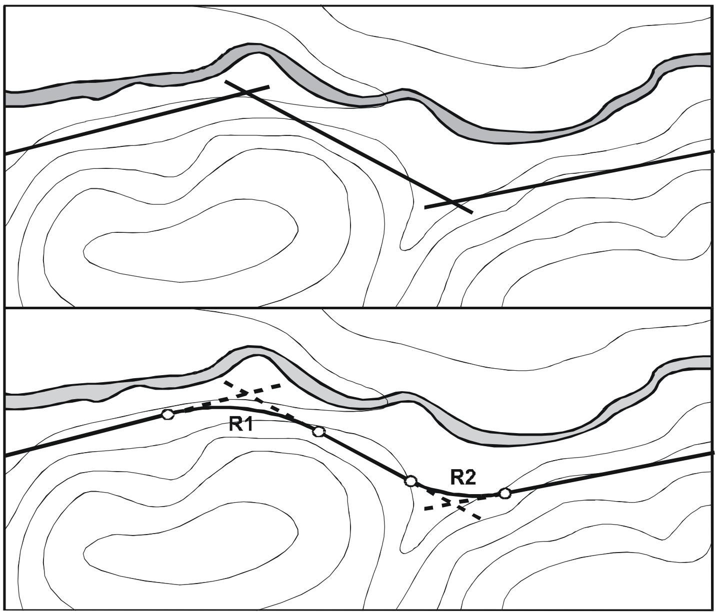


Figure 6-6: Route planning by tangent method

Arc method (see Figure 6.7)

The other method starts by selecting arcs with radii that fit in well with the scale and form of the landscape. These arcs are then linked together with transition curves, i.e. clothoids of larger arcs. It is important in this strategy to avoid dilemma sight distances and to create sufficient overtaking sight distances. This method is more likely to result in a route which is well adapted to the terrain.

The road location and alignment procedure should start on the map by sketching suitable alignment alternatives. Impressions and notations from the landscape and its characteristic forms and properties should be used to create alternatives anchored in the landscape. The straight line is normally not in harmony with the landscape.

The terrain adaptation must, as already stressed, be combined with the partly contradictive requirement to create sufficient overtaking sight distances and to avoid dilemma sight distances. Left-hand bends, even with large radii, will rarely have sufficient sight distance for safe overtaking.

The designer should mentally visualize the three dimensional form. This could be supported by using simple profile sketches to analyse the phasing between horizontal and vertical alignment. Intersections, interchanges etc should be considered already in this stage.

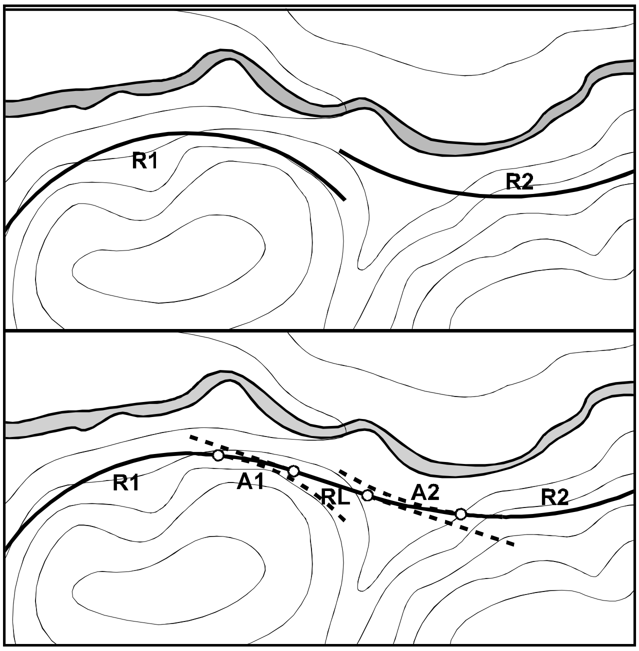


Figure 6-7: Route planning by arc method

<u>Step 3: Detailed design</u>

Having found a route in harmony with the terrain using the sketch technique above horizontal geometric elements can be calculated and a first profile produced. Technical requirements should also be checked such as:
- minimum horizontal and vertical elements;
- combined elements for visual guidance;
- stopping, dilemma and overtaking sight distances; and,
- speed profiles.

Always check what percentage of the road will have overtaking sight distance. This will have a major impact on safety and level-of-service at medium high traffic flows.

The coordination between vertical and horizontal alignment should be checked using perspective images. It is important to learn how to select points for perspectives and how to interpret perspective images. The perspective images could be used to judge if minor or major adjustments are needed.

6.2 Horizontal Alignment

6.2.1 General

The design elements of a horizontal alignment are the tangent (straight section), the circular curve, the transition curve (spiral curve) and the superelevation sections.

6.2.2 Tangent Section

From an aesthetic point of view, tangent sections may often be beneficial in flat country but are less in rolling or mountainous terrain. From a safety standpoint, they provide better visibility and more passing opportunities. However, long tangent sections increase the danger from headlight glare and usually lead to excessive speeding. In hot climate areas, long tangents have been shown to increase driver fatigue and hence cause accidents. This issue needs to be addressed in the course of the horizontal alignment design. The maximum length of a tangent section therefore should not exceed 4.0 kilometers.

6.2.3 The Circular Curve

When a vehicle moves in a circular path, it is forced radially outward by centrifugal force. The centrifugal force is counterbalanced by superelevation of the roadway and/or the side friction developed between the tires and the road surface. It is recommended that curves are designed such that it is necessary for vehicles traveling at the design speed to steer into a bend. For any given curve and speed, superelevation may be introduced to enable a component of the vehicle's weight to reduce the frictional need.

For calculation of the minimum horizontal radius, R_{min}, for a particular design speed, the following equation shall be used:

$$R_{min} = \frac{V_D^{\,2}}{127(e+f)}$$

Where

VD = Design Speed (km/h)

e = Cross fall of road or the maximum super-elevation (%/100) the value of e may represent the simple removal of adverse cross fall or include super-elevation (e = +ve for cross slopes sloping towards the inside of the curve and otherwise -ve).

f = Coefficient of side friction force developed between the vehicle's tires and road pavement

The side friction factor may be considered to be the lateral force developed by the driver on a level road. The technical evidence indicates that lateral accelerations, and hence side friction factors, increase with reduced radii of curvature and increased speed. Side friction coefficients are dependent on vehicle speed, condition of texture or road way surface, weather conditions, and type and condition of tires. The range is considerable and values of "f" found from road measurements have varied from just over 0.1 for high speed roads to over 0.5 on lower speed roads. The results of empirical studies have indicated 0.22 as a value of "f" above which passengers experience some discomfort.

Example of calculating minimum radius:

V_D = 80 km/hr
e_{max} = 7%
f = 0.14
find minimum radius.

Table 6-1 shows minimum radii for horizontal curves on non-urban roads based on various limiting values of e (4, 6, 7, & 8%) and f.

Design Speed [km/hr]	Maximum e [%]	Limiting value of f	(e/100 + f)	Calculated Radius [m]	Rounded Radius [m]
30	4.00	0.17	0.21	33.7	35
40	4.00	0.17	0.21	60.0	60
50	4.00	0.16	0.20	98.4	100
60	4.00	0.15	0.19	149.2	150
70	4.00	0.14	0.18	214.3	215
80	4.00	0.14	0.18	280.0	280
90	4.00	0.13	0.17	375.2	375
100	4.00	0.12	0.16	492.1	490
110	4.00	0.11	0.15	635.2	635
120	4.00	0.09	0.13	872.2	870
30	6.00	0.17	0.23	30.8	30
40	6.00	0.17	0.23	54.8	55
50	6.00	0.16	0.22	89.5	90
60	6.00	0.15	0.21	135.0	135
70	6.00	0.14	0.20	192.9	195
80	6.00	0.14	0.20	252.0	250
90	6.00	0.13	0.19	335.7	335
100	6.00	0.12	0.18	437.4	440
110	6.00	0.11	0.17	560.4	560
120	6.00	0.09	0.15	755.9	755
30	7.00	0.17	0.24	29.5	30
40	7.00	0.17	0.24	52.5	55
50	7.00	0.16	0.23	85.6	85
60	7.00	0.15	0.22	128.8	130
70	7.00	0.14	0.21	183.7	185
80	7.00	0.14	0.21	240.0	240
90	7.00	0.13	0.20	318.9	320
100	7.00	0.12	0.19	414.4	415
110	7.00	0.11	0.18	529.3	530
120	7.00	0.09	0.16	708.7	710
30	8.00	0.17	0.25	28.3	30
40	8.00	0.17	0.25	50.4	50
50	8.00	0.16	0.24	82.0	80
60	8.00	0.15	0.23	123.2	125
70	8.00	0.14	0.22	175.4	175
80	8.00	0.14	0.22	229.1	230
90	8.00	0.13	0.21	303.7	305
100	8.00	0.12	0.20	393.7	400
110	8.00	0.11	0.19	501.5	500
120	8.00	0.09	0.17	667.0	670

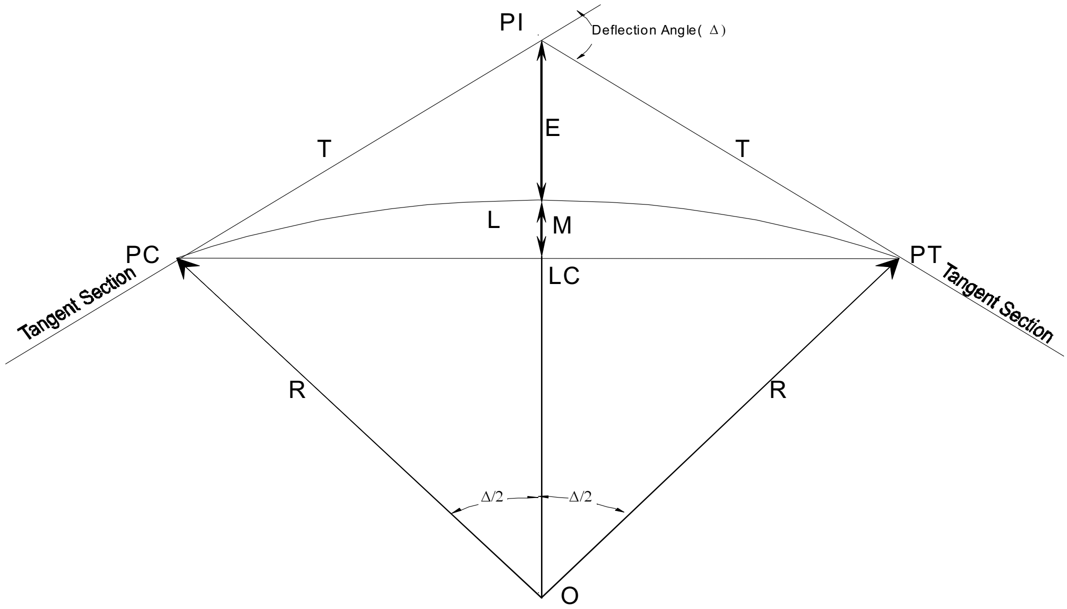


Figure 6-8: Elements of circular curve

1. PI is the point of the intersection of the two tangents.
2. T is the straight line in a road.
3. Δ is the deflection angel formed by the intersection of the two tangents at the PI.
4. L is the length of the arc (curve) between the PC and PT.
5. LC is the long chord between the Pc and the PT.
6. E is the external distance from the PI to the center of the arc.
7. M is the Middle ordinate, the distance from the middle of the arc to the mid point of the Long Chord.
8. PC (point of curvature) is the point where the tangent ends and the curve begins.
9. PT (point of tangency) is the point where the curve ends, and the tangent starts.
10. R is the radius. It is the distance from the center of the circle (O) to any point on the circumference.
11. Degree of curve (D)

By Arch Definition

D is the central angel subtended by a 20meters arc.

According to the above definition:

$$\frac{20}{2\pi R} = \frac{D}{360}$$

$$D = \frac{20x360}{2\prod R}$$

$$D = \frac{1145.916}{R}$$

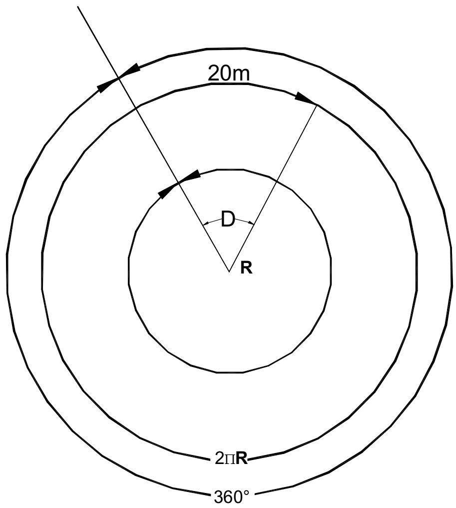


Figure 6-9: Degree of curve and radius of curve by arc definition

12. R (Radius of curve) by Arc definition

$$R = \frac{1145.916}{D}$$

13. Tangent (Tangent Distance)

T = R tan (∆/2)

14. External Distance (E)

E = R [1/Cos(∆/2) – 1]

OR

E = R [Sec (∆/2) – 1]

15. Curve Length (L)

$$L = \frac{20\Delta}{D}$$

16. Middle Ordinate (M)

    M = R [1 – Cos (∆/2)]

17. Long Chord (LC)

    LC = 2R sin (∆/2)

18. Point of Curvature (C)

    PC Station = PI station – T (tangent)

19. Point of Tangency (PT)

    PT Station = PC station + L (Length of Arc)

EXAMPLE 1: -

A curve has a deflection angle of $\Delta = 20^0\ 18'\ 02''$, and a Degree of Curve of $4^0\ 00'$. The point of intersection (PI) is 5+053.87. Calculate radius (R), tangent (T), external distance (E), curve length (L), point of curvature (PC), and point of tangent (PT).

$\Delta = 20^0\ 18'\ 02$
$D = 4^0$

$$R = \frac{1145.916}{4} = 286.479$$

$$T = R \tan\left(\frac{\Delta}{2}\right)$$

$$= 286.479 \tan\left(\frac{23^0 18' 02''}{2}\right)$$

= **59.07**

$$E = R[\mathrm{Sec}\left(\frac{\Delta}{2}\right) - 1]$$

= 286.479 [(1/cos 10^0 09' 01")) – 1]
= **4.55**

L = (20 x ∆)/D = (20 x 23^0 18' 02")/4
= (20 x 23.30)/4 = **116.50**

$PI_{sta.}$ = 5 + 053.87

$PC_{sta} = PI_{sta} - T$ = 5 + 053.87 – 59.07 = 4 + 994.8
$PT_{sta} = PC_{sta} + L$ = 4 + 994.8 + 116.50 = 5 + 111.3

The horizontal curvature over a particular road section should be as consistent as possible. Isolated sharp curves on an otherwise straight alignment are dangerous (see Figure 6-4). Particular care should be taken to avoid sharp curves at the ends of long straight sections.

i) Reverse Curves, Broken-Back Curves and Compound Curves

Curves are more frequent in rugged terrain. Tangent sections are shortened, and a stage may be reached where successive curves are dealt in isolation. There are three cases of successive curves: (see Figure 6.3):

a) Reverse Curve: a curve followed by another curve in the opposite direction
b) Broken-Back Curve: a curve followed by another curve in the same direction
c) Compound curve: curves in the same direction, with different radius size, but without any intervening tangent section between them. Desirable maximum ratio of radii 1.5:1; maximum 1.75:1;

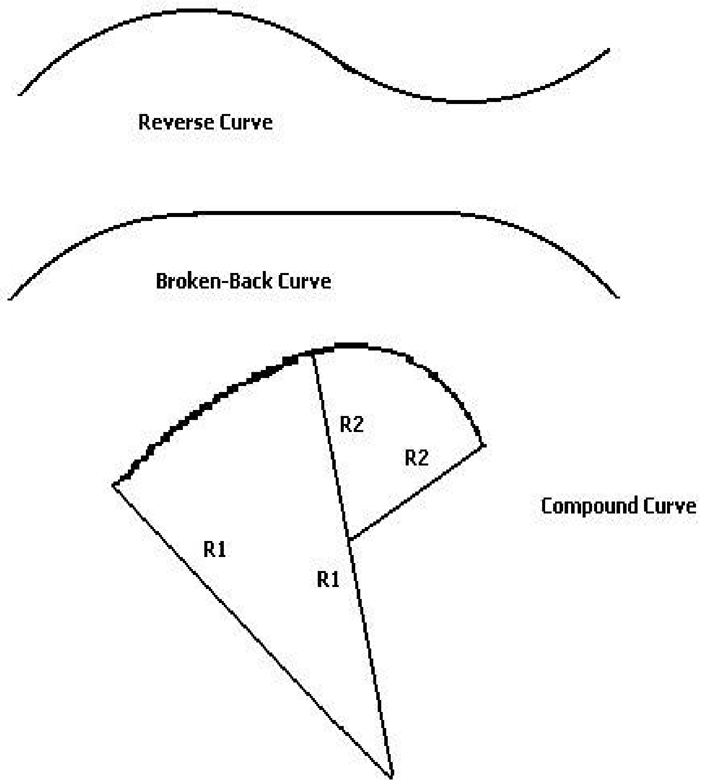


Figure 6-10: Reverse Curves, Broken-Back Curves, and Compound Curves

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.

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.

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.

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. Where the use of compound curves cannot be avoided, the radius of the flatter circular arc preferably should not be more than 50 percent greater than the radius of the sharper arc; i.e. R1 should not exceed 1.5 R2. A compound arc on this basis is suitable as a form of transition from either a flat curve or a tangent to a sharper curve, although a spiral transition curve is preferred (see Section Switchbacks iii).

ii)Isolated Curves

Long tangent roadway segments, joined by an isolated curve designed at or near the minimum radius result in unsafe operations, as a driver will anticipate drivable speeds in excess of the design speed. Good design practice is to avoid the use of minimum standards in such conditions. For isolated curves, the minimum horizontal curve radius as shown in Tables 4-3 through 4-9 in section 4 shall be increased by 50 percent. This will result, generally, in the ability to negotiate the curve at a speed approximately 10 km/h higher than the design speed.

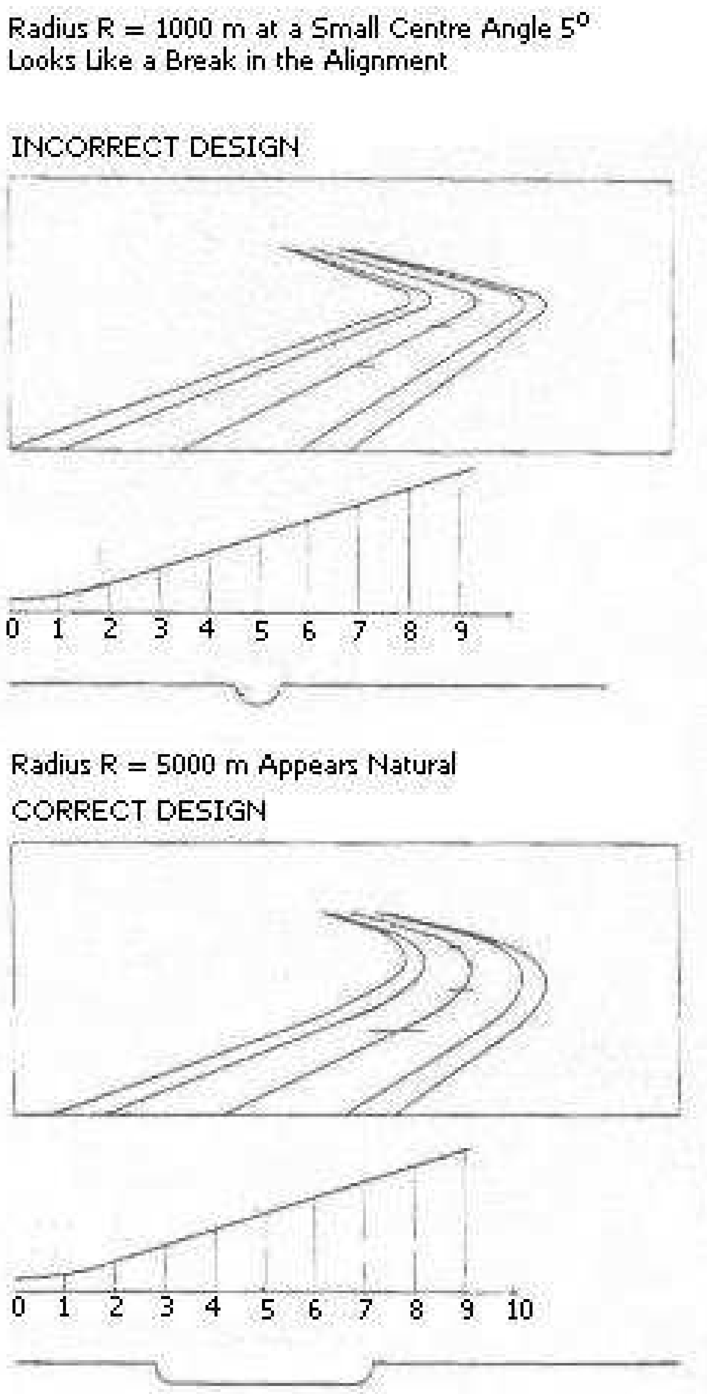


Figure 6-11:Isolated curves

iii) Switchback Curves

Switchback or hairpin curves are used where necessary in traversing mountainous and escarpment terrain. 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.

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, as indicated in Figures 5-1 through 5-5. These figures show that the minimum outer radii for design vehicles DV2 through DV5 are 12.8m, 12.8m, 13.7m, and 13.7m, respectively. Minimum inner radii are 8.5m, 7.4m, 5.8m, and 2.9m, respectively.

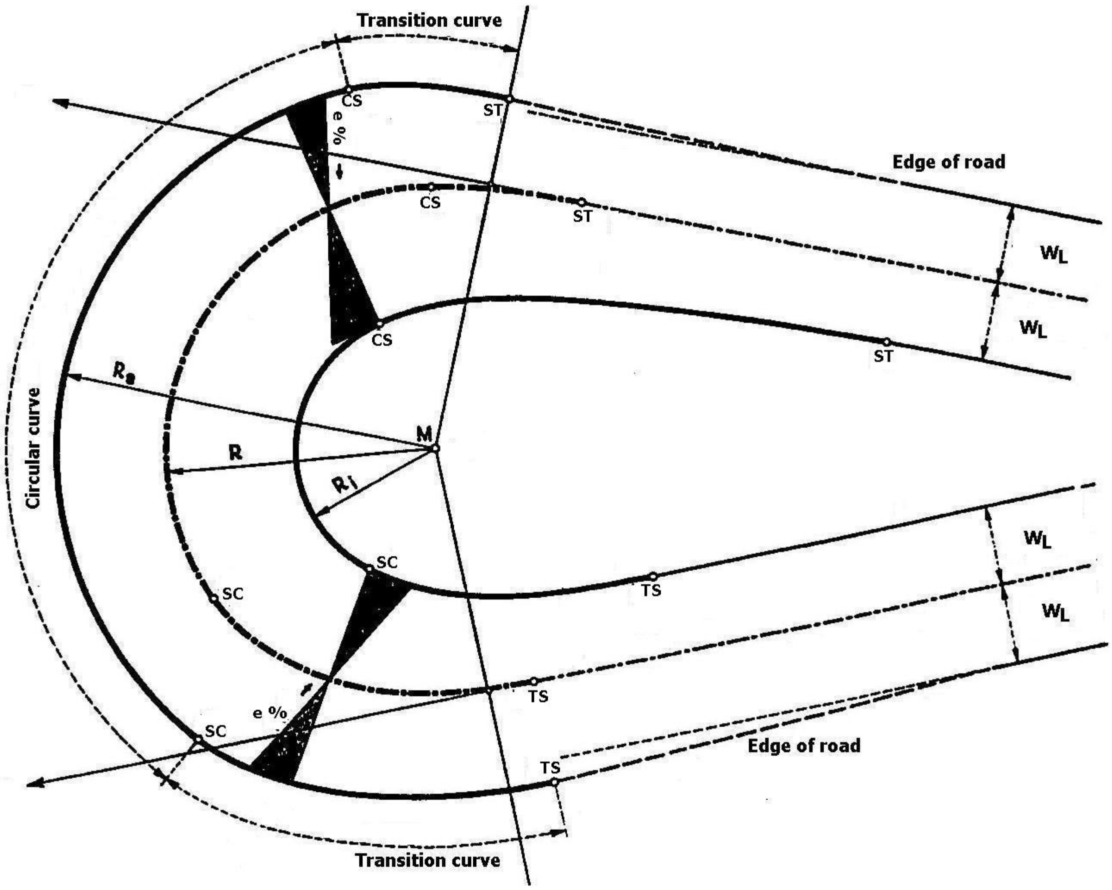


Figure 6-12: Switch back curve

iv) Transition Curves

The characteristic of transition (spiral or clothoid) curve is that it has a constantly changing radius. Transition curves may be inserted between tangents and circular curves to reduce the abrupt introduction of the lateral acceleration. They may also be used to link straights or two circular curves.

In practice, drivers employ their own transition on entry to a circular curve and transition curve contribute to the comfort of the driver in only a limited number of situations. However, they also provide convenient sections over which superelevation or pavement widening may be applied, and can improve the appearance of the road by avoiding sharp discontinuities in alignment at the beginning and end of circular curves. For large radius curves the rate of change of lateral acceleration is small and transition curves are not normally required.

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. By definition the radius at any point of the spiral varies inversely with the distance measured along the spiral.

The following equation is used for computing the minimum length of spiral.

Where:

$$L = \frac{0.0702V^3}{RC}$$

L = minimum length of spiral, [m];

V = speed, km/h;

R = curve radius, [m]; and,

C = rate of increase of centripetal acceleration, [m/s3]

The factor C is an empirical value indicating the comfort and safety involved. The value C=1 generally is acceptable for railroad operation, but values ranging from 1 to 3 have been used for roads. A more practical control for the length of spiral is that in which it equals the length required for superelevation runoff.

Transition curves are required if the following relationship is fulfilled:

$$R < \frac{V^3}{432}$$

Where:

R = Radius of curve [m]; and,

V = Design speed [km/hr]

In all other cases where the above is not fulfilled, transition curve is not required.

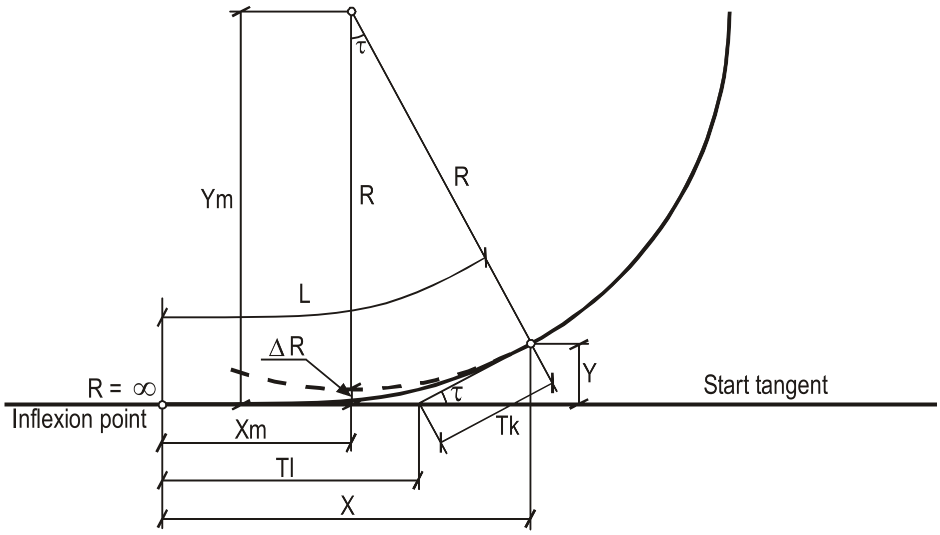
v) Clothoids

The clothoid is the name given to one form of transition curve. This is the most commonly used form of transition in highway design and is defined mathematically as follows:

$A^2 = R - L$

where A = clothoid parameter (m)
R = final radius (m). Start point in the straight tangent with R=∞
L = clothoid length (m)

$A^2 = R \times L$



R = final radius
ΔR = circular shift
τ = direction change
Tl = long tangent
Tk =short tangent
Xm = abscissa of centre point
Ym = centre Y-coordinatet
X = final clothoide X-coordinate
Y = final clothoide Y-coordinate

Figure 6-13:Clothoid elements

The parameter of the clothoid and the length of the transition curve recommended are decided by the rate of increase of centripetal acceleration (m/sec^3) k, see formula below. K should normally be in the interval 0.3 to 0.6 with a standard value of 0.45 m/sec^3.

$$A = \sqrt{v^3 / k}$$

where A = clothoid parameter (m)
v = design speed (m/s)
k = rate of increase of centripetal acceleration, (0,45 m/s^3)
(adapted from Swedish and German design guidelines)

The recommended transition length should apply to the following requirements:

- at least as long as the length needed for the change of superelevation;
- not exceeding the rate of increase of centripetal acceleration; and,
- aesthetic balance between parameters and arc lengths.

Minimum lengths due to rate of change of centripetal acceleration are given in Figure 6-14.

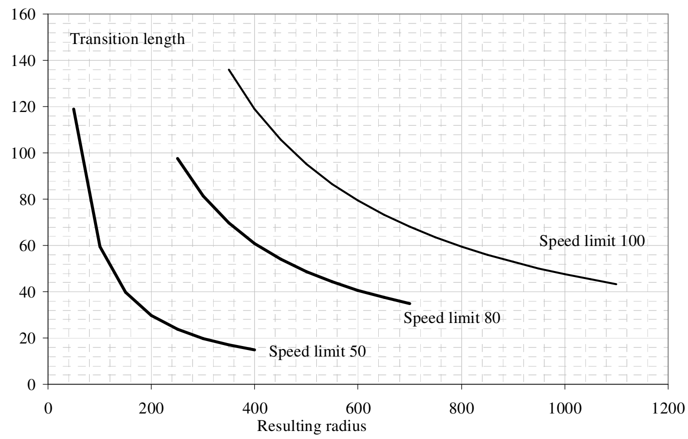


Figure 6-14: Minimum length due to rate of change of centripetal acceleration

Aesthetic considerations recommend that

- R/3 < A < R
- 0.1R < L < R

A simple clothoid could be used as transition curve between a tangent and a circle. The circular displacement ΔR should be at least 0.25 m to avoid kinks.

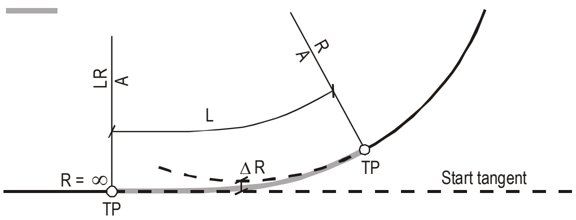


Figure 6-15: Simple clothoid

The compound curve consists of two arcs with a connecting clothoid. The major circle must surround the minor circle with separate centre points.

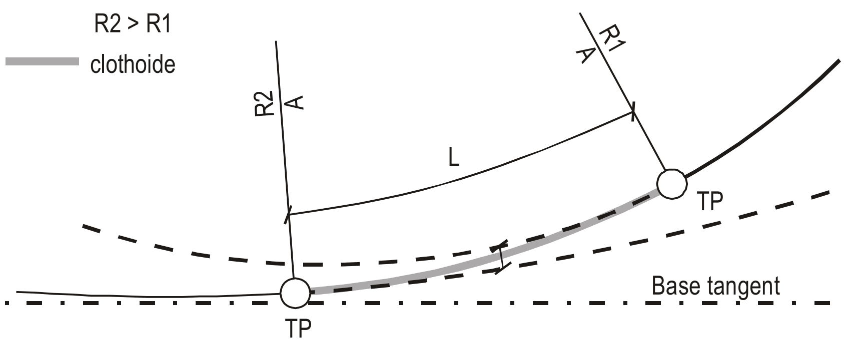


Figure 6-16: Compound curve

The compound curve could be substituted with three circles with $R_1 < R_2 < R_3$.

The S-curve is a combination of two clothoids between two reverse circles. The relationship between the clothoid parameters could be controlled by aesthetic aspects on harmonic alignment, differences in size of superelevation and design of superelevation transition. With A < 200 the ratio A1 / A2 $\leq$ 1.5 is recommended, see also X. A1 / A2 > 1.5 could give an unbalanced design.

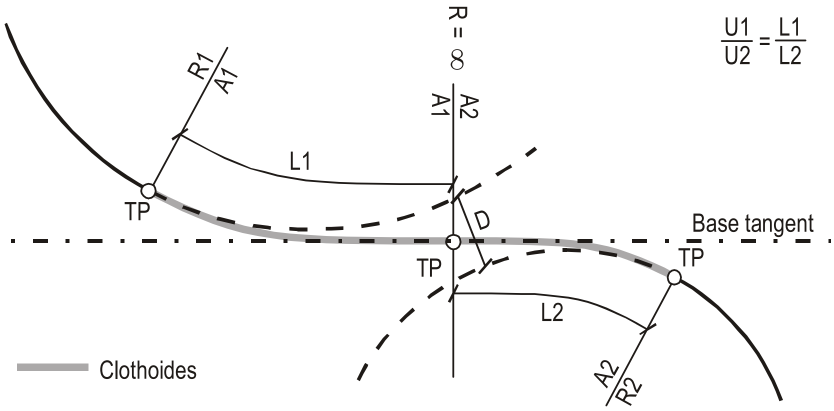


Figure 6-17: S-curve

vi) Superelevation

The normal cross-fall on a road will result in a vehicle on the outside lane of a horizontal curve needing to develop high levels of frictional force to resist sliding; the amount of increase being dependent on speed, curve radius and cross-fall. In order to achieve the necessary cornering stability, it is recommended that adverse cross-fall is removed. The identification of speed and radius combinations at which this should occur is rather subjective as there is no evidence linking adverse cross-fall to accident risk.

For small radius curves and at higher speeds, the removal of adverse cross-fall alone will be insufficient to reduce frictional needs to an acceptable level, and cross-fall should be increased by the application of superelevation. A minimum radius is reached when the maximum acceptable frictional superelevation derived forces have been developed.

On unpaved roads, the cross-fall is designed to remove rainwater quickly and effectively, and will be dependent on local conditions and materials. Values of superelevation lower than the value of the cross-fall will fail to drain the surface, whilst higher values will be likely to result in erosion. Therefore, on unpaved roads, it is preferable that the maximum superelevation will therefore be the elimination of adverse crossfall.

The practical limitations to the use of higher values or rate of superelevation are that lower friction values may prevail due to any thin layer of mud on the pavement surface, oil spots, and possible water on pavement surface of curve with poor surface drainage, which with high speeds results in hydroplaning.

In urban areas where traffic congestion or extensive marginal development acts to curb top speeds, it is common practice to utilize a low maximum rate of superelevation, usually 4 percent. Similarly, either a low maximum rate of superelevation or no superelevation is employed within important intersection areas or where there is a tendency to drive slowly because of turning and crossing movements, warning devices, and signals. Superelevation is a requirement for all standards of roads.

vii) Superelevation Runoff

Where transition (spiral or clothoid) curves are used superelevation should be applied over the length of the transition curves. Otherwise it should be introduced such that two thirds are applied on the straight prior to the start of the circular curve. Superelevation runoff is the length of road needed to accomplish the change in cross slope from a section with adverse crown removed to a fully superelevated section, or vice versa. Tangent runout is the length of road needed to accomplish the change in cross slope from a normal section to a section with the adverse cross slope removed, or vice versa.

Length of runoff is directly proportional to the total superelevation, which is the product of the lane width and superelevation rate. Accordingly the length of superelevation runoff is computed as follows:

L=i x Lw x ratio

Where:

L=length of superelevation runoff

i=superelevation slope (i/100)

Lw=lane width

ratio=maximum relative gradient (slope) ratio for profiles between the edges of carriageway and centerline.

However the length obtained by the above relationship should not be less than the minimum length of runoff which is approximately the distance traveled in 2 seconds at the design speed. This minimum length should be provided from the point of view of general appearance and to avoid undesirably abrupt edge-of-pavement profiles.

The maximum relative gradients (slopes) for profiles between the edges of two lane carriageway at different design speed are given in the table below:

Table 6-2: Maximum relative gradient (slope) ratio for profiles between edges of two lane carriageway and centerline and design speed

Design speed [km/hr]	Maximum relative gradient (slope) ratio for profiles between edges of two lane carriageway and centerline [%].
30	133:1
40	143:1
50	150:1
60	167:1
70	182:1
80	200:1
90	210:1
100	210:1
110	238:1
120	250:1

Among the available methods of profile design in attaining superelevation, the method which attains the superelevation by revolving the carriageway with normal cross slopes about the centerline profile is recommended.

In horizontal alignment design with transition curves/spirals, the superelevation runoff is effected over the whole length of the transition curve (spiral curve). The length of runoff as can be seen on Figure 6-6 is the spiral length with the tangent to spiral (TS) at the beginning and the spiral to curve (SC) at the end. The length of the runoff/spiral (without including tangent runout) should not be less than that given in Tables 6-4 – 6.9 for the respective design speed, radius of curve, and maximum superelevation. The values given in these tables are for 3.65-m lanes. For other lane widths the length vary in proportion of the actual lane width to 3.65 m. This means that shorter lengths are required for lane widths less than 3.65 m. However, considering uniformity and practical use of the empirically derived values it is suggested to use the values of 3.65 m for all cases.

Superelevation runoff length for wider carriageway should be greater than those for a two lane rural road. Superelevation runoff length for roads having more than two lanes, expressed in terms of length of superelevation of two-lane road, is as given in the table below.

Table 6-3: Length of superelevation runoff for multi lane roads expressed in the length of two lane road

No. of lanes	Length of superelevation runoff expressed as the length of superelevation of two lane road
2	1.0
3	1.2
4	1.5

The change in cross slope begins by removing the adverse cross slope from the lane or lanes on the outside of the curve on a length of tangent just ahead of TS (the tangent runout). 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. This rate of removal should preferably be the same as the rate used to effect the superelevation runoff.

Between the TS and SC (the superelevation runoff) the traveled way is rotated about the center of the road to reach the full superelevation at the SC. This procedure is reversed on leaving the curve. By this design the whole of the circular curve has full superelevation, as shown in Figure 6.6.

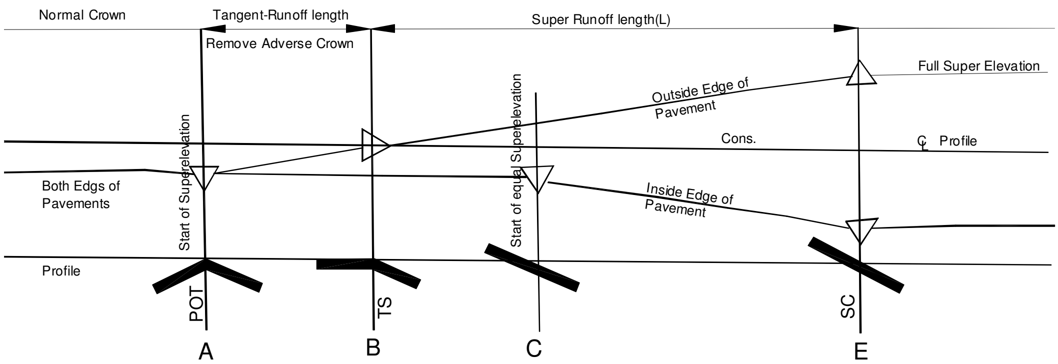


Figure 6-18: Spiral Curve Transition

In design of curves without spirals the superelevation runoff is considered to be that length beyond the tangent runout. Empirical methods are employed to locate the superelevation runoff length with respect to the point of curvature (PC).

Current design practice is to place approximately two-thirds of the runoff on the tangent approach and one-third on the curve, as shown in Figure 6.7.

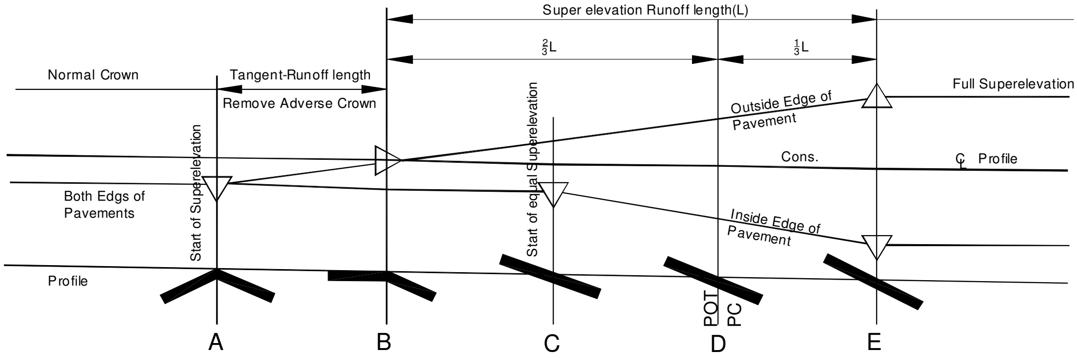


Figure 6-19: Circular Curve Transition

EXAMPLE 1:-

A Simple curve has a deflection angle of Δ =52° 27’ 32”, a radius of R =300 m, a design speed of V=50km/hr, Roadway width Rw=7m, Normal crown slope of NC=2.5%, and a maximum superelevation rate of e_{max}=7%. The point of intersection (PI) is at station 10+684.231. Calculate Degree of Curvature (D), Tangent distance(T), curve length (Lc), Point of Curvature (PC), Point of Tangent (PT), and Points on superelevation runoff.

D=1145.916/300=3.81972
T=R*tan(Δ/2)=300*tan(52.4589/2)=147.81
Lc=20*Δ/D=20*52.459/3.81972=274.674
PC=PI-T=10+684.231-147.81=10+536.421
PT=PC+Lc=10+536.421+274.674=10+811.095

From table FSE = 4.2%, Ls = 28m
Start of superelevation = PC-2/3*Ls-Lra

Where
FSE, full superelevaation
Ls, runoff length
Lra, Length of remove adverse crown
=NC*Ls/FSE
=2.5*28/4.2
=16.67 m

Start of superelevation=10+536.421-(2/3*28)-16.67
=10+501.081

Start of runoff=PC-2/3*Ls
=10+536.421-18.67
=10+517.751

Start of equal superelevation = Start of runoff + Lra
=10+517.751+16.67
=10+534.421

Start of FSE =PC+1/3*Ls
=10+536.421+1/3*28
=10+545.754

End of FSE=PT-1/3*Ls
=10+811.095-1/3*28
=10+801.762

End of runoff=PT+2/3*Ls
=10+811.095+18.67
=10+829.765

End of equal superelevation = End of runoff-Lra
=10+8219.765-16.67
=10+813.095

End of superelevation =PT+2/3*Ls+Lra
=10+811.095+2/3*28+16.67
=10+846.435

EXAMPLE 2:-

A curve has a deflection angle of Δ=23° 18’ 02”, a radius of R=250,a design speed of V=80km/hr,Road way width Rw=7m,Normal Crown slope of NC=2.5%,and a maximum superelevation rate of e_{max}=7%. The point of intersection (PI) is 30+270.759. Calculate Degree of Curvature (D), Central angle of spiral arc (θs), length of spiral (Ls), total tangent distance (T),curve length (L), point of change from tangent to spiral (TS), point of change from spiral to curve (SC), point of change from curve to spiral (CS), point of change from spiral to tangent (ST),and Points on superelevation runoff.

If $R>V^3/432$ then transition curve is not required
$V^3=80^3/432=1185.185>250$ a transition curve is required
From table 6-7 FSE=7%, NC=2.5%; Ls=49m

D=1145.916/250=4.5836

θs=Ls*D/40
=49*4.5836/40=5.615

T= total tangent length = R*tan(Δ/2)+Ls/2

T =250*tan(23.3/2)+49/2=76.045

L=20*(Δ-2* θs)/D=20*(23.3-2*5.615)/4.5386=53.188

TS=PI-T=30+270.759 - 76.045=30+194.714

SC=TS+Ls=30+194.714+49 =30+243.714

CS=SC+L=30+243.714+53.188=30+296.902

ST=CS+Ls=30+296.902+49=30+345.902

Start of superelevation = TS-Lra

Where

Lra Length of remove adverse crown
=NC*Ls/FSE
=2.5*49/7
=17.5

Start of superelevation = 30+194.714-17.5
= 30+177.214

Start of equal superelevation = TS+Lra
= 30+194.714+17.5
= 30+212.214

Start of full superelevation = TS + Ls
= 30+194.714 + 49
= 30+243.714

End of full superelevation = Start of full superelevation + Curve length
= 30+243.714 + 53.188
= 30+296.902

End of equal superelevation =ST-Lra
=30+345.902-17.5
=30+328.402

End of superelevation =ST+Lra
=30+345.902+17.5
=30+363.402

Tables 6-4 – 6.9 give both superelevation rates and length of runoff for horizontal curves at different speeds for 4 percent, 7 percent, and 8 percent maximum superelevation and normal crown slope of both 2.5 and 4 percent.

vii) Shoulder Superelevation

Figure 6-8 depicts shoulder superelevation rates corresponding to carriageway superelevation rates. The figure shows that on the low side (inner shoulder) of superelevated curves, the shoulder superelevation matches the roadway superelevation. On the high side (outer shoulder), the superelevation is set such that the grade break between the roadway and the shoulder is 8 percent. An exception to this occurs at a maximum superelevation of 8 percent, where the resultant shoulder superelevation would be an undesirable flat configuration. Here the superelevation is set at -1% to drain the shoulder. On paved road with unsealed shoulders, the shoulders should drain away from the paved area to avoid loose material being washed across the road.

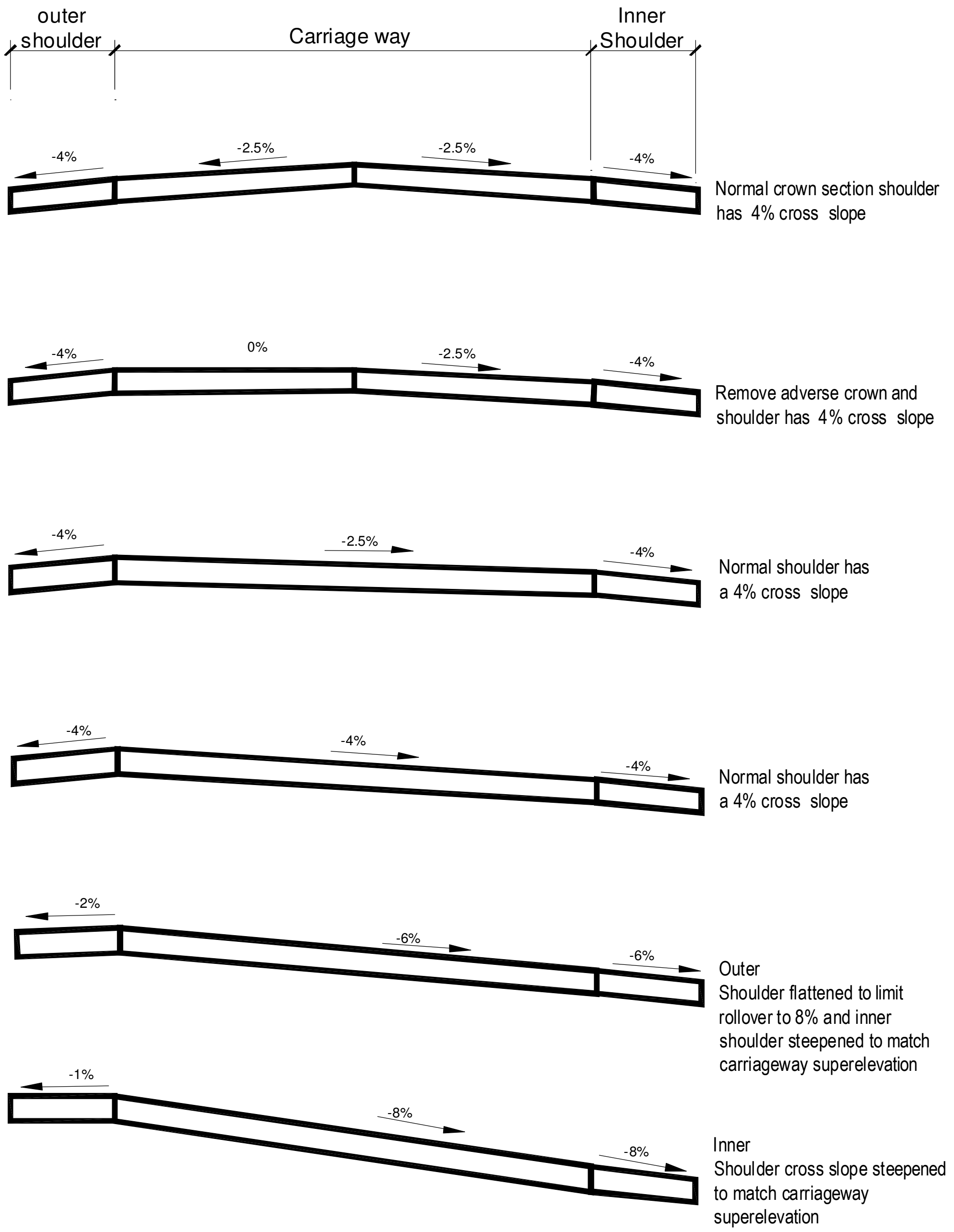


Figure 6-20: Shoulder Superelevation (for Surfaced Roads)

Table 6-4: Superelevation rates, length of runoff and other values for design elements related to design speed and horizontal curve (e_{max}=4%, NC= 2.5%)

	Vd=30km/h		Vd=40km/h		Vd=50km/h		Vd=60km/h		Vd=70km/h		Vd=80km/h		Vd=90km/h		Vd=100km/h		Vd=110km/h		Vd=120km/h	
R(m)	**e(%)**	**L(m)**	**E(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**
7000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
5000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
3000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	56	RC	61	RC	67
2500	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	50	RC	56	RC	61	RC	67
2000	NC	0	NC	0	NC	0	NC	0	NC	0	RC	44	RC	50	RC	56	RC	61	2.8%	67
1500	NC	0	NC	0	NC	0	NC	0	RC	39	RC	44	RC	50	2.6%	56	2.9%	61	3.4%	67
1400	NC	0	NC	0	NC	0	NC	0	RC	39	RC	44	RC	50	2.7%	56	3.0%	61	3.5%	67
1300	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	RC	50	2.8%	56	3.1%	61	3.6%	67
1200	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	2.6%	50	2.9%	56	3.3%	61	3.7%	67
1000	NC	0	NC	0	NC	0	RC	33	RC	39	2.5%	44	2.9%	50	3.2%	56	3.6%	61	3.9%	67
900	NC	0	NC	0	RC	28	RC	33	RC	39	2.7%	44	3.0%	50	3.4%	56	3.7%	61	4.0%	67
800	NC	0	NC	0	RC	28	RC	33	2.5%	39	2.8%	44	3.2%	50	3.5%	56	3.9%	61	**Rmin**	**870**
700	NC	0	NC	0	RC	28	RC	33	2.7%	39	3.0%	44	3.4%	50	3.7%	56	4.0%	61		
600	NC	0	RC	22	RC	28	RC	33	2.9%	39	3.2%	44	3.6%	50	3.9%	56	**Rmin**	**635**		
500	NC	0	RC	22	RC	28	2.7%	33	3.1%	39	3.5%	44	3.8%	50	4.0%	56				
400	NC	0	RC	22	2.5%	28	3.0%	33	3.4%	39	3.7%	44	4.0%	50	**Rmin**	**490**				
300	RC	17	RC	22	2.8%	28	3.3%	33	3.8%	39	4.0%	44	**Rmin**	**375**						
250	RC	17	2.6%	22	3.1%	28	3.6%	33	3.9%	39	**Rmin**	**280**								
200	RC	17	2.8%	22	3.3%	28	3.8%	33	**Rmin**	**215**										
175	RC	17	2.9%	22	3.5%	28	3.9%	33												
150	2.5%	17	3.1%	22	3.7%	28	4.0%	33												
140	2.6%	17	3.2%	22	3.8%	28	**Rmin**	**150**												
130	2.6%	17	3.3%	22	3.8%	28														
120	2.7%	17	3.4%	22	3.9%	28														
119	2.7%	17	3.4%	22	3.9%	28														
110	2.8%	17	3.5%	22	4.0%	28														
100	2.9%	17	3.6%	22	4.0%	28														
90	3.0%	17	3.7%	22	**Rmin**	**100**														
80	3.2%	17	3.8%	22																
70	3.3%	17	3.9%	22																
60	3.5%	17	4.0%	22																
50	3.7%	17	**Rmin**	**60**																
40	3.9%	18																		
30	**Rmin**	**35**																		

e_{max}= 4%
R =radius of curve
V =assumed design speed
E =rate of superelevation
L =minimum length of runoff(does not include tangent run out)
NC =normal crown section
RC =remove adverse crown,superelevation at normal crown slope
Width of lane 3.5 m
Normal crown slope 2.5%; Adverse crown slope 1.5%

Table 6-5: Superelevation rates, length of runoff and other values for design elements related to design speed and horizontal curve (e_{max}=4%, NC=4%)

	Vd=30km/h		Vd=40km/h		Vd=50km/h		Vd=60km/h		Vd=70km/h		Vd=80km/h		Vd=90km/h		Vd=100km/h		Vd=110km/h		Vd=120km/h	
R(m)	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**
7000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
5000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
3000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	56	RC	61	RC	67
2500	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	50	RC	56	RC	61	RC	67
2000	NC	0	NC	0	NC	0	NC	0	NC	0	RC	44	RC	50	RC	56	RC	61	RC	67
1500	NC	0	NC	0	NC	0	NC	0	RC	39	RC	44	RC	50	RC	56	RC	61	RC	67
1400	NC	0	NC	0	NC	0	NC	0	RC	39	RC	44	RC	50	RC	56	RC	61	RC	67
1300	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	RC	50	RC	56	RC	61	RC	67
1200	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	RC	50	RC	56	RC	61	RC	67
1000	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	RC	50	RC	56	RC	61	RC	67
900	NC	0	NC	0	RC	28	RC	33	RC	39	RC	44	RC	50	RC	56	RC	61	RC	67
800	NC	0	NC	0	RC	28	RC	33	RC	39	RC	44	RC	50	RC	56	RC	61	**Rmin**	**870**
700	NC	0	NC	0	RC	28	RC	33	RC	39	RC	44	RC	50	RC	56	RC	61		
600	NC	0	RC	22	RC	28	RC	33	RC	39	RC	44	RC	50	RC	56	**Rmin**	**635**		
500	NC	0	RC	22	RC	28	RC	33	RC	39	RC	44	RC	50	RC	56				
400	NC	0	RC	22	RC	28	RC	33	RC	39	RC	44	RC	50	**Rmin**	**490**				
300	RC	19	RC	22	RC	28	RC	33	RC	39	RC	44	**Rmin**	**375**						
250	RC	19	RC	22	RC	28	RC	33	RC	39	**Rmin**	**280**								
200	RC	19	RC	22	RC	28	RC	33	**Rmin**	**215**										
175	RC	19	RC	22	RC	28	RC	33												
150	RC	19	RC	22	RC	28	RC	33												
140	RC	19	RC	22	RC	28	**Rmin**	**150**												
130	RC	19	RC	22	RC	28														
120	RC	19	RC	22	RC	28														
119	RC	19	RC	22	RC	28														
110	RC	19	RC	22	RC	28														
100	RC	19	RC	22	RC	28														
90	RC	19	RC	22	**Rmin**	**100**														
80	RC	19	RC	22																
70	RC	19	RC	22																
60	RC	19	4.0%	22																
50	RC	19	**Rmin**	**60**																
40	RC	19																		
30	**Rmin**	**35**																		

e_{max}= 4%
R =radius of curve
V =assumed design speed
e =rate of superelevation
L =minimum length of runoff(does not include tangent run out)
NC =normal crown section
RC =remove adverse crown, superelevation at normal crown slope
Width of lane 3.5 m
Normal crown slope = 4.0%; Adverse crown slope = 1.5%

Table 6-6: Superelevation rates, length of runoff and other values for design elements related to design speed and horizontal curve (e_{max}=7%, NC=2.5%)

	Vd=30km/h		Vd=40km/h		Vd=50km/h		Vd=60km/h		Vd=70km/h		Vd=80km/h		Vd=90km/h		Vd=100km/h		Vd=110km/h		Vd=120km/h	
R(m)	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**
7000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
5000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
3000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	56	RC	61	RC	67
2500	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	50	RC	56	RC	61	2.8%	67
2000	NC	0	NC	0	NC	0	NC	0	NC	0	RC	44	RC	50	2.6%	56	2.9%	61	3.4%	67
1500	NC	0	NC	0	NC	0	NC	0	RC	39	RC	44	2.8%	50	3.3%	56	3.8%	61	4.4%	67
1400	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	2.9%	50	3.5%	56	4.0%	61	4.7%	67
1300	NC	0	NC	0	NC	0	RC	33	RC	39	2.6%	44	3.1%	50	3.7%	56	4.2%	61	5.0%	67
1200	NC	0	NC	0	NC	0	RC	33	RC	39	2.8%	44	3.3%	50	4.0%	56	4.5%	61	5.3%	67
1000	NC	0	NC	0	RC	28	RC	33	2.7%	39	3.3%	44	3.9%	50	4.6%	56	5.2%	61	6.1%	67
900	NC	0	NC	0	RC	28	RC	33	3.0%	39	3.5%	44	4.2%	50	4.9%	56	5.6%	61	6.5%	67
800	NC	0	NC	0	RC	28	2.6%	33	3.3%	39	3.9%	44	4.6%	50	5.3%	56	6.1%	61	6.9%	67
700	NC	0	RC	22	RC	28	2.9%	33	3.6%	39	4.3%	44	5.0%	50	5.8%	56	6.5%	61	7.0%	67
600	NC	0	RC	22	2.5%	28	3.3%	33	4.1%	39	4.8%	44	5.5%	50	6.3%	56	6.9%	61	**Rmin**	**700**
500	NC	0	RC	22	2.9%	28	3.8%	33	4.6%	39	5.3%	44	6.1%	50	6.8%	56	7.0%	61		
400	RC	17	2.6%	22	3.5%	28	4.4%	33	5.3%	39	6.0%	44	6.7%	50	7.0%	56	**Rmin**	**500**		
300	RC	17	3.3%	22	4.2%	28	5.2%	33	6.1%	39	6.8%	47	7.0%	51	**Rmin**	**400**				
250	RC	17	3.8%	22	4.7%	28	5.7%	33	6.6%	42	7.0%	49	**Rmin**	**300**						
200	2.9%	17	4.3%	22	5.3%	28	6.3%	37	7.0%	44	**Rmin**	**250**								
175	3.2%	17	4.6%	23	5.6%	29	6.6%	39	**Rmin**	**200**										
150	3.6%	17	4.9%	25	6.0%	32	6.9%	40												
140	3.8%	18	5.1%	25	6.2%	33	7.0%	41												
130	3.9%	18	5.2%	26	6.4%	33	**Rmin**	**140**												
120	4.1%	19	5.4%	27	6.6%	34														
119	4.2%	19	5.5%	27	6.6%	35														
110	4.3%	20	5.7%	28	6.7%	35														
100	4.5%	21	5.9%	29	6.9%	36														
90	4.8%	22	6.1%	31	7.0%	37														
80	5.0%	23	6.4%	32	**Rmin**	**90**														
70	5.3%	25	6.7%	33																
60	5.7%	27	6.9%	35																
50	6.2%	29	7.0%	35																
40	6.7%	31	**Rmin**	**50**																
30	7.0%	33																		
	Rmin	**30**																		

e_{max}= 7%
R =radius of curve
V =assumed design speed
e =rate of superelevation
L =minimum length of runoff(does not include tangent run out)
NC =normal crown section
RC =remove adverse crown,superelevation at normal crown slope
Width of lane = 3.5 m
Normal crown slope = 2.5%; Adverse crown slope = 1.5%

Table 6-7: Superelevation rates, length of runoff and other values for design elements related to design speed and horizontal curve (e_{max}=7%, NC=4%)

	Vd=30km/h		Vd=40km/h		Vd=50km/h		Vd=60km/h		Vd=70km/h		Vd=80km/h		Vd=90km/h		Vd=100km/h		Vd=110km/h		Vd=120km/h	
R(m)	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**
7000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
5000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
3000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	56	RC	61	RC	67
2500	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	50	RC	56	RC	61	RC	67
2000	NC	0	NC	0	NC	0	NC	0	NC	0	RC	44	RC	50	RC	56	RC	61	RC	67
1500	NC	0	NC	0	NC	0	NC	0	RC	39	RC	44	RC	50	RC	56	RC	61	4.4%	67
1400	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	RC	50	RC	56	RC	61	4.7%	67
1300	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	RC	50	RC	56	4.2%	61	5.0%	67
1200	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	RC	50	RC	56	4.5%	61	5.3%	67
1000	NC	0	NC	0	RC	28	RC	33	RC	39	RC	44	RC	50	4.6%	56	5.2%	61	6.1%	67
900	NC	0	NC	0	RC	28	RC	33	RC	39	RC	44	4.2%	50	4.9%	56	5.6%	61	6.5%	67
800	NC	0	NC	0	RC	28	RC	33	RC	39	RC	44	4.6%	50	5.3%	56	6.1%	61	6.9%	67
700	NC	0	RC	22	RC	28	RC	33	RC	39	4.3%	44	5.0%	50	5.8%	56	6.5%	61	7.0%	67
600	NC	0	RC	22	RC	28	RC	33	4.1%	39	4.8%	44	5.5%	50	6.3%	56	6.9%	61	**Rmin**	**700**
500	NC	0	RC	22	RC	28	RC	33	4.6%	39	5.3%	44	6.1%	50	6.8%	56	7.0%	61		
400	RC	19	RC	22	RC	28	4.4%	33	5.3%	39	6.0%	44	6.7%	50	7.0%	56	**Rmin**	**500**		
300	RC	19	RC	22	4.2%	28	5.2%	33	6.1%	39	6.8%	47	7.0%	51	**Rmin**	**400**				
250	RC	19	RC	22	4.7%	28	5.7%	33	6.6%	42	7.0%	49	**Rmin**	**300**						
200	RC	19	4.3%	22	5.3%	28	6.3%	37	7.0%	44	**Rmin**	**250**								
175	RC	19	4.6%	23	5.6%	29	6.6%	39	**Rmin**	**200**										
150	RC	19	4.9%	25	6.0%	32	6.9%	40												
140	RC	19	5.1%	25	6.2%	33	7.0%	41												
130	RC	19	5.2%	26	6.4%	33	**Rmin**	**140**												
120	4.1%	19	5.4%	27	6.6%	34														
119	4.2%	19	5.5%	27	6.6%	35														
110	4.3%	20	5.7%	28	6.7%	35														
100	4.5%	21	5.9%	29	6.9%	36														
90	4.8%	22	6.1%	31	7.0%	37														
80	5.0%	23	6.4%	32	**Rmin**	**90**														
70	5.3%	25	6.7%	33																
60	5.7%	27	6.9%	35																
50	6.2%	29	7.0%	35																
40	6.7%	31	**Rmin**	**50**																
30	7.0%	33																		
	Rmin	**30**																		

e_{max}= 7%
R =radius of curve
V =assumed design speed
e =rate of superelevation
L =minimum length of runoff(does not include tangent run out)
NC =normal crown section
RC =remove adverse crown,superelevation at normal crown slope
Width of lane = 3.5 m
Normal crown slope = 4%; Adverse crown slope = 1.5%

Table 6-8: Superelevation rates, length of runoff and other values for design elements related to design speed and horizontal curve (e_{max}=8%, NC=2.5%)

	Vd=30km/h		Vd=40km/h		Vd=50km/h		Vd=60km/h		Vd=70km/h		Vd=80km/h		Vd=90km/h		Vd=100km/h		Vd=110km/h		Vd=120km/h	
R(m)	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**
7000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
5000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
3000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	56	RC	61	RC	67
2500	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	50	RC	56	RC	61	2.9%	67
2000	NC	0	NC	0	NC	0	NC	0	NC	0	RC	44	RC	50	2.6%	56	3.0%	61	3.5%	67
1500	NC	0	NC	0	NC	0	NC	0	RC	39	RC	44	2.8%	50	3.4%	56	3.9%	61	4.6%	67
1400	NC	0	NC	0	NC	0	RC	33	RC	39	2.5%	44	3.0%	50	3.6%	56	4.1%	61	4.9%	67
1300	NC	0	NC	0	NC	0	RC	33	RC	39	2.7%	44	3.2%	50	3.8%	56	4.4%	61	5.2%	67
1200	NC	0	NC	0	NC	0	RC	33	RC	39	2.9%	44	3.4%	50	4.1%	56	4.7%	61	5.6%	67
1000	NC	0	NC	0	RC	28	RC	33	2.8%	39	3.4%	44	4.0%	50	4.8%	56	5.5%	61	6.5%	67
900	NC	0	NC	0	RC	28	RC	33	3.1%	39	3.7%	44	4.4%	50	5.2%	56	6.0%	61	7.1%	67
800	NC	0	NC	0	RC	28	2.7%	33	3.4%	39	4.1%	44	4.8%	50	5.7%	56	6.6%	61	7.6%	67
700	NC	0	RC	22	RC	28	3.0%	33	3.8%	39	4.5%	44	5.3%	50	6.3%	56	7.2%	61	8.0%	70
600	NC	0	RC	22	2.6%	28	3.4%	33	4.3%	39	5.1%	44	6.0%	50	6.9%	56	7.7%	64	**Rmin**	**655**
500	NC	0	RC	22	3.0%	28	3.9%	33	4.9%	39	5.8%	44	6.7%	50	7.6%	59	8.0%	67		
400	RC	17	2.7%	22	3.6%	28	4.7%	33	5.7%	39	6.6%	46	7.5%	55	8.0%	62	**Rmin**	**500**		
300	RC	17	3.5%	22	4.5%	28	5.6%	33	6.7%	43	7.6%	53	**Rmin**	**305**	**Rmin**	**395**				
250	RC	17	4.0%	22	5.1%	28	6.2%	37	7.4%	47	7.9%	56								
200	3.0%	17	4.6%	23	5.8%	30	7.0%	41	7.9%	50	**Rmin**	**230**								
175	3.4%	17	5.0%	25	6.2%	32	7.4%	43	8.0%	51										
150	3.8%	18	5.4%	27	6.7%	35	7.8%	45	**Rmin**	**175**										
140	4.0%	18	5.6%	28	6.9%	36	7.9%	46												
130	4.2%	19	5.8%	29	7.1%	37	8.0%	47												
120	4.4%	21	6.0%	30	7.4%	39	**Rmin**	**125**												
119	4.4%	21	6.0%	30	7.4%	39														
110	4.7%	22	6.3%	31	7.6%	40														
100	5.0%	23	6.6%	33	7.8%	41														
90	5.2%	24	6.9%	34	8.0%	42														
80	5.5%	26	7.2%	36	8.0%	42														
70	5.9%	28	7.5%	38	**Rmin**	**80**														
60	6.4%	30	7.9%	39																
50	6.9%	32	8.0%	40																
40	7.5%	35	**Rmin**	**50**																
30	8.0%	37																		
	Rmin	**30**																		

e_{max}= 8%
R =radius of curve
V =assumed design speed
e =rate of superelevation
L =minimum length of runoff(does not include tangent run out)
NC =normal crown section
RC =remove adverse crown,superelevation at normal crown slope
Lane width = 3.5 m
Normal crown slope = 2.5%; Adverse crown slope 1.5%

Table 6-9: Superelevation rates, length of runoff and other values for design elements related to design speed and horizontal curve (e_{max}=8%, NC=4%)

	Vd=30km/h		Vd=40km/h		Vd=50km/h		Vd=60km/h		Vd=70km/h		Vd=80km/h		Vd=90km/h		Vd=100km/h		Vd=110km/h		Vd=120km/h	
R(m)	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**	**e(%)**	**L(m)**
7000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
5000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
3000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	56	RC	61	RC	67
2500	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	50	RC	56	RC	61	RC	67
2000	NC	0	NC	0	NC	0	NC	0	NC	0	RC	44	RC	50	RC	56	RC	61	RC	67
1500	NC	0	NC	0	NC	0	NC	0	RC	39	RC	44	RC	50	RC	56	RC	61	4.6%	67
1400	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	RC	50	RC	56	4.1%	61	4.9%	67
1300	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	RC	50	RC	56	4.4%	61	5.2%	67
1200	NC	0	NC	0	NC	0	RC	33	RC	39	RC	44	RC	50	4.1%	56	4.7%	61	5.6%	67
1000	NC	0	NC	0	RC	28	RC	33	RC	39	RC	44	4.0%	50	4.8%	56	5.5%	61	6.5%	67
900	NC	0	NC	0	RC	28	RC	33	RC	39	RC	44	4.4%	50	5.2%	56	6.0%	61	7.1%	67
800	NC	0	NC	0	RC	28	RC	33	RC	39	4.1%	44	4.8%	50	5.7%	56	6.6%	61	7.6%	67
700	NC	0	RC	22	RC	28	RC	33	RC	39	4.5%	44	5.3%	50	6.3%	56	7.2%	61	8.0%	70
600	NC	0	RC	22	RC	28	RC	33	4.3%	39	5.1%	44	6.0%	50	6.9%	56	7.7%	64	**Rmin**	**655**
500	NC	0	RC	22	RC	28	RC	33	4.9%	39	5.8%	44	6.7%	50	7.6%	59	8.0%	67		
400	RC	19	RC	22	RC	28	4.7%	33	5.7%	39	6.6%	46	7.5%	55	8.0%	62	**Rmin**	**500**		
300	RC	19	RC	22	4.5%	28	5.6%	33	6.7%	43	7.6%	53	**Rmin**	**305**	**Rmin**	**395**				
250	RC	19	RC	22	5.1%	28	6.2%	37	7.4%	47	7.9%	56								
200	RC	19	4.6%	23	5.8%	30	7.0%	41	7.9%	50	**Rmin**	**230**								
175	RC	19	5.0%	25	6.2%	32	7.4%	43	8.0%	51										
150	RC	19	5.4%	27	6.7%	35	7.8%	45	**Rmin**	**175**										
140	RC	19	5.6%	28	6.9%	36	7.9%	46												
130	4.2%	19	5.8%	29	7.1%	37	8.0%	47												
120	4.4%	21	6.0%	30	7.4%	39	**Rmin**	**125**												
119	4.4%	21	6.0%	30	7.4%	39														
110	4.7%	22	6.3%	31	7.6%	40														
100	5.0%	23	6.6%	33	7.8%	41														
90	5.2%	24	6.9%	34	8.0%	42														
80	5.5%	26	7.2%	36	8.0%	42														
70	5.9%	28	7.5%	38	**Rmin**	**80**														
60	6.4%	30	7.9%	39																
50	6.9%	32	8.0%	40																
40	7.5%	35	**Rmin**	**50**																
30	8.0%	37																		
	Rmin	**30**																		

e_{max}= 8%
R =radius of curve
V =assumed design speed
e =rate of superelevation
L =minimum length of runoff(does not include tangent run out)
NC =normal crown section
RC =remove adverse crown, superelevation at normal crown slope
Lane width = 3.5 m
Normal crown slope = 4%; Adverse crown slope 1.5%

6.2.4 Widening on Curve

Widths should be increased on certain horizontal curves (a) to allow for the swept paths of trucks; (b) to allow drivers to maneuver when approaching other vehicles; (c) to make operating conditions on curves comparable to those on tangents. Widening is needed on certain curves due to (a) The vehicle or truck occupies a greater width because rear wheels generally track inside front wheel (off-tracking) in rounding curves, or (b) the driver experience difficulty in steering their vehicles in the center of the lane.

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 centered in lane and reduce the likelihood of either colliding with an oncoming vehicle or driving onto the shoulder.

Widening is also required (for design classes I, II, III, A, & B) at high fills for the psychological comfort of the driver. Widening for curve and high fill embankment shall be added where both cases apply. The height of fill is measured from the edge of the shoulder to the toe of the slope.

The following levels of widening are recommended.

Table 6-10: Widening on curves and on high fill

<table>
<tr><th rowspan="3">Radius of Curve [m]</th><th colspan="2">Curve widening</th><th colspan="2">Fill widening</th></tr>
<tr><th rowspan="2">Single lane</th><th rowspan="2">Two lane</th><th>Height of fill [m]</th><th>Widening [m]</th></tr>
<tr><td>0.0 – 3.0</td><td>0.3</td></tr>
<tr><td>20-40</td><td>0.6</td><td>1.5</td><td>3.0 – 6.0</td><td>0.6</td></tr>
<tr><td>41 – 60</td><td>0.6</td><td>1.2</td><td>6.0 – 9.0</td><td>0.9</td></tr>
<tr><td>61 – 120</td><td>0.0</td><td>0.9</td><td>Over 9.0</td><td>0.9</td></tr>
<tr><td>121 – 250</td><td>0.0</td><td>0.6</td><td></td><td></td></tr>
<tr><td>>250</td><td>0.0</td><td>0.0</td><td></td><td></td></tr>
</table>

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.

6.2.5 Sight distance on horizontal curves

Another element of horizontal alignment is the site distance across the inside of curve. Where there are sight obstruction (such as walls, cut slope, buildings and longitudinal barriers) on the inside of curves, a design to provide adequate sight distance may require adjustment in the normal road cross section or change in alignment if the obstruction could not be removed.

The sight distance is measured along the line of sight which is the chord line of the inside lane.

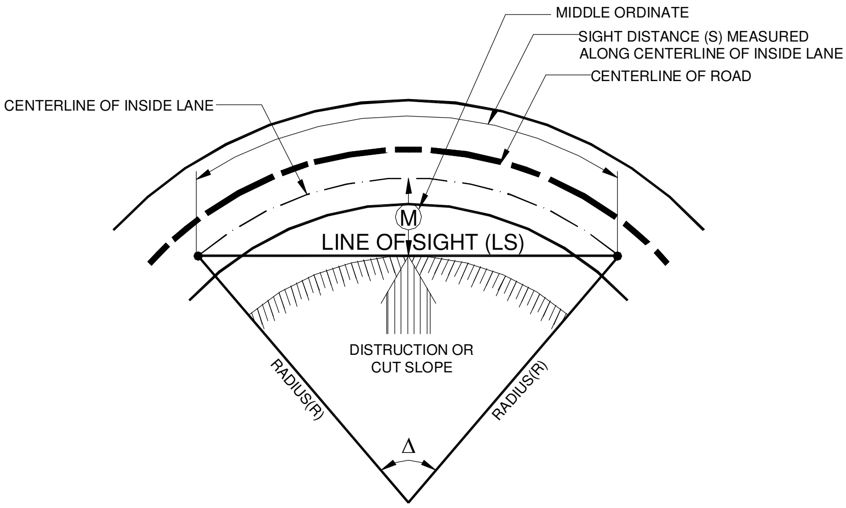


Figure 6-21: Sight distance on horizontal curve

Line of sight (LS) = 2R Sin(Δ/2)
Where Δ is the maximum deflection angle subtended by line of sight and the radius (^{o})
Middle ordinate (M) = R(1 – Cos (Δ/2)) [a]
Middle ordinate (M) = R(1 – Cos(28.65S/R)) [b]
S = stopping sight distance for the corresponding design speed

In order to satisfy the required stopping sight distance for any design speed, the middle ordinate calculated per the relationship [b] should not be less than that of [a]. This means that the available sight distance should not be less than the required sight distance for the specific design speed.

Example:

Design speed 80 km/hr, Radius 500 m
Maximum deflection angle subtended by the line of sight and radius of curve
$\Delta = 25^0$
Calculate the line of sight (LS), and the middle ordinate (M)
Line of sight (LS) = 2 x 500 x Sin(25/2) = 216.44meters
Middle ordinate (M) = 500 x (1 – Cos(25/2)) = 11.85meters
Middle ordinate (M) = 500 x (1 – Cos(28.55x126 /500)) = 3.94
11.85>>3.94 design is OK.

6.3 Vertical Alignment

The two major aspects of vertical alignment are vertical curvature, which is governed by sight distance and comfort criteria and gradient which is related to vehicle performance and level of service.

Vertical curves are required to provide smooth transitions between consecutive straight gradients. The simple parabola is recommended for these. The parabola provides a constant rate of change of curvature and hence acceleration and visibility, along its length and it has the form:

$$r = \frac{g_2 - g_1}{L} \qquad y = \frac{rx^2}{2} + g1x + BVC_{elevation}$$

Where

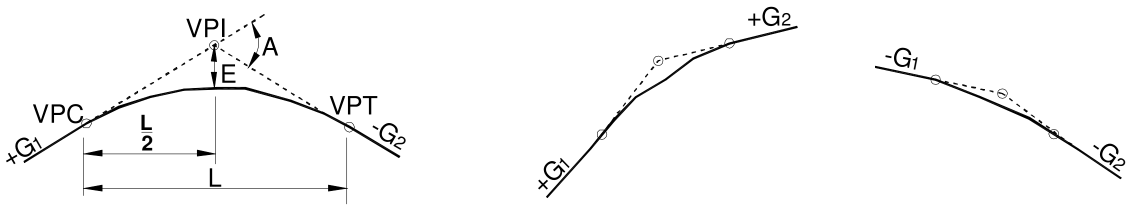
- r = rate of change of grade per section (%)
- g_1 = starting grade (%)
- g_2 = ending grade (%)
- L = length of curve (horizontal distance [m])
- y = elevation of a point on the curve
- x = distance in stations from the BVC (beginning of vertical curve) [meters/100]
- $BVC_{elevation}$ = elevation of beginning of the vertical curve
- EVC = end of the vertical curve

A related formula is:

$$y = \frac{G * L}{200}\left[\frac{x}{L}\right]^2$$

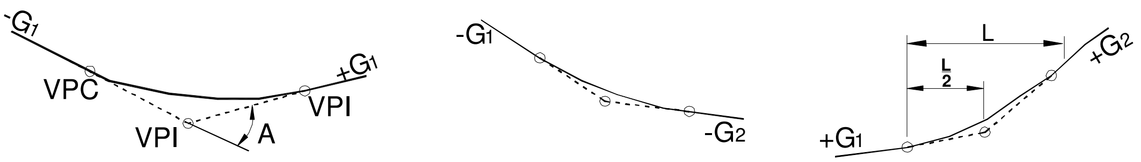
Where

- y = vertical distance from the tangent to the curve [meters]
- x = horizontal distance from the start of the vertical curve [meters]
- G = algebraic difference in gradients [%]
- L = length of vertical curve [meters]



G_1, and G_2, Tangent grades in persent
A, Algebric difference
L, Length of vertical curve
VPC, Vertical point of curvature
VPI, Vertical point of Intersection
VPT, Vertical point of Tangency

Figure 6-22: Crest vertical Curves



G_1, and G_2, Tangent grades in persent
A, Algebric difference
L, Length of vertical curve
VPC, Vertical point of curvature
VPI, Vertical point of Intersection
VPT, Vertical point of Tangency

Figure 6-23: Sag vertical curves

The minimum length of curve to be introduced with respect to sight distance is given by the following relationship:

$$L = KG$$

Where

- L = minimum length of curve [m]
- K = constant
- G = algebraic difference in gradients [%]

The constant K, which is defined as the ratio of L:G. It is the distance over which a one percent change in gradient takes place. It is therefore useful in determining the horizontal distance from the vertical point of curve (VPC) to the apex of crest curve with opposite grades or the lowest point of sag curve with opposite grades. These points (apex of crest curve and lowest point of sag curve) where the slope is zero are situated at a distance from the VPC equal to K times the approach gradient.

6.3.1 Crest Vertical Curves

The minimum lengths of crest curves have been designed to provide sufficient sight distance during daylight conditions. Longer lengths would be needed to meet the same visibility requirements at night on unlit roads. Even on a level road, low meeting beam headlight illumination may not even show up small objects at the design stopping sight distances. However, it is considered that these longer lengths of curve are not justified as high objects and vehicle tail lights will be illuminated at the required stopping sight distances on crest curves. Vehicles will be identified by the approaching illumination and drivers should be more alert at night and/or be traveling at reduced speed.

The greater sight distances required to provide safe overtaking opportunities are not easily provided on crest curves. If full overtaking sight distance cannot be obtained, the design should aim to reduce the length of crest curves to provide the minimum stopping sight distance, thus increasing overtaking opportunities on the gradients on either side of the curve.

Two conditions exist when considering length of vertical curves. The first is where sight distance is less than the length of the vertical curve, and the second is where sight distance extends beyond the vertical curve.

The basic formulas for length of parabolic vertical crest curve in terms of algebraic difference in grade when stopping sight distance is taken as a control are:

When S is less than L,
$$L = \frac{GS^2}{100\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2}$$

When S is greater than L,
$$L = 2S - \frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{G}$$

Where: L= length of vertical curve [m]
S= Stopping sight distance [m]
G= algebraic difference in grades, [%]
h_1= height of eye above roadway surface, 1.07 m.
h_2= height of object above roadway surface, 0.15 m.

When the height of eye and height of object are 1.07 m and 0.15 m respectively, as it is used in the stopping sight distance the above formulae become:

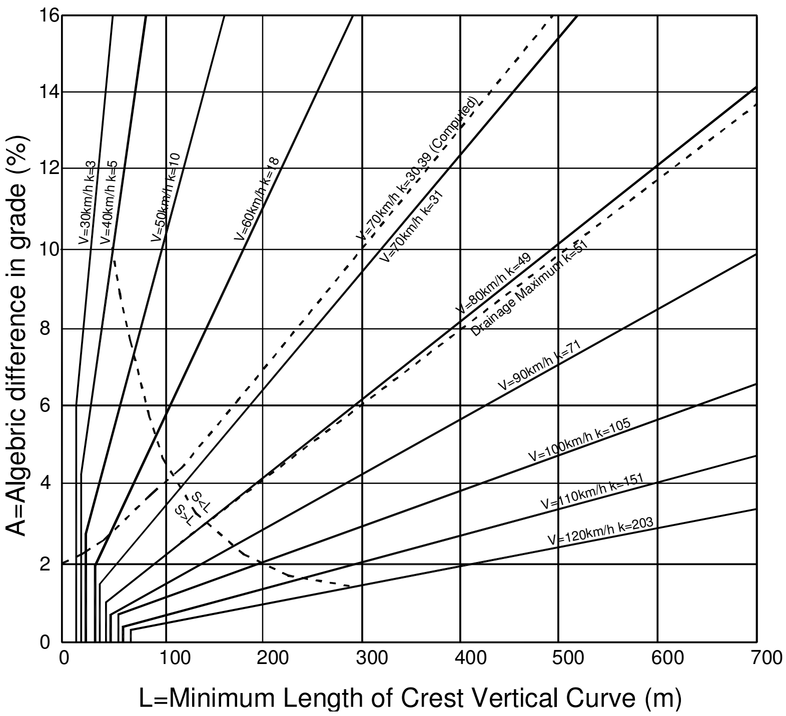
Where S is less tha L,
$$L = \frac{GS^2}{404}$$

Where S is greater than L,
$$L = 2S - \frac{404}{G}$$

Table 6-11: K value for crest vertical curve based on stopping sight distance as design control

Design Speed [km/hr]	Assumed Speed for Condition [km/hr]	Coefficient of friction f	Stopping sight distance for Design [m]	Rate of vertical curvature, K(length [m] per % of G)	
				Computed	Rounded for Design
30	30-30	0.40	29.7-29.7	2.18-2.18	3-3
40	40-40	0.38	44.4-44.4	4.88-4.88	5-5
50	47-50	0.35	57.5-62.8	8.18-9.76	9-10
60	55-60	0.33	74.3-84.6	13.66-17.72	14-18
70	63-70	0.31	94.2-110.8	21.96-30.39	22-31
80	70-80	0.30	112.9-139.5	31.55-48.17	32-49
90	77-90	0.30	131.3-168.8	42.67-70.53	43-71
100	85-100	0.29	157.0-205.2	61.01-104.23	62-105
110	91-110	0.28	179.6-246.5	79.84-150.40	80-151
120	98-120	0.28	203.1-285.8	102.10-202.18	103-203

Table 6-11 is K value for crest vertical curve calculated based on stopping sight distance as design control and Table 6-12 gives K value for crest vertical curve which



has been calculated taking passing sight distance as design control. The resulting length of crest vertical curves are given in Figure 6-24 & 6-25 respectively.

Figure 6-24: Minimum length of crest vertical curve (upper range), stopping sight distance as a design control

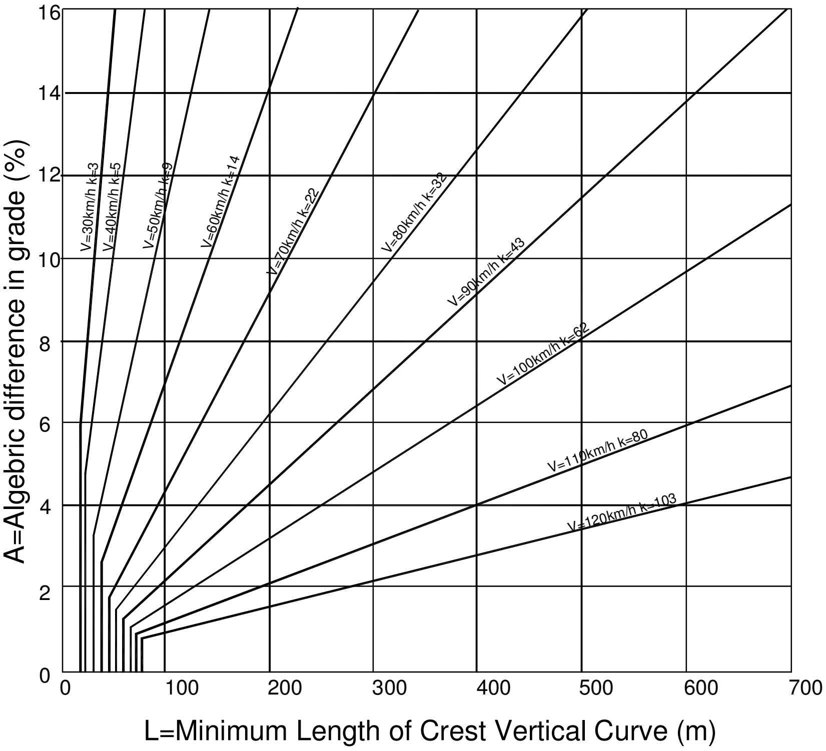


Figure 6-25: Minimum length of crest vertical curve (lower range), stopping sight distance as design control

The basic formulas for length of parabolic vertical crest curve in terms of algebraic difference in grade when passing sight distance is taken as a control and height of eye 1.07 m., and height of object 1.3 m., are:

When S is less than L,

$$L = \frac{GS^2}{946}$$

Where S is passing sight distance [m]
When S is greater than L,

$$L = 2S - \frac{946}{G}$$

Where S is passing sight distance [m]

For minimum passing sight distances, the required lengths of crest vertical curves are substantially longer than those for stopping sight distances.

The need to see the road surface itself is applicable in particular circumstances such as a vertical curve on the approach to a ford or drift where a driver may have to stop because of the presence of surface water.

Table 6-12: K value for crest vertical curve based on stopping sight distance as design control

Design Speed [km/hr]	Minimum Passing sight distance for Design [m]	Rate of vertical curve, K, Rounded for Design (length (m) per % of G	
		Computed	Rounded for Design
30	217	49.78	50
40	285	85.86	86
50	345	125.82	126
60	407	175.10	176
70	482	245.59	246
80	541	309.39	310
90	605	386.92	387
100	670	474.52	475
110	728	560.24	561
120	792	663.07	664

6.3.2 Sag Vertical Curves

Sight distances have been based on the characteristics of car drivers as, although braking distances are greater with trucks, they will usually be traveling more slowly and the eye height of truck drivers is about 1.0m higher. Requirements are related to rates of deceleration available with an emergency stop. Skid resistance values are dependent on tire, road surface conditions and speed, and vary substantially.

It is necessary in the design that adequate sight distance will be available on sag curves in daylight. Lengths of sag vertical curves are to be based on at least four different criteria.

These are:

a) head light sight distance;
b) rider comfort;
c) drainage control; and,
d) a rule-of-thumb for general appearance.

Among all of these it is recommended to use the head light sight distance criterion for determination of minimum length of sag curve from the point of view that it is more logical for general use and especially in connection to safety on roads. Furthermore, for overall safety a sag vertical curve should be long enough so that the light beam distance is nearly the same as the stopping sight distance. Generally headlight height to be used is 0.60 m above the road surface and 1-degree (10) upward divergence of the light beam. Thus it is pertinent to use stopping sight distance for different design speed and S as the distance between the vehicle and the point where the 1-degree angle of light ray intersects the surface of the road. In other words S is the distance of light beam.

When S is less than L: $L = GS^2/(200(h_1 + S\tan\theta))$

When S is greater than L: $L = 2S - 200(h_1 + S\tan\theta)/G$

Where:

L length of sag vertical curve, [m]
S light beam distance = stopping sight distance, [m]
G algebraic difference in grades, [%]
h1 headlight height (0.6 m)
θ angle of divergence of headlight beam (10)

By inserting $\theta = 1^0$ and $h_1 = 0.6$ m the above relationship will be as follows:

When S is less than L:	$L=GS^2/(120+3.5S)$

When S is greater than L:	$L= 2S-((120+3.5S)/G)$

For overall safety on roads, a sag vertical curve should be long enough so that the light beam distance is nearly the same as the stopping sight distance. Accordingly, it is pertinent to use stopping sight distances for different design speeds as the S value in the above relationships. The resulting computed and rounded values of K selected as a design control based on the range of stopping sight distances for various design speeds are given in Table 6-13. The resulting lengths of vertical curves for the upper and lower range of stopping sight distances for different design speeds are given in Figures 6-26 and 6-27 respectively.

Table 6-13: K value for sag vertical curve based on stopping sight distance as design control

Design Speed [km/hr]	Assumed Speed for Condition [km/hr]	Coefficient of friction f	Stopping sight distance for Design [m]	Rate of vertical curvature, K(length [m] per % of G)	
				Computed	Rounded for Design
30	30-30	0.40	29.7-29.7	3.94-3.94	4-4
40	40-40	0.38	44.4-44.4	7.16-7.16	8-8
50	47-50	0.35	57.5-62.8	10.29-11.61	11-12
60	55-60	0.33	74.3-84.6	14.53-17.20	15-18
70	63-70	0.31	94.2-110.8	19.73-24.18	20-25
80	70-80	0.30	112.9-139.5	24.74-31.99	25-32
90	77-90	0.30	131.3-168.8	29.75-40.09	30-41
100	85-100	0.29	157.0-205.2	36.82-50.24	37-51
110	91-110	0.28	179.6-246.5	43.09-61.83	44-62
120	98-120	0.28	203.1-285.8	49.65-72.91	50-73

Riding on sag vertical curve is comfortable when the centrifugal acceleration does not exceed 0.3 m/sec^2. The general expression for such a criterion is:

$$L=GV^2/395$$

Where L=length of vertical curve; G=algebraic difference in grades [%]; and V is the design speed [km/hr].

The length of vertical curve required to satisfy comfort factor at various design speed is only about 50% of that required to satisfy the head light distance requirement for the normal range of design conditions.

Drainage affects design of some sag vertical curves where curbed sections are used. The criterion is providing a minimum grade of 0.30 percent within the 15m of the level point. This criterion is also applicable for crest vertical curves as well.

For general and good appearance, as a rule-of-thumb for length of vertical curve L=30G, and K=30 could be used.

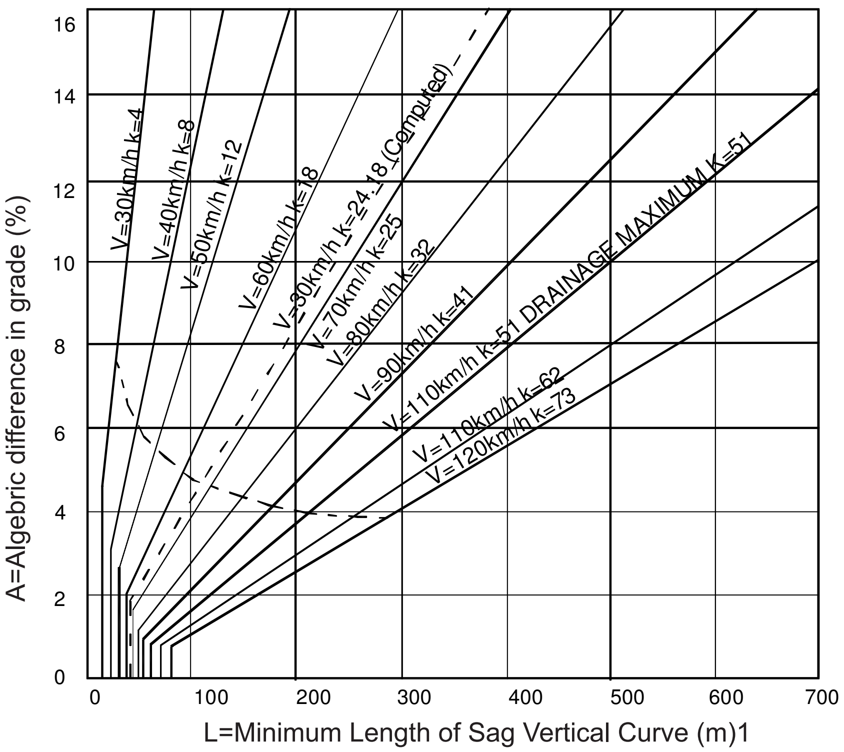


Figure 6-26: Minimum length of Sag Vertical Curve (upper range) stages sag distance design control.

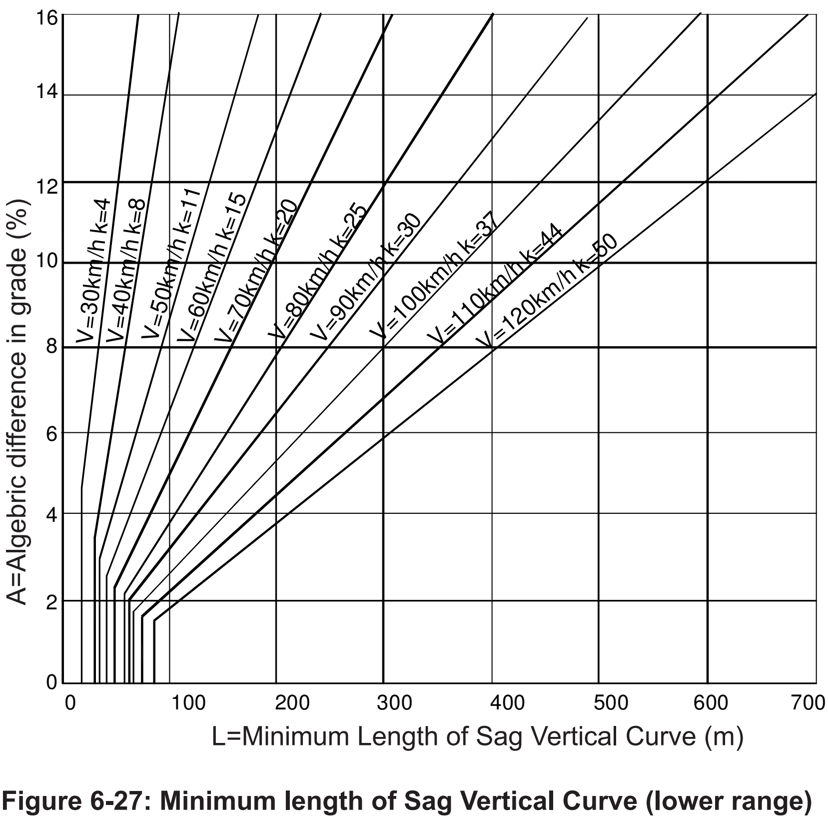


Figure 6-27: Minimum length of Sag Vertical Curve (lower range)

6.4 Phasing Of Horizontal And Vertical Alignment

Six basic forms for the space curve can be defined. There are a number of long-established rules on how to combine these elements in different terrain situations. Some of these are summarized here.

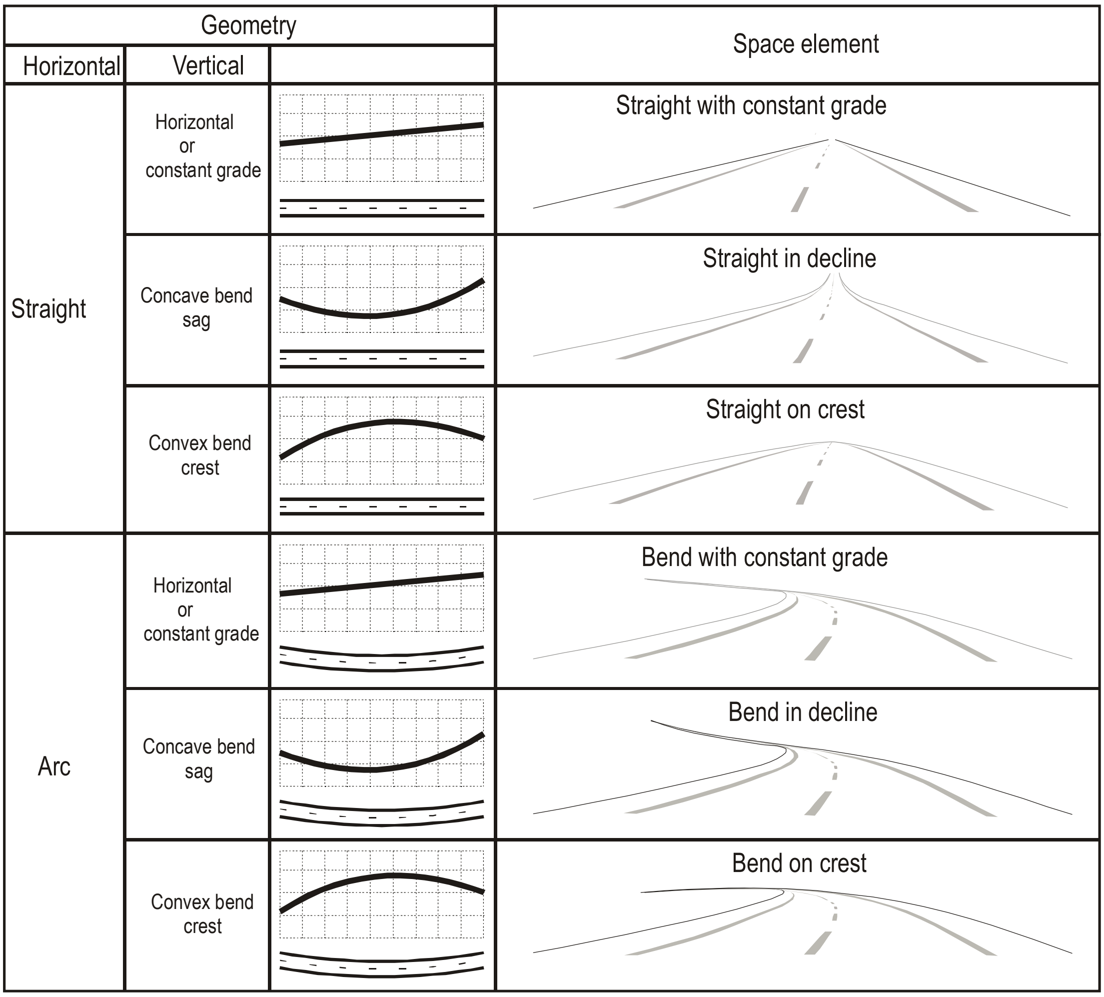


Figure 6-28: Basic space elements

A straight, especially not a long one, should not be preceded by a small arc, see section 6.2.3. A transition curve is recommended in this case. It could be a clothoid or a bigger arc with a radius at least twice as big ($R_2/R_1 \leq 2.0$).

The clothoid gives a form that facilitates the driver to choose his lateral position in the lane in the curve reducing short-cutting. It also gives a smoother alignment. The narrower the road the more important is the use of transition curves to create lane designs used by the driver and to create a harmonic road design adapted to the surroundings.

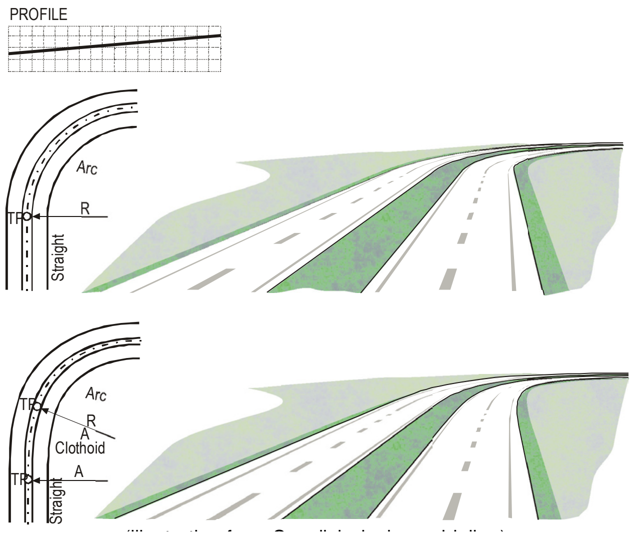


Figure 6-29: Comparison of eye impression at straight - arc and straight - clothoid - arc

It is important that deviations between the elements of the alignment are large enough. The minor change in direction the larger is the arc needed to avoid kinks in the alignment.

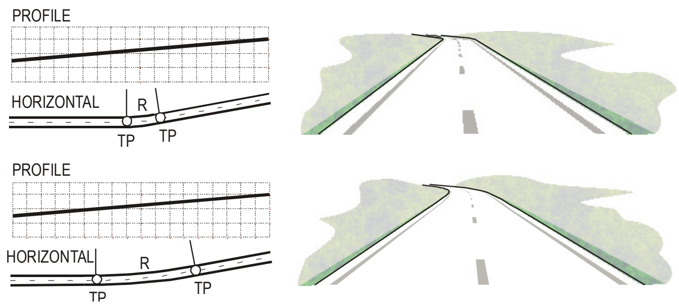


Figure 6-30: Road with and without optical kink

It is also important that vertical elements arc lengths are large enough to avoid kinks, especially for concave vertical curves (sags). The illustration in Figure 6-31 shows a comparison of transitions with from – 2 % to + 3 % with a 210 m long vertical radius with R 4000 m and a 900 m long vertical curve with 18000 m radius

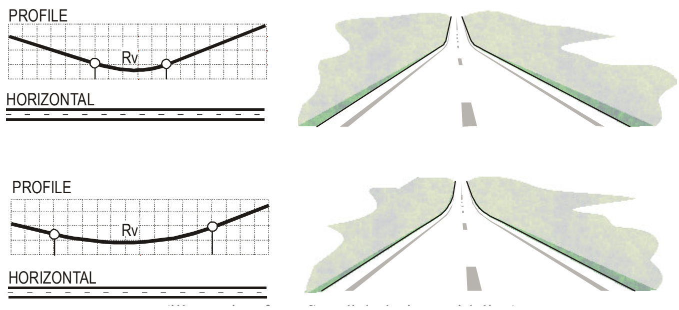


Figure 6-31: Comparison of alternative transitions from - 2% to + 3%

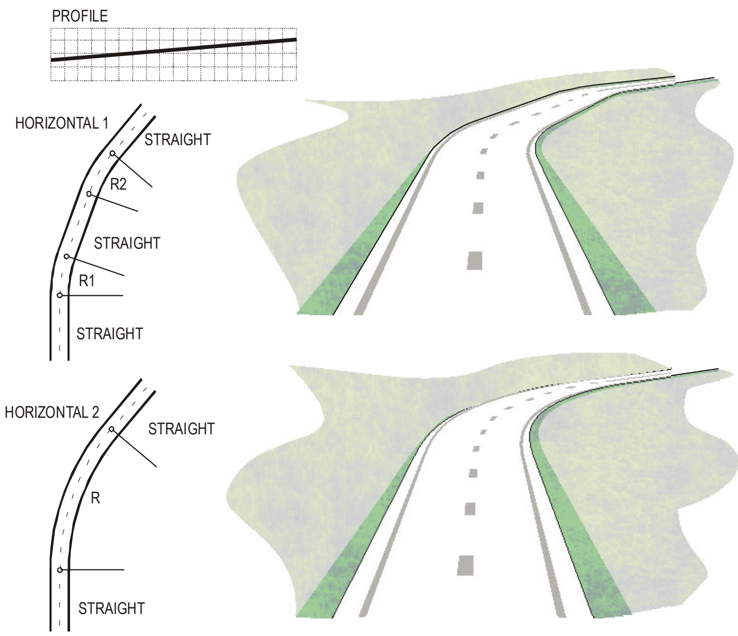


Figure 6-32: A short straight between two curves should be avoided

The combination of horizontal curves and vertical grades is critical, see Figure 6-33. A horizontal curve could terminate anywhere on the vertical curve, case A in Figure 6-33, and should preferably start in the upper part, case D. A later start is acceptable, case E, and could be improved with a transition curve case F. The start tangent point of a horizontal curve should be located on the upper part of the first sag curve in the grade, not on the lower part.

A start of the horizontal curve before the sag, case B, could be interpreted as if the sag is a horizontal contra curve in the large horizontal curve. A start of the sag in the beginning of the horizontal curve creates a kink. A sag within a horizontal curve should have a large arc.

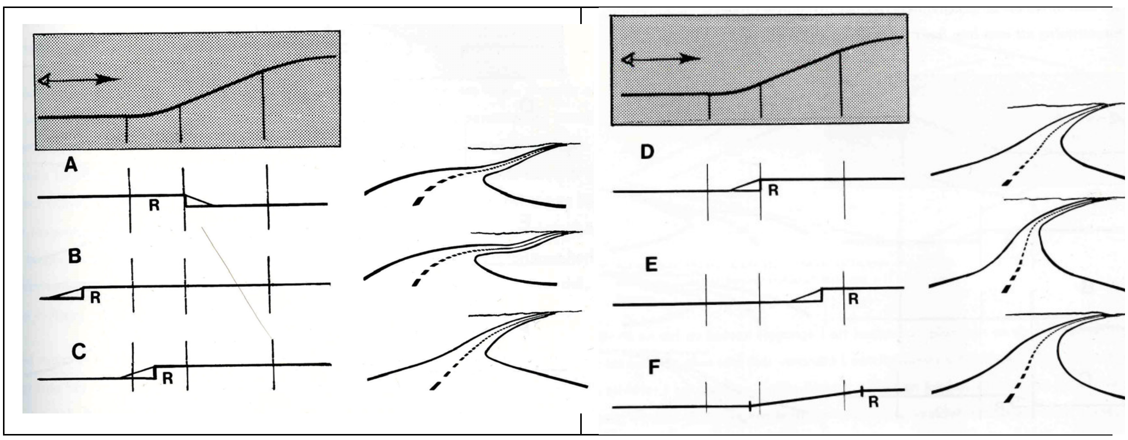


Figure 6-33: Illustration of combination of horizontal curves and vertical grades

Another critical design is the S-curve. The ratio between the horizontal radius Rh and the vertical radius Rv should be as small as possible, at least with Rh / Rv < 1/5 to 1/10. Moreover, the changeover should be vertically and horizontally reasonably coordinated. The vertical curve should be small or large in order to maximize overtaking sight distances.

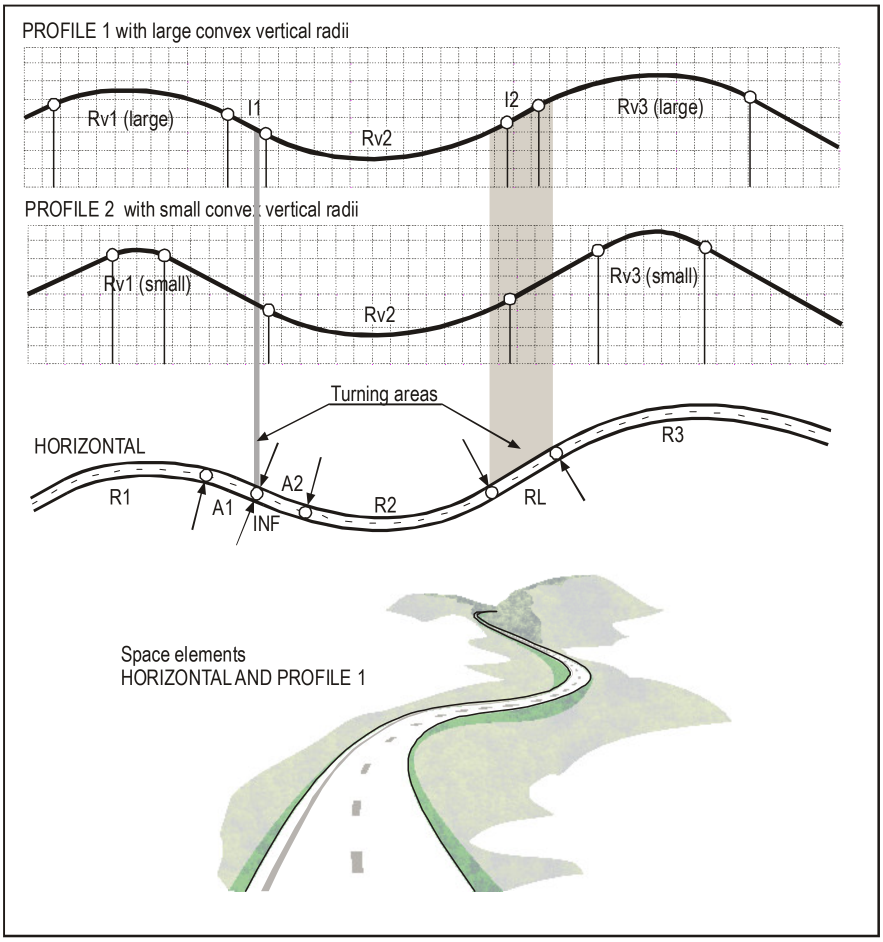


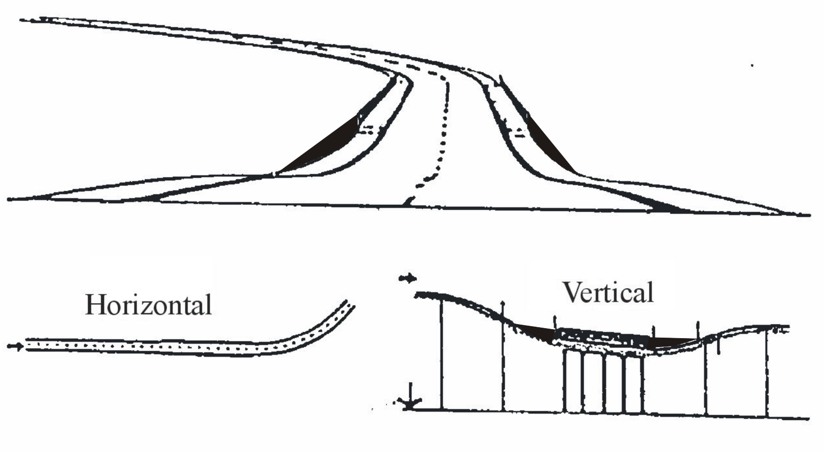
Figure 6-34: Coordination of vertical and horizontal elements in turning area

Some dangerous alignment combinations to avoid are:

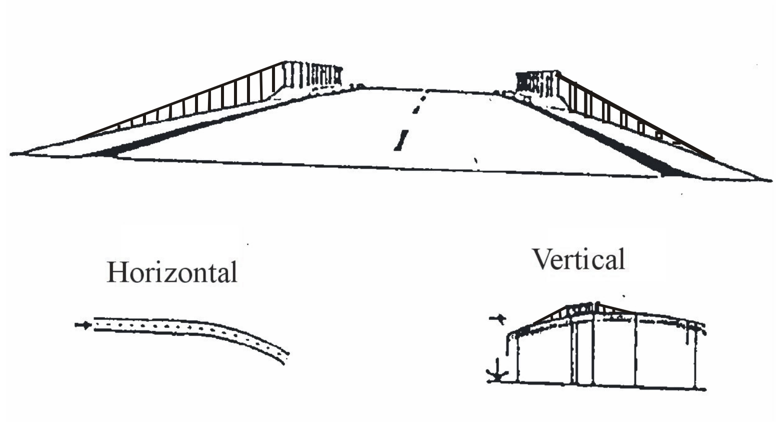
- sharp horizontal curves after sharp crests – the horizontal curve should start before the crest to give visual guidance; and,
- consecutive horizontal curves with diminishing radii.

Bridges are important to coordinate in the chosen harmonic alignment. Straights should, if possible, be avoided, see the two upper parts of Figure 6-35 below. Bridges in curves improve the opportunity for drivers and passengers to experience and enjoy

Bridge not adapted to the road spline – ”board effect”



Bridge on straight and crest does not create any bridge experience



Bridge in horizontal curve improves the experience of the bridge

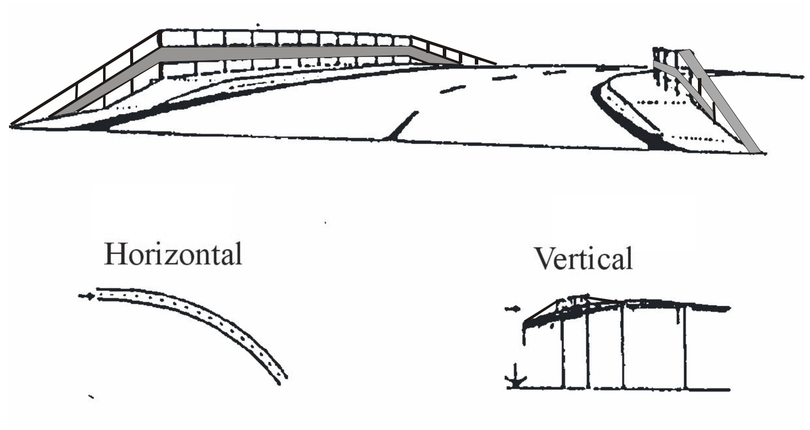


Figure 6-35: Example of adaptation of bridge alignment

Note: *the bridge parapet shown is indicative only -, see section 9.4 for advice on the design of bridge parapets*

Larger bridges, crossing rivers or valleys, should preferably be located and aligned to give the driver a view of the bridge and not only the surface - see Figure 6-36.

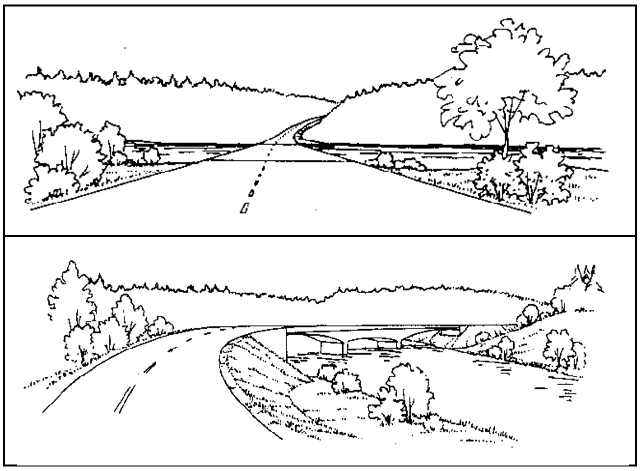


Figure 6-36: Bridge alignment without and with a bridge view

6.4.1 Alignment Defects due to Mis-phasing

Phasing of the vertical and horizontal curves 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 the 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 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.

When the horizontal and vertical curves are adequately separated or when they are coincident, no phasing problem occurs and no corrective action is 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.

6.4.2 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 such that vertical and horizontal curves begin at a common station and end at a common station. In some cases, depending on the curvature, it is sufficient if only one end of each of the curves is at a common station.

Cases of mis-phasing fall into several types. These are described below together with the necessary corrective action for each type.

6.4.2.1 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 Figures 6-38b and c.

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.

6.4.2.2 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 6-37d.

Corrective action consists of increasing the separation between the curves, or making the curves concurrent, as in Figure 6-37a.

6.4.2.3 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 6-37e. The corrective action is to make both ends of the curves coincident as in Figure 6-37a, or to separate them.

6.4.2.4 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.

6.4.2.5 Other Mis-Phasing

Other types of mis-phasing are also indicated in Figure 6-37:

A sag curve occurs between two horizontal curves in the same direction in Figure 6-37g. This illustrates the need to avoid broken back curves in design (see Section 6.2.3 (i)).

A double sag curve occurs at one horizontal curve in Figure 6-37h. This illustrates the effect in this case of a broken back vertical alignment on design.

Figure 6-37i 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.

6.4.3 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.

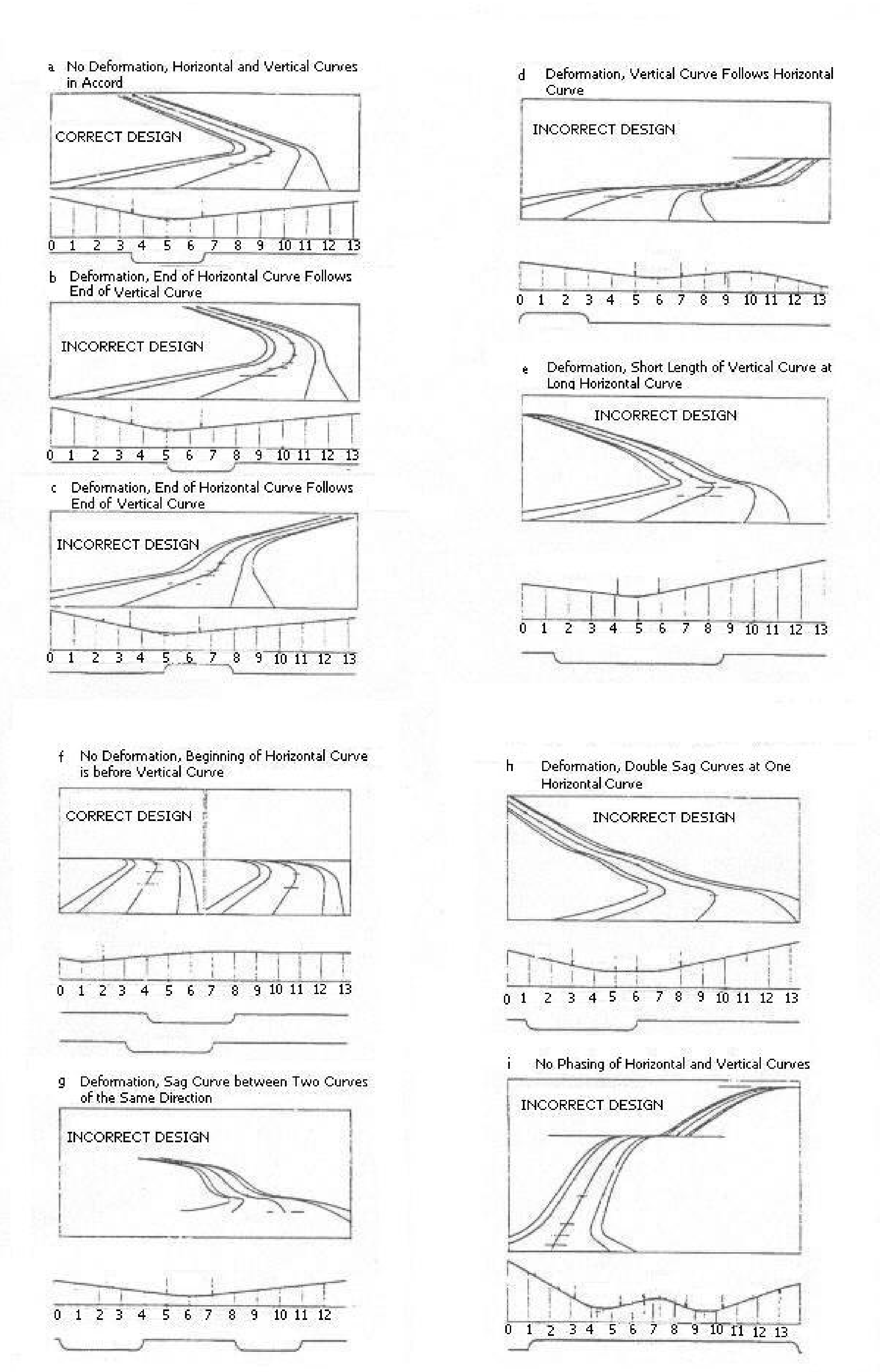


Figure 6-37: Phasing of Horizontal and Vertical Curves

6.5 Gradients

6.5.1 Maximum Gradients

Vehicle operations on gradients are complex and depend on a number of factors: severity and length of gradient; level and composition of traffic; and the number of overtaking opportunities on the gradient and in its vicinity.

For very low levels of traffic flow represented by only a few four-wheel drive vehicles other references advocate a maximum traversable gradient of up to 18 percent. Small commercial vehicles can usually negotiate an 18 per cent gradient, whilst two-wheel drive trucks can successfully manage gradients of 15-16 per cent except when heavily laden.

However, in a country where the vehicle fleet is composed of a high percentage of vehicles that are underpowered and poorly maintained roads with high grades are avoided and underutilized by traffic due to an inability to ascend high grades. In such cases the choice is left to make a limit on gradients based on the design vehicle of existing fleet, although this translates into an added cost to develop the road infrastructure.

Maximum vertical gradient is therefore and extremely important criterion that greatly affects both the serviceability performance and cost of the road. These performance considerations

Table 6-14: Maximum Gradients (Desirable)

Topography	Maximum gradient [%] for design speed [km/hr]								
	40	50	60	70	80	90	100	110	120
Flat	-	-	5	4	4	3.5	3	3	3
Rolling	-	7	6	5.5	5	4.5	4.5	-	-
Mountainous	10	9	8	7	-	-	-	-	-

Table 6-15: Absolute maximum Gradients

Topography	Maximum gradient [%] for design speed [km/hr]								
	40	50	60	70	80	90	100	110	120
Flat	-	-	-	6	6	5.5	5	4	4
Rolling	-	9	8	7.5	7	6.5	-	-	-
Mountainous	12	11	10	9	-	-	-	-	-

When gradients of 10 percent or greater are reached, consideration should be given to the possibility of paving these steep sections to enable sufficient traction to be achieved, as well as for pavement maintenance reasons. However, this is clearly not practical for all classes of roads, particularly at lower traffic volumes. There may be cases where paving greater than 10 percent will be economical. This depends on the standard and the service of the road to be provided.

As traffic flows increase, the economic dis-benefits of more severe gradients, measured as increased vehicle operating and travel time costs, are more likely to result in economic justification for reducing the severity and/or length of a gradient. On the higher design classes of road, the lower maximum recommended gradients reflect these economics. However, a separate economic assessment of alternatives to long or severe gradients should be undertaken where possible or necessary.

Standards for desirable maximum gradients were set to assure user comfort and to avoid severe reductions in the design speed. If the occasional terrain anomaly is encountered that requires excessive earthworks to reduce the vertical alignment to the desirable standard an absolute maximum gradient can be used. Employment of a gradient in excess of the desirable maximum can only be authorized through the employment of a Departure from Standard (see Section 1).

6.5.2 Minimum Gradients

The minimum gradient in cuttings in order to avoid standing water in the road side ditches is 0.3 - 0.5 percent. However, flat and level gradients on uncurbed paved roads are acceptable when the cross slope and carriageway elevation above the surrounding ground is adequate to drain the surface laterally. With curbed roads or streets, adequate longitudinal gradients should be provided to facilitate surface drainage.

6.5.3 Gradients Through Villages

In many instances the natural grade level is flat through villages. However the natural grade in the villages and towns should be used as much as possible for the road traversing them in order not to affect existing properties. The adjacent roadside ditches in such circumstances can readily become clogged and ineffective. It is also the case that they are deliberately blocked to provide access to adjacent property or to channel flow for agricultural use. These practices lead to saturation of the sub-grade and hence pavement failure, and should be avoided.

6.5.4 Critical Length of Gradient

Maximum gradient by itself is not a complete control and the length of a particular grade must be also checked. Critical length may be defined at the maximum length of a specific upgrade on which a loaded truck can operate without an unreasonable reduction in speed. A reduction 15 km/hr. or more could be considered to be as “un reasonable”.

Critical length of gradient is considered to be the maximum length of a designated upgrade upon which a loaded truck can operate without unreasonable reduction in speed. Critical length of gradient is, to some extent, dependent on the gradient of the approach; a downhill approach will allow vehicles to gain momentum and increase the critical length. In general, the critical length of gradient should decrease, as gradient increases. This is shown in Table 6-16 below. Where it is necessary to exceed the critical length of gradient on heavily trafficked roads, it is desirable to provide either with safe passing distances on the rise, or a climbing lane for heavy vehicles.

Table 6-16: Maximum desirable length of gradient

Gradient [%]	Maximum desirable length of gradient [m]
12	300
11	350
10	400
9	450

6.6 Climbing And Overtaking Lanes

Where longitudinal gradients are too long and /or too steep that they result in significant increase in the speed differences between cars and heavy vehicles and both traffic safety and road capacity is may be adversely affected. A climbing lane is an effective means of reducing the impact of a steep gradient. A climbing lane is an additional outside lane added to a single or dual carriageway in order to improve overtaking opportunities, capacity and safety (see Figure 6-38). It is inserted into the carriage way by means of entry and exit tapers to the left of the continuous lane so that slow moving vehicles have to merge into the faster traffic at the termination point. Climbing lane has the effect of reducing congestion in the through lanes by removing slower moving vehicles from the traffic stream. It also enhances road safety by reducing the speed differential in the through lane. An overtaking lane serves the same objectives without steep gradient. Climbing lanes are obviously more cost-effective than overtaking lanes as the probability of an overtaking need normally is higher and the overtaking length is shorter.

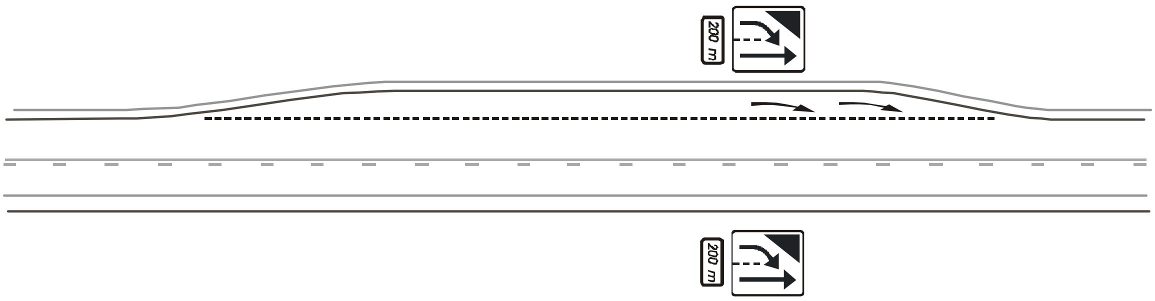


Figure 6-38: Climbing Lane

The requirements for climbing lanes are therefore based on road standard, speed and traffic volume.

The 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 will increase with increase 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 following guidelines are to be used to determine whether the effect of such gradients will be sufficiently severe to warrant the design and provision of climbing lanes:-

a) Climbing lanes will not be required on (i) roads with AADT < 2000 pcu in design year 10, and (ii) on all design class III , A, B and C roads even if the AADT exceeds 2000 pcu in design year 10.

b) Where passing opportunities are limited on the gradients, then climbing lanes must be considered on design class I, & II roads with traffic flows in design year 10 is in the range of 2000 pcu < AADT < 6000 pcu.

c) Climbing lanes will normally required on roads with AADT ≥ 6000 pcu in design year 10.

Climbing lanes should be considered if the design truck speed decreases more than 20 km/h under the truck speed limit, normally 80 km/h in rural conditions. This gives the threshold criteria shown in Table 6-17: on combinations of grade and length which fulfill these criteria. (example: an average grade of 2% requires a length of 1500 m before the design vehicle has dropped its speed 20 km/h). The speed profile method below could be used to assess truck speed profiles more thoroughly.

Table 6-17: Maximum Gradients (Desirable)

Average grade (%)	Minimum length (m)
2	1500
3	500
4	300

Long steep down grades are very dangerous due to risk of overheating and brake failures. It is recommended that down grades longer than 1000 m with average grades over 5% be reviewed for the need of descending lanes.

The climbing (and descending) lane is sometimes not effectively used, especially when traffic flows are heavy, because the drivers of slower vehicle fear that they will not be allowed to merge with the faster vehicles when the climbing lane ends. The position of the lane-drop must therefore allow the slower vehicle to gain enough speed to merge. As the climbing lane will be used largely by trucks and buses, with design truck speed less than 60 km/hr, it is desirable that it must be as wide as the through lane but in no case less than 3.0 m wide with entry and exit tapers according to the table below. At lane-drops the sight distance should exceed that required for “no overtaking centerline markings. The tapers shall not be considered as part of the climbing lanes. They must be clearly marked and, where possible, should end on level or downhill sections where speed differences between different classes of vehicles are lowest to allow safe and efficient merging maneuvers. Furthermore, it must be constructed in such a way that it can be immediately recognized as an extra lane in one direction. The center line of the normal two lane road should be clearly marked in the required manner as for any two-lane road, including yellow barrier lines for no-passing zone. Adequate signs at the beginning of the upgrade such as “Slower Traffic Keep Left” or “Trucks Use Left Lane” may be used to direct slow-moving vehicles into the climbing lane.

Table 6-18: Absolute Maximum Gradients

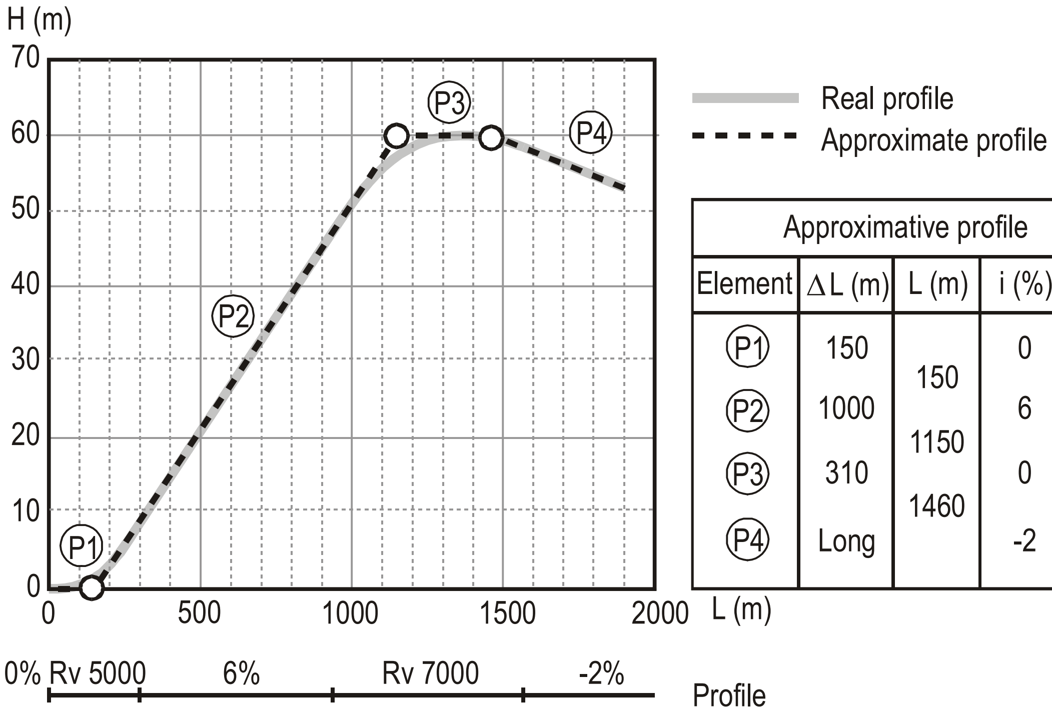
Speed limit (km/h)	Entry taper (m)	Exit taper (m)
80 100	150 200	200 300

There is a problem in the application of a climbing lane in mountainous and escarpment terrain. Here due to heavy earthworks, the carriageway and shoulder widths may have been reduced, and thus a climbing lane will increase the roadway width. Consideration must be given to a balance between the benefits to traffic and the initial construction cost. In sections requiring heavy side cut, the provision of climbing lanes may be unreasonably high in relation to the benefits. Reduced level of service over such sections could be sometimes taken as an alternative.

Overtaking lanes can be used to improve overtaking sight distances on flatter sections. The length of an overtaking lane should be in the range 1000 m to 2000 m.

Speed profile

The speed profile graph can be used to assess truck speed behavior on combined vertical alignments as shown in the following example.



Approximative profile			
Element	ΔL (m)	L (m)	i (%)
P1	150	150	0
P2	1000	1150	6
P3	310	1460	0
P4	Long		-2

Figure 6-39: Example profile

Calculate the speed profile for a heavy truck with power-mass ratio 5 and a start speed of 80 km/h on the following profile with a 6 % uphill preceded by a straight and a 5000 m sag and proceeded by a 7000 m crest and 2 % downhill. Approximate the profile to straights and grades by extending the vertical curve tangents as illustrated in the figure above.

The truck speed will be affected from the start point of section P2. This element, 1000 m with grade 6 %, will give the speed performance according to the segment AB on the + 6 %-curve starting at 80 km/h with a length of 1000 m giving an end speed of some 28 km/h (crawling speed for – 6 %).

The truck will start accelerating on the third element P3, 310 m and flat, with speed behaviour given by the segment CD on the 0 %-curve starting from some 28 m/h reaching some 56 km/h.

The truck will obtain its start speed after some 400 m on the final downhill with speed behavior according to segment EF on the -2 % curve.

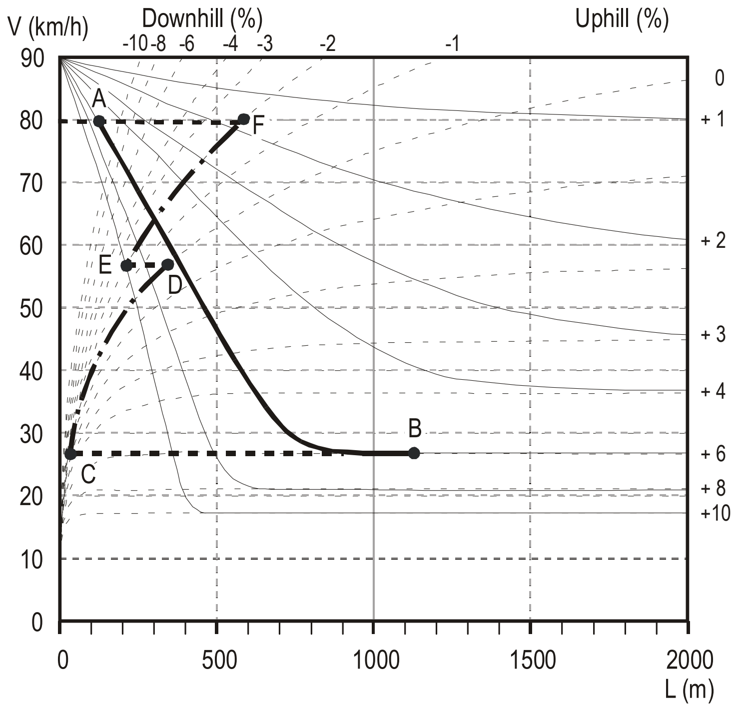


Figure 6-40: Example to assess truck speed profile

The resulting speed profile is illustrated below with proposed location of the crawling lane, entry and exit tapers.

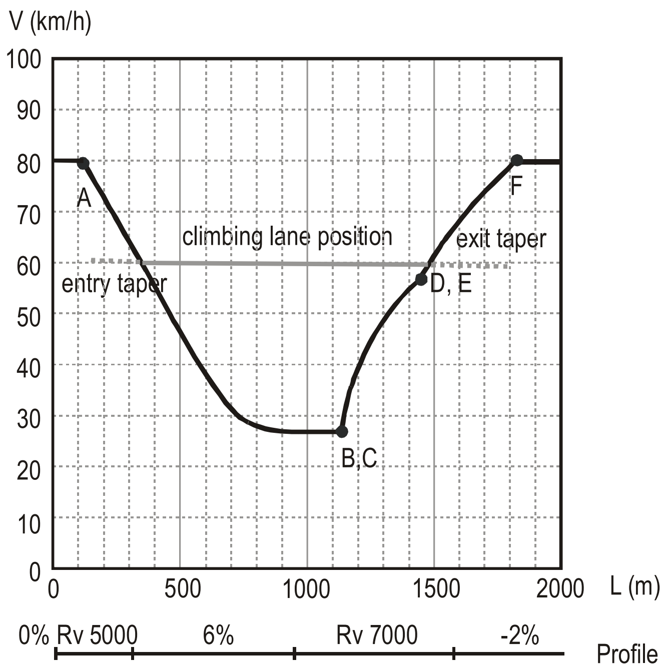


Figure 6-41: Resulting heavy truck speed profile

6.7 Sight Distances

6.7.1 General

The ability to see ahead is of the utmost importance in the safe and efficient operation of vehicles or roads. Sight distances of sufficient length should be provided so that drivers can control the operation of their vehicles to avoid striking an unexpected object on the traveled way.

6.7.2 Stopping Sight Distance

Stopping sight distance is the minimum sight distance available on a road that enables a vehicle traveling at or near the likely top design speed to stop before reaching an object in its path. To ensure safety, sufficient sight distance must always be available to enable a vehicle traveling at the design speed to stop before reaching an object in its path.

Stopping sight distance is the sum of two distances:

a) the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied also called as brake reaction distance (0.278 x prt x V), where prt is perception reaction time in seconds.

b) the distance traveled by the vehicle after the brakes are applied to stop or the distance required to stop the vehicle from the instant brake application begins. It is also called as braking distance ($V^2/254(f+G)$)

Therefore stopping sight distance is calculated using the following relationship:

$$SSD = 0.278 \times prt \times V + \left(\frac{V^2}{254(f+G)} \right)$$

Where:

SSD = stopping sight distance [m]
prt = perception reaction time [sec]
V = Vehicle speed [km/hr]
f = coefficient of longitudinal friction [ ]
G = percent grade, + for up grade and – for down grade [%/100]

Although trucks as a whole and especially the larger and heavier units require longer stopping distance from a given speed than do passenger cars, the minimum stopping sight distance is always derived from passenger cars. This is because of the fact that, truck drivers are always able to see substantially farther due to their higher position in the vehicle and in nearly all cases trucks travel slower than passenger cars.

In computing stopping sight distance, perception reaction time of 2.5 sec, eye height of 1.07 m above the road surface and object height is taken as 0.15 m above the road surface are taken.

The table below shows calculated stopping sight distances on level ground for wet pavement condition and assumed speed conditions.

Table 6.19: Stopping sight distance on level ground for wet pavement condition

Design Speed [km/hr]	Assumed Speed for Condition [km/hr]	Brake Reaction		Coefficient of friction for wet pavement condition (f)	Breaking distance on level [m]	Stopping sight distance for design [m]
		Time [sec]	Distance [m]			
30	30-30	2.5	20.8-20.8	0.40	8.9-8.9	29.7-29.7
40	40-40	2.5	27.8-27.8	0.38	16.6-16.6	44.4-44.4
50	47-50	2.5	32.6-34.7	0.35	24.8-28.1	57.5-62.8
60	55-60	2.5	38.2-41.7	0.33	36.1-42.9	74.3-84.6
70	63-70	2.5	43.8-48.6	0.31	50.4-62.2	94.2-110.8
80	70-80	2.5	48.6-55.6	0.30	64.3-84.0	112.9-139.5
90	77-90	2.5	53.5-62.5	0.30	77.8-106.3	131.3-168.8
100	85-100	2.5	59.0-69.4	0.29	98.1-135.8	157.0-205.2
110	91-110	2.5	63.2-76.4	0.28	116.4-170.1	179.6-246.5
120	98-120	2.5	68.1-83.3	0.28	135.0-202.5	203.1-285.8

Effect of grade on stopping sight distances for 3%, 5% and 7% upgrades and down grades are shown on the table below.

Table 6.20: Effect of grade on stopping sight distance

Design Speed [km/hr]	Stopping sight distance for downgrades [m]			Assumed speed for condition [km/hr]	Stopping sight distance for upgrades [m]		
	3%	5%	7%		3%	5%	7%
30	30.4	30.7	31.6	30	29.1	28.7	28.4
40	45.8	46.3	48.1	40	43.2	42.4	41.8
50	65.5	66.5	69.9	47	55.6	54.4	53.4
60	88.9	90.6	96.2	55	71.6	69.6	68.0
70	117.5	120.1	129.0	63	89.7	87.2	84.9
80	148.9	152.5	165.2	70	107.1	103.8	100.8
90	180.7	185.2	201.2	77	124.2	120.2	116.6
100	220.9	227.0	248.5	85	148.0	142.7	138.1
110	267.0	274.9	303.3	91	168.4	162.0	156.4
120	310.2	319.6	353.4	98	190.1	182.7	176.1

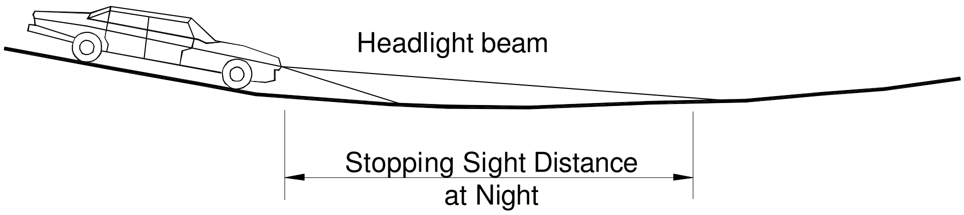


Figure 6-42: Stopping sight distance on sag

6.7.3 Passing Sight distance

Passing sight distance is the minimum sight distance on two-lane road that is required to enable the driver of one vehicle safely pass another vehicle without interfering with the speed of an oncoming vehicle traveling at the design speed.

In computing passing sight distances, driver's eye height is taken as 1.07 meters above the road surface and object height for passing sight distance is taken as 1.3 meters above the road surface.

The minimum passing sight distance for a two-lane road is about four times as great as the minimum stopping sight distance at the same speed. The available sight distance needs to be checked separately for both stopping and passing sight distances and for each direction of travel.

Passing or overtaking opportunities without creating dilemma zones could be maximized by providing an auxiliary lane for exclusive use of traffic in one direction. This may take the form of a dual carriage way, a climbing lane in the case of up hill terrain or an overtaking lane in flat terrain. Overtaking lane or passing lane are normally provided in areas where construction costs are low and where there is an absence of passing opportunities or short passing opportunities. As recommended for climbing lanes, the entrance taper to a passing lane could be 100 meters in length and the length of the exit tapper double this to allow adequate time for merging vehicles to find a gap in the through flow. Seeing that both the entrance and the exit tapers signal a change in operating conditions on the road, it is recommended that decision sight distance should be available at entrances and exits.

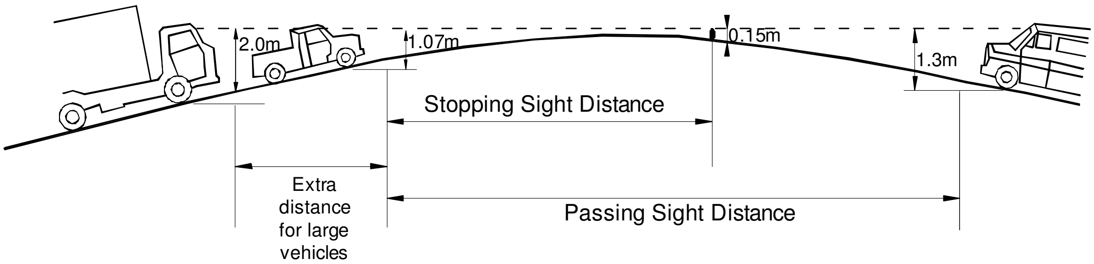


Figure 6-43: Stopping and passing sight distances on crest curve

The minimum sight distance for a two-lane road is determined as the sum of four distances and as can be seen in figure 6-44.

d1=distance traveled during perception reaction time and during initial acceleration to the point of encroachment on the right lane or during the time of initial maneuver.

d2=distance traveled while the passing vehicle occupies the right lane, finishes its overtaking maneuver and return to the left lane.

d3=distance between the passing vehicle at the end of its maneuver and the opposing vehicle.

d4=distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the right lane, or 2/3 of d2 above.

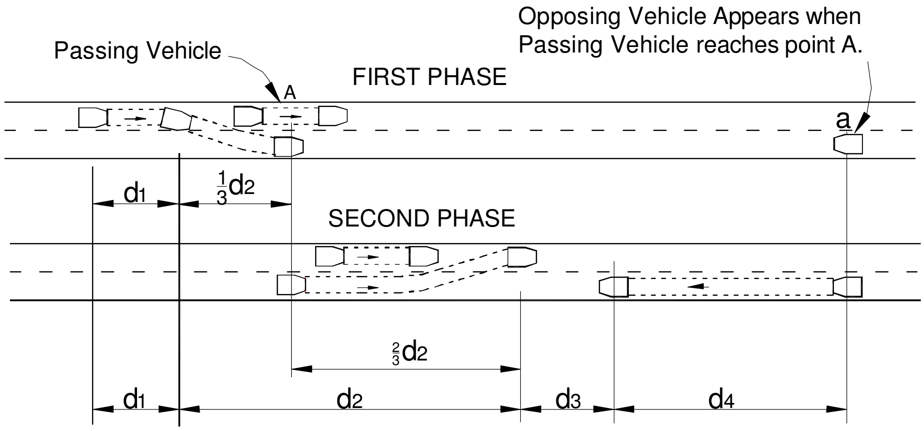


Figure 6-44: Passing or overtaking maneuver on two-lane rural road

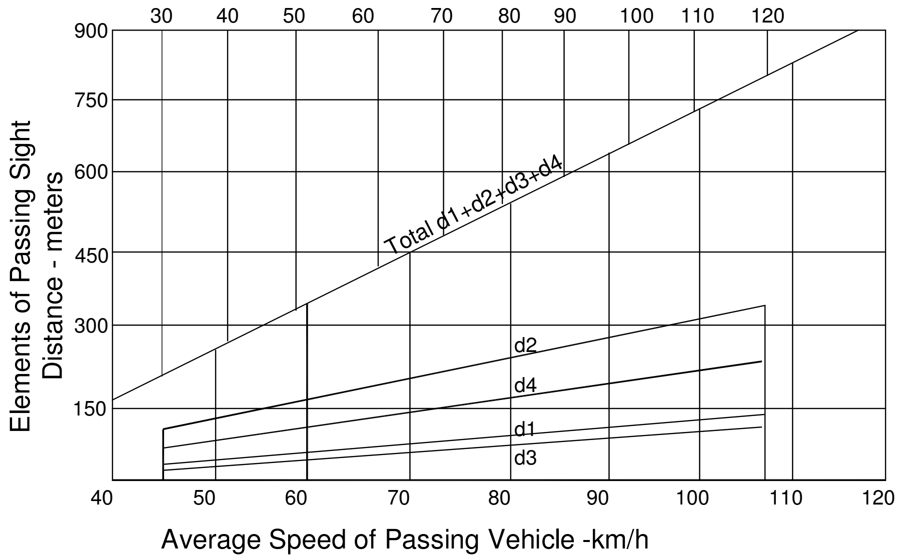


Figure 6-45: Elements of and total passing sight distance on two-lane rural road

The distance traveled during the initial maneuvering period (d1):

$$d_1 = 0.278t_1\left(v - m + \frac{at_1}{2}\right)$$

Where:

t1=time of initial maneuver, [sec]
a=average acceleration [km/h/sec]
v=average speed of passing vehicle [km/h]
m=difference in speed of passed vehicle and passing vehicle [km/hr]

Distance while passing vehicle occupies right lane, finishes its overtaking maneuver and returns to the left lane (d2):

$$d_2=0.278vt_2$$

Where:

t_2=time passing vehicle occupies right lane [sec]
v=average speed of passing vehicle [km/h]

Clearance length (d3): safe clearance distance between the opposing and passing vehicles at the end of the maneuvers is dependent on passing speed.

Table 6.21: Elements of Safe Passing Sight Distance – Two Lane Roads

Speed Group [km/h]	50-65	66-80	81-95	96-110
Average passing speed [km/hr]	56.2	70.0	84.5	99.80
Initial maneuver:				
a = average acceleration[km/h/s][a]	2.25	2.30	2.37	2.41
t_1 = time [s][a]	3.6	4.0	4.3	4.5
d_1 = distance traveled [m]	45	65	90	110
Occupation of right lane, finish maneuver and return to left lane:				
t_2 = time [s][a]	9.3	10.0	10.7	11.3
d_2 = distance traveled [m]	145	195	250	315
Clearance length:				
d_3 = clearance length [m][a]	30	55	75	90
Opposing vehicle:				
d_4 = distance traveled [m]	95	130	165	210
Total distance, d_1+d_2+d_3+d_4 [m]	**315**	**445**	**580**	**725**

Distance traveled by an opposing vehicle (d4):

$$d_3 = \frac{2}{3} d_2$$

The minimum passing sight distance (PSD) is therefore equal to:

$$PSD = d_1 + d_2 + d_3 + d_4$$

Values of minimum passing sight distance (PSD) for design on a two-lane road for various design speeds are given in the table 6.22 below.

Table 6.22: Minimum passing sight distance for design

Design Speed [km/h]	**Assumed speed [km/h]**		
	Passed vehicle [km/h]	**Passing vehicle [km/h]**	**Minimum passing sight distance for design [m]**
30	29	44	217
40	36	51	285
50	44	59	345
60	51	66	407
70	59	74	482
80	65	80	541
90	73	88	605
100	79	94	670
110	85	100	728
120	91	106	792

Effect of Grade on Passing Sight Distance.

The sight distances required to permit vehicles traveling upgrade to pass with safety are greater than those required on level roads because of reduced acceleration of passing vehicle (which increases the time of passing) and the likelihood of opposing vehicle speeding up (which increases the distance traveled by it). If passings are to be performed safely on upgrades, the passing sight distance should be greater than the derived minimum.

Passing is easier for the vehicle traveling downgrade because the overtaking vehicle can accelerate more rapidly than on the level and thus can reduce the time of passing, but the overtaken vehicle can accelerate easily so that a situation akin to a racing context may result.

6.7.4 Decision Sight Distance

Decision sight distance is the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard or its potential threat, select an appropriate speed and path, and initiate and complete the required safety maneuver safely and efficiently. Because decision sight distance gives drivers additional margin for error and affords them sufficient length to maneuver their vehicles at the same or reduced speed rather than to just stop, its values are substantially greater than stopping sight distances.

6.7.5 Meeting Sight Distance

Meeting sight distance is the distance required to enable the drivers of two vehicles traveling in opposite directions, on a two-lane rural road, with insufficient width for passing, to bring their vehicles to a safe stop after becoming visible to each other. It is the sum of the respective stopping sight distances for the two vehicles plus 10.0 m safety distance.

SECTION 7: CROSS SECTION ELEMENTS

7.1 Introduction

A road cross section will normally consist of the road way, carriageway, shoulders, curbs, drainage features, median and earthwork profiles.

Road way is the portion of the road, consisting of the shoulders, the carriageways and the median.

Carriageway is the portion of the road used for the movement of the vehicles exclusive of shoulders.

Earthwork profiles are the side and back slopes of the road cross section.

7.2 Headroom And Lateral Clearance

Headroom is the required height to allow traffic to pass safely under objects restricting the height. It should normally be provided over the full width of the carriageway.

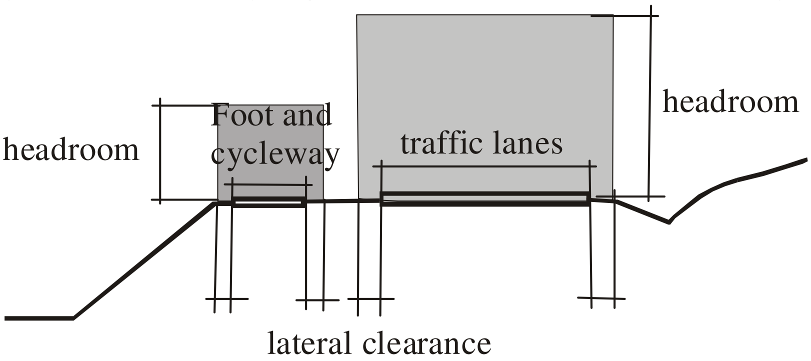


Figure 7-1: Headroom and lateral clearance

The maximum legal height for a vehicle in Uganda is 4.0m. In determining the headroom standard, allowance must be made for:

- The road surface being raised during pavement overlay work
- The possibility of an overbridge collapsing if hit by a vehicle
- The need to allow for occasional oversized vehicles.

The recommended headroom under bridge structures should be 5.0 m on class A, B and C roads and 4.5m on lower road classes. The headroom should be 6m under high-power cables and 5 m under low-power cables. The minimum headroom over footways and cycleways should be 2.5m. An addition to the normal headroom is needed at crests with radii below 700m – see table 7-1.

Table 7-1:Additions to headroom at crests

	Crest radius (m)				
	100	**200**	**300**	**400**	**500-700**
Addition (m)	0.12	0.06	0.04	0.03	0.02

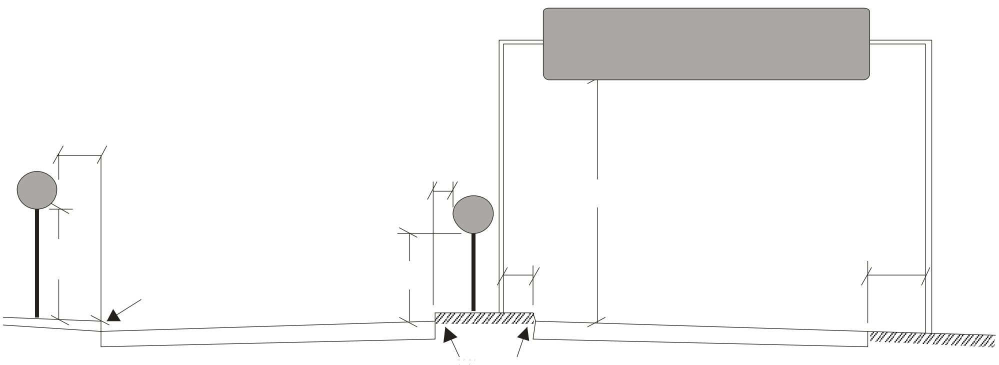
Lateral clearance, see figure 7-1, is the minimum permitted distance between the edge of the traffic lane, the footway/path or cycle way and the nearest fixed object. Fixed objects must not be so close as to discourage the driver from making full use of the traffic lane. The recommended lateral clearances are given in table 7.2.

Table 7-2: Minimum lateral clearances for traffic lanes, foot- and cycleways

Impacting object	Speed limit				Footways and cycle-ways 1)	Foot-paths1)
	30	50	80	100		
height lower than 0.2 m	0.00	0.00	0.25	0.25	0.00	0.00
height higher than 0.2 m	0.30	0.60	1.50	2.00	0.30	0.15
guardrail	0.30	0.60	0.60	0.60	0.00	0.00
roofs etc	≥1.00	≥1.00	≥1.00	≥1.00	0.50	0.50

1) including hard shoulders for walking and cycling

Minimum headrooms and clearances for traffic signs are given in the MOWHC Traffic Signs Manual – see Figure 7-2.



	Minimum (mm)	Desirable (mm)
A	600	1000
B	300	600
C	300	600
D	1000	1800
E	2100	2100
F	5000[1)]	5200
1) maximum allowed vehicle height 4.00 m F Could be decreased to 4.5 m on category C and D roads		

Figure 7-2: Headrooms and clearances for traffic signs

7.3 Road And Lane Width

Road width should be minimised so as to reduce the costs of construction and maintenance whilst being sufficient to carry the traffic loading efficiently and safely.

Lane width and condition of the road surface have the greatest influence and effect on safety and comfort of driving than other features of road. In cases of paved road when we say lane width it is the net width excluding width of edge strip, edge line and center line markings. Lane widths of 2.80, for Design Class Paved III and Gravel B, to 3.65, for Design Class Paved Ia, are used. Higher design speed requires wider lane width than relatively lower design speeds. The capacity of a road is affected by the width of the lane. Narrow lanes force drivers to operate their vehicles closer to each other laterally than they would normally desire. Restricted clearances have the same effect. In capacity sense the effective width of travelled way is reduced when adjacent obstructions such as retaining walls, headwall of structures, and parked cars restrict the lateral clearance. When continuous two-way right-turn lanes are provided, a lane width of 3.0 to 3.6 m could be provided. For road class Paved Ia a 3.6 m right turn lane is considered from the 5.0 m median width.

A lateral/edge strip is a portion of the road way contiguous with the traveled way for recovery of errant vehicles, for lateral support of base and surface course. It also permits drivers meeting or passing other vehicles to drive on the very edge of the roadway without leaving the surface. The width of the edge strip depends on the design speed.

Table 7-3: Width of edge strip and edge line

Design Speed [km/h]	width of edge strip [m]	edge line [m]
120	0.75	0.15
100	0.45	0.15
80	0.45	0.10
60	0.20	0.10
<60	-	-

For access roads with low volumes of traffic design class E roads, single lane operation is adequate as there will be only a small probability of vehicles meeting, and the few passing manoeuvres can be undertaken at a much reduced speeds using either passing places or shoulders. As long as sight distances are adequate for safe stopping, these manoeuvres can be performed without hazard, and the overall loss in efficiency brought about by the reduced speeds will be small as only a few such manoeuvres will be involved. It is not cost-effective to widen the running surface in such circumstances and a basic width of 4.0 m will normally suffice. Single lane road is dealt with and further discussed in Section 7.12.

7.4 Shoulders

Shoulder is the portion of the roadway contiguous to with the carriageway for the under listed purposes or functions. Shoulders are recommended for all Design Classes and may be paved when the carriageway is paved. This has a number of advantages. It will prevent and protect the carriageway pavement from edge failure and ravelling. It will accommodate a very heavy pedestrians and other non-motorized traffic that would otherwise, especially during inclement weather, use the road way and interrupt vehicular traffic. It will provide a better surface for vehicles parking and vehicles requiring immediate repair. The width of shoulder varies from 1.2 for Design Class Gravel C up to 2.5 m for Design Class Paved Ia. Shoulders are intended to perform the following purposes:

- To provide additional maneuvering space on roads of low classification and traffic flows;

- To provide parking space at least partly off the carriage for vehicles which are broken down;
- Safety margin to enable drivers to recover control;
- To enable non-motorized traffic (pedestrian and cyclist) to travel with minimum encroachment on the carriageway;
- To provide lateral support of pavement structures; and,
- To act as barrier for moisture egression.

It is recommended that all shoulders of paved roads be paved, though exceptions may be made for very low volume roads. Regardless of the width, a shoulder should be continuous. Paved shoulders are a safety feature and they also contribute to structural integrity and lower maintenance costs. A continuous paved shoulder provides an area for bicyclists to operate without obstructing faster moving motor vehicle traffic. Shoulders intended for use by pedestrian and cyclists must be at least 1.5 m wide. Where there is a lot of pedestrian and cycle traffic the shoulder may be widened to 2.0 metres, but it would be much safer to provide a separate footway/cycle way. Furthermore, shoulders on structures should have the same width as those on the rest section of the road way. The narrowing or absence of shoulders, especially on structures, may cause serious operating and safety problems. Long, high-cost structures usually warrant detailed special studies to determine feasible dimensions. Reduced width shoulders may be considered in rare justifiable cases. Use of different surfacing material can make the shoulder more visible, and this helps drivers to avoid straying onto it by mistake. However, it is not recommended to make the surface of the shoulder rougher than that of the traffic lane, because this will discourage pedestrians and cyclists from using it.

All shoulders should be sloped sufficiently to rapidly drain surface water but not to the extent that vehicular use would be restricted. Paved shoulders normally have same slope as the traffic lanes and should be sloped from 3 to 6 percent, gravel and crushed-rock shoulders from 4 to 6 percent and turf shoulders up to 8 percent.

Much smaller clearances will sometimes be necessary at specific locations such as on bridges, although a minimum of 1.0m will remain desirable. Minimum overall widths in such circumstances should be sufficient to allow the passage of traffic without an unacceptable reduction in speed, which will depend on the length of the reduced width section and levels of motorized and non-motorized traffic flow. Separate facilities should be provided for pedestrians where possible.

7.5 Normal Cross Fall

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 down ward towards both edges. These are also called as cross fall or camber. Cross fall 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-fall 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.

The normal cross fall for paved carriageway on tangent sections and on very flat curves with larger radius, shall be 2.5 percent maximum. Higher values up to 3 percent could be used in areas of intense rainfall in order to facilitate roadway drainage, even though undesirable from the point of view of operation. On high type two-lane carriageway the crown slope of as low as 2 percent is accepted for all other conditions. The normal cross fall for unpaved roads should be 4 percent.

7.6 Side Slope and Back Slopes

Side-slopes should be designed to insure the stability of the roadway and to provide a reasonable opportunity for recovery of an out-of-control vehicle. The design of side slopes is of great importance for traffic safety, as this can determine whether drivers that run off the road are able to recover control. The shallower the slope, the safer it will be. And the transition from the shoulder to the foreslope must be smooth enough to prevent the vehicle becoming airborne. A safe transition is also needed between the foreslope and the back slope so as to avoid causing the vehicle to rollover.

Three regions of the roadside are important when evaluating the safety aspects: the top of the slope (hinge point), the side slope, and the toe of the slope (intersection of the fore slope with level ground or with a back slope, forming a ditch). Figure 7-1 illustrates these three regions.

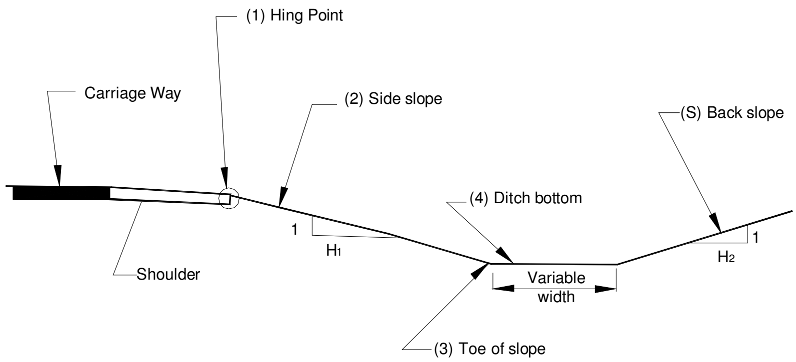


Figure 7-3: Roadside regions

The hinge point contributes to loss of steering control because the vehicle becomes airborne in crossing this point. The side slope region is important in the design of high slopes where a driver could attempt a recovery maneuver or reduce speed before impacting the ditch area. Safe transition between the side and back slopes should be provided as there is a chance of a vehicle reaching the ditch is somewhat on the higher side. Rounding at the hinge point though not necessary from a vehicular rollover can significantly reduce the hazard potential. Similarly, rounding at the toe of the slope is also beneficial. In general rounded slopes reduce the chances of an errant vehicle becoming airborne, there by reducing the hazard of encroachment and affording the driver more control over the vehicles.

Embankment or fill slopes parallel to the flow of traffic may be defined as recoverable, non-recoverable, or critical. Recoverable slopes include all embankment slopes, 1:4 (1 vertical to 4 horizontal) or flatter. Motorists, who encroach on recoverable slopes, can generally stop their vehicles or slow them enough to return to the roadway safely. Fixed obstacles, such as culvert head walls, should not extend above the embankment within the clear zone distance.

A non-recoverable slope is defined as one which is traversable, but from which most motorists will be unable to stop or to return to the roadway easily. Typically, vehicles on such slopes typically can be expected to reach the bottom. Embankments between 1:3 and 1:4 generally fall into this category.

Since a high percentage of encroaching vehicles will reach the toe of these slopes, the clear zone distance extends beyond the slope, and a clear runout area at the base is desirable.

A critical slope is one on which a vehicle is likely to overturn. Slopes steeper than 1:3 generally fall into this category.

The selection of a side slope and back slope is dependent on safety considerations, depth/height of cut or fill, characteristic of soil or natural ground material, and economic considerations. Further, the guideline in this section may be most applicable to new construction or major reconstruction. On maintenance and rehabilitation projects, the primary emphasis is placed on the roadway itself. It may not be cost-effective or practical because of environmental impacts or limited right-of-way to bring these projects into full compliance with the side slope recommendations provided in this guide.

Table 7-4 indicates the side slope ratios recommended for use in the design according to the height of fill and cut, and the type of material.

However, this table should be used as a guide only, particularly as concerns applicable standards in rock cuts, where a controlling influence is cost. When the embankment (fill) height is greater than about 1.0m, the 1:4 foreslopes recommended in the table becomes uneconomic. This is because a large amount of fill material will be needed and the structure will extend over a large area – thus increasing land acquisition costs. In these circumstances the foreslope is best determined by the natural angle of repose and erodibility of the material (often 1:1.5). Where steep foreslopes have to be used, consider installing safety barrier Note also that certain soils that may be present at subgrade may be unstable at 1:1.5 side slopes, and for these soils a gentler slopes may be required. Slope configuration and treatments in areas with identified slope stability problems should be addressed as a final design issue.

Table 7-4: Slope Ratio Table (Vertical to Horizontal ratio)

Material type	Height of Slope (m)	Fill side slope V:H	Cut		Type of slope
			Back Slope	Side Slope	
			V:H		
Earth or Soil	0.00 -1.00 1.00 -3.00 over 3.00	1 : 4 1 : 2 1 : 1.5	1 : 3 1 : 2 1 : 1.5	1 : 4* 1 : 2 1 : 1.5	Recoverable Critical Critical
Compacted Lateritic Soil	0.00 – 1.00 1.00 – 3.00 over 3.00	1 : 4 1 : 2 1 : 1.5	1 : 2 1 : 1.5 1 : 1	1 : 4* 1 : 2 1 : 1.5	Recoverable Critical Critical
Hard Rock	0.00 – 2.00 over 2.00	1 : 1.25 1 : 1	3 : 1 4 : 1	1 : 1.25 1 : 1	Critical Critical
Weathered Rock	0.00 – 2.00 over 2.00	1 : 1.5 1 : 1	3 : 1 3 : 1	1 : 1.5 1 : 1	Critical Critical
Decomposed Rock	0.00 – 1.00 1.00 – 3.00 over 3.00	1 : 4 1 : 2 1 : 1.5	1 : 3 1 : 2 1 : 1.5	1 : 4 1 : 2 1 : 1.5	Recoverable Critical Critical

*in hilly, mountainous and difficult areas 1:4 could be reduced to 1:2

7.7 Drainage Channels

Good drainage is essential to protect the road from damage. Drainage channels include (a) road side channel running parallel to the road and in cut sections to remove water from the road cross section; (b) toe of slope channel to convey the water from any cut section and from adjacent slopes to the natural watercourse; (c) intercepting channels placed back of the top of cut slopes to intercept surface water; and, (d) chutes to carry collected water down steep cut or fill slopes.

Drainage channels perform the vital function of collecting and conveying surface water from the road right-of-way. Therefore, drainage channels should have adequate capacity for the available peak runoff, should provide for unusual storm water with minimum damage to the road, and should be located and shaped to avoid creating a potential conflict with traffic. Channels should be protected from erosion with the lowest cost protective lining that will withstand the flow velocities expected. Channel should be kept clean and free of material that would lower the capacity of the channel. Channel deterioration can reduce channel capacity and overflow may occur.

Minimum ditch dimensions is given as follows. Minimum depth of ditches should be 0.6m in mountainous and escarpment terrain, and 1.0m elsewhere, using a “U-ditch” configuration with bed width of 0.5m. The side slope and back slope of ditches should generally conform to the applicable slope ratio of the road given in Table 7-4. The capacity of ditches should be checked against the available flood discharge and should be ascertained that it can accommodate without being over-flooded or over-topped. In case that the capacity is found to be low, then the size should be revised by either increasing the bed width or making deeper or both.

Side drains should be avoided in areas with highly expansive clay soils such as black cotton soils. Where this is not possible, they shall be kept at a minimum distance of 4-6m from the toe of the embankment. The ditch in such cases should have a trapezoidal, flat-bottom configuration. The minimum recommended embankment height in such flat, marshy and black cotton soil area is 1.5m.

In populated areas deep and open road side drain channel are hazardous to vehicles & pedestrians. It is therefore recommended to use covered drain channel or under drain system depending on the size of the town, design class of the road, as well as the construction and maintenance cost of said drainage.

Key points to consider in the design of safe side drains are:

- There should be sufficient discharge points and culverts to ensure that the drain never gets very deep;
- Open drains are best located outside the clear zone;
- With open drains, the slope next to the road should as much as possible be flat enough to reduce the risk of errant vehicles overturning;
- In built-up areas channel drains deeper than 250mm should be covered or under-drain system be used for the safety and convenience of both pedestrians and vehicles;
- The drain should terminate or discharge in a satisfactory manner without risk of causing erosion or other problems; and,
- The drain should be capable of being cleaned and maintained easily.

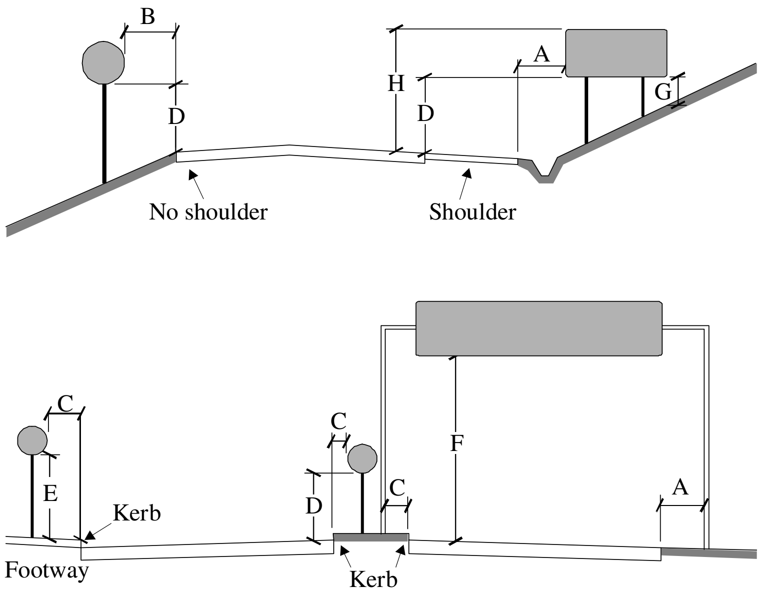
It is not always easy to design drains that can cope with the expected flow and yet are safe, affordable and easy to maintain, so compromises are often required.

7.8 Clear Zone

The clear zone is a safety zone adjacent to the traffic lanes. It is an unobstructed, relatively flat area provided beyond the edge of carriageway for the recovery of errant vehicles. The clear zone must be kept free of rigid objects (such as posts, trees etc with a diametreover 0.10m) and other hazards, such as steep slopes, open drains, etc. Elimination of roadside furniture or its relocation to less vulnerable areas are options in the development of safer roadsides. If the hazard cannot be removed or relocated from the clear zone it should be shielded by safety barrier. If shielding by safety barriers is not possible, for whatever reason, consideration should be given to delineating the feature so that it is readily visible to a motorist. Once a vehicle has left the roadway, an accident may occur. The end result of an encroachment depends upon the physical characteristics of the roadside environment. Flat, traversable, stable slopes will minimize overturning accidents, which are usually severe.

For adequate safety, it is desirable to provide an unencumbered roadside recovery area that is as wide as practical on a specific road section. A clear zone of 3 m or more from the edge of the carriageway appropriately graded and having gentle slopes and rounded cross-sectional design, is desirable. At existing pipe culverts, box culverts and bridges, the clearance cannot be less than the roadway width; if this clearance is not met, the structure must be widened. New pipe and box culvert installations, and extensions to same, must be designed with a clearance at least the width of shoulder from the edge of the carriageway.

Minimum desirable and maximum horizontal clearance to road signs as well as overhead clearance between overhead structures shall be as per the Ministry Traffic Sign Manual and as reproduced in figure 7-4.



	Minimum (mm)	Desirable (mm)	Maximum (mm)
A	600	1000	2500
B	1000	1500	2500
C	300	600	
D	1000	1800	2500
E	2100	2100	2500
F	5000	5200	
G	750		
H			5000

Figure 7-4: Heights and clearances

Horizontal clearance from guard rails, guide posts, and marker posts in rural section shall be a minimum of 0.6 m from the shoulder edge (or carriageway edge if there is no shoulder). However there must be at least 0.6 m between the back of the post and the break of fill slope in order to have sufficient ground support for the post.

Maximum allowed vehicle height in Uganda is 4.0 m and minimum vertical clearance of 5.0 m should be allowed for the design. . This minimum vertical clearance could be reduced to 4.5 on Class D and E roads.

It is obvious that the need for clear zones increases with speed and curvature. The following clear zone widths (Table 7-5), measured from the edge of the traffic lane, are considered to give an acceptable standard of safety. Traffic volume is also a factor, as, generally, the higher the traffic volume the greater the frequency of run-off-road incidents –which supports the use of wider clear zone widths.

Table 7-5: Clear zone widths

Speed limit	Standard	
	Desired	Minimum
70	5 m	3 m
80	6 m	4 m
100	9 m	6 m

The clear zone widths given in Table 7-5 should be increased at sharp bends on high-speed roads by a correction factor to be obtained from Figure 7-6 depending on the radius of curve.

Foreslopes steeper than 1:3 cannot be counted as part of the clear zone because they are too steep. Slopes that can be traversed safely by out-of-control vehicles need to be at least 1:4 or gentler. Slopes between 1:3 and 1:4 are marginal; the normal practice is that half the width of these slopes is counted as part of the clear zone – see Figure 7.5.

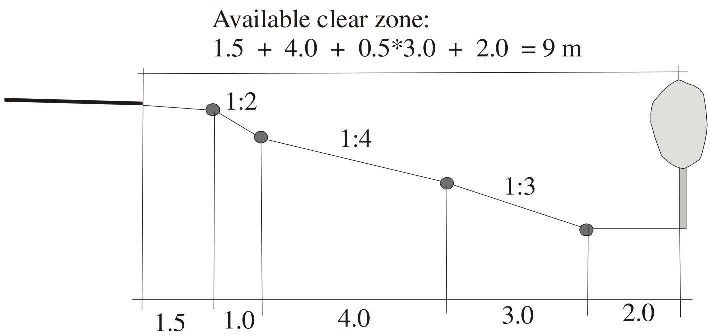


Figure 7-5: Example how to calculate clear zones

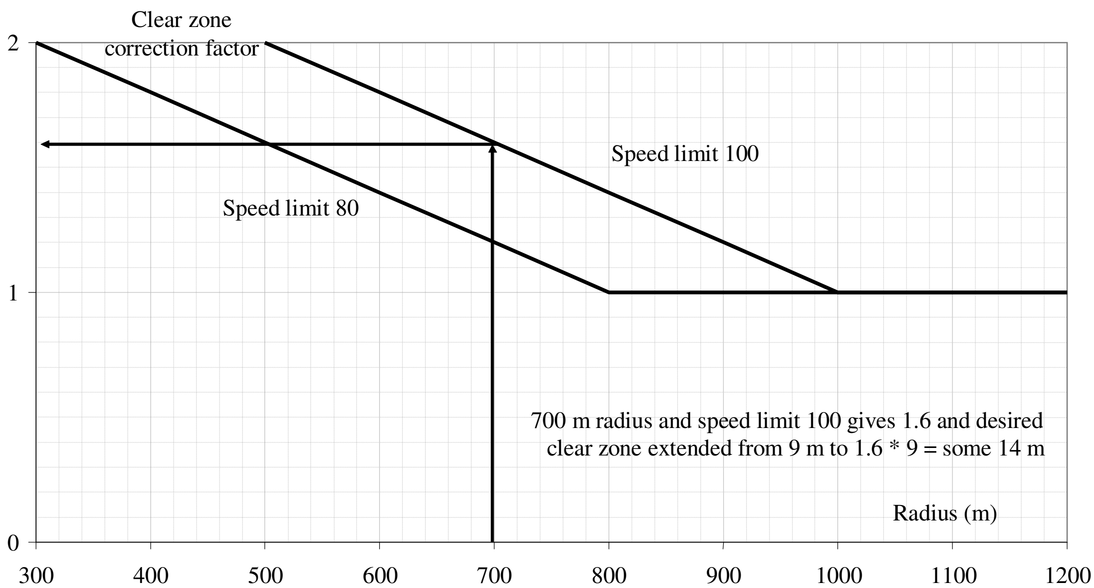


Figure 7-6: Clear zone correction factor for bends

7.9 Side Roads and Culverts

Side roads are often built up on a little embankment so that they enter the main road on the same level. This embankment can be an obstacle to vehicles that run off the road. And the culvert carrying the main road drain under the side road will often have a large, solid headwall. Where there is a culvert under the main road, the culvert headwall is often close to the edge of the carriageway, especially if the road has been widened at some stage. These are hazards. With side roads it is best to try and construct gentle embankment slopes and move the culvert further away from the main road. In the case of the culvert under the main road it should be extended in order to move the ends away from the carriageway edge. It is also important to assess whether culverts really need large solid headwalls. It may be possible to provide a smooth opening instead.

7.10 Right-of-way

Right-of-ways, or road reserves, are provided in order to accommodate the ultimate planned roadway, including all cross sectional elements and to enhance the safety, operation and appearance of the roads. The width of right-of-way depends on class of the road, the cross section elements of the road, topography and other physical controls together with economic considerations. Although it is desirable to acquire sufficient right-of-way to accommodate all elements of the cross section and appropriate border areas, right-of-way widths should be limited to a practical minimum in both rural and developed areas affecting the economy of the inhabitants.

Figure 7-7 shows the cross-sectional elements to be considered when determining right of way. A uniform width of right-of-way may be convenient, but there are special cases where additional right-of-way may be desirable. These special cases could be locations where the side slopes extend beyond the normal right-of-way, where greater sight distance is desirable, at intersections and junctions and for environmental considerations. In all cases the right-of-way should always be determined and shown on the final design plans of road projects.

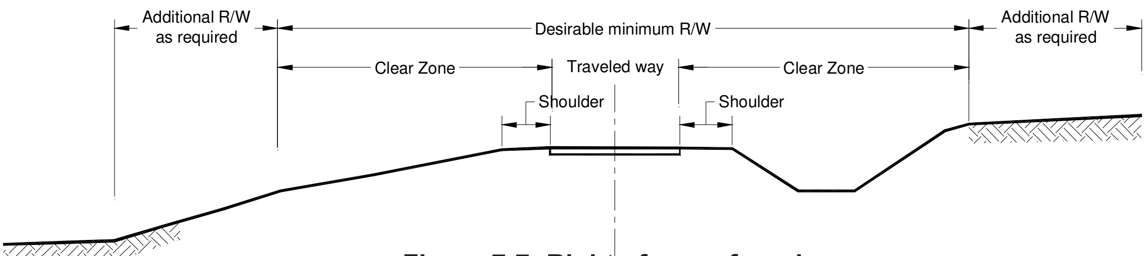


Figure 7-7: Right of way of road

Road reserve widths applicable for the different road classes are given in Tables 4-3 up to 4-9. In mountainous or escarpment terrain, a cut section may be of such depth that the right-of-way width is exceeded from the top of cut on one side to the other top of cut.

Additional areas required for outlets etc., should be provided in a manner that will not endanger the future integrity of the drainage facility and will provide adjoining land owners restricted use of this land after completion of the road.

Reduced widths should be adopted only when these are found necessary for economic, financial or environmental reasons in order to preserve valuable land, resources or existing development or when provision of the desirable width would incur unreasonably high costs because of physical constraints. In such cases, it is recommended that the right-of-way should extend a minimum of a nominal 3 metres from the edges of the road works. However, where this occurs, it is advisable to restrict building activity along the road to prevent overcrowding, to preserve space for future improvements, and to allow provision for sight distances at curves.

7.11 Four-lane And Divided Roads

In the case that the traffic volumes could not be accommodated by a two lane rural road then there will be a need to increase the roadway to a four-lane facility when a certain volume is reached.

It is also the case that some cities and villages have included a four-lane roadway as a feature in their master plans.

Four lane and divided roads are required when the design traffic volume is sufficient to justify their use. They are also frequently used in urban/peri-urban areas.

As discussed earlier a minimum median width of 4.8 or about 5.0 metres is required to allow the provision of right-turning lanes outside of the adjacent carriageway, and to avoid having a turning passenger vehicle from distributing the vehicles on the through lanes.

7.12 Single Lane Roads

For low traffic volume roads (<50ADT), single lane operation is adequate as there will be only a small probability of vehicles meeting, and the few passing maneuvers can be undertaken at very reduced speeds using either the shoulder or passing bays. In such cases adequate sight distances should be provided for safe stopping, these maneuvers can be performed without hazard, and the overall loss in efficiency brought about by the reduced speeds will be small, as only a few such maneuvers will be involved.

The lowest design classes or road will not allow passing and overtaking to occur on the carriageway and passing must be performed using shoulders. In such cases the width of roadway including shoulder should be enough to allow two design vehicles to pass, i.e. a minimum of 6.0 metres width, and vehicles would be expected to stop or slow to a very low speed.

In the case where shoulders are not provided passing and overtaking maneuvers are to be undertaken on passing bays. The increased width of road at passing bays should be enough to allow two design vehicles pass safely. In such cases it is normal that passing bays should be located every 300 to 500 metres depending on the terrain and geometric conditions. However, adjacent passing bays must be inter-visible and appropriately placed, inter-visible passing bays are essential to ensure the free flow of traffic. Account should be taken of sight distances, the likelihood of vehicles meeting between passing bays and the potential difficulty of reversing. In general, passing bays should be constructed as the most economic locations as determined by terrain and ground conditions, such as transitions from cuttings to embankment, rather than at precise intervals.

The length of individual passing bays will vary with local conditions and the type of design vehicle but, generally, a length of 20 metres including tapers will cater for most commercial vehicles.

7.13 Median

A median is highly desirable on road of four or more lanes. A median is defined as the portion of a divided road separating the carriageway for traffic opposing directions. The median width is expressed as the dimension between the through-lane edges and includes the right shoulders, if any. It includes the inner shoulders and the central island.

The principal functions of a median are to separate opposing traffic, provide a recovery area for out-of –control vehicles, provide a stopping area in case of emergencies, allow space for speed changes and storage of right-turning and U-turning vehicles, minimize headlight glare, provide width for future lanes, and to provide a refuge for pedestrian crossing the road in case of urban and populated areas. For maximum efficiency, a median should be highly visible both night and day and contrast with the through traffic lanes.

Medians may be depressed, raised, or flush with the carriageway. Medians should be as wide as feasible but dimension in balance with other components of the cross section.

In determining median width, consideration should be given to the possible need for median barrier. Where possible, median width should be such that a median barrier is not warranted. In general, the median should be as wide as practical. Where the median is less than 9.0 m wide consider installing a median barrier. However, economic factor and available of land (right-of-way), and also terrain often limit the width of median. The minimum width of median is as narrow as 1.2 to 1.8 m. Where provision for right turn lane is required the minimum width of median will be 4.8 – 5.0 m. In some cases for future upgrading of the road a central reserve of minimum 12.0 m could be introduced to serve as median. The 12m central reserve could accommodate in the future two lanes each having 3.5m in addition to a 5.0 m median.

Figure 7-8 gives recommendations on median design and width on high-speed rural dual carriageway roads.

Medians on urban dual carriageways should normally be designed to function as a refuge for pedestrians. The median should have a minimum width of 2.0m, but this can be reduced to an absolute minimum of 1.2m where space is very restricted. A 2.0 m width will also give sufficient space for most signs, signals and lighting columns. Median barriers should not normally be necessary on urban dual carriageways with speed limits of less than 80km/h.

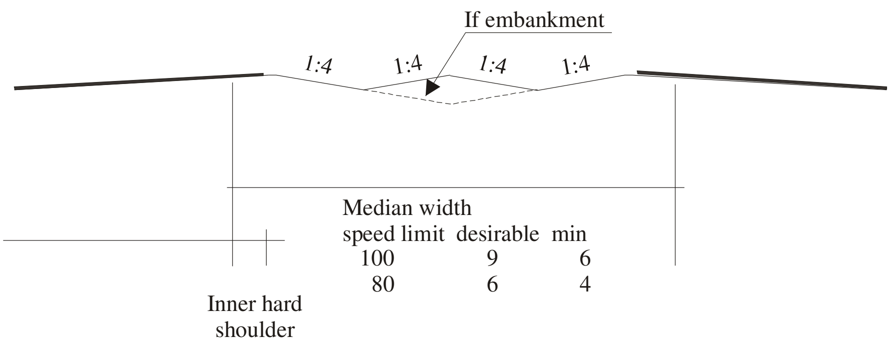


Figure 7-8: Median designs at speed limit 80 and 100 km/h

7.14 Cross-section Over Bridges And Culverts

The safety and operational capacity of bridges and culverts will be affected if the road cross-section is not maintained across these structures. The key points to consider with respect to cross-sections over bridges and culverts are:

• Any significant narrowing of the traffic lane or shoulder is dangerous, especially on high-speed roads;

• When roads are being upgraded and widened, the bridges and culverts will normally need to be widened as well;

• If the shoulder is not continued across the structure, vulnerable road users will move out into the traffic lane in front of fast-moving vehicles and there will be a risk of collisions;

• Footways are conventionally provided on structures with parapets, but the accident risks need to be assessed carefully, especially where footways can only be provided by omitting the shoulders; take account of the relative volumes of pedestrians and cyclists, the speed and volume of motorised traffic and the length of the span;

• Where footways are provided they should be a minimum of 1m wide, and they will usually be quite high (150-200mm) in order to better protect the bridge parapet, in which case the ends must be stepped or ramped down and flared away from the edge of the traffic lane;

• It is best to segregate the vulnerable road users from vehicles by means of a safety barrier

– see Figure below – and this is the preferred solution for bridges on Class A and Class B roads;

• Where, exceptionally, a single lane bridge is planned the traffic lane should be a maximum of 3.7 m wide between kerbs in order to avoid confusion over whether the bridge is for one-way or two-way traffic.

See Section 12 for advice on the design of bridge parapets

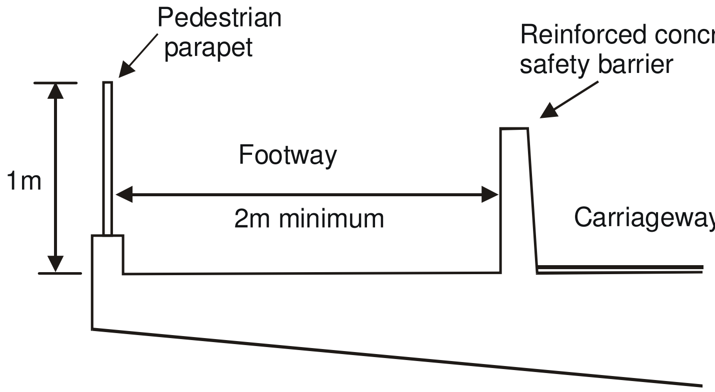


Figure 7-9: Segregated footway on bridges

(Note: if cyclists are likely to use the footway increase the parapet height to 1.4 m)

7.15 Footways and Cycleways

The conventional practice is to assume that pedestrians and cyclists can use the shoulders, but it is much safer for them to be on a separate footway, or combined footway/cycleway. At high flows there can be conflicts between cyclists and pedestrians, but these are not as dangerous as conflicts with motor vehicles. Combined footways/cycleways should be 3.0m wide (2.0m absolute minimum). It is important for footway and cycleway surfaces to be at least as smooth as the adjacent traffic lanes and shoulders – preferably smoother. See also Section 11.

7.15.1 Footways and cycleways in rural areas

The footway/cycleways should be separated from the carriageway by a grass strip or similar, at least 2.0m wide. Gradients should be gentle, preferably less than 4%. On embankments the footway/cycleway can be benched into the fore slope. The planting of shade trees can help encourage people to use the facility.

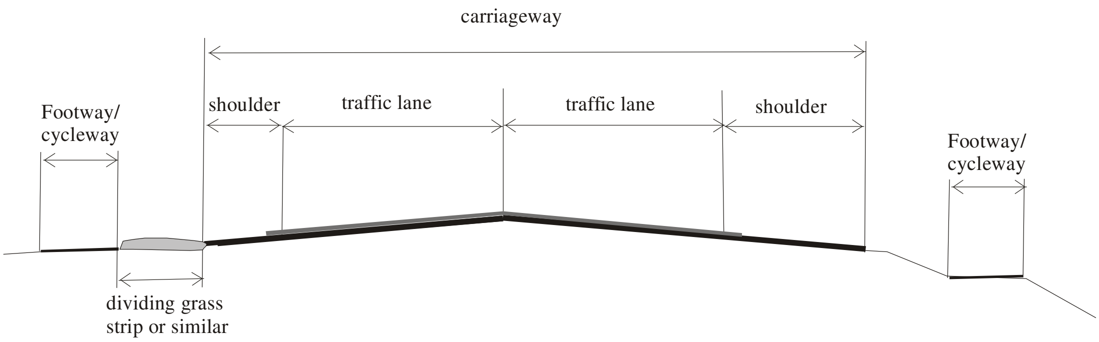


Figure 7-10: Separate foot and cycleway on rural roads

7.15.2 Footways and cycleways in built-up areas

Raised, kerbed footways should be provided in the larger built-up areas. Cycleways, where necessary, should be constructed behind the footway.

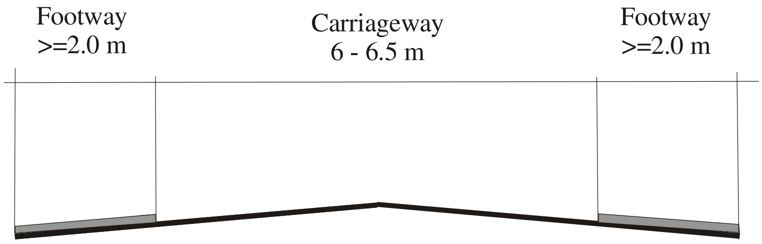


Figure 7-11: Raised, kerbed footway in urban areas

A simpler and cheaper alternative is to have the footway at the same level as the traffic lane, but separated by a barrier kerb or low wall – see Figure 7-12. This means it can function as a combined footway/cycleway. Gaps are left in the separator to allow drainage and access to roadside premises. The separators should be painted white to make them more visible at night, and care should be taken to avoid starting the separator where speeds are high or visibility is poor. If necessary, fit reflectors to the end of the separator.

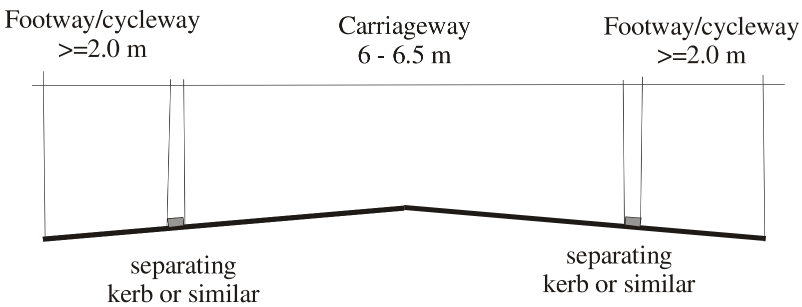


Figure 7-12: Footway/cycleway on physically separated shoulders

7.16 Service Roads

In the larger trading centres and towns it is recommended that service roads be provided. A typical service road design is illustrated below. The local access traffic is kept separate from the through traffic, and the service road provides space for parking, unloading and loading, bus stops, and informal trading.

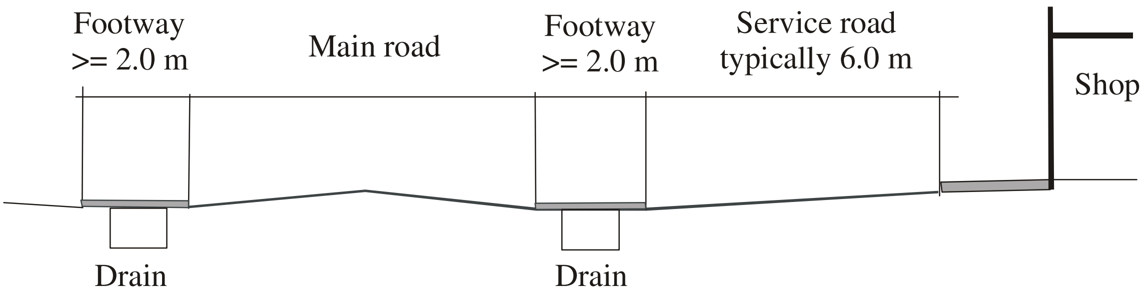


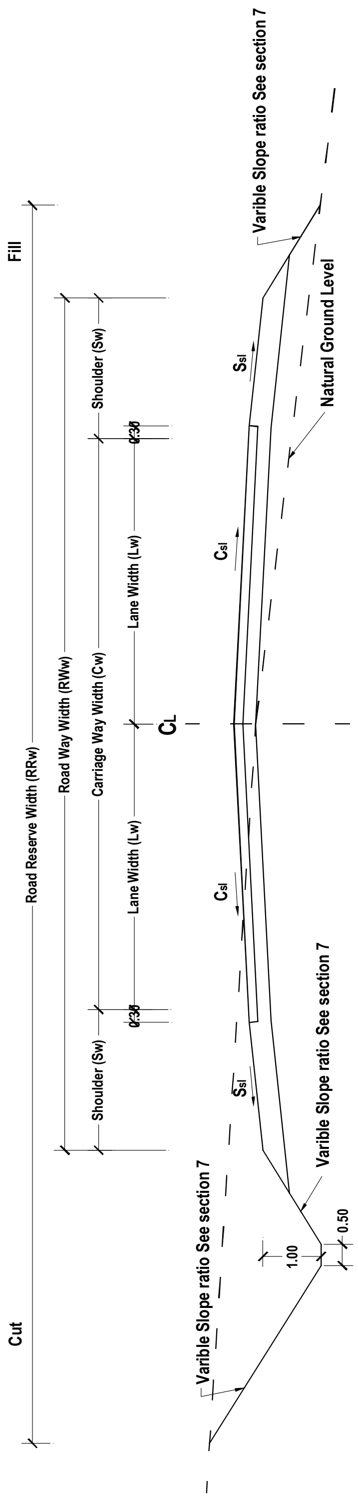
Figure 7-13: Service roads

7.17 Typical Cross Sections

Typical cross sections for road classes Paved Ia, Ib, II, III, and Gravel A, B, & C for both Town and Rural (Normal) sections are given in the following figures (7-14 – 7-18). It is very dangerous for pedestrians and cyclists to use the shoulder of high-speed roads. Therefore the separate foot and cycleways are required to be introduced. The introduction of footways and cycleways in rural and urban areas, when they are required to be introduced, shall be as per Section 7.15 of this manual.

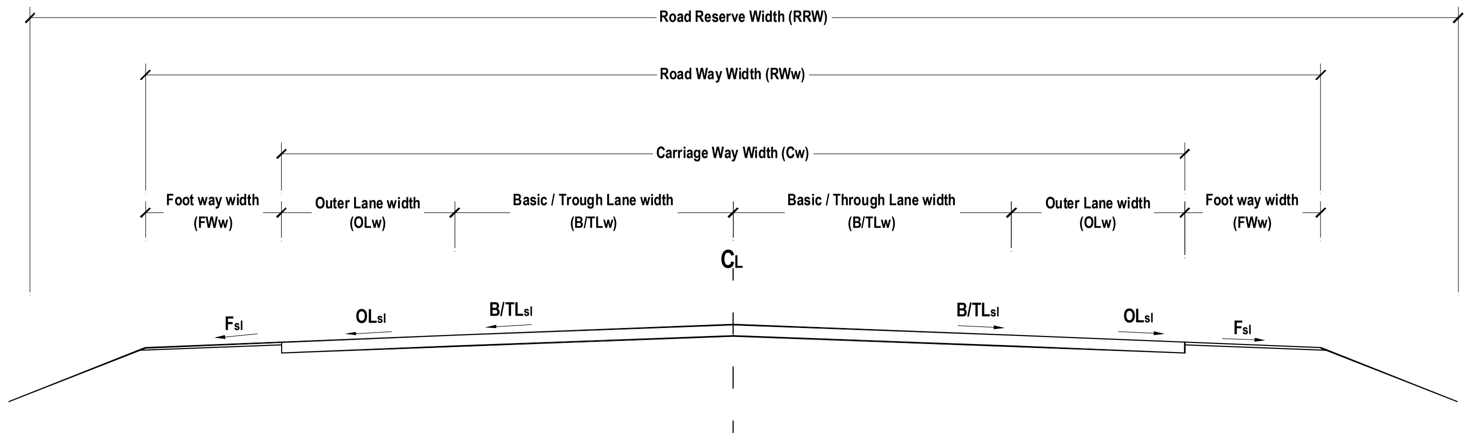
It is preferable that high class roads the primary function of which, are to provide mobility (such road design classes of Paved Ia & Ib), do not pass through towns and populated areas.

In roads passing through towns, in addition to the basic or through lanes, an outer lane on both sides of the road are to be introduced in order to avoid disruption of non-stopping or passing traffic by stopping vehicles in the towns. Provision of parking lane on higher class of roads in towns may have adverse impact on safety of pedestrians. Therefore segregation of through traffic from parking and footways is preferable for safety



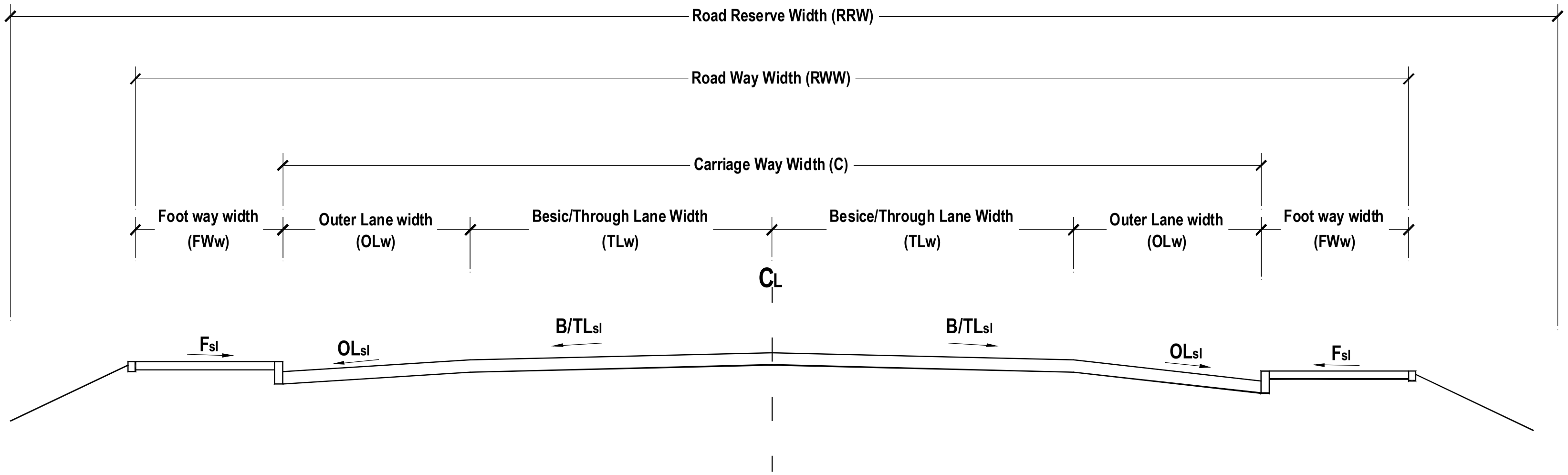
Design Class of Road	Dimension(m)						Slope (%)	
	RRw	RWw	Cw	No of Lane	Lw	Sw	Csl	Ssl
Paved Ib	40.0	11.0	7.0	2	3.5	2.0	2.5	4.0
Paved II	30.0	10.0	6.0	2	3.0	2.0	2.5	4.0
Paved III	30.0	8.6	5.6	2	2.8	1.5	2.5	4.0
Gravel A	30.0	10.0	6.0	2	3.0	2.0	4.0	4.0
Gravel B	25.0	8.6	5.6	2	2.8	1.5	4.0	4.0
Gravel C	15.0	6.4	4.0	1	4.0	1.2	4.0	4.0

Figure 7-14: Typical normal cross section and values of various cross section elements for Road Design Classes Bitumen Ib, II, III, Gravel A, B, and C



Design Class of Road	Dimension (m)							Slope (%)		
	RRw	RWw	Cw	No of lane	B/TLw	OLw	FWw	B/TLsl	OLsl	Fsl
Gravel A	30	17.0	12.0	2	3.0	3.0	2.5	4.0	4.0	4.0
Gravel B	25	14.6	10.6	2	2.8	2.5	2.0	4.0	4.0	4.0
Gravel C	20	13.0	9.0	1	4.0	2.5	2.0	4.0	4.0	4.0

Figure 7-16: Typical town cross section and values of various cross section elements for Design Classes Paved Ib, II and III roads



Design Class of Road	Dimension(m)						Slope		
	RRw	RWw	C	B/TLw	OLw	FWw	B/TL(%)	OLsl	Fsl (%)
Paved Ib	40	19.0	14.0	3.5	3.5	2.5	2.5	3.0	3.0
Paved II	30	17.0	12.0	3.0	3.0	2.5	2.5	3.0	3.0
Paved III	30	15.6	10.6	2.8	2.5	2.5	2.5	3.0	3.0

Figure 7-16: Typical town cross section and values of various cross section elements for Design Classes Paved Ib, II and III roads

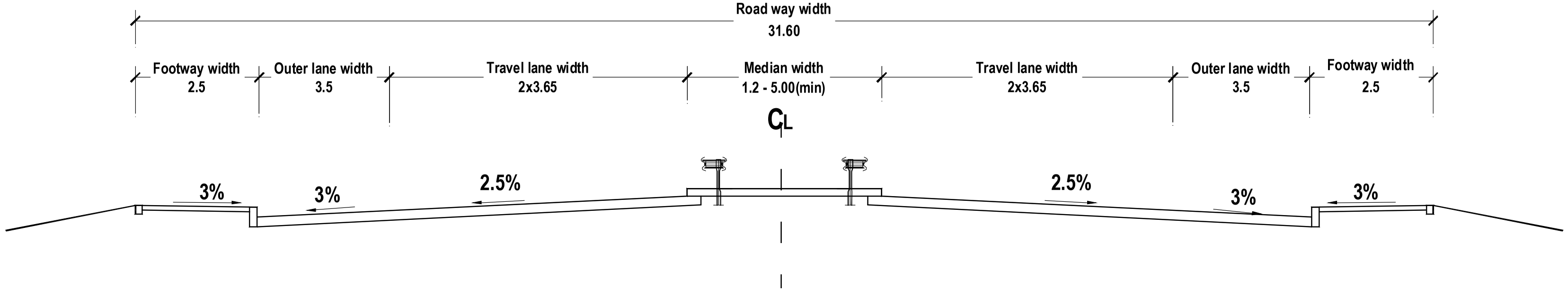


Figure 7-17: Typical Normal Cross Section for Design Class Paved Ia Road

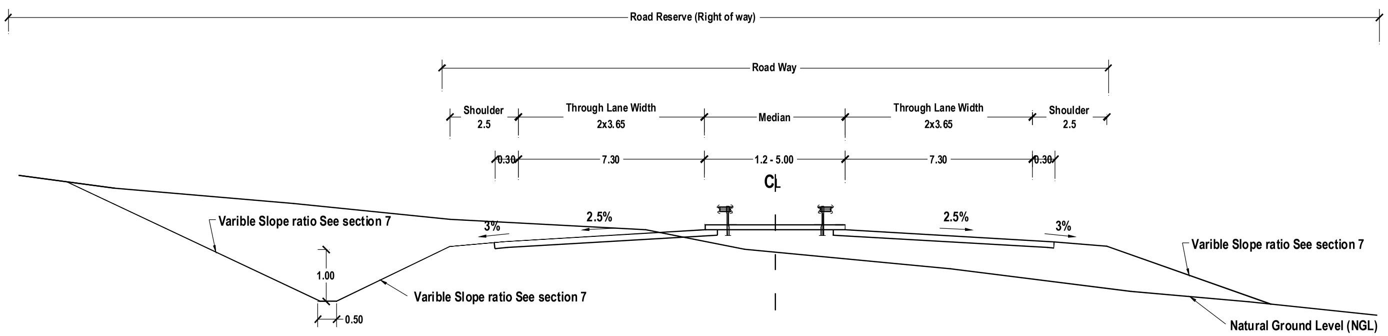


Figure 7-18: Typical Town Cross Section for Design Class Paved Ia Road

SECTION 8 : AT GRADE INTERSECTIONS

8.1 General

A junction, or intersection, is the general area where two or more roads join, or cross, within which are included the carriageway(s) and roadside facilities for traffic movement in that area. They are the most critical element in the road network. Because they involve users sharing road space (unless they are grade-separated intersections) their capacity is limited and there is an inherent risk of collisions.

An intersection is an important part of a road because, to a great extent, the extent, efficiency, safety, speed, cost of operation, and capacity depend on its design. Each intersection involves through-or cross-traffic movements on one or more of the roads concerned and may involve turning movements between these roads. Junctions (or intersections) are necessary to allow vehicles to access to and exit from the road. The greatest degree of traffic conflict occurs at junctions and vehicle accidents predominate at these locations. 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 junctions and from a traffic-safety aspect these lightly trafficked junctions require as much attention as do those junctions where heavier conflicting traffic movements occur. Good junction 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 junctions to limit inherent hazard and maintain acceptable traffic capacity is of great importance. To accomplish this, the layout and operation of the junction should be obvious to the driver, with good visibility between conflicting movements. Furthermore, the number of junctions 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.

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.

8.2 Intersection Types

Different at-grade junction (intersection) types will be appropriate under different circumstances depending on traffic flows, speeds, and site limitations.

8.2.1 An Access

An access shall be defined as the intersection of an unclassified road with a classified road and shall generally be provided within the road reserve boundary of the classified road. Access roads 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 junction standards. This may occur, for example, at an entrance to an industrial development or factory site. 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 3m. The lay out and location of the access must satisfy the visibility requirement for “stop” conditions given in Figure 8-12. A drainage pipe 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.

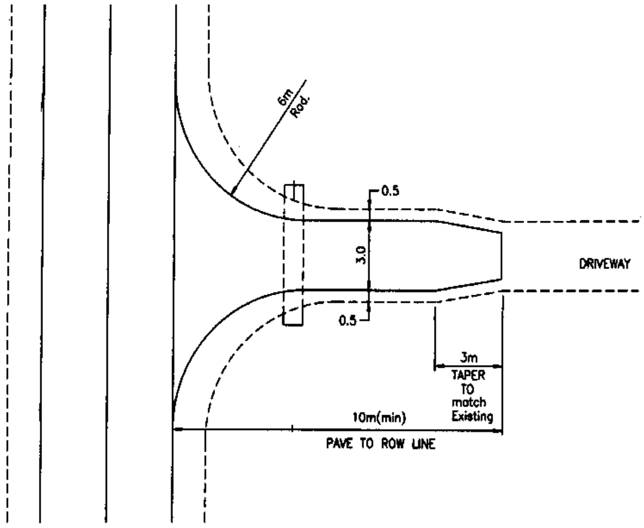


Figure **8-1: Typical Access**

8.2.2 Junction/Intersection

A junction is the intersection of two or more classified roads on the same surface at grade. At grade intersections can be classified in to two main intersection categories based on the type of control used. For each category, there are a number of intersection types.

These types of at-grade intersection are shown in the table below:

Table 8-1: Types of at-grade Intersection

Intersection category	**Traffic control**		**Intersection types**
	Major road	**Minor road**	
Priority intersection	Priority	Stop or give way sign	A Unchannelised T-intersection B Partly Channelised T-intersection C Channelised T-intersection
Control intersection	Traffic signals or give way sign		D Roundabout E Signalised intersection

8.2.2.1 Priority Intersections

Priority intersections will be adequate in most rural situations. Three types of T intersections are given below:

Unchannelised T-intersection (A)

The unchannelised design is suitable for intersections where there is a very small amount of turning traffic. It is the simplest design and has no traffic islands.

Partly Channelised T-intersection (B)

The partly channelised design is for intersections with a moderate volume of turning traffic. It has a traffic island in the minor road arm. In urban areas, the traffic island would normally be kerbed in order to provide a refuge for pedestrians crossing the road.

Channelised T-intersection (C)

The fully channelised design is for intersections with a high volume of turning traffic or high –speeds. It has traffic islands in both the minor road and the main road.

Typical priority intersections in rural areas is shown in Figure 8-2.

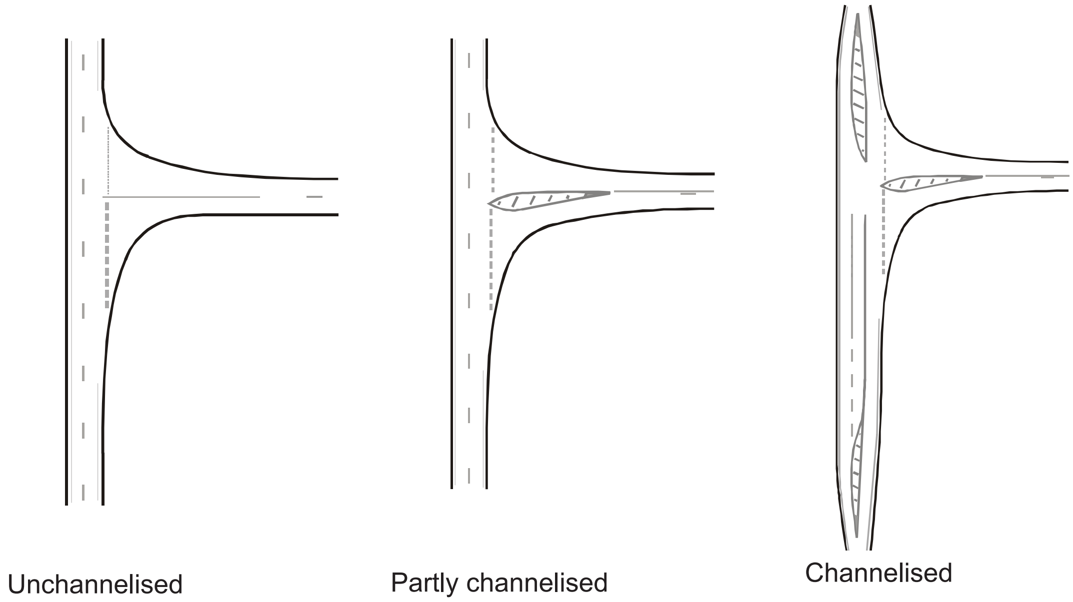


Figure 8-2: Typical T-intersections

The crossroads form of priority intersection must not be used. It has a very high number of conflict points, and has a much higher accident risk than any other kind of intersection. Existing crossroads should, where possible, be converted to a staggered intersection, or roundabout, or be controlled by traffic signals.

8.2.2.2 Control intersections

Control intersections are mostly used in towns and trading centers. However, roundabouts can be used in rural areas in intersections between major roads or other intersections with high traffic volumes. There are two types of control intersections:

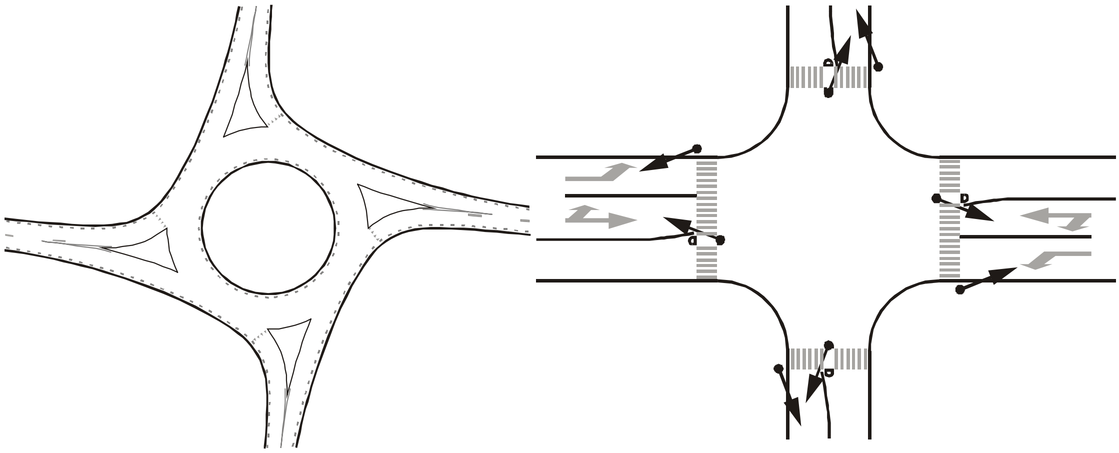
Roundabout (D)

Roundabouts are controlled by the rule that all entry traffic must give way to circulating traffic. The ratio of minor road incoming traffic to the total incoming traffic should preferably be at least 10 to 15%. Roundabouts can be of normal size, i.e. with central island radius 10 m or more, or small size, i.e. with central island radius less than 10 m.

Signalised intersection (E)

Signalised intersections have conflicts separated by traffic signals. No conflicts are allowed between straight through traffic movements.

Typical design of control intersections is shown in Figure 8.3.



Roundabout Signalised intersection

Figure 8-3: Typical designs for control intersections

8.2.3 Junction Maneuvers

Three basic movements or maneuvers occur at junctions, namely merging, cutting and cutting and merging. These maneuvers are illustrated below.

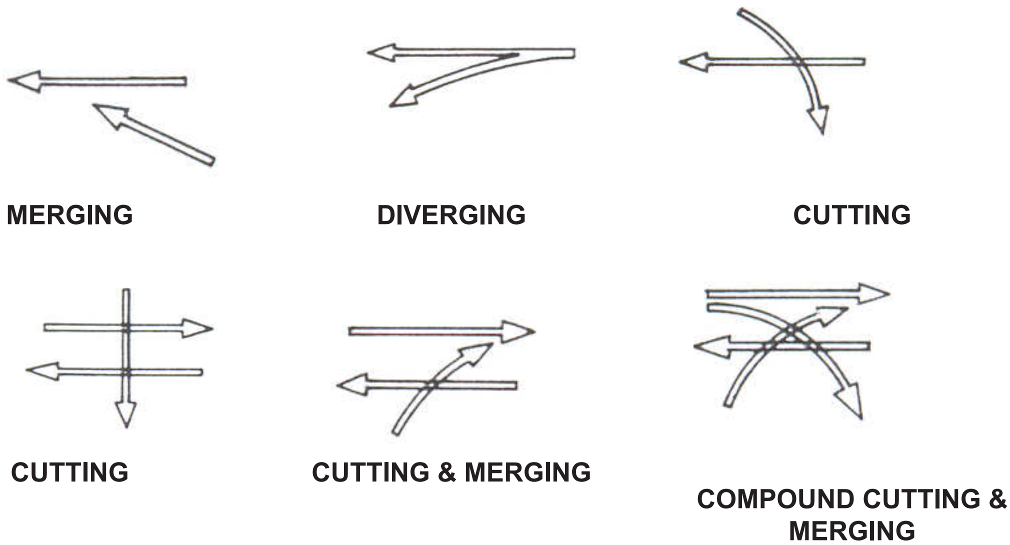


Figure 8-4: Junction Maneuvers

8.2.4 Junction Design Speed

The Junction Design Speed, which is the principal design parameter upon which the geometrical layout and capacity of a junction is based, is the design speed of the major road in the vicinity of the junction. This design speed will not necessarily be the same as the average major road design speed but may be higher or lower. Therefore the designer must give careful consideration to the selection of the appropriate Junction Design Speed as this will greatly affect both the safety and efficiency of the junction and the construction cost.

For reasons of economy, the Junction Design Speed shall not be more than 20 km/h higher than the average design speed of the major road.

For safety reasons, the Junction Design Speed should never be less than 20 km/h lower than the average design speed for the major road.

8.2.5 The Major Road

At T-junctions, the through road, which will usually be carrying the higher traffic volume, shall be considered the major road.

At four, or more, leg junctions, the major road shall be the road with the higher sectional characteristics as determined by:

(a) Right of way regulations on adjoining junctions

(b) Vehicle operating speeds

(c) Traffic volumes

The longer the section of road with continuous priority, higher vehicle operating speeds and/or traffic volumes, the higher rated the sectional characteristics shall be, regardless of the administrative classification of the road.

8.3 Design Requirements

The design of at-grade junctions must take account of the following basic requirements:

- Safety;
- operational comfort;
- capacity; and,
- economy.

8.3.1 Safety and Operational Comfort

A junction is considered safe when it is perceptible, comprehensible and maneuverable. These three requirements can generally be met by complying with the following guidelines.

(a) Perception

(i) The junction should be sited so that the major road approaches are readily visible.

(ii) Early widening of the junction approaches.

(iii) The use of traffic islands in the minor road to emphasize a “yield” or “stop” requirement.

(iv) The use of early and eye-catching traffic signs.

(v) Optical guidance by landscaping and the use of road furniture, especially where a junction must be located on a crest curve.

(vi) The provision of visibility splays which ensure unobstructed sight lines to the left and right along the major road.

(vii) The angle of intersection of the major and minor roads should be between 70 and 110 degrees.

(viii) The use of single lane approaches is preferred on the minor road in order to avoid mutual sight obstruction from two vehicles waiting next to each other to turn or cross the major road.

(b) Comprehension

(i) The right of way should follow naturally and logically from the junction layout.

(ii) The types of junctions used throughout the whole road network should be as much as possible similar.

(iii) The provision of optical guidance by the use of clearly visible kerbs, traffic islands, road markings, road signs and other road furniture.

(c) Maneuverability

(i) A11 traffic lanes should be of adequate width for the appropriate vehicle turning characteristics. To accommodate truck traffic, turning radii shall be 15 meters minimum.

(ii) The edges of traffic lanes should be clearly indicated by road markings.

(iii) Traffic islands and kerbs should not conflict with the natural vehicle paths.

The operation of the junction depends principally upon the frequency of gaps that naturally occur between vehicles in the main road flow. These gaps should be of sufficient duration to permit vehicles from the minor road to merge with, or cross, the major road flow. In consequence junctions are limited in capacity, but this capacity may be optimized by, for example, canalization or the separation of maneuvers.

8.3.2 Capacity

The operation of uncontrolled junctions depends principally upon the frequency of gaps which naturally occur between vehicles in the main road flow. These gaps should be of sufficient duration to permit vehicles from the minor road to merge with, or cross, the major road flow. In consequence junctions are limited in capacity, but this capacity may be optimized by, for example, channelisation or the separation of maneuvers.

8.3.3 Economy

An economical junction design. generally results from a minimization of the construction, maintenance and operational costs.

Delay can be an important operational factor and the saving in time otherwise lost may justify a more expensive, even grade separated, junction.

Loss of lives, personal injuries and damage to vehicles caused by junction-accidents are considered as operational “costs” and should be taken into account.

The optimum economic return may often be obtained by a phased construction, for example by constructing initially an at-grade junction which may later be grade separated.

8.4 Selection of Intersection Type

8.4.1 General

These selection guidelines mainly deal with traffic safety. Other important impacts such as capacity / road user costs, environmental issues, investment and maintenance costs should also be taken into consideration. Capacity, delays, queue lengths, road user costs and also exhaust emissions could be estimated using standard software such as Oscady, Picady and Arcady (UK) SIDRA (Australia) or Capcal (Sweden) but they have not been calibrated for Ugandan conditions, so they could give misleading results. Some traffic flow threshold values for capacity are given in Figure 8.6.

The safety requirement for intersections can be defined as an interval where the expected number of accidents should not exceed a desired level and must not exceed a maximum level. If the expected number of accidents does not exceed the desired level, a priority intersection should be selected. If the number exceeds the maximum level, a control intersection should be selected. Between the two defined levels, a control intersection should be considered. The traffic flow threshold values presented in the following figures 8.5 and 8.7 are based on this concept using general European traffic safety research results on the relationship between speed and incoming traffic flows on the major and minor road.

The selection is divided into two steps; selection of intersection category (priority or control) and selection of intersection type. It is based on the following assumptions:

- Priority intersections can be safe and give sufficient capacity for certain traffic volumes and speed limits;

- If a priority intersection is not sufficient for safety and capacity, the major road traffic must also be controlled; and,

- Depending on location, traffic conditions and speed limits, different types of priority or control intersection should be selected.

8.4.2 Selection of Intersection Category

Safety

The selection of intersection category 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 8-5 are for T-intersections on 2-lane roads with 50, 80 and 100 km/h speed limit. The diagrams are, as already stated, based on general European experience on relationships between speed, safety and traffic flows. They are judged reasonable to be used in Uganda until sufficient local research is available.

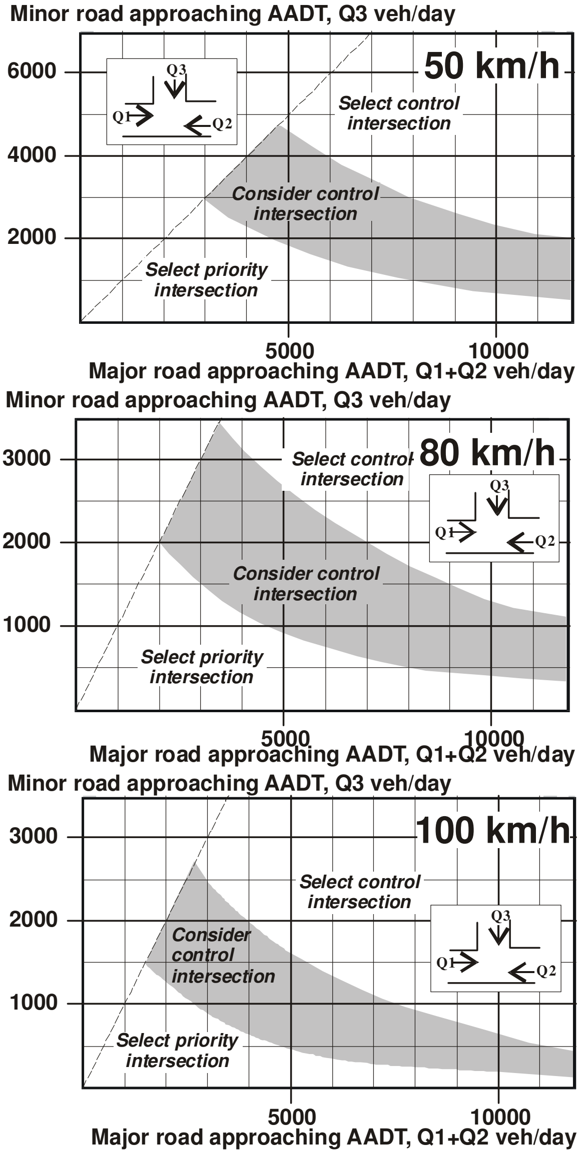


Figure 8-5: Selection of intersection category as to safety

Capacity

The selection of intersection category based on safety should be checked for capacity. It can be made by using diagrams with the relationships between the capacity and the approaching traffic volumes during the design hour (DHV in pcu/design hour). The diagrams shown in Figure 8-6 are for T-intersections on 2-lane roads with 50, 80 and 100 km/h speed limit. The desired level refers to a degree of saturation (actual traffic flow/capacity) of 0.5. The acceptable level refers to a degree of saturation of 0.7.

The diagrams are based on Swedish capacity studies with findings similar to other European countries. It is judged reasonable to be used in Uganda until sufficient Ugandan research is available. Capacity could be checked more in detail using standard capacity software as already stated with the general drawback that Ugandan capacity studies are as yet not available.

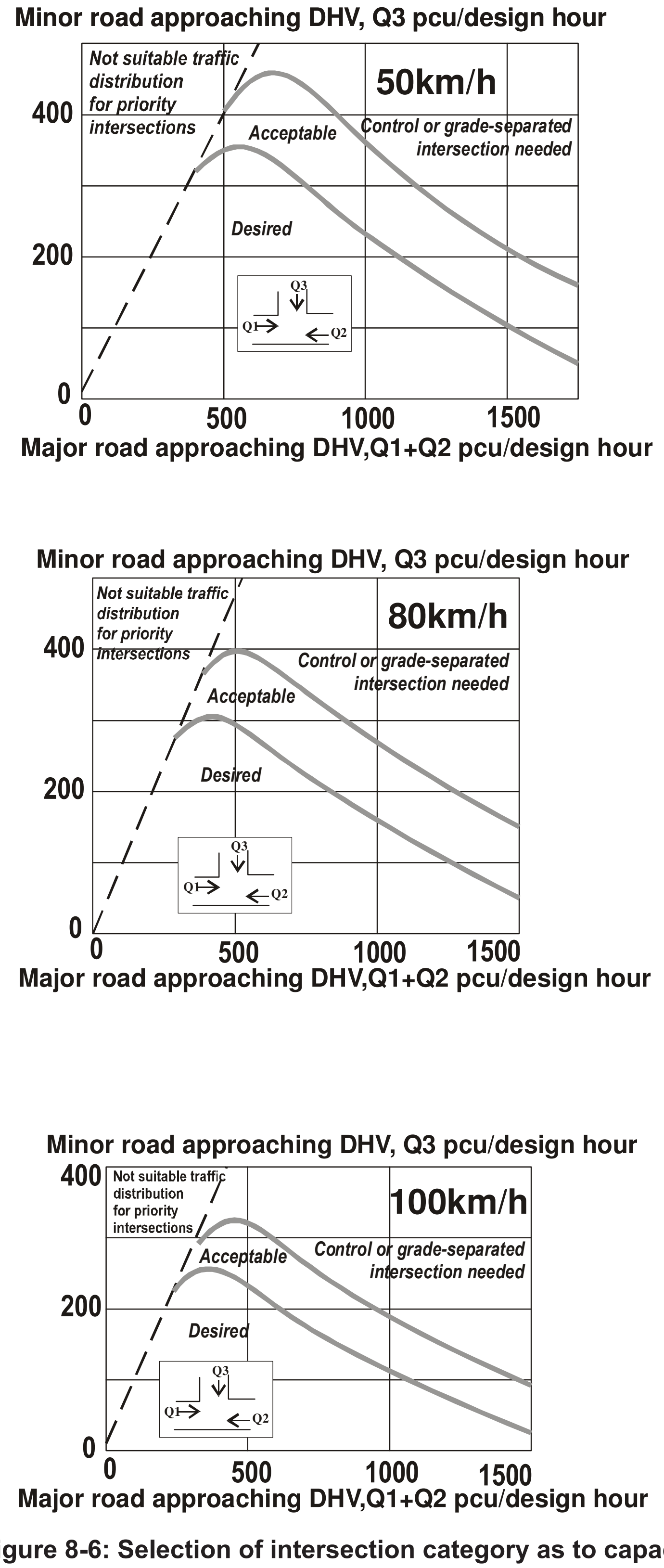


Figure 8-6: Selection of intersection category as to capacity

8.4.3 Selection of Intersection Type

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.

The diagrams are based on general European findings on safety effects of right turn lanes. It is judged reasonable to be used in Uganda until sufficient Ugandan statistics are available. Note however they are only a starting point for determining the most appropriate form of intersection.

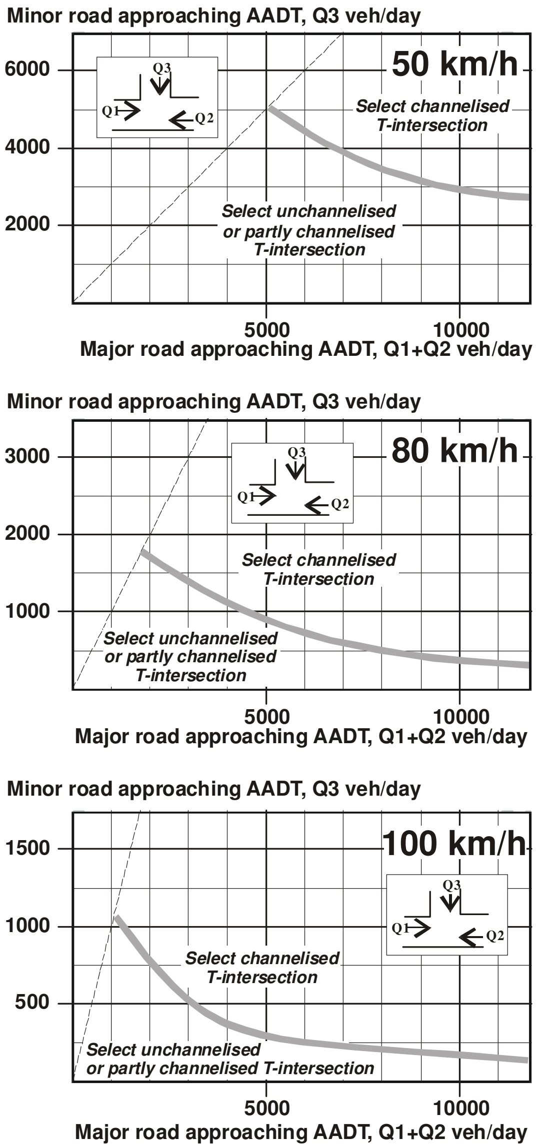


Figure 8-7: Selection of priority intersection type as to safety

Partly channelised 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.

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 Kenyan and UK experience.

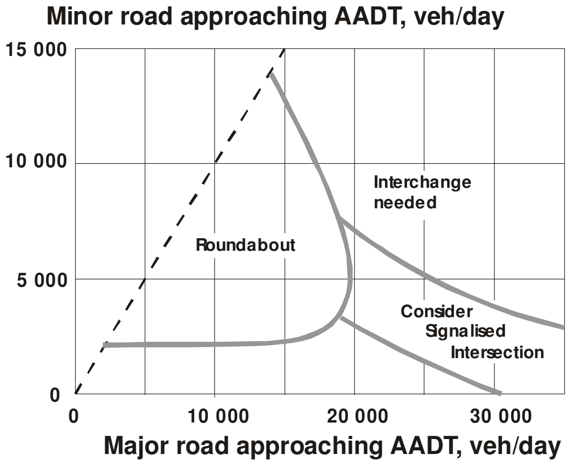


Figure 8-8: Selection of control intersection type

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.

8.5 Junction Design Procedure

Designing an intersection is a highly complex task and it is advisable to follow a standard procedure. The design procedure should be used for new intersections as well as for upgrading of existing intersections. There should be one procedure for preliminary design and one for detailed design.

The objective with the preliminary design is to select the intersection type and location and to make a draft intersection drawing and traffic control plan. The objective with the detailed design is to do the geometric design and to make a detailed intersection drawing and traffic control plan. As with all road projects, it is recommended that a safety audit be done before the scheme is finalised and built.

The procedure to be used for junction design involves four basic steps which are as follows:

(i) Data collection (see Section 8.5.1).

(ii) Define the major road (Section 8.2.5) and determine the junction design speed (Section 8.2.4).

(iii) Select intersection category and type and check that it offers adequate safety and capacity for the predicted traffic maneuvers (Section 8.4).

(iv) Refine and modify the basic junction layout to meet the safety and operational requirements outlined in Section 8.3.1. This is done by applying the principles of junction design which are described in detail in Section 8.6 under the following headings:
(a) Distance between adjoining junctions
(b) Visibility splays
(c) Turning lanes
(d) Central reserves
(e) Traffic islands and minor road widening
(f) Alignment islands and minor road widening

8.5.1 Data Collection

The following data will be required to ensure that a safe, economic and geometrically satisfactory design is produced:

(a) A plan to a scale of at least 1:500, showing all topographical details.

(b) Characteristics of the crossing or joining roads, i.e. horizontal and vertical alignments, distances to adjoining junctions, cross-sectional data, vehicle operating speeds, etc.

(c) Characteristics of the predicted volumes and compositions of the various traffic streams.

(d) Other factors affecting the design, such as topographical or geotechnical peculiarities, locations of public utilities, pedestrian movements, adjacent land usage, etc.

(e) Traffic accident data, especially where the reconstruction of an existing junction is involved.

8.5.2 Basic Junction Layout and Capacity

The basic junction layout (intersection type) for both single and dual carriageway roads is the T-junction with the major road traffic having priority over the minor road traffic. Crossroads, although not recommended, may also be used but only on single carriageway roads where traffic flows are very low and where site conditions will not permit the use of staggered T-junctions. Selection of intersection category and type is to be carried out as outlined in section 8.4.

Where staggered T-junctions are used to replace a crossroads, the right-left stagger as indicated in Figure 8-9 is preferred to the left-right stagger and the minimum stagger should be 50m. On traffic grounds this is because in the latter case opposing queues of right turning vehicles from the major road will have to wait side by side with the consequent possibility of the whole junction locking.

In a left/right staggered intersection the minimum distance should be at least 100 meters and should be longer in order to allow the provision of right turn lanes in the major road. The length of right turn lane depends on the junction design speed and traffic turning right in pcu/hr.

Where there is a lot of cross traffic (from one minor road to the other) the right/left stagger is preferable. This is because, once the driver has turned into the main road, he can proceed to the exit without impeding other traffic. The left/right stagger involves vehicles turning right out of the main road across the path of oncoming traffic, and this is a particularly hazardous manoeuvre.

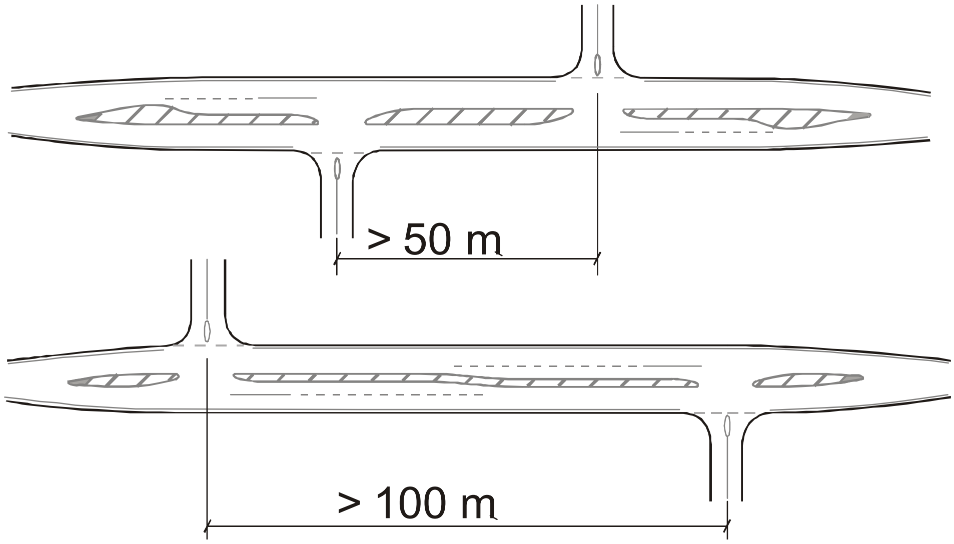


Figure 8-9: Right/Left and Left/Right staggered intersections and their respective minimum distances

Where more complex junction layouts involving the intersection of four or more roads are encountered, these should be simplified by realigning the approaches, to safer, more comprehensible and manoeuvrable layouts. Examples of such simplifications are given in Figure 8-10.

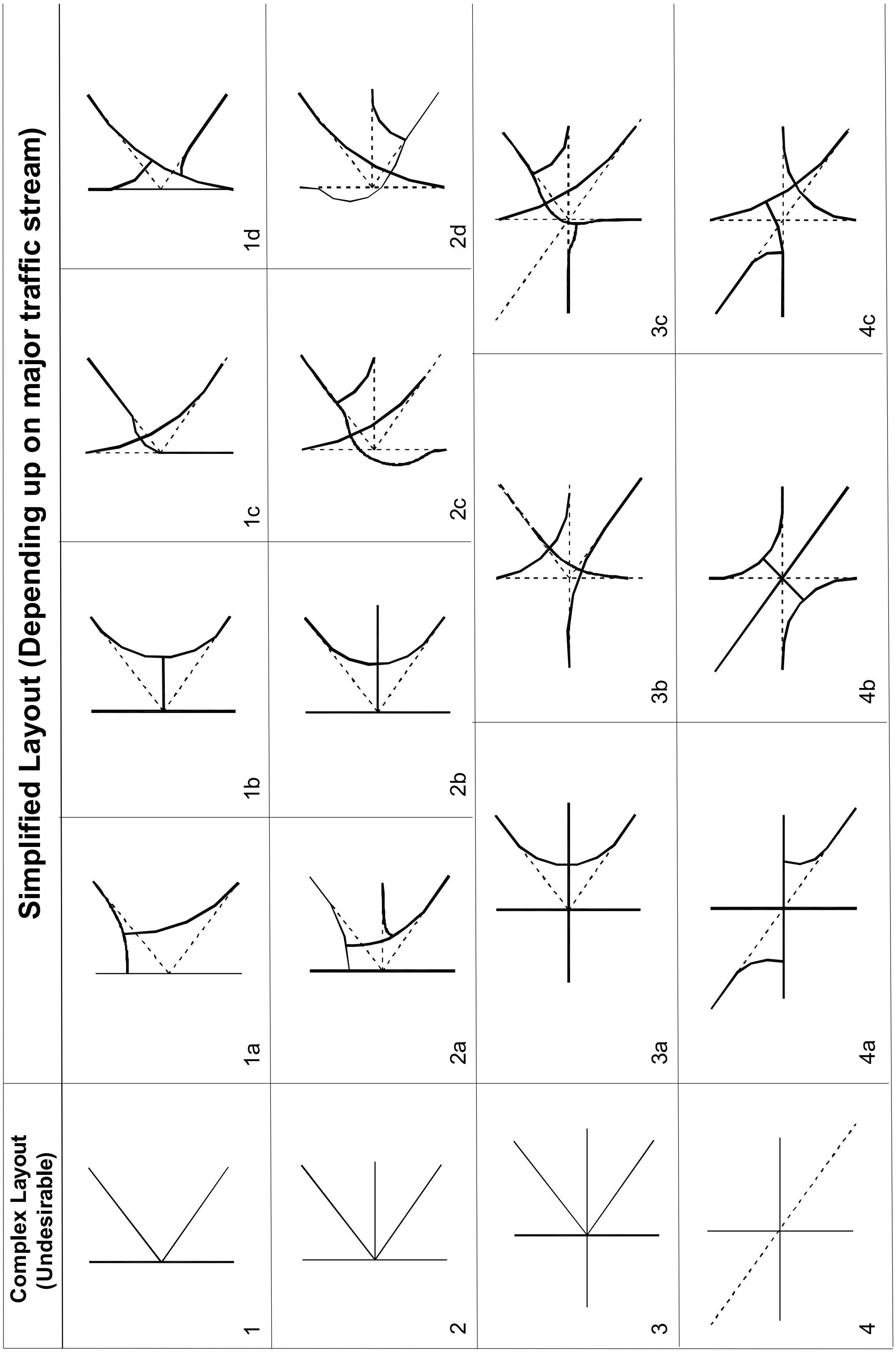


Figure 8-10: Example of the Simplification of Complex Junctions

8.6 Principles Of Junction Design

The basic principles of good intersection design are:

- minimize the number of conflict points, and thus the risk of accidents;
- give priority to major traffic movements, through alignment, signing and traffic control;
- separate conflicts in space or time;
- control the angle of conflict; crossing streams of traffic should intersect at a right angle or near right angle;
- define and minimise conflict areas;
- define vehicle paths;
- ensure adequate sight distances;
- control approach speeds using alignment, lane width, traffic control, or speed limits;
- provide clear indication of right-of-way requirements;
- minimise roadside hazards;
- provide for all vehicular and non-vehicular traffic likely to use the intersection, including goods vehicles, public service vehicles, pedestrians and other vulnerable road users;
- simplify the driving task, so that road users have to make only one decision at a time; and,
- minimise road user delay.

Having selected the basic junction layout and checked that it offers sufficient capacity, it is necessary to adapt this basic layout in accordance with the following principles to ensure that a safe, economic and geometrically satisfactory design will be produced.

8.6.1 Distance between Adjoining Junctions

The minimum distance between consecutive junctions 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 centerlines equal to the passing sight distance appropriate for the Junction Design Speed plus half the length of the widened major road sections at each junction, or

(ii) A grouping of minor road junctions into pairs to form staggered T-junctions and a distance between pairs as in (i) above.

8.6.2 Visibility Splays

At major/minor priority junctions visibility splays to the standards described below should be provided at all new junctions and aimed at for existing junctions.

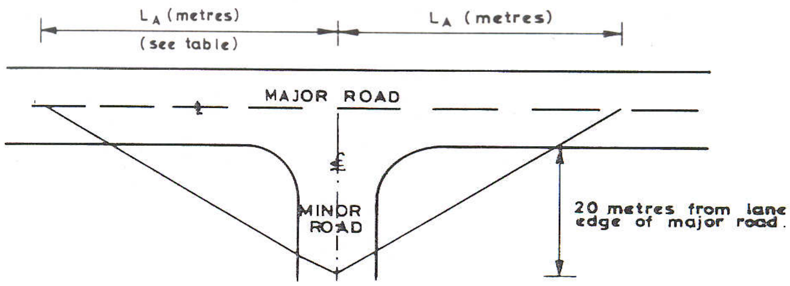
The visibility splays for both the. “Approach Conditions” (Figure 8-11) and the “Stop Conditions” (Figure 8-12) should be provided.

On the minor road, particularly where the approach to the junction is on a horizontal curve, the visibility of traffic signs is essential and a visibility splay in accordance with Figure 8-8 must be provided.

Where site conditions make it impossible to improve an existing junction to these standards, at least the visibility splays for the “Stop Condition” must always be provided.

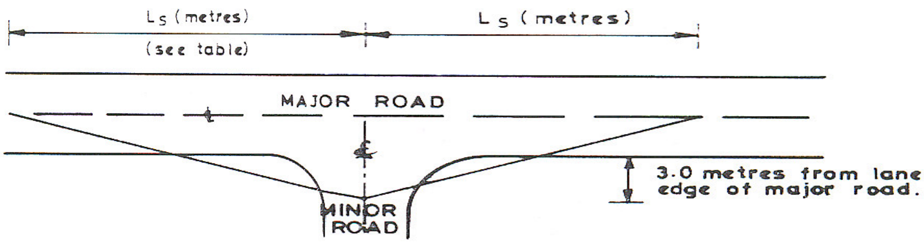
8.6.3 Turning Lanes

Left and right turning lanes are of particular value on the higher speed roads when a vehicle slowing down to turn and leave the major road and may impede following vehicles.



Junction Design speed (km/h)	40	50	60	70	80	90	100	120
Length L_A (metres)	110	135	160	185	210	235	260	310

Figure 8-11: Visibility Splays for “Approach” or “Yield” Conditions



Junction Design speed (km/h)	40	50	60	70	80	90	100	120
Length L_S (metres)	120	145	185	230	280	340	400	500

Figure 8-12: Visibility Splays for “Stop” Conditions

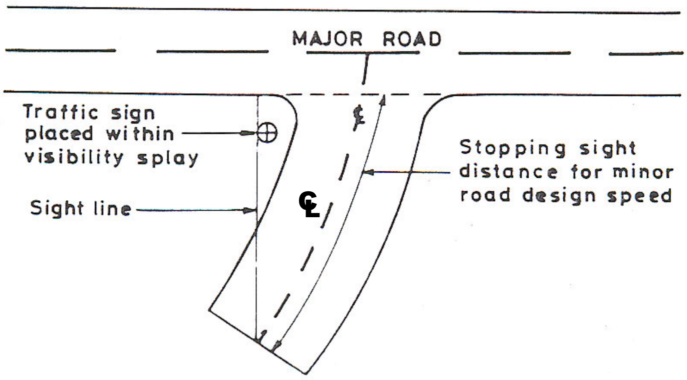


Figure 8-13: Minor Road Approach Visibility Requirements

(i) Left Turn Lanes

Left turn lanes, comprising diverging sections and deceleration sections, shall be provided under any of the following conditions:

(a) On dual carriageway roads.

(b) When the Junction Design Speed is 100 km/h or greater and the A.A.D.T. on the major road in Design Year 10 is greater than 2000 p.c.u.

(c) When the A.A.D.T. of the left turning traffic in Design Year 10 is greater than 800 p.c.u.

(d) Where junctions are sited on left-hand bends and perception of the junction for major road traffic would be greatly improved by its inclusion.

(e) On four or more lane undivided highways.

The minimum lengths for diverging sections are given in Table 8.2 and shall be formed by direct tapers.

Table 8-2: Minimum Length of Diverging Section (Lc)

JUNCTION DESIGN SPEED (km/h)	120	100	80	≤70
LENGTH OF DIVERGING SECTION (m)	60	50	40	30

The minimum lengths for deceleration sections are dependent upon the Junction Design Speed, the exit radius from the major road into the minor road and the approach gradient of the major road. Where left turn lanes are required, the exit radius shall be 25 meters and the minimum length of deceleration sections shown in Table 8-3 shall be used.

The lengths given in Table 8-3.apply for approach gradients of -2% to +2%; where approach gradients greater than 2% are encountered the lengths from Table 8-3 shall be multiplied by the adjustment factor given in Table 8-4 but the adopted lengths must never be less than 30 meters.

The width of the deceleration lane shall be the same as the major road approach lane.

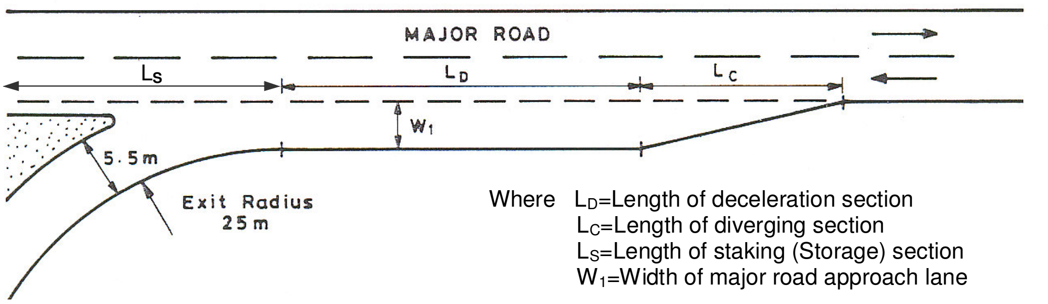
Table 8-3: Minimum Lengths for Left Turn Deceleration LD (for approach gradient of -2% to 2%).

JUNCTION DESIGN SPEED (km/h)	120	100	8O	≤ 70
LENGTH OF DECELERATION SECTION (m)	110	70	50	30

Note: Where vehicles are required to stop before entering the minor road, the deceleration section length should be equal to that used for a right turn lane as given in Table 8-5.

Table 8-4: Adjustment Factors for Approach Gradient

	% DOWNGRADE				% UPGRADE			
	6	5	4	3	3	4	5	6
ADJUSTMENT FACTOR	1.4	1.3	1.25	1.2	0.9	0.9	0.85	0.8



Note: For details of traffic islands and carriageway edges to minor road (See section 8.6.6)

Figure 8-14: Layout for Left Turn Lane

<u>ii) Right Turn Lanes</u>

A separate lane for right turning traffic (i.e. traffic turning right from the major road into the minor road) shall be provided under any of the following conditions:

(a) On dual carriageway roads.

(b) When the Junction Design Speed is 100 km/h or greater and the A.A.D.T, on the major road in Design Year 10 is greater than 1500 p.c.u.

(c) When the ratio of the major road flow being cut to the right turning flow exceeds the values given on Figure 8-15.

(d) On four, or more lane undivided roads.

A right turn lane will consist of a diverging (taper) section, a deceleration section and a storage section.

The minimum lengths for diverging sections are as for left turn lanes and are given in Table 8-2 and shall be formed by direct tapers.

The minimum lengths for deceleration sections for approach gradients of -2% to +2% are given in Table 8-5. For approach gradients greater than 2% the adjustment factors given in Table 8-4 for left turn lanes shall be applied but the adopted lengths shall never be less than 30 meters.

Table 8-5: Minimum Length for Right Turn Deceleration Section (for approach gradients of -2% to 2%)

JUNCTION DESIGN SPEED (km/h)	120	100	80	70	60	50	40
LENGTH OF DECELERATION SECTION (m)	160	105	85	75	70	50	30

The lengths of storage sections for right turning traffic are given in Table 8-6.

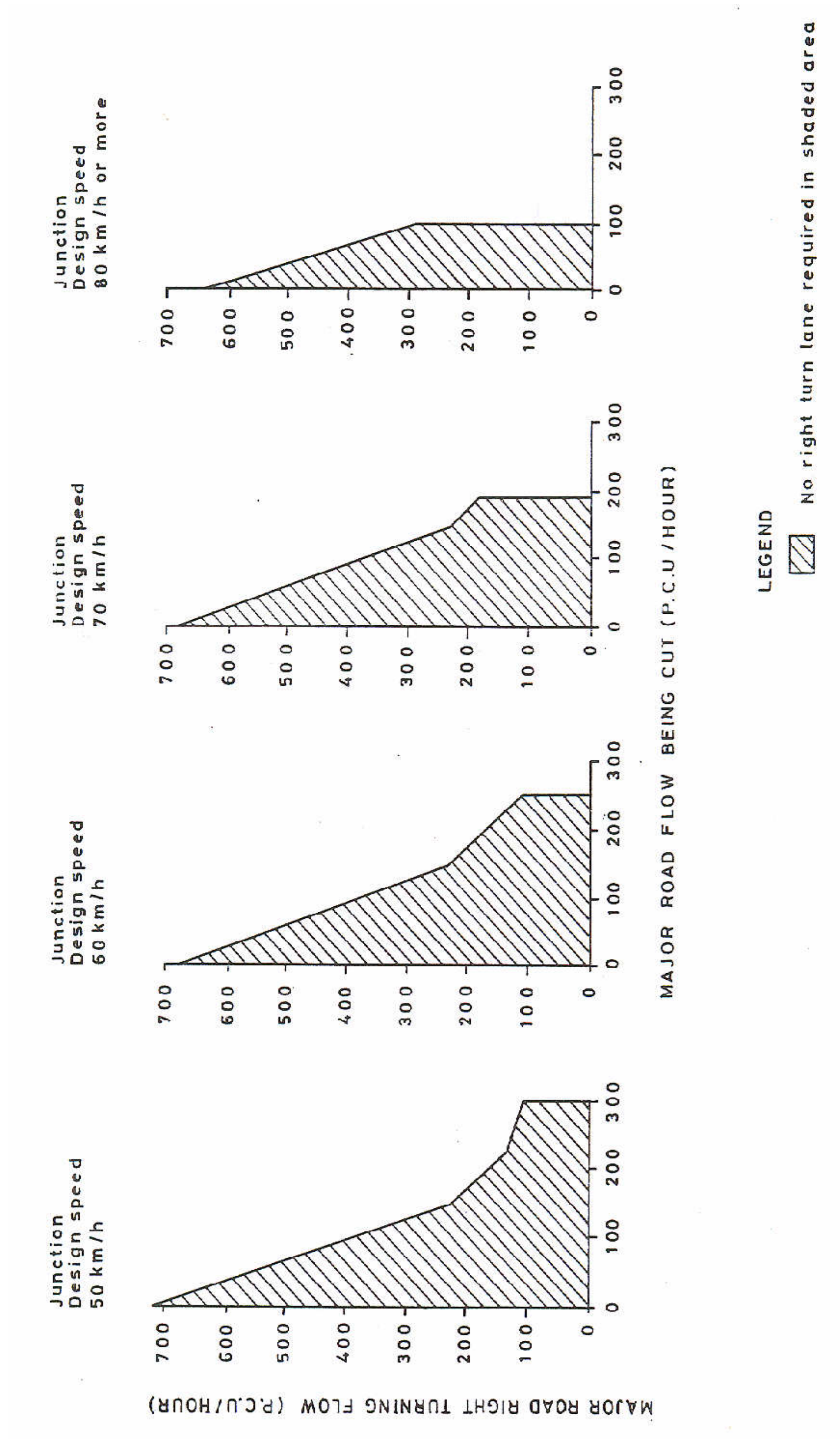


Figure 8-15: Criteria for Determining the Provision of Right Turn Lanes

Table 8-6: Length of Storage Section for Right Turning Traffic.

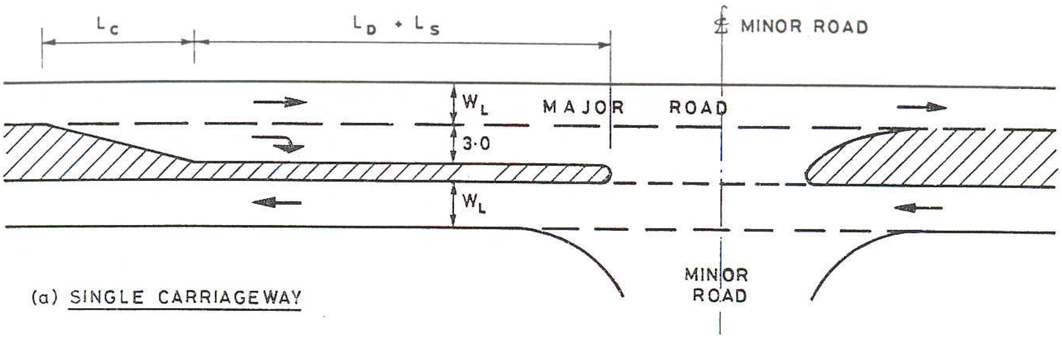
Traffic Turning Right [p.c.u./h]	Length of Storage Section [m]
0 - 150	20
151 - 300	40
over 300	N x 9.75 (where N is number of p.c.u. turning right per two minutes)

The width of the deceleration and storage sections shall be 3.0 meters. For details of carriageway widening to accommodate right turning lanes, see Section 8.6.7.

Provision of right turn lanes can be made for both the major and minor road. On single road way roads where a right turn lane is to be provided, a painted central reserve shall always be used.

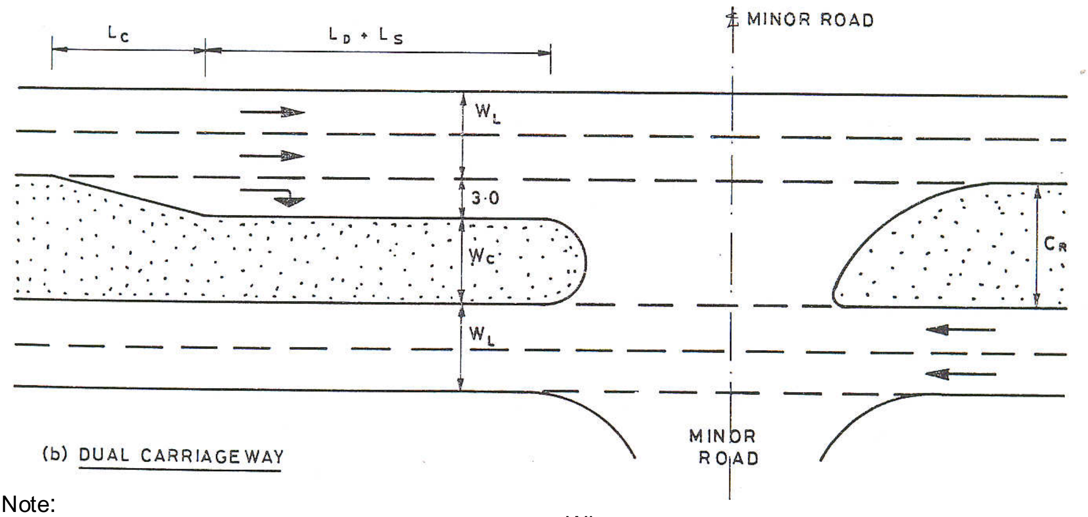
In order to accommodate a right turn lane on a single carriageway road, the road has to be widened to provide the required width. The widening shall be designed so that the through lanes are given smooth and optically pleasing alignment. The width of the through lanes at the junction shall be the same as the approach lanes. The widening shall be provided by the deviation of both through lanes from the centerline. This shall be achieved by introducing a taper of 100 meter length at the beginning and ending of the widening.

Figures 8-16(a) and (b) show typical layouts for right turn lanes to single and dual carriageway roads respectively.



Note:
1. Central reservation to be formed by Road markings (see section 8.6.5)
2. For details of minor road widening and traffic islands (see section 8.6.6)
3. For details of major road alignment and widening (see section 8.6.7).

Where LC = Length of diverging section
LD = Length of deceleration section
LS = Length of storage section
WL = Width of through carriageway



Note:

1. Edges of central reserve may be kerbed in vicinity of junction (section 8.6.5) if raised krebs are used they must be set back 0.25m from lane edges (see section 8.6.6).
2. For details of minor road widening and Traffic islands (see section 8.6.6)

Where LC = Length of diverging section
LD = Length of deceleration section
LS = Length of storage section
WL = Width of through carriageway
WC = Width dependent upon width of central reserve
CR = Normal central reserve width

Figure 8-16: Layouts for Right Turn Lanes

8.6.4 Major Road Cross Section

Excessive intersection widths should be avoided in order to discourage high speeds and overtaking. If section of major road is made across the full width of right turn lane it will have the view shown in Figure 8-17.

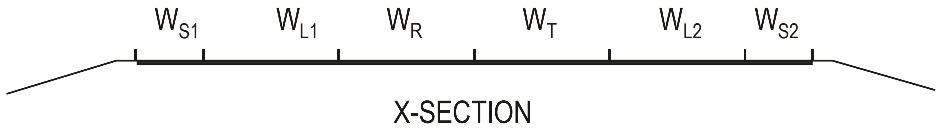


Figure 8-17: Major road cross section

The maximum cross-section width is:

$$\mathbf{W = W_{S1} + W_{L1} + W_R + W_T + W_{L2} + W_{S2}.}$$

- through lanes width, $\mathbf{W_{L1}}$ and $\mathbf{W_{L2}}$, should normally be unchanged through the intersection. However, if they are ≥ 3.5 m on the approaches to the intersection, they could be slightly narrowed to discourage high speeds and overtaking, otherwise the width should be kept.
- the right turning lane width, $\mathbf{W_R}$,should normally be 3.0 m.
- the traffic island width, $\mathbf{W_T}$, depends on island type:
  - island created with road markings: normally 0.35 m for double centre line
  - kerbed island: space needed for:

    Pass left side only traffic sign (sign no. R75.1), 0.4 to 0.9 m
    lateral clearances, minimum 0.3 m
    an inner hard shoulder, if needed, in the opposite direction, 0.25 to 0.5 m for an edge line

The total width will vary from minimum 1.2 m to 2.0 m.

- paved shoulder widths, $\mathbf{W_{S1}}$ and $\mathbf{W_{S2}}$, are as per the design class of road, should be narrowed in two lane roads to 0.5 m in order to discourage overtaking in the intersection. Separate footways should be provided for pedestrians so that they do not have to walk on the shoulder.

Where there are many long vehicles turning right into the main road consider extending the width of the island so that it provides them with some protection if the driver decides to make the turn in two stages (i.e. crosses one major road traffic direction at a time).

8.6.5 Central Reserves

The widening of the central reserve of a dual carriageway in the vicinity of a junction may be required to allow more space for crossing vehicles to wait in safety. A width of 10 metres will normally provide the appropriate balance between safety and cost.

To ensure that vehicles can turn right without difficulty to, or from, a major road, the gap in the central reserve should extend beyond the continuation of both kerb lines of the minor road to the edge of the major road. Normally an extension of 3.0 meters will be sufficient but each layout should be checked. The ends of the central reserve should be curved to ease the paths of turning vehicles.

On single carriageway roads where a right turn lane is to be provided, a hatched central reserve shall always be used unless lighting is provided, in which case the central reserve may be kerbed.

On dual carriageway roads the central reserve in the vicinity of junctions should be edged with flush kerbs unless lighting is provided, in which case raised kerbs may be used.

8.6.6 Traffic Islands and Minor Road Widening

Traffic island is a defined area between traffic lanes for the control of vehicle movements and which may also be used as a pedestrian refuge. It may take the form of an area delineated by barrier kerbs or a pavement area marked by paint or a combination of these. Traffic islands should be provided where necessary, at major/minor priority junctions, for the following reasons:

(i) To assist traffic streams to intersect or merge at suitable angles.

(ii) To control vehicle speeds.

(iii) To provide shelter for vehicles waiting to carry out certain maneuvers such as turning right.

(iv) To assist pedestrians to cross.

Islands are either elongated or triangular in shape and are situated in areas not normally used as vehicle paths, the dimensions depending upon the particular junction or bus layout. Traffic islands bordered by raised kerbs should not be used in the major road unless lighting is provided but can be used without lighting in the minor road. To enable raised islands to be clearly seen they should have an area of at least 4.5m2 and where necessary additional guidance should be given by carriageway markings in advance of the nose supplemented, if necessary, by speed humps.

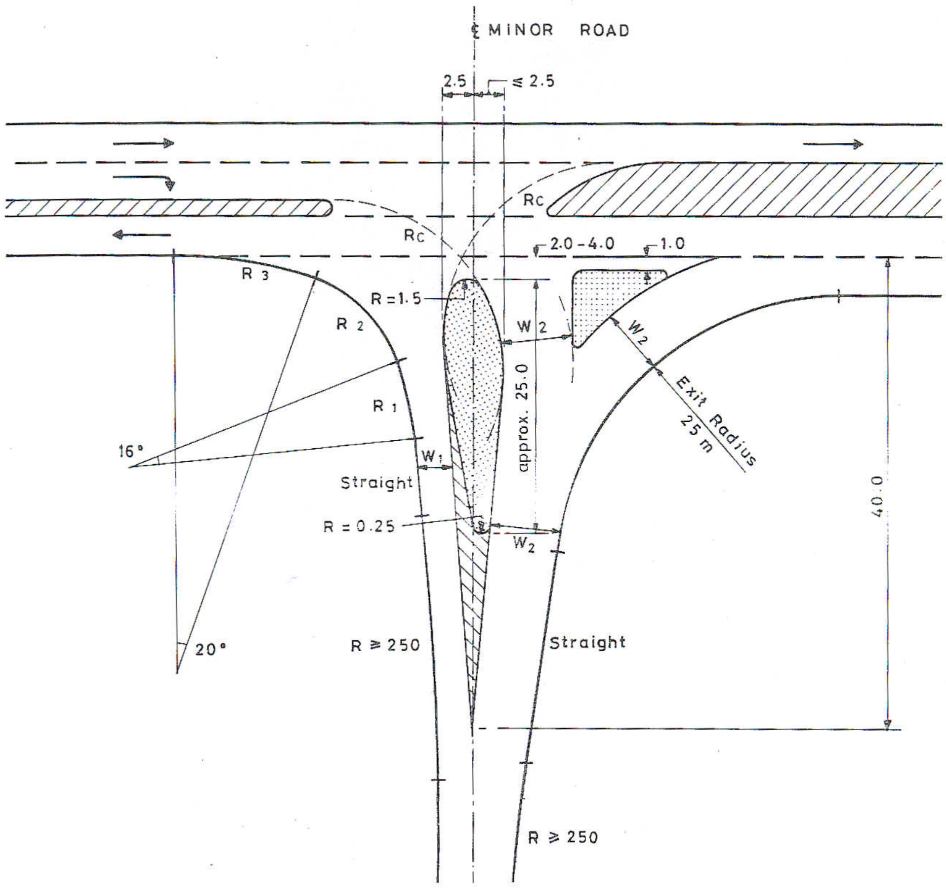
The layout of an island is determined by the edges of the through traffic lanes, turning vehicles and the lateral clearance to the island sides. The edges of all raised islands parallel to traffic lanes must be set back from the traffic lane edges by a minimum of 0.30 meters.

Generally two basic layouts for traffic islands and minor road widening will be used but each junction should be carefully checked to ensure that adequate clearance is given for the types of vehicles expected to use the junction; see Section 5 for details of design vehicle turning characteristics.

Junction Layout Type A (channelised intersection), as shown on Figure 8-18, is to be used whenever a separate right turn lane is required in accordance with the requirements of Section

8.6.3(ii). It should be noted that this layout also makes provision for a left turn lane. However, the left turn lane may be omitted if the conditions described in Section 8.6.3(i) for its provision are not met; in such cases the exit radius should be amended to comply with the triple radius exit curve shown for Junction Layout Type B and the triangular island omitted.

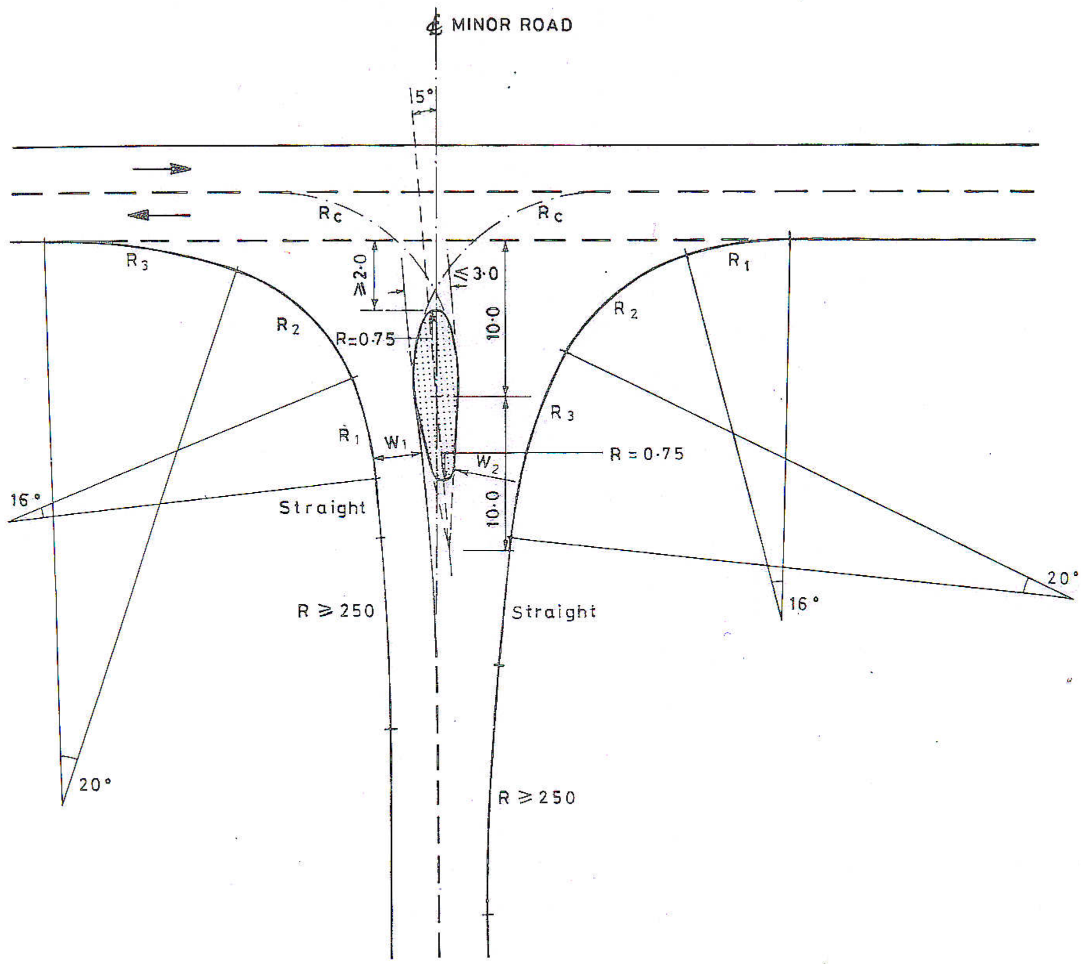
Junction Layout Type B (partly channelised interesection), as shown on Figure 8-19, is to be used whenever a separate right turn lane is not required. The layout shown on Figure 8-18 does not include a left turn lane but such a lane may be included if the conditions for its provision, as described in Section 8.6.3 (i), are met; in such cases the triple radius exit curve should be replaced by the 25 meter exit radius and an additional traffic island as shown for Junction Layout Type A.



NOTES:

1. R_C = Central radius dependent up on vehicle turning characteristics (minimum turning radius) recommended value 15m
2. The ratio R_1:R_2:R_3 to be 2:1:3 and the recommended value for R_2 is 12.0m.
3. W_1 shall equal minor road lane width but shall not be less than 3.0m.
4. W_2 shall equal 5.5m (Excluding offsets to raise kerbs)
5. For detail of major road widening see section 8.6.7

Figure 8-18:Junction Layout Type A



NOTES:

1. R_C = Central radius dependent up on vehicle turning characteristics
2. The ratio R_1:R_2:R_3 to be 2:1:3 and R_2 will be dependent of vehicle turning characteristics and proportion of large vehicles recommended range for R_2 is 8-12.0m.
3. W1 shall equal minor road lane width
4. W_2 shall be dependent up on vehicle turning characteristics.

Figure 8-19:Junction Layout Type A

8.6.7 Alignment and Widening of the Major Road

In order to accommodate a right turn lane on a single carriageway road the carriageway has to be widened to provide the required width. The width of the through lanes at the junction shall be the same as the approach lanes.

The widening shall be designed so that the through lanes are given smooth and optically pleasing alignments.

On straight alignments, the widening shall be provided by the deviation of the through lane opposite the minor road. This deviation shall be effected so as to avoid the appearance of an unsightly “bulge” in the horizontal alignment. This shall be achieved by introducing radii of 7,500m at the beginning and end of the widening, 1 in 45 tapers and a radius of between 5,000 and 10,000 meters as shown diagrammatically on Figure 8-20(a).

On curved alignments, a smooth alignment for the through lanes can be achieved by widening on the inside of the carve. This is done by introducing transition curves which approximate to 1 in 45 tapers as shown on Figure 8-20(b).

It is advisable to lengthen the island, if the intersection is located on a crest or in a horizontal curve, as this will make the intersection more visible to approaching traffic.

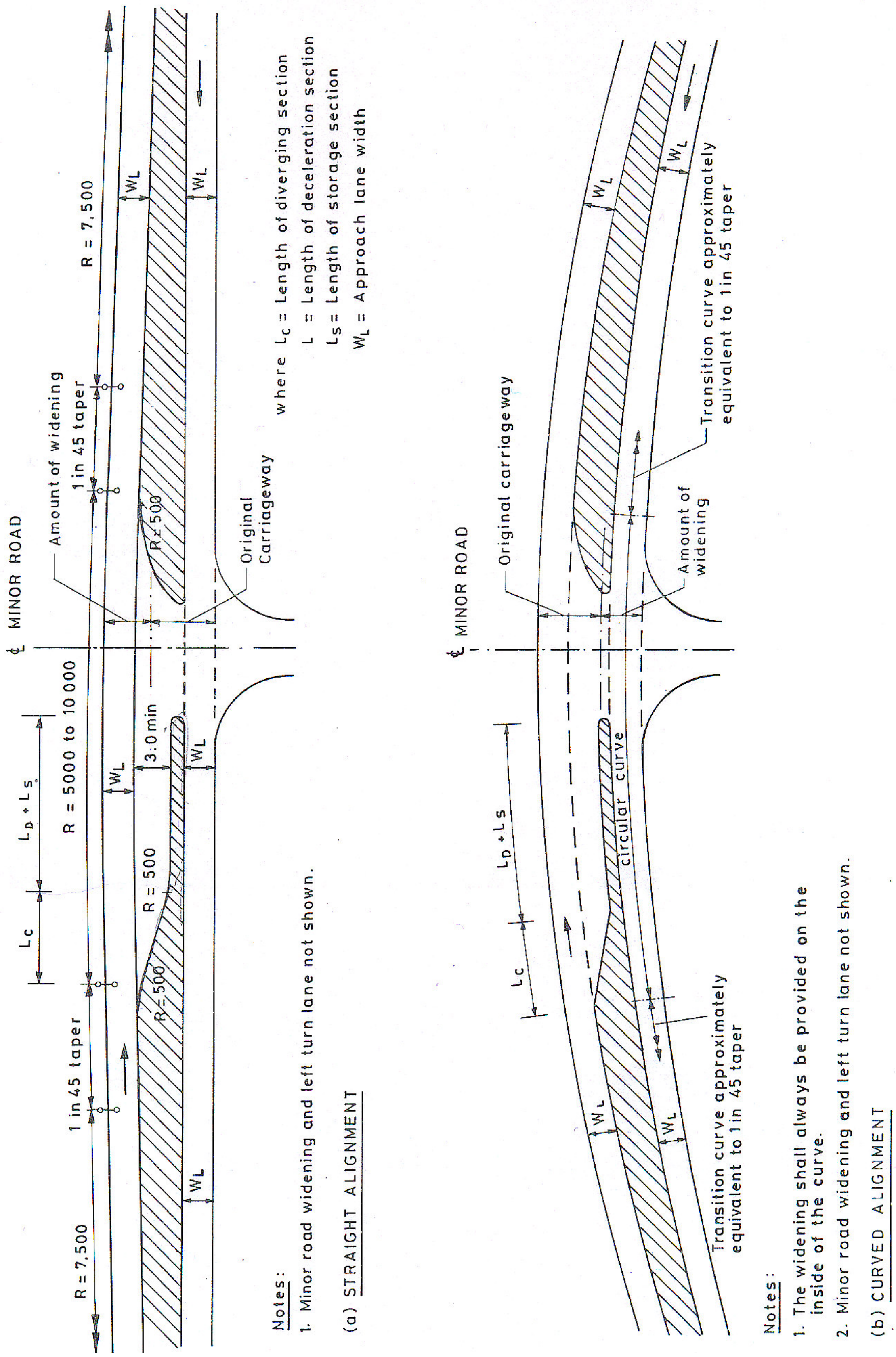


Figure 8-20: Application of Widening to the Major Road

8.6.8 Checklist for Junction Design

- ❑ 1. Will the junction be able to carry the expected/future traffic levels without becoming overloaded and congested?
- ❑ 2. Have the traffic and safety performance of alternative junction designs been considered?
- ❑ 3. Is the route through the junction as simple and clear to all users as possible?
- ❑ 4. Is the presence of the junction clearly evident at a safe distance to approaching vehicles for all directions?
- ❑ 5. Are warning and information signs placed sufficiently in advance of the junction for a driver to take appropriate and safe action given the design speeds on the road?
- ❑ 6. On the approach to the junction, is the driver clearly aware of the actions necessary to negotiate the junction safely?
- ❑ 7. Are turning movements segregated as required for the design standard?
- ❑ 8. Are drainage features sufficient to avoid the presence of standing water?
- ❑ 9. Is the level of lighting adequate for the junction, location, pedestrians, and the design standard?
- ❑ 10. Are the warning signs and markings sufficient, particularly at night?
- ❑ 11. Have the needs of pedestrian and non-motorized vehicles been met?
- ❑ 12. Are sight lines sufficient and clear of obstructions including parked and stopped vehicles?
- ❑ 13. Are accesses prohibited a safe distance away from the junction?
- ❑ 14. Have adequate facilities such as footpaths, refuges, and crossings, been provided for pedestrians?
- ❑ 15. Does the design, road marking and signing clearly identify rights of way and priorities?
- ❑ 16. Is the design of the junction consistent with road types and adjacent junctions?
- ❑ 17. Are the turning lanes and tapers where required of sufficient length for speeds and storage?

Date: ....

Designer..

Signature ....

8.7 Design of Roundabouts

The following design items concerning roundabouts are covered in this section:

1. General requirements;
2. Design principles;
3. Sight distances;
4. Central island and circulating carriageway;
5. Entries;
6. Exits;
7. Combination of entries and exits; and,
8. Pedestrian and cycle crossings.

8.7.1 General Requirements

The following features are generally considered necessary for a roundabout to perform safely and efficiently:

- it must be easily seen and identified when drivers are approaching it;
- the design must encourage drivers to enter the intersection slowly (<50km/h) and keep a low speed throughout – this is crucial to safety;
- the layout must be simple and easy to understand;
- it must be clearly signed and marked;
- adequate, but not excessive sight distance, must be provided at all entry points to enable the driver to observe the movements of conflicting vehicles, pedestrians and cyclists; and,
- lighting is very beneficial for safety.

8.7.2 Design Principle

Speed control

Approaching vehicles must be slowed down to 50km/h or less and this can be achieved in various ways including the use of a large centre island, offsetting the entry roads, and deflecting the entry roads sharply to the left as they join the circulating carriageway. In practice all roundabouts must be designed with some entry deflection.

To keep the speed down to 50 km/h or less, the roundabout should be designed so that it is not possible to drive through it on a path (see Figure 8.30) with a radius of more than 100m.

Number and alignment of entry roads

Roundabouts work best with four arms or entries, but they can also be used where there are three or five entries. More than five legs should not be considered.

Ideally, the entry roads should be equally-spaced around the perimeter with a minimum angle of 600 between them.

In three arm intersections, the angles between the entry roads can be adjusted by displacement of the central island from the intersection point of the centre lines of the connecting roads or by deflection of the road alignments.

In five arm intersections, the space for the extra connection can be created by making the central island elliptical or by increasing the radius of the central island to at least 20 m. However, elliptical central islands can be confusing.

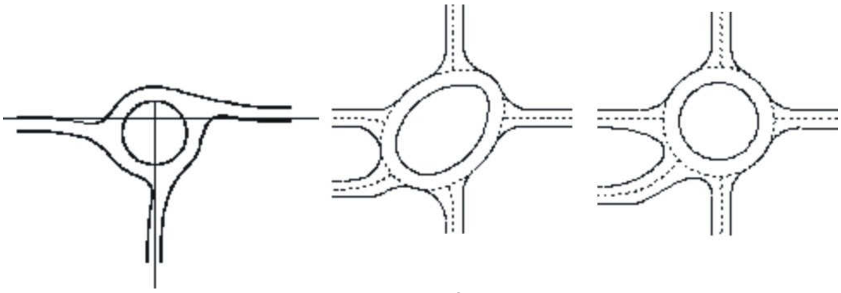


Figure 8-21: Three and five arm roundabout

8.7.3 Visibility and Sight Distances

Roundabouts should preferably be located where, approaching drivers can have a good overview of the roundabout with its entries, exits and circulating carriageway. Stopping sight distance must be available on all approaches.

Roundabouts can be made more visible from a distance by moulding the centre island into a dome and by planting the centre with shrubs not thicker than 100 mm in diameter. Advance direction signs (map-type with an accurate diagram of the roundabout approach and layout) are essential. Large chevron signs must be installed on the centre island opposite each approach.

The visibility splays shown in Figure 8.22 must be provided to allow drivers to judge whether it is safe to enter the roundabout. It must be possible to see vehicles at the preceding entry and the following exit as well as the nearest parts of the circulating carriageway. However, drivers should not be able to see the preceding entry from more than 15m before the “give way” line, as this might encourage excessive approach speeds.

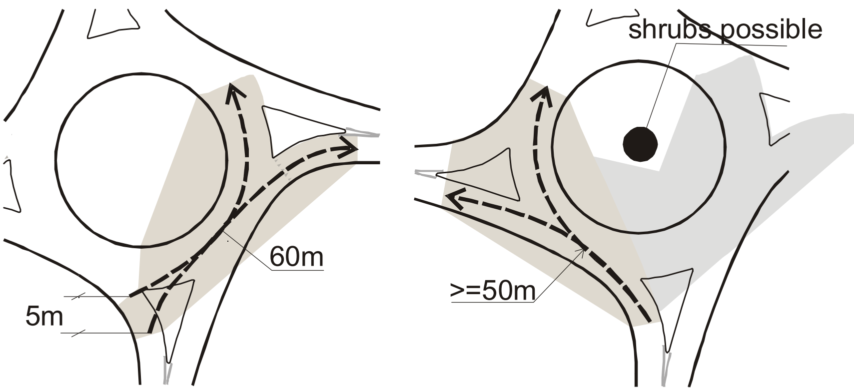


Figure 8-22: Required visibility for entering a roundabout

Once in the roundabout drivers must be able to see the area shown in Figure 8.23. Signs and landscaping on the centre island should be designed and located so that they do not obstruct the view more than absolutely necessary as illustrated to the right above.

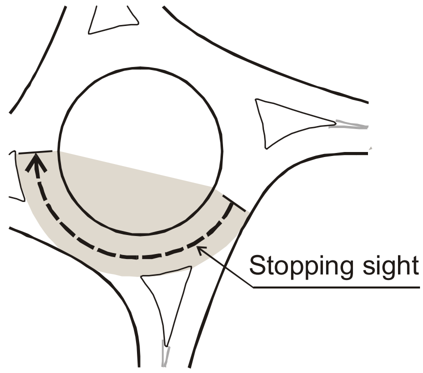


Figure 8-23: Required visibility for drivers within a roundabout

8.7.4 Centre Island and Circulating Carriageway

Definitions

The dimensions of roundabouts are defined by the following radii and widths shown in Figure 8.24:

- Edge of carriageway radius, Re
- Central island radius, Rc
- Inner central island radius, Ri
- Circulating carriageway width, B and,
- Traversable area (small roundabouts only).

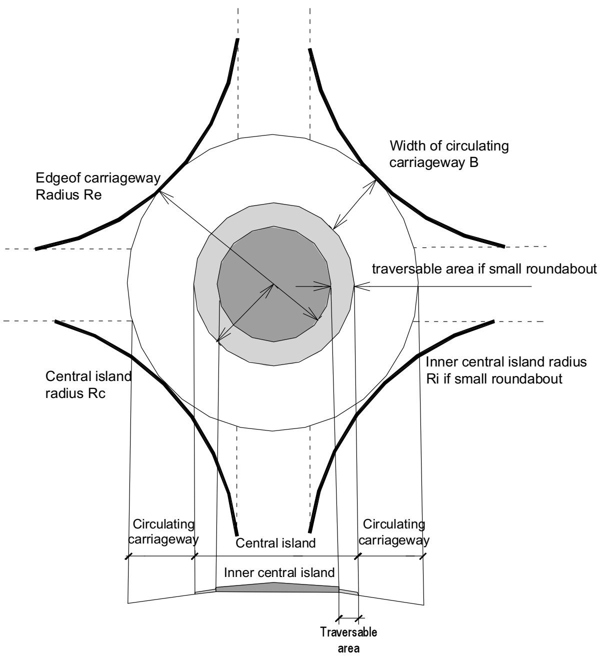


Figure 8-24: Roundabout radii and widths

Normal roundabouts (carriage way radius ≥ 18 m and centre island radius ≥ 10 m)

Central island radius

The central island radius should normally be between 10 metres and 25 metres. It is difficult to control speeds if the roundabout is larger than this, and this would mean that cyclists and other vulnerable road users would be at risk. In most cases the size of the site will determine the size of the roundabout.

There is no simple relation between the central island radius and safety and the possible capacity. The capacity is also depending on other design parameters, such as the width of the circulating roadway and the entry and exit angles and widths.

Width of the circulating carriageway

The width of the circulating carriageway depends on whether it is to be one lane or two lane. Normally, one lane roundabouts are designed for a semi-trailer, and two lane roundabouts are designed for a semi-trailer and a passenger car.

For normal (central island radius 10 metres or greater) one lane roundabouts and two lanes roundabouts the central island radius, the edge of carriageway radius and the width of the circulating carriageway are determined by the diagram in Figure 8.25.

Check that the circulating carriageway is no more than about 1.2 x the maximum entry width. Very wide carriageways encourage unsafe speeds.

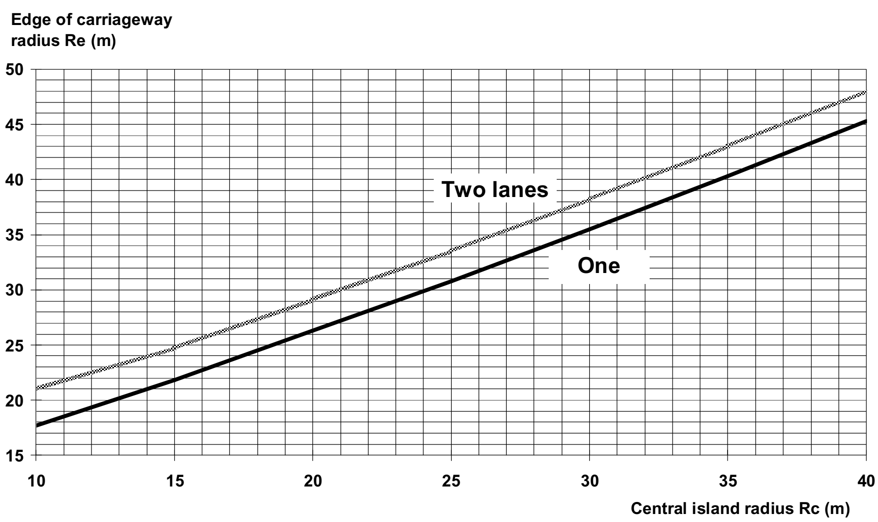


Figure 8-25:Radius of central island and circulating carriageway radius in normal roundabouts

(consequence of the design vehicles used based on International agreements on vehicle manoeuvrability)

Small roundabouts (carriageway radius < 18 m)

Where space is limited, such as in built-up areas, a slightly different design of roundabout is needed in order to accommodate long trucks without sacrificing speed controlling features.

Island radius

Small roundabouts shall have an inner central island radius of at least 2 metres.

Width of the circulating carriageway

The problem with small roundabouts is that it is difficult to control car speeds because the circulating carriageway has to be very wide in order to accommodate semi-trailers and long vehicles. The solution is to build a centre island with an outer fringe which is traversable by long vehicles – see Figure 8.24. The traversable area should be a maximum of 40mm high, have a rough surface (to discourage light vehicles), and be edged with a mountable kerb. The intention is that light vehicles will go around the outside of the traversable area, thus forcing the drivers to travel slowly. Drivers of long vehicles will be able to negotiate the roundabout by letting the rear wheels cross the traversable area.

Advice on central island radii and traversable area are given in Figure 8.26.

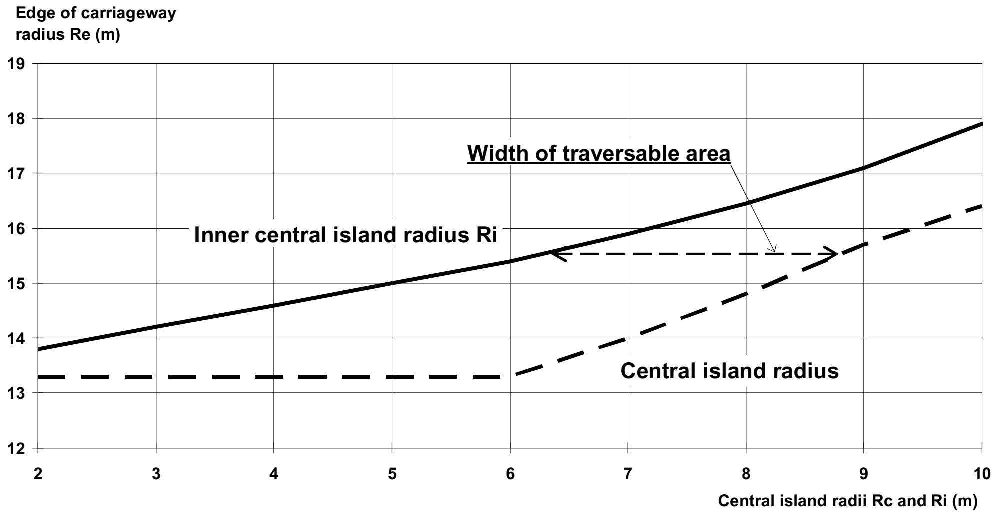


Figure 8-26: Roundabout radii in small roundabouts

8.7.5 Entries

Number of entry lanes

One lane roundabouts are preferred from a safety viewpoint. For higher traffic volumes, a 2-lane circulating roadway may be necessary. The diagram Figure 8.27 shows the need for two lanes.

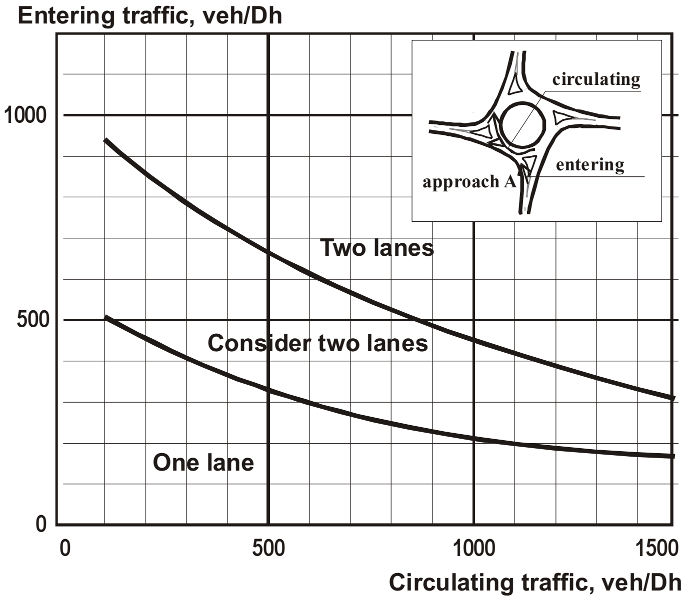


Figure 8-27:Number of entry lanes

The need for two lanes must be checked for each entry's entering and circulating flows during the design hour. If two lanes are necessary for one entry, the whole roundabout should be designed with two lanes. An alternative design to increase the capacity for one entry can be to use a separate left turn lane as shown in Figure 8.28. With this arrangement care must be taken to ensure good visibility and signing at the merge – otherwise cyclists and other vulnerable road users could be put at risk.

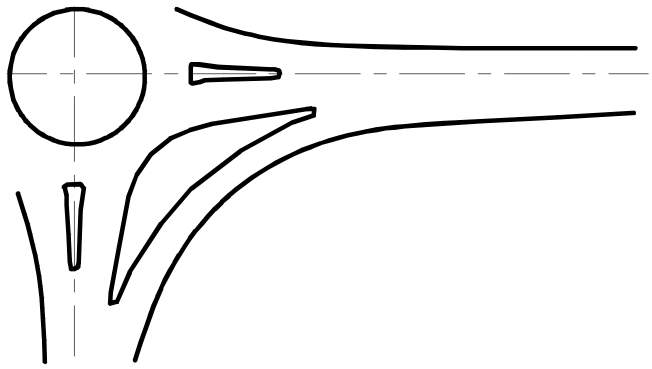


Figure 8-28: Alternative design to increase the capacity for an entry

Approach alignment

As previously stated, entry deflection is essential in order to reduce the speed of approaching vehicles to 50km/h or less. The size of the deflection is dependent on the alignment of the entry and should normally be at least one lane wide (3.5 m). Figure 8.29 shows one way of achieving entry deflection. Avoid making the deflection too sharp as this could cause vehicles to overturn or overshoot (i.e. driver unable to stop at the "give way" line).

The entry road must be level with the circulating carriageway for a distance of at least 15 m before the "give way" line.

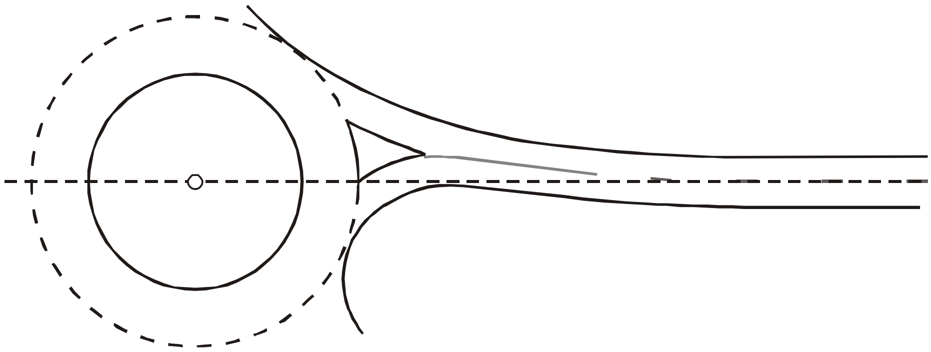


Figure 8-29: Design of approach deflection

Entry Radius

The entry radius should normally be in the range 15 – 20 m. It should never be less than 10m. Large entry radii will result in inadequate entry deflection and must not be used.

Entry width

The entry width is depending on the main entry radius. The entry widths in Table 8.7 should normally be used for one and two lanes roundabouts respectively. The transition to normal lane width should be at least 30 metres long.

Table 8-7: Entry Widths

Number of lanes	Design vehicle(s)	Entry width	
		Entry radius ≤15 m	Entry radius >15 m
1	Semi-trailer	6.5 m	6.0 m
2	Semi-trailer + passenger car	10.0 m	9.5 m

8.7.6 Exits

Number of exit lanes

The number of exit lanes can be decided according to Table 8.8.

Table 8-8: Number of Exist Lanes

Exiting traffic veh / Dh	Number of lanes
< 750	One
750 – 1 050	Consider two
1 050 – 1 500	Two

Exit curve

The exit should be designed to give smooth traffic flow. The main radius in the exit curve should be between 50 and 100 metres for a normal roundabout. If there is a pedestrian crossing on the exit the radius should be 50m or smaller, in order to control speeds.

Exit width

The exit widths in Table 8.9 should normally be used for one and two lanes roundabouts respectively. The transition to normal lane width should be 75 - 100 metres long.

Table 8-9: Exit Widths

Number of lanes	Design vehicle(s)	Exit width
1	Semi-trailer	5.5 m
2	Semi-trailer + passenger car	7.0 m

8.7.7 Combination of Exit and Entry Curves

Driving paths

The alignment of the connecting roads can make it necessary to adjust the exit and entry curve radii. If larger radii than normal are used, it must be checked that all possible 2 metres wide driving paths for passenger cars fulfill the requirement R1<=R2<=R3<=100 metres to achieve speed control, see Figure 8.30

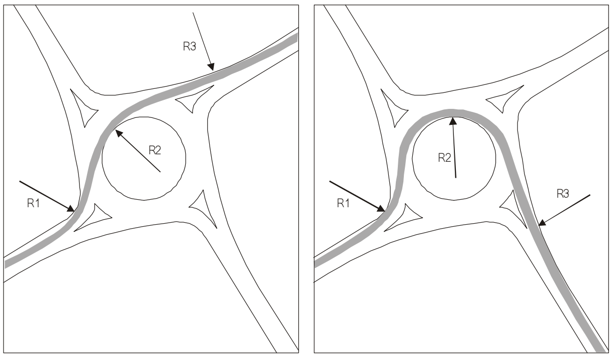


Figure 8-30: Driving paths for passenger cars

8.7.8 Alignment between Entry and Exit

It is preferable to avoid reverse curvature between the entry and the following exit - see Figure 8.31. For roundabouts with big central islands or long distances between entry and exit, this can be difficult to avoid. If possible, the alignment of the connecting roads should be adjusted.

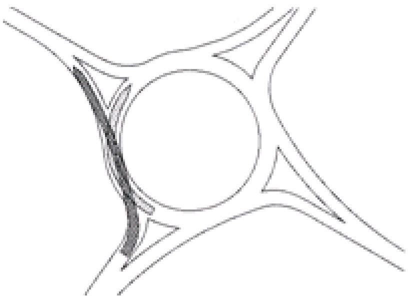


Figure 8.31: Alignment between entry and exit

8.7.9 Pedestrian and Cycle Crossings

Pedestrian/cycle crossings are normally placed according to one of the two alternatives shown in Figure 8.32.

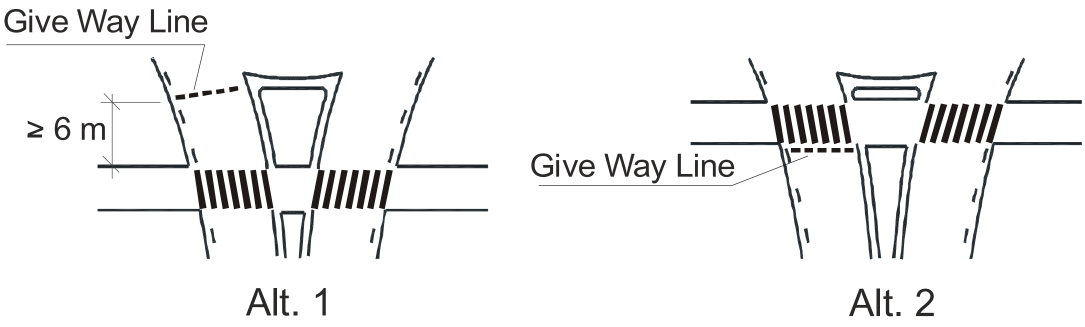


Figure 8-32: Location of pedestrian crossings

In Alt 1 the give way line is placed after and in Alt 2 before the pedestrian crossing. The advantages and disadvantages with the Alt 1 compared to Alt 2 are mainly the following:

With a distance between the crossing and the give way line, vehicles can yield for the pedestrian crossing and the roundabout separately. This improves capacity. The traffic safety effects are questioned. Some traffic safety researchers claim Alt 2 to be superior.

With the pedestrian crossing at a distance from the roundabout, an exiting vehicle can give way to a pedestrian without blocking the roundabout with obvious capacity advantages.

A disadvantage is that the traffic island may have to be extended and widened to accommodate pedestrians and cyclists. Another disadvantage is that pedestrians have to make a bigger detour.

8.7.10 Capacity of Roundabouts

The capacity of a roundabout is governed by the capacity of each individual weaving section.

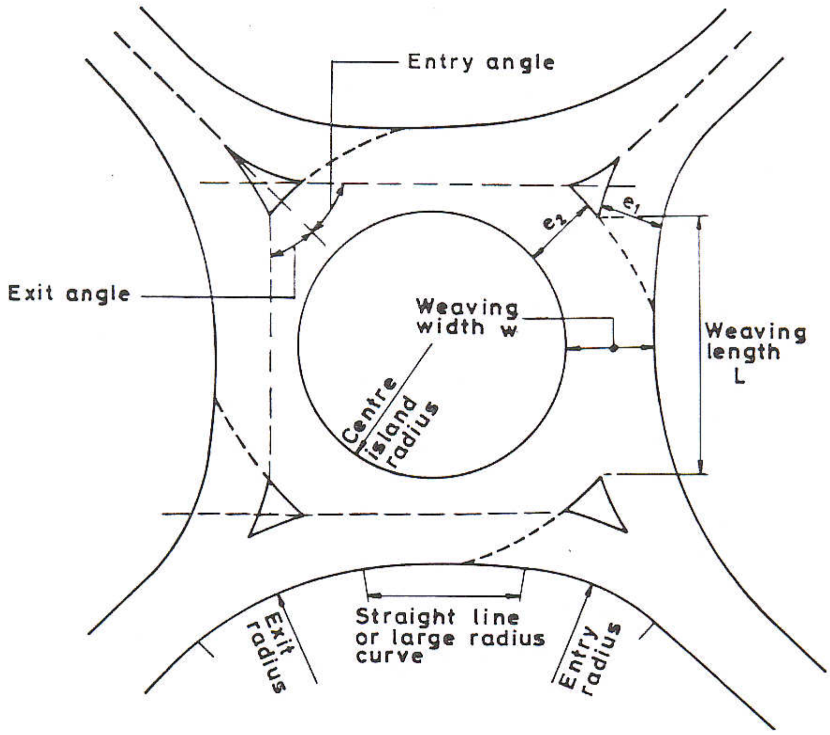
The capacity of a weaving section between entries, when designed in accordance with the foregoing principles, is calculated from the formula:

$$Q_p = \frac{240w\left(1+ e/w\right)}{1+ w/L}$$

where:

Q_p = the practical capacity [p.c.u./hour];
w = the width of weaving section (the difference between edge of carriageway (Re) and central island radius (Rc) [m];
e = the average width of entries to the weaving section [m]; and,
L = the length of weaving section [m].

The actual design capacity of the weaving section should not exceed 85% of the practical capacity, Qp.



The following steps may be followed in laying out a trial geometry for a roundabout:

1. Select the general design criteria to be used;
2. Select the appropriate design vehicle for the site;
3. Adopt a minimum design vehicle turning radius;
4. Determine from traffic flows the number of lanes required on entry, exit and circulation;
5. Identify the needs of pedestrians;
6. Identify the location of controls such as right-of-way boundaries, utilities, access requirements, and establish the space available;
7. Select a trial central island diameter and determine the width needed of the circulating carriageway;
8. Draw the roundabout;

9. Check that the size and shape is adequate to accommodate all intersecting legs with sufficient separations for satisfactory traffic operations;
10. Lay out the entrance/exit islands;
11. Check the achievement of adequate deflection (Figure 8-30). Adjust as required;
12. Check site distances at approaches and exits;
13. Layout lane and pavement markings;
14. Layout lighting plan; and,
15. Layout sign plan.

8.8 Design Of Signalised Intersection

8.8.1 Introduction

This section deals with geometric layout of signalised intersections and connections to the signal control strategy. Close co-operation is necessary with the signal control and lighting engineers throughout the design process, especially in the early stages, to optimize and coordinate geometric layout and signal control strategy.

Signal control at an intersection, properly designed, can enhance traffic safety and efficiency by reducing congestion and conflicts between different vehicle movements. The major advantages compared to priority-controlled intersections are:

- the maximum waiting time is fixed and known (if capacity is not reached);
- the available capacity is distributed fairly between approaches; and,
- the driver on the minor road does not have to make a judgment on when it is safe to proceed.

Most of the safety problems that arise with signalised intersections are related to drivers passing the signal at red, and rear-end collisions at signal changes from green to red. This has implications for signal visibility and timings.

Primary conflicts between motor vehicles must be separated in a signalised intersection, see Figure 8.34 below. Motor vehicles passing a steady green signal or green arrow signal must not encounter any primary conflicts, but lower order conflicts, i.e. with turning vehicles, may be acceptable in some circumstances.

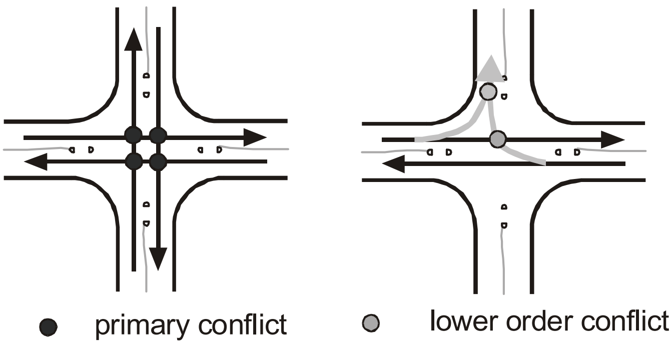


Figure 8-34: Primary motor vehicle conflicts

The control strategy of a signalised intersection is called the phases or the stage sequence. An example of a stage sequence for a T-intersection with a protected right turn (controlled by a green arrow) - is shown in Figure 8.35.

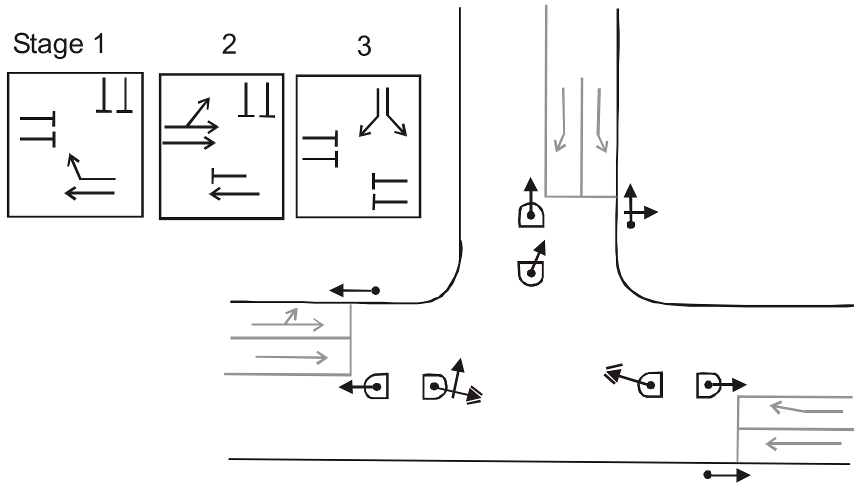


Figure 8-35: Protected right turn stage sequence

The control strategy can work on fixed timings or be vehicle-actuated - adapting to traffic conditions by detectors. Vehicle actuated (demand-responsive) signals are much more efficient, and because of this drivers are more likely to comply with them. Each stage has a minimum and maximum green time. There should always be an intergreen period between conflicting stages to allow for safe stage changes. The length of the intergreen depends mainly on the size of the intersection, speed limit and whether pedestrians and cyclists are involved. The time period between two consecutive starts of the same stage is the cycle time.

8.8.2 Control Strategy and Layout

Signalised intersections should normally be restricted to roads with a speed limit 50 km/h. Where signals are needed on roads with speeds higher than 50km/h additional equipment is needed to ensure safety. Signals should never be installed on roads where the speed limit is higher than 70km/h.

Protected right turns are preferable from a safety viewpoint. They give positive control and are easy to understand. The disadvantage is that they use up significant intersection capacity, so waiting times are longer.

Pedestrian crossing signals may be provided at signalised intersections. They should have their own stage, during which there should be no conflicts with vehicle movements.

The number of traffic lanes with permitted traffic directions and signal control type with stages and location of signal heads should be decided due to capacity, traffic safety, road user costs, environmental and other impacts and investment and maintenance costs. The capacity analysis should be based on expected traffic volumes during the design hour, normally both morning and evening peaks.

The following safety requirements must be coordinated with the geometric layout:

- Protected right turns (i.e. without conflicts) must have right turn lanes

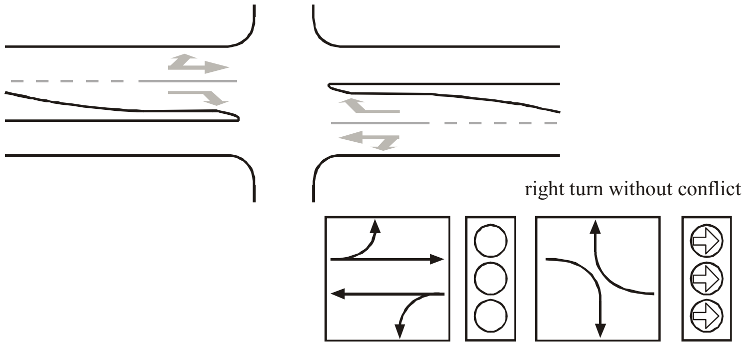


Figure 8-36: Right turn lanes with protected right turn

- Permissive right turns (i.e. with conflict with opposing straight forward traffic) can have separate lanes.

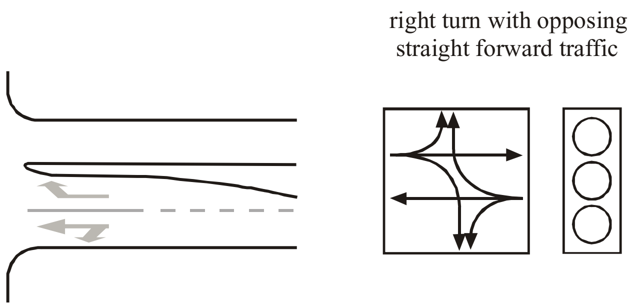


Figure 8-37: Right turn lanes with permissive right turn

8.8.3 Visibility

Each traffic lane shall have clear vision of at least one primary signal head associated with its particular movement from the desirable stopping sight distance, 70 m at 50 km/h and 110 m at 70 km/h speed limit. It is also important that the desirable stopping sight distance is available to all possible queue tails given by the capacity calculation. The warning sign for traffic signals must be used where the visibility is marginal.

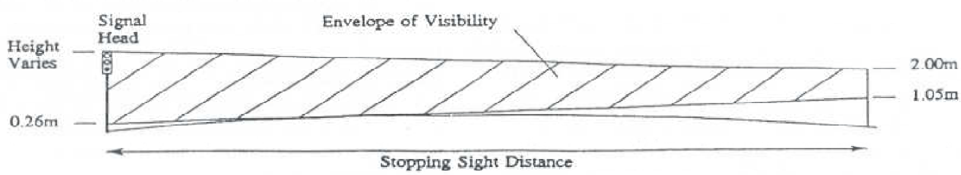


Figure 8-38: Visibility requirements on intersection approach

The intersection intervisibility zone is defined as the area bounded by measurements from a distance of 2.5 m behind the stop-line extending the full carriageway width for each arm, as indicated in Figure 8.39. Designers should aim to achieve the greatest level of intervisibility within this zone to permit manoeuvres to be completed safely once drivers, cyclists and pedestrians have entered the zone.

Visibility along the intersecting road must be at least equal to the standards for “STOP” signs, as set out in Figure 8.12. This is to ensure a minimum level of safety when the signals are out of order.

Minor obstructions to visibility caused by slender projections such as lighting columns, sign supports, signal posts, controller cabinet and guardrails may be unavoidable. When placing signs, street furniture and planting, consideration should be given to ensure that their obstructive effect is minimised.

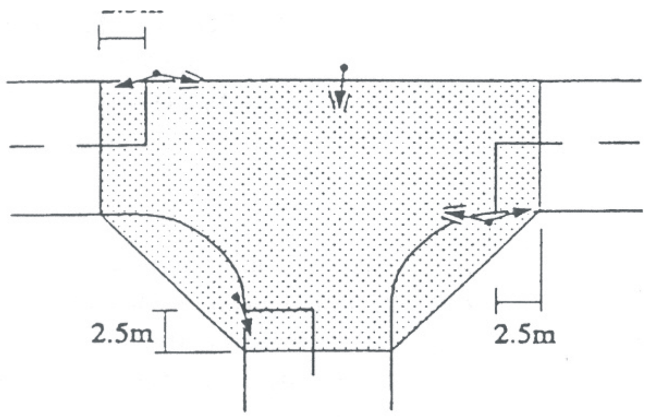


Figure 8-39: Intervisibility zone without pedestrian crossing

8.8.4 Lane Design

Traffic lanes should normally be 3.0 to 3.5 m wide. Nearside (kerb) lanes that are well-used by cyclists should be widened to 4 m if possible. The lane width can be narrowed to 2.75 m, if space is very limited, but only if there are few trucks or buses.

The required lane lengths depend on estimated queue lengths to be decided based on the capacity analysis.

Entry lanes for right turners are needed, as already stated, if protected right turns are to be used. Additional entry lanes for through traffic will improve capacity and level-of-service, but the larger intersection area can result in the need to set longer intergreen periods.

The entry taper L in of a kerbed entry lane should be minimum 30 m (taper 1:10) to allow a design semi-trailer to cope with it. Tapers can be narrowed to 1:5 to allow more queuing space within the same total length.

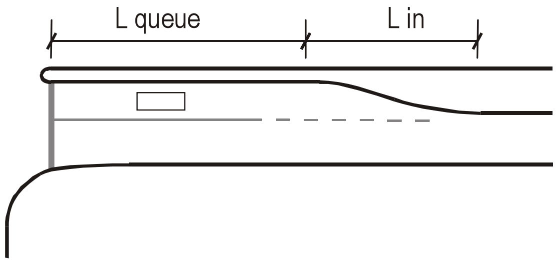


Figure 8-40: Right turn lane design

Minimum design measurements for a right turn with a ghost island are shown in Figure 8.41.

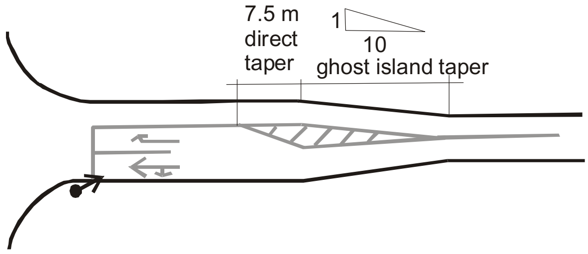


Figure 8-41: Ghost island layout

Opposing right turns, especially permissive right turns (i.e. with opposing traffic) on the main road, should be aligned opposite each other to improve visibility to meeting vehicles, to avoid, if possible, safety problems as shown in Figure 8.42.

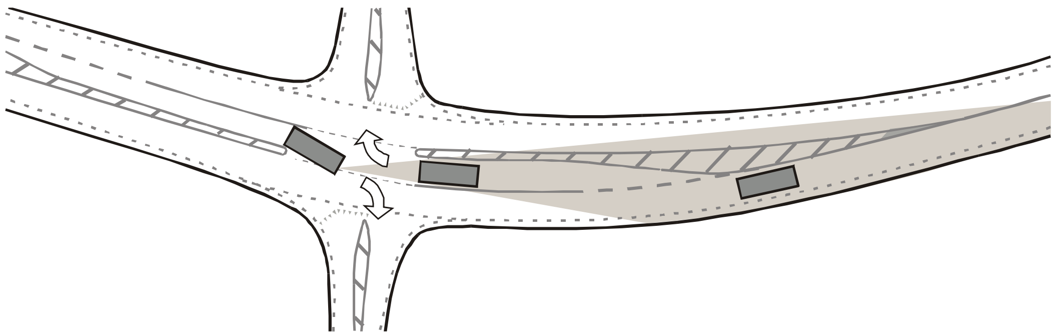


Figure 8-42: Sight shadow design problem at permissive right turns

The number of straight ahead entry and exit lanes should be balanced in order to reduce conflicts caused by traffic merging or diverging within the intersection visibility zone. Lane drops should take place beyond the visibility zone over a distance of at least 100 m for a single lane reduction. The lane drop may be carried out on either the nearside or offside dependant on traffic conditions.

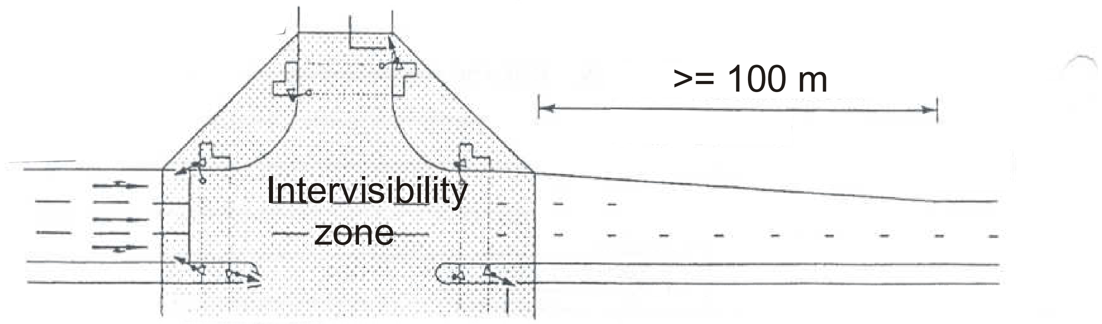


Figure 8-43: Lane drop design principles

Slip lanes (for left turners) can be signalised or uncontrolled ("give way" signs and markings). They can be used when left turn manoeuvres for large vehicles have to be facilitated, see example below. Uncontrolled slip lanes improve the efficiency of the traffic signal control, as intergreens can be decreased, especially at high left turn volumes. Uncontrolled traffic should be separated with a triangular separation island.

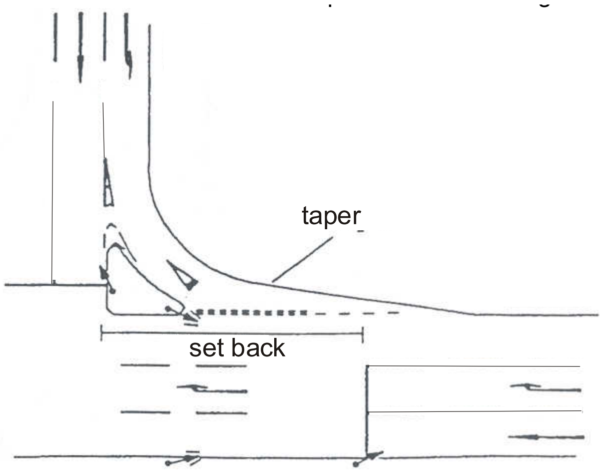


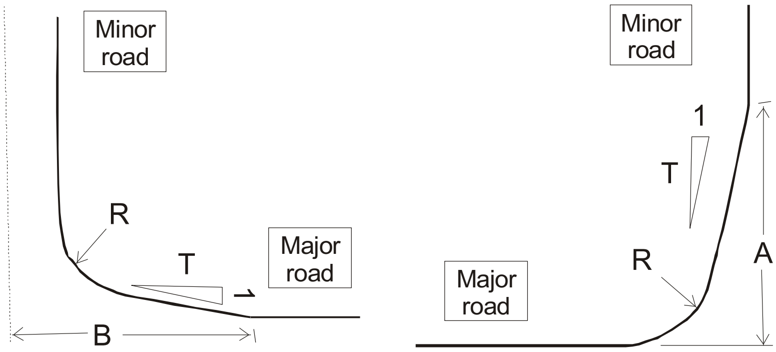
Figure 8-44: Left turn slip lane with taper to facilitate large vehicles

If left turn slip lanes are used, a consistent design approach should be adopted for ease of understanding. Uncontrolled slip lanes can be confusing for pedestrians. Uncontrolled and controlled pedestrian crossings should not be mixed within the same intersection.

8.8.5 Swept Paths and Corner Curves

Corner curves and channel width design depend on what design vehicle and design level-of-service is chosen, see Section 5.

Signalised intersections with very low volumes of large trucks and buses could have simple 6 m corner radiuses to minimise the intersection area and optimise the signal control strategy. The radius should be increased to 10 m if 12 m rigid trucks or buses are common. The following combinations of tapers and corner radii can be used in urban areas to accommodate semi trailers, see Figure 8-45.



	R	T	A	B
Urban	10	5	30	30
Rural*	15	10	25	25
* or triple radii curves, see section 6.6.2				

Figure 8-45: Combinations of tapers and corner radii

It is essential to ensure that adequate turning radii are provided for the swept paths of all types of vehicles using the intersection. Swept paths must be checked for all permitted turning movements to control locations of traffic islands, signals etc, see examples below. The example on the left indicates that there is an unnecessary taper; the example on the right indicates that the stop-line must be set back.

Simple swept path templates are not recommended for checking whether semi trailers can negotiate intersections. The use of specialist computer software (such as AUTOTURN) gives a much more accurate simulation.

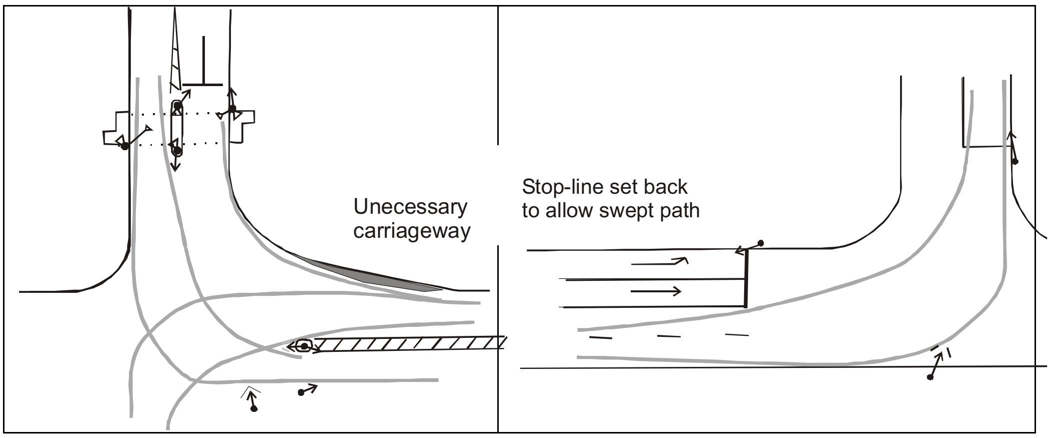


Figure 8-46: Examples of swept path checks

Nosings of central reserves and pedestrian refuges should be set back a minimum of 1.5 m, measured from a line extended from the edge of the intersecting roads. Minimum clearances should be provided, see Section 7, and must be controlled if the superelevation is over 2.5%.

8.8.6 Signals

There should be at least two signals visible from each approach, and stop-line usually comprising a primary and a secondary signal (see also the Traffic Signs Manual, Volume 1). Where separate signalling of turning movements is used this advice applies to the approach lane(s) associated with each turning movement. One signal post can then display information for more than one turning movement.

The primary signal should be located to the left of the approach a minimum of 1 m beyond the stop line and in advance of crossing marks for pedestrians if any. The secondary signal should be located within a 30 degree angle on a maximum distance of 50 m with priorities as shown in the figure below.

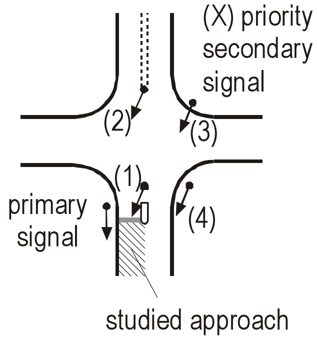


Figure 8-47: Signal location advice

The primary signal should preferably be located 0.8 to 1.0 metre from the edge of the carriageway with 0.3 and 2.0 m as minimum and maximum. Recommended locations in relation to the stop-line and a pedestrian crossing are shown below.

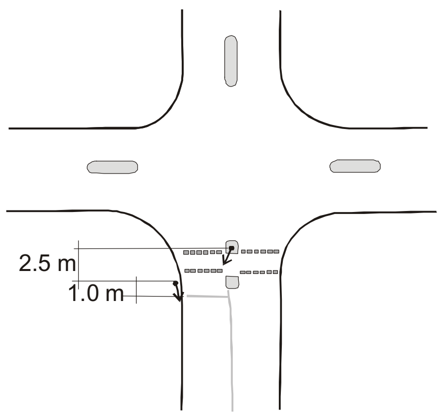


Figure 8-48: Primary signal location advice

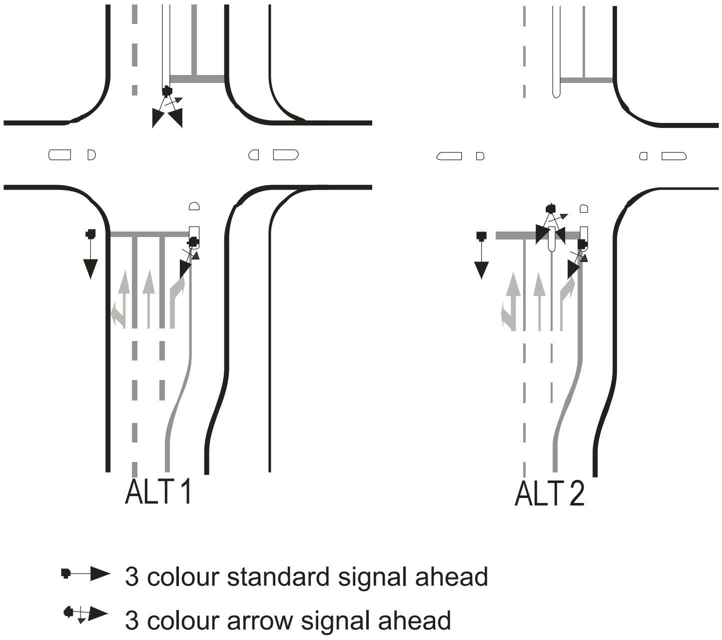


Figure 8-49: Alternative signal locations for right turn lanes

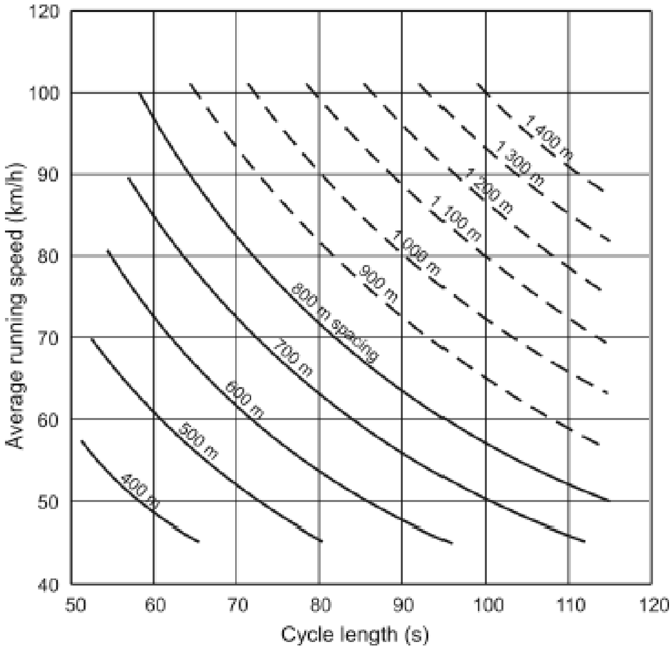
The standard traffic signal head width is 300 mm (with 450 mm as oversize), which results in island width requirements, including clearances, of 0.3 to 0.6 m from 0.9 m to 1.65 m. Wider islands can be needed if they are also to serve as pedestrian refuges.

8.8.7 Spacing of Signalized Intersections

Designers seldom have influence on the spacing of roadways in a network as it is largely predicated by the original or developed land use. Nevertheless, the spacing of any type of intersection impact significantly on the operation, level of service and capacity of a roadway. It then follows that intersection spacing should, inter alia, be based on road function and traffic volume. The Road Agency should therefore play a role in the determination of the location of individual intersections. This is of particular concern when the provision of a new intersection on an existing road is being considered.

Along signalized roads, intersection spacing should be consistent with the running speed and signal cycle lengths, which are variables in themselves. If the spacing of the intersections is based on acceptable running speeds and cycle lengths, signal progression and an efficient use of the roadway can be achieved. All these variable are combined in a chart given in Figure 8-50, allowing the selection of suitable spacing between signalized intersection.

From figure 8-50 it can be seen that the minimum spacing is 400 m. Where spacing closer than this minimum exists, a number of alternative action can be considered. Among these alternatives are two-way flows can be converted to one-way operation or minor connecting roads can be closed or diverted, and channelisation can be used to restrict turning movements.



Note:
Dashed lines indicate reduced platooning and signal progression benefits for signal spacing greater than 800 m

Figure 8-50: Desirable signal spacings

8.8.8 Pedestrian and cyclist facilities

Pedestrian crossings should be perpendicular to the edge of the carriageway to assist intervisibility and to benefit visually impaired people. The footway should have a dropped kerb.

Minimum measures for pedestrian refuges for pedestrian crossings timed to permit crossing in one movement is shown below. The normal width should be 2.5 m, with 1.5 m as the absolute minimum.

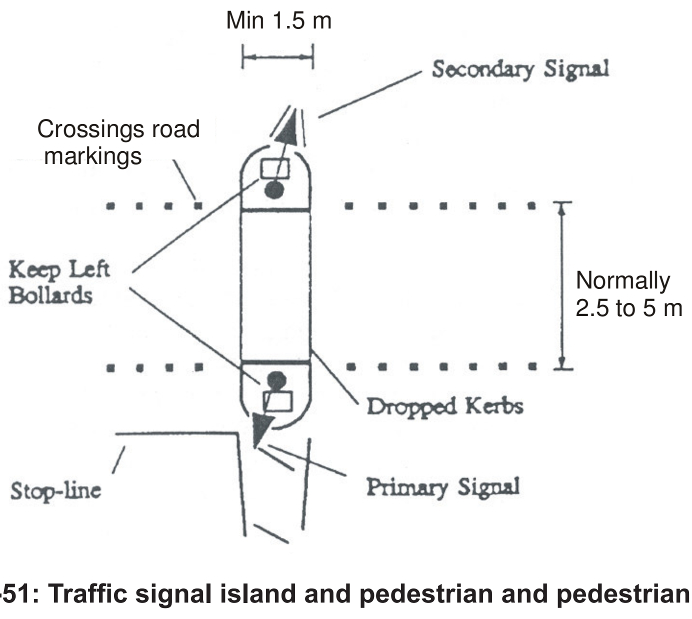


Figure 8-51: Traffic signal island and pedestrian and pedestrian refuge

Pedestrian phases should preferably not have conflicts with turning traffic. This could be arranged with staggered pedestrian crossings as illustrated below.

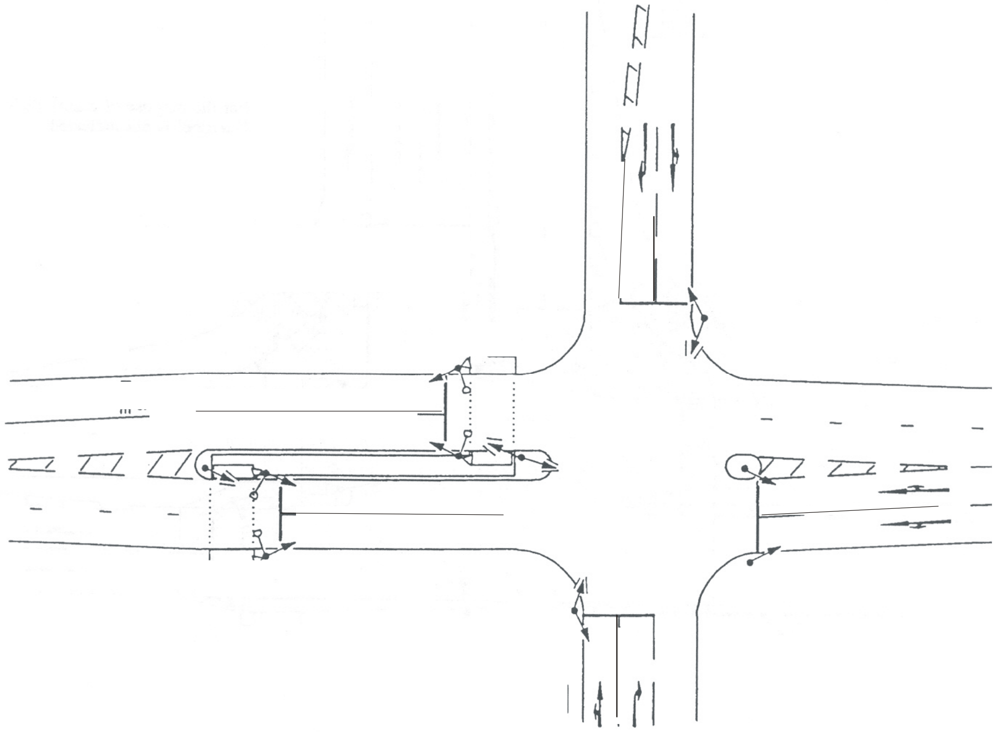


Figure 8-52: Examle of a signal-controlled intersection with staggered pedestrian crossing.

SECTION 9: INTERCHANGES

9.1 Introduction

9.1.1 General

The principal difference between interchanges (graded separated intersection) and other forms of intersection is that, in interchanges, crossing movements are separated in space whereas, in the latter case, they are separated in time. At-grade intersections accommodate turning movements either within the limitations of the crossing roadway widths or through the application of turning roadways whereas the turning movements at interchanges are accommodated on ramps. The ramps replace the slow turn through an angle of skew that is approximately equal to 900 by high-speed merging and diverging manoeuvres at relatively flat angles.

The various types of interchange configuration are illustrated in Section 9.7. Each basic form can be divided into sub-types. For example, the Diamond Interchange is represented by the narrow diamond, the wide diamond and the split diamond. The most recent development in the Diamond interchange form is the Single Point Diamond Interchange. This form is also referred to as the Urban Interchange.

Historically, the type of interchange to be applied at a particular site would be selected as an input into the design process. In fact, like the cross-section, the interchange is the aggregation of various elements. A more sensible approach is thus to select the elements appropriate to a particular site in terms of the topography, local land usage and traffic movements and then to aggregate them into some or other type of interchange.

9.1.2 Design Principles

Manoeuvres in an interchange area occur at high speeds close to the freeway and over relatively short distances. It is therefore important that drivers should experience no difficulty in recognising their route through the interchange irrespective of whether that route traverses the interchange on the freeway or diverts to depart from the freeway to a destination that may be to the left or the right of the freeway. 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:

- select a suitable speed and accelerate or decelerate to the selected speed within the available distance;
- select the appropriate lane and carry out the necessary weaving manoeuvres to effect lane changes if necessary; and
- 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.

Interchange exits and entrances should always be located on the left. Right-hand side entrances and exits are counter to driver expectancy and also have the effect of mixing high-speed through traffic with lower-speed turning vehicles. The problem of extracting turning vehicles from the median island and providing sufficient vertical clearance either over or under the opposing freeway through lanes is not trivial. The application of right-hand entrances and exits should only be considered under extremely limiting circumstances. Even in the case of a major fork where two freeways are diverging, the lesser movement should, for preference, be on the left.

Route continuity substantially simplifies the navigational aspects of the driving task. For example, if a driver simply wishes to travel on a freeway network through a city from one end to the other it should not be necessary to deviate from one route to another.

Uniformity of signing practice is an important aspect of consistent design and reference should be made to the Uganda Traffic Signs Manual.

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 freeway should take place clear of the freeway itself. In this manner, drivers will be required to take two binary decisions, (Yes/No) followed by (Left/Right), as opposed to a single compound decision. This spreads the workload and simplifies the decision process, hence improving the operational efficiency of the entire facility. Closely spaced successive off-ramps could be a source of confusion to the driver leading to erratic responses and manoeuvres.

Single entrances are to be preferred, also in support of operational efficiency of the interchange. Merging manoeuvres by entering vehicles are an interruption of the free flow of traffic in the left lane of the freeway. Closely spaced entrances exacerbate the problem and the resulting turbulence could influence the adjacent lanes as well.

From the standpoint of convenience and safety, in particular prevention of wrong-way movements, interchanges should provide ramps to serve all turning movements. If, for any reason, this is not possible or desirable, it is nevertheless to be preferred that, for any travel movement from one road to another within an interchange, the return movement should also be provided.

Provision of a spatial separation between two crossing streams of traffic raises the problem of which to take over the top - the perennial Over versus Under debate. The choice of whether the crossing road should be taken over or under the freeway 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 freeway. These are:

- Exit ramps on up-grades assist deceleration and entrance ramps on downgrades assist acceleration and have a beneficial effect on truck noise;
- Rising exit ramps are highly visible to drivers who may wish to exit from the freeway;
- 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 freeway or perhaps to change lanes with a view to the impending departure from the freeway;
- Dropping the freeway into cut reduces noise levels to surrounding communities and also reduces visual intrusion;
- For the long-distance driver on a rural freeway, a crossing road on a structure may represent an interesting change of view; and,
- 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 freeway.

The other design principles, being continuity of basic lanes, lane balance and lane drops are discussed in Section 9.5 as matters of detailed design.

9.2 Interchange Warrants

9.2.1 Traffic Volumes

With increasing traffic volumes, a point will be reached where all the options of temporal separation of conflicting movements at an at-grade intersection have been exhausted. One of the possible solutions to the problem is to provide an interchange.

The elimination of bottlenecks by means of interchanges can be applied to any intersection at which demand exceeds capacity and is not necessarily limited to arterials. Under these circumstances, it is necessary to weigh up the economic benefits of increased safety, reduced delay and reduced operating and maintenance cost of vehicles against the cost of provision of the interchange. The latter includes the cost of land acquisition, which could be high, and the cost of construction. As the construction site would be heavily constricted by the need to accommodate traffic flows that were sufficiently heavy to justify the interchange in the first instance, the cost of construction could be significantly higher than on the equivalent green field site.

9.2.2 Freeways

The outstanding feature of freeways is the limitation of access that is brought to bear on their operation. Access is permitted only at designated points and only to vehicles travelling at or near freeway speeds. As such, access by means of intersections is precluded and the only permitted access is by way of interchanges. Crossing roads are normally those that are high in the functional road hierarchy, e.g. arterials, although, if these are very widely spaced, it may be necessary to provide an interchange serving a lower order road, for example a collector.

It follows that the connection between two freeways would also be by means of an interchange, in which case reference is to a systems interchange as opposed to an access interchange.

9.2.3 Safety

Some at-grade intersections exhibit high crash rates that cannot be lowered by improvements to the geometry of the intersections or through the application of control devices. Such situations are often found at heavily travelled urban intersections. Crash rates also tend to be high at the intersections on heavily travelled rural arterials where there is a proliferation of ribbon development.

A third area of high crash rates is at intersections on lightly travelled low volume rural locations where speeds tend to be high. In these cases, low-cost interchanges such as the Jug-handle layout may be an adequate solution to the problem.

9.2.4 Topography

The topography may force a vertical separation between crossing roads at the logical intersection location. As an illustration, the through road may be on a crest curve in cut with the crossing road at or above ground level. If it is not possible to relocate the intersection, a simple Jug-handle type of interchange as illustrated in Figure 9.13 may be an adequate solution to the problem.

9.3 Weaving

The Highway Capacity Manual (2000) 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 weave are illustrated in Figures 9.1, 9.2 and 9.3. A 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.

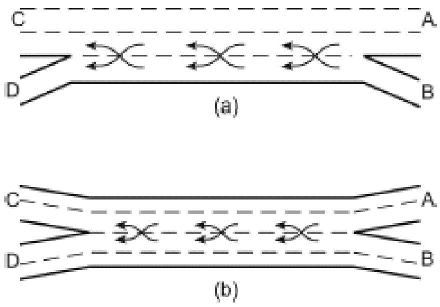


Figure 9.1:Type A weaves:
(a) ramp-weave
(b) major weave with crown line

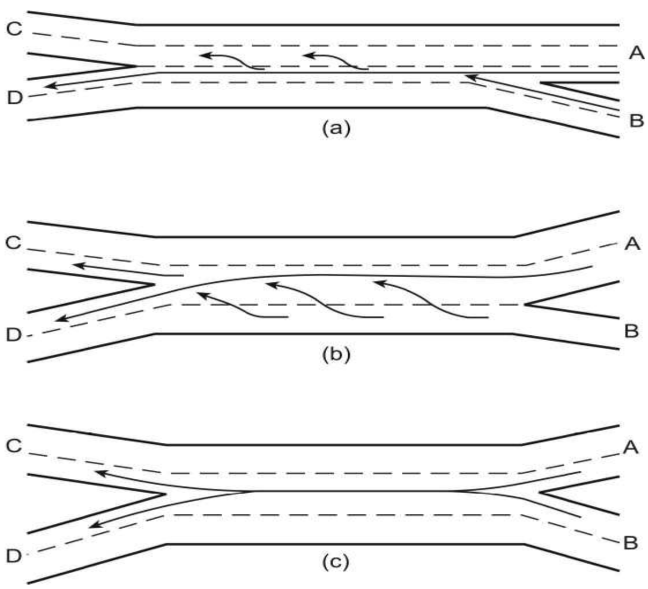


Figure 9.2: Type B weaves:
(a) major weave with lane balance at exit
(b) major weave with merging at entrance
(c) major weave with merging at entrance and lane balance at exit

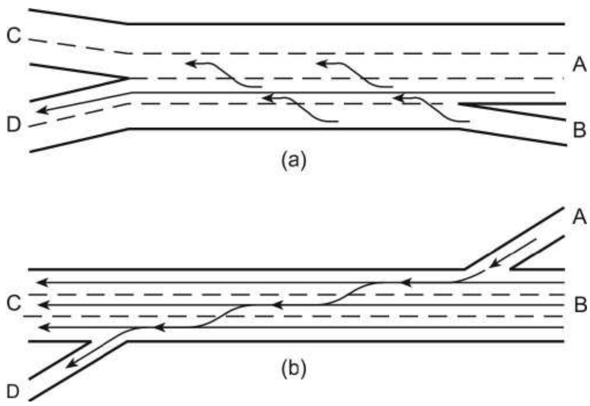


Figure 9.3: Type C weaves:
(a) major weave without lane balance
(b) two-sided weave

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 weave.

9.4 Location and Spacing of Interchanges

The location of interchanges is based primarily on service to adjacent land. On rural freeways bypassing small communities, the provision of a single interchange may be adequate, with larger communities requiring more. The precise location of interchanges would depend on the particular needs of the community but, as a general guide, would be on roads recognised as being major components of the local system.

Rural interchanges are typically spaced at distances of eight kilometres apart or more. 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 freeways than on rural freeways. As drivers are accustomed to taking a variety of alternative actions in rapid succession a spacing of closer than eight kilometres can be considered.

At spacings 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. For a common centreline-to-centreline spacing, the weaving length available between two diamond interchanges is significantly different from that between a Par-Clo-A followed by a Par-Clo-B. Weaving distance is defined in the Highway Capacity Manual 2000 and other sources as the distance between the point at which the separation between the ramp and the adjacent lane is 0.5 metres to the point at the following off-ramp at which the distance between ramp and lane is 3.7 m as illustrated in Figure 9-4.

If this definition is adopted, the weaving length becomes a function of the rates of taper applied to the on- and off-ramps. Reference to the Yellow Line Break Point (YLBP) distance is totally unambiguous and is the preferred option.

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 freeway. The third criterion is that of turbulence, which is applied to the merge-diverge situation.

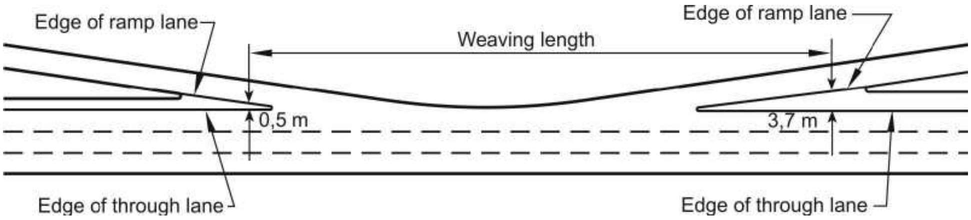


Figure 9.4: Weaving distance

(1) The distance required to provide adequate sign posting which, in turn, influences the safe operation of the freeway, is used to define the minimum distance between ramps. The minimum distances to be used for detailed design purposes, as measured between Yellow Line Break Points, for different areas and interchange types should be not less than the values stated in Table 9.1.

Table 9.1: Interchange Spacing in terms of Signage Requirements

Configuration	Urban areas	Rural areas
Access to access	1 300 m	2 200 m
Access to systems	2 100 m	3 300 m
Systems to access	1 400 m	2 200 m
Systems to systems	2 500 m	3 800 m

(2) In exceptional cases, where it is not possible to meet the above requirements, relaxation of these may be considered. It is, however, necessary to ensure that densities in the freeway left hand lane are not so high that the flow of traffic breaks down. Densities associated with LOS E would make it difficult, if not actually impossible, for drivers to be able to change lanes. Formulae according to which densities can be estimated are provided in the Highway Capacity Manual (2000). Drivers need time to locate a gap and then to position themselves correctly in relation to the gap while simultaneously adjusting their speed to that required for the lane change. The actual process of changing lanes also requires time.

(3) In the case of the merge-diverge maneuver, turbulence caused on the left lane of the freeway by a close succession of entering and exiting vehicles becomes an issue. According to some studies this turbulence manifests itself over a distance of roughly 450 metres upstream of an off-ramp and downstream of an on-ramp. A spacing of 900 metres would suggest that the entire length of freeway between interchanges would be subject to turbulent flow. The likelihood of breakdown in the traffic flow would thus be high and the designer should ensure that space is available for one area of turbulence to subside before onset of the next.

(4) In off-peak periods, vehicles would be moving between interchanges at the design speed or higher. The geometry of the on- and off-ramps should be such that they can accommodate maneuvers at these speeds. Increasing the taper rates or reducing the length of the speed change lanes purely to achieve some or other hypothetically acceptable Yellow Line Break Point distance does not constitute good design.

(5) The spacing between successive interchanges will have an impact on traffic operations on the crossing roads and vice versa. If the crossing road can deliver vehicles to the freeway faster than they can carry out the merge, stacking of vehicles will occur on the on-ramp with the queue possibly backing up on to the crossing road itself. Stacking can also occur on an off-ramp if the crossing road ramp terminal cannot accommodate the rate of flow arriving from the freeway. The queue could conceivably back up onto the freeway, which would create an extremely hazardous situation.

It should be realised that relaxations below the distances recommended under (1) above will result in an increase in the driver workload. Failure to accommodate acceptable levels of driver workload in relation to reaction times can be expected to result in higher than average crash rates. Some studies demonstrate that, at spacings between noses of greater than 2 500 metres, the crash rate is fairly constant, i.e. the presence of the following interchange is not a factor in the crash rate. At spacings of less than 2500 m between noses, the crash rate increases until, at about 500 m between noses, the crash rate is nearly double that of the 2500 m spacing. This is illustrated in Figure 9.2 below

The question that must be addressed is the benefit that the community can expect to derive in exchange for the cost of the higher accident rate. By virtue of the fact that freeway speeds tend to be high, there is a high probability that many of the accidents would be fatal. It is therefore suggested that the decision to reduce the interchange spacing below those listed in Table 9.1 should not be taken lightly.

It would be necessary to undertake a full-scale engineering analysis of the situation that would include:

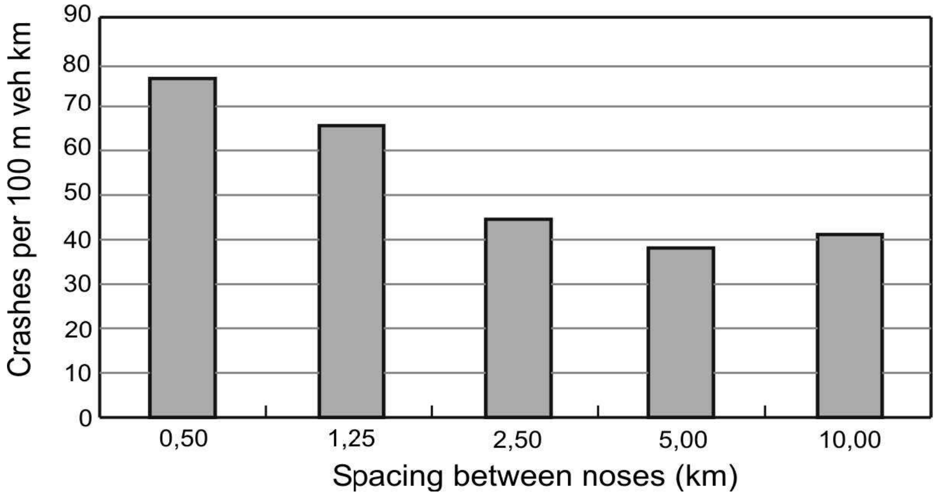


Figure 9.5: Relationship between Interchange Spacing and Accident Rate

- estimation of future traffic volumes at a ten to twenty year time horizon, comprising weaving and through volumes in the design year;
- calculation of traffic densities;
- assessment of the local geometry in terms of sight distances, and horizontal and vertical alignment;
- development of a sign sequence; and
- a form of benefit/cost analysis relating community benefits to the decrease in traffic safety.

Density offers some indication of the level of exposure to risk and, for want of any better measure, it is suggested that a density higher than 22 vehicles/kilometre/lane, corresponding to LOS D, would not result in acceptable design. It would be necessary to pay attention to remedial actions to prevent interchange constraints, such as inadequate ramp capacity, signalling or crossroad volumes, causing back up onto the freeway.

In summary: Spacings of interchanges in terms of their YLBP distances should desirably be in accordance with Table 9.1. If these spacings are not achievable and an interchange is absolutely vital for service to the community and adjacent land uses, relaxations may be considered but then, to minimise the risk of crashes, the density calculated according to the above-mentioned formulae should not exceed 22 vehi-cles/kilometre/lane, which corresponds to LOS D.

9.5 Basic Lanes and Lane Balance

Basic lanes are those that are maintained over an extended length of a route, irrespective of local changes in traffic volumes and requirements for lane balance. Alternatively stated, the basic number of lanes is a constant number of lanes assigned to a route, exclusive of auxiliary lanes.

The number of basic lanes changes only when there is a significant change in the general level of traffic volumes on the route. Short sections of the route may thus have insufficient capacity, which problem can be overcome by the use of auxiliary lanes. In the case of spare capacity, reduction in the number of lanes is not recommended because this area could, at some future time, become a bottleneck. Unusual traffic demands, created by accidents, maintenance or special events, could also result in these areas becoming bottlenecks.

The basic number of lanes is derived from consideration of the design traffic volumes and capacity analyses. To promote the smooth flow of traffic there should be a proper balance of lanes at points where merging or diverging manoeuvres occur. In essence, there should be one lane where the driver has the choice of a change of direction without the need to change lanes.

At merges, the number of lanes downstream of the merge should be one less than the number of lanes upstream of the merge. This is typified by a one-lane ramp merging with a two-lane carriageway that, after the merge, continues as a two-lane carriageway as is the case on a typical Diamond Interchange layout. This rule precludes a two-lane ramp immediately merging with the carriageway without the addition of an auxiliary lane.

At diverges, the number of lanes downstream of the diverge should be one more than the number upstream of the diverge. The only exception to this rule is on short weaving sections, such as at Cloverleaf Interchanges, where a condition of this exception is that there is an auxiliary lane through the weaving section.

When two lanes diverge from the freeway, the above rule indicates that the number of freeway lanes beyond the diverge is reduced by one.

This can be used to drop a basic lane to match anticipated flows beyond the diverge. Alternatively, it can be an auxiliary lane that is dropped.

Basic lanes and lane balance are brought into harmony with each other by building on the basic lanes, adding or removing auxiliary lanes as required. The principle of lane balance should always be applied in the use of auxiliary lanes. Operational problems on existing roadways can be directly attributed to a lack of lane balance and failure to maintain route continuity.

The application of lane balance and coordination with basic number of lanes is illustrated in Figure 9.6.

9.6 Auxiliary Lanes

As in the case of the two-lane two-way road cross-section with its climbing and passing lanes, and the intersection with its right- and left-turning lanes, the auxiliary lane also has its role to play in the freeway cross-section and the interchange. In a sense, the application of the auxiliary lane in the freeway environment is identical to its application elsewhere. It is added to address a local operational issue and, as soon as the need for the auxiliary lane is past, it is dropped. Important features to consider in the application and design of the auxiliary lane are thus:

- The need for an auxiliary lane;
- The terminals; and
- Driver information.

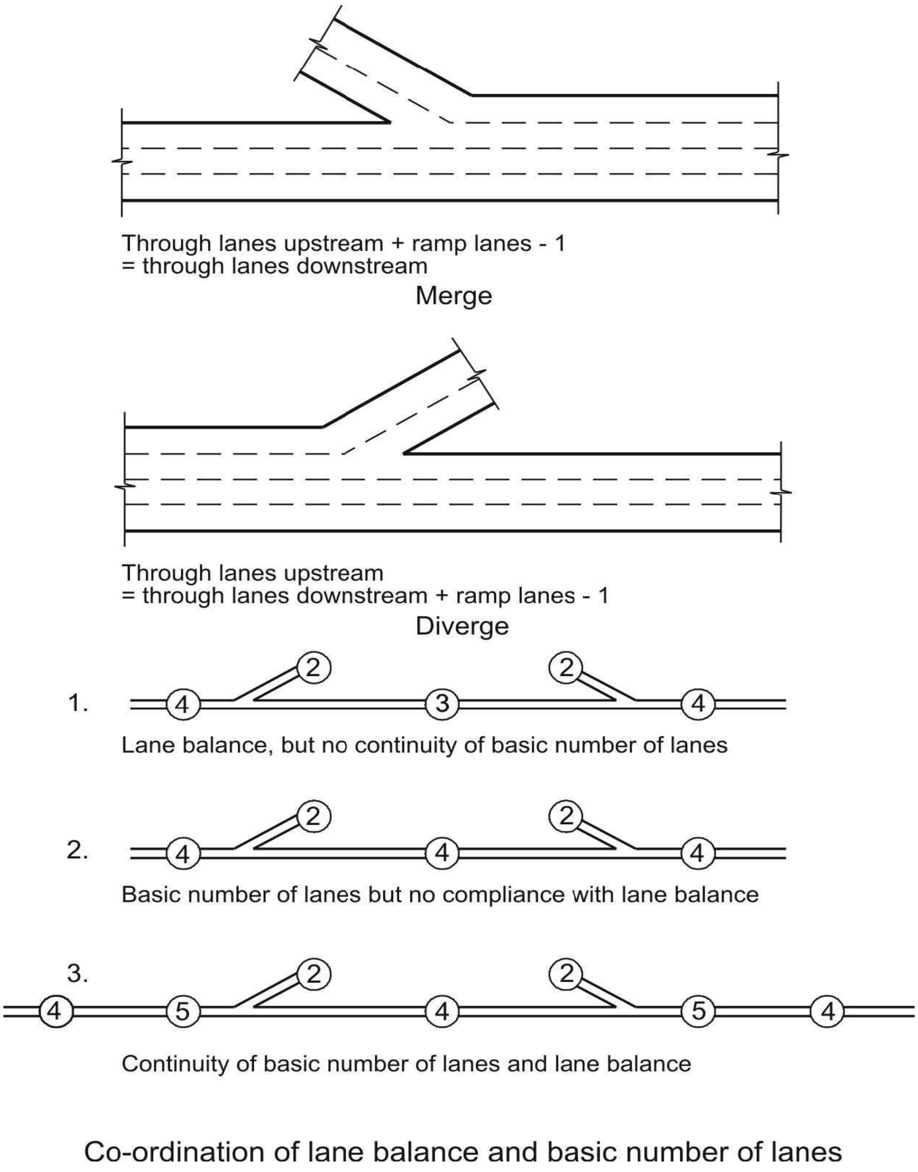


Figure 9.6: Coordination of lane balance with basic number of lanes

9.6.1 The Need for an Auxiliary Lane

Auxiliary lanes are normally required on freeways either as:

- climbing lanes; or
- to support weaving; or
- to support lane balance.

The climbing lane application is similar to that discussed in Section 6 in respect of two-lane two-way roads whereas the weaving and lane balance applications are unique to the freeway situation.

Climbing lanes

Ideally, maximum gradients on freeways are in the range of three to four per cent ensuring that most vehicles can maintain a high and fairly constant speed. However, in heavily rolling country it is not always possible to achieve this ideal without incurring excessive costs in terms of earthworks construction. Because of the heavy volumes of traffic that necessitate the provision of a freeway, lane changing to overtake a slow-moving vehicle is not always easy and, under peak flow conditions, may actually be impossible. Speed differentials in the traffic stream are thus not only extremely disruptive but may also be potentially dangerous. Both conditions, i.e. disruption and reduction in safety, require consideration.

If a gradient on a freeway is steeper than four per cent, an operational analysis should be carried out to establish the impact of the gradient on the Level of Service. A drop through one level, e.g from LOS B through LOS C to LOS D, would normally suggest a need for a climbing lane.

As crash rates increase exponentially with increasing speed differential. For this reason, international warrants for climbing lanes normally include a speed differential in the range of 15 to 20 km/h. A truck speed reduction of 20 km/h is taken to warrant for climbing lanes. If, on an existing freeway, the measured truck speed reduction in the outermost lane is thus 20 km/h or higher, the provision of a climbing lane should be considered. In the case of a new design, it will be necessary to construct a speed profile of the truck traffic to evaluate the need for a climbing lane.

Weaving

In the urban environment, interchanges are fairly closely spaced and local drivers are very inclined to use freeways as part of the local circulation system - a form of rat-running in reverse and as undesirable as the normal form of rat-running where the higher order road is bypassed through the use of local residential streets as long-distance urban routes. To ensure that the freeway is not unduly congested because of this practice, an auxiliary lane can be provided between adjacent interchanges resulting in Type A weaving as described in Section 9.3.

If a large number of vehicles are entering at the upstream interchange, it may be necessary to provide a two-lane entrance ramp. Some of these vehicles may exit at the following interchange but those wishing to travel further will have to weave across traffic from still further upstream that intends exiting at the following interchange and then merge with through traffic on the freeway. The auxiliary lane is then extended beyond the downstream interchange to allow a separation between the two manoeuvres. Similarly, a large volume of exiting vehicles may necessitate a two-lane exit, in which case the auxiliary lane should be extended upstream. Type B weaving thus comes into being. The desired length of the extension of the auxiliary lane beyond the two interchanges is normally assessed in terms of the probability of merging vehicles locating an acceptable gap in the opposing traffic flow.

Lane balance

As discussed in Section 9.5, lane balance requires that:

- In the case of an exit, the number of lanes downstream of the diverge should be one more than the number upstream; and,
- In the case of an entrance, the number of lanes downstream of the merge should be one less than the number upstream.

This is illustrated in Figure 9.6.

Single-lane on- and off-ramps do not require auxiliary lanes to achieve lane balance in terms of the above definition. It should be noted that, unless two-lane on- and off-ramps are provided, the Type A weave is actually a violation of the principles of lane balance.

To achieve lane balance at an exit, three lanes upstream of the diverge should be followed by a two-lane off-ramp in combination with two basic lanes on the freeway. The continuity of basic lanes requires that the outermost of the three upstream lanes should be an auxiliary lane.

If all three upstream lanes are basic lanes, it is possible that traffic volumes beyond the off-ramp may have reduced to the point where three basic lanes are no longer necessary. Provision of a two-lane exit would thus be a convenient device to achieve a lane drop while simultaneously maintaining lane balance. The alternative would be to provide a single-lane off-ramp, carrying the basic lanes through the interchange and dropping the outside lane some distance beyond the on-ramp terminal. In view of the additional construction costs involved, this approach is not recommended. If a two-lane on-ramp is joining two basic lanes, lane balance will require that there be three lanes beyond the merge. The outside lane of the three could become a new basic lane if the increase in traffic on the freeway merits it. On the other hand, it is possible that the flow on the ramp is essentially a local point of high density and that two basic lanes are all that are required downstream of the on-ramp. In this case, the outside lane could be dropped as soon as convenient.

9.6.2 Auxiliary Lane Terminals

An auxiliary lane is intended to match a particular situation such as, for example, an unacceptably high speed differential in the traffic stream. It follows that the full width of auxiliary lane must be provided over the entire distance in which the situation prevails. The terminals are thus required to be provided outside the area of need and not as part of the length of the auxiliary lane.

Entering and exiting from auxiliary lanes require a reverse curve path to be followed. It is thus suggested that the taper rates discussed in Section 6 be employed rather than those normally applied to on- and off-ramps. The entrance taper should thus be about 100 metres long and the exit taper about 200 metres long.

9.6.3 Driver Information

The informational needs of drivers relate specifically to needs with regard to the exit from the auxiliary lane and include an indication of:

- the presence of a lane drop;
- the location of the lane drop; and,
- the appropriate action to be undertaken

9.7 Interchange Types

9.7.1 General

There is a wide variety of types of interchanges that can be employed under the various circumstances that warrant the application of inter-changes. The major determinant of the type of interchange to be employed at any particular site is the classification and characteristics of the intersecting road. Intersecting roads are typically freeways or urban arterials but may also be collectors.

In the case of freeways as intersecting roads, reference is made to systems interchanges. Systems interchanges exclusively serve vehicles that are already on the freeway system.

Access to the freeway system from the surrounding area is via interchanges on roads other than freeways, for which reason these interchanges are known as access interchanges. Service areas, providing opportunities to buy fuel, or food or simply to relax for a while are typically accessed via an interchange. In some instances, the services are duplicated on either side of the freeway, in which access is via a left-in/left-out configuration. The requirements in terms of deflection angle, length of ramp and spacing that apply to interchange ramps apply equally to left-in/left-out ramps. In effect, this situation could be described as being an interchange without a crossing road.

The primary difference between systems and access/service interchanges is that the ramps on systems interchanges have free-flowing terminals at both ends, whereas the intersecting road ramp terminals on an access interchange are typically in the form of at-grade intersections.

Interchanges can also be between non-freeway roads, for example between two heavily trafficked arterials. In very rare instances there may even be an application for an interchange between a major and a local road, as suggested above in the case where local topography may force a grade separation between the two roads.

In addition to the classification and nature of the intersecting road, there are a number of controls guiding the selection of the most appropriate interchange form for any particular situation. In the sense of context sensitive design, these include;

- Safety;
- Adjacent land use;
- Design speed of both the freeway and the intersecting road;
- Traffic volumes of the through and turning movements;
- Traffic composition;
- Number of required legs;
- Road reserve and spatial requirements;
- Topography;
- Service to adjacent communities;
- Environmental considerations, and,
- Economics.

The relative importance of these controls may vary from interchange to interchange. For any particular site, each of the controls will have to be examined and its relative importance assessed. Only after this process will it be possible to study alternative interchange types and configurations to determine the most suitable in terms of the more important controls.

While the selection of the most appropriate type and configuration of interchange may vary between sites, it is important to provide consistent operating conditions in order to match driver expectations.

9.7.2 Systems Interchanges

As stated above, at-grade intersections are inappropriate to systems interchanges and their avoidance is mandatory. For this reason, hybrid interchanges, in which an access interchange is contained within a systems interchange, are to be avoided.

Hybrid interchanges inevitably lead to an unsafe mix of high and low speed traffic. Furthermore, signposting anything up to six possible destinations within a very short distance is, at best, difficult. Selecting the appropriate response generates an enormous workload for the driver so that the probability of error is substantial. Past experience suggests that these interchange configurations are rarely successful.

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. Various forms of systems interchanges are illustrated below.

Four-legged interchanges

The fully directional interchange illustrated in Figure 9.7 (i) provides single exits from all four directions and directional ramps for all eight turning movements. The through roads and ramps are separated vertically on four levels. Partially directional interchanges allow the number of levels to be reduced. The Single Loop Partially-directional Interchange, illustrated in Figure 9.7 (ii), and the Two Loop arrangement, illustrated in Figure 9.7 (iii) and (iv), require three levels.

The difference between Figures 9.7 (iii) and (iv) is that, in the former case, the freeways cross and, in the latter, route continuity dictates a change in alignment. Loop ramps are normally only used for lighter volumes of right-turning traffic. A three-loop arrangement is, in effect, a cloverleaf configuration, with one of the loops being replaced by a directional ramp and is not likely to occur in practice, largely because of the problem of weaving discussed below.

The principal benefit of the cloverleaf is that it requires only a simple one-level structure, in contrast to the complex and correspondingly costly structures necessary for the directional and partially directional configurations. The major weakness of the cloverleaf is that it requires weaving over very short distances. Provided weaving volumes are not high and sufficient space is available to accommodate the interchange, the cloverleaf can, however, be considered to be an option. If weaving is required to take place on the main carriageways, the turbulence so created has a serious effect on the flow of traffic through the interchange area. The cloverleaf also has the characteristic of confronting the driver with two exits from the freeway in quick succession. Both these problems can be resolved by providing collector-distributor roads adjacent to the through carriageways.

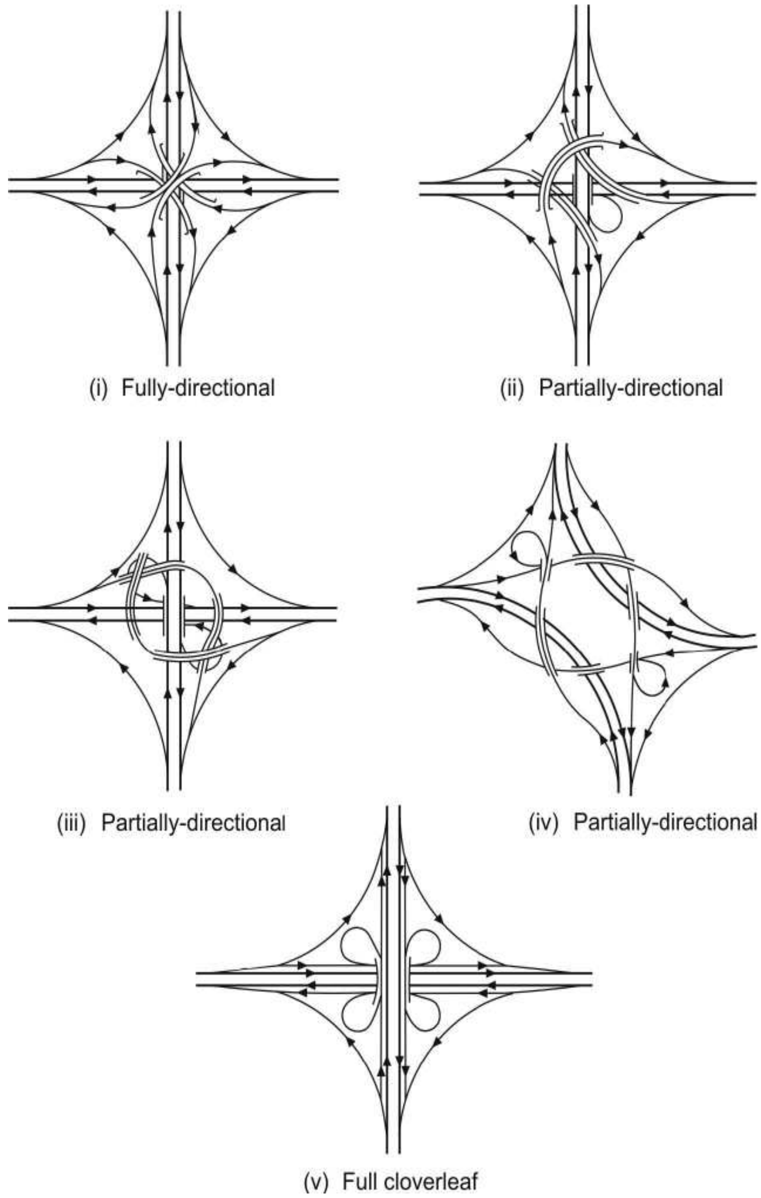


Figure 9-7: Four-legged Systems Interchanges

Three-legged interchanges

Various fully-directional and partially-directional three legged interchanges are illustrated in Figure 9.8. In Figure 9.8 (i), one single structure providing a three-level separation is required. Figure 9.8 (ii) also requires three levels of roadway but spread across two structures hence reducing the complexity of the structural design.

It is also possible with this layout to slightly reduce the height through which vehicles have to climb. Figure 9.8 (iii) illustrates a fully-directional interchange that requires only two but widely separated structures. If North is assumed as being at the top of the page, vehicles turning from West to South have a slightly longer path imposed on them so that this should, ideally be the lesser turning volume.

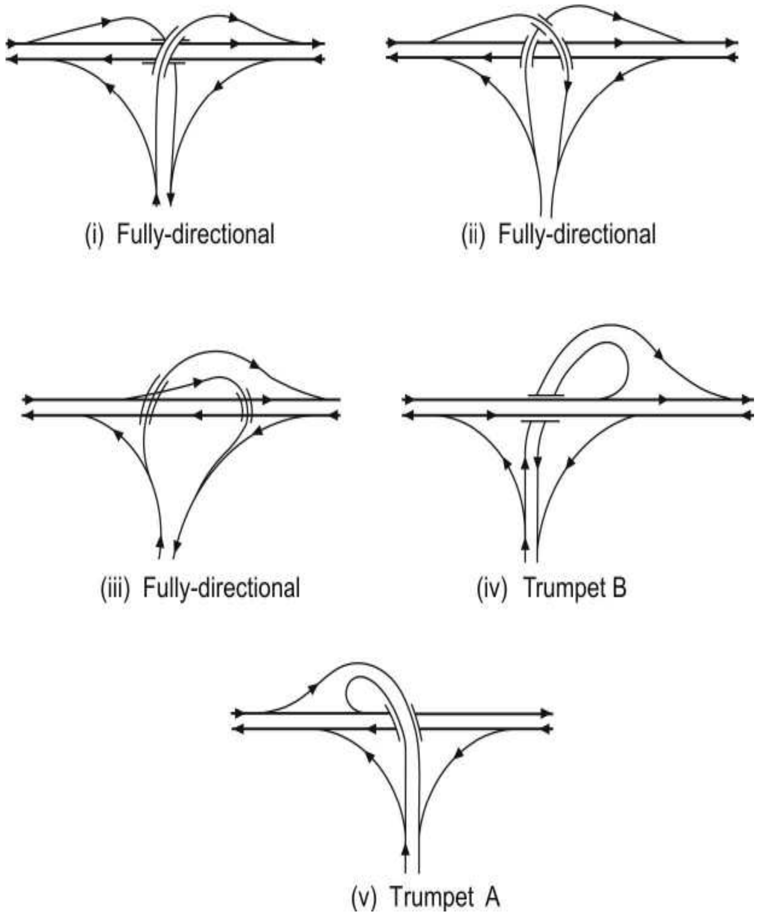


Figure 9-8: Three legged Systems Interchanges

Figures 9.8 (iv) and (v) show semi-directional interchanges. Their names stem from the loop ramp located within the directional ramp creating the appearance of the bell of a trumpet. The letters “A” and “B” refer to the loop being in Advance of the structure or Beyond it. The smaller of the turning movements should ideally be on the loop ramp but the availability of space may not always make this possible.

9.7.3 Access and Service Interchanges

In the case of the systems interchange, all traffic enters the interchange area at freeway speeds. 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 freeway at or near freeway speeds. Similarly, exiting vehicles should be provided with deceleration lanes to accommodate the possibility of a stop at the crossing road.

As previously discussed, there is distinct merit in the crossing road being taken over the freeway as opposed to under it. One of the advantages of the crossing road being over the freeway is that the positive and negative gradients respectively support the required deceleration and acceleration to and from the crossing road. The final decision on the location of the crossing road is, however, also dependent on other controls such as topography and cost.

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. Each has a variety of possible configurations.

Trumpet interchanges used to be considered suitable in cases where access was to 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.

Diamond Interchanges

There are three basic forms of Diamond:

- The Simple Diamond;
- The Split Diamond; and the,
- Single Point Interchange.

The Simple Diamond is easy for the driver to understand and economical in its use of space. The major problem with this configuration is that the right turn on the crossing road can cause queuing on the exit ramp. In extreme cases, these queues can extend back onto the freeway, creating a hazardous situation. Where the traffic on the right turn is very heavy, it may be necessary to consider placing it on a loop ramp. This is the reverse of the situation on systems interchanges where it is the lesser volumes that are located on loop ramps. It has the advantage that the right turn is converted into a left-turn at the crossing road ramp terminal. By the provision of auxiliary lanes, this turn can operate continuously without being impeded by traffic signals.

The Simple Diamond can take one of two configurations: the Narrow Diamond and the Wide Diamond.

The Narrow Diamond is the form customarily applied. In this configuration, the crossing road ramp terminals are very close in plan to the freeway shoulders to the extent that, where space is heavily constricted, retaining walls are located just outside the freeway shoulder breakpoints. Apart from the problem of the right turn referred to above, it can also suffer from a lack of intersection sight distance at the crossing road ramp terminals. This problem arises when the crossing road is taken over the freeway and is on a minimum value crest curve on the structure. In addition, the bridge balustrades can also inhibit sight distance. In the case where the crossing road ramp terminal is signalised, this is less of a problem, although a vehicle accidentally or by intent running the red signal could create a dangerous situation.

The Wide Diamond was originally intended as a form of stage construction, leading up to conversion to a full Cloverleaf Interchange. The time span between construction of the Diamond and the intended conversion was, however, usually so great that, by the time the upgrade became necessary, standards had increased to the level whereby the loop ramps could not be accommodated in the available space. The decline in the popularity of the Cloverleaf has led to the Wide Diamond also falling out of favour.

The Wide Diamond has the problem of imposing a long travel distance on right-turning vehicles but is not without its advantages. The crossing road ramp terminals are located at the start of the approach fill to the structure. To achieve this condition, the ramps have to be fairly long so that queues backing up onto the freeway are less likely than on the Narrow Diamond. The crossing road ramp terminals are also at ground level, which is a safer alternative than having the intersections on a high fill. Finally, because the ramp terminals are remote from the structure, intersection sight distance is usually not a problem.

The Split Diamond can also take one of two forms: the conventional Split and the transposed Split. This configuration is normally used when the crossing road takes the form of a one-way pair. The problems of sight distance and queues backing up are not normally experienced on Split Diamonds and the most significant drawback is that right-turning vehicles have to traverse three intersections before being clear of the interchange. It is also necessary to construct frontage roads linking the two one-way streets to provide a clear route for right-turning vehicles.

The transposed Split has the ramps between the two structures. This results in a very short distance between the entrance and succeeding exit ramps, with significant problems of weaving on the freeway. Scissor ramps are the extreme example of the transposed Split. These require either signalisation of the crossing of the two ramps or a grade separation. The transposed Split has little to recommend it and has fallen into disuse, being discussed here only for completeness of the record.

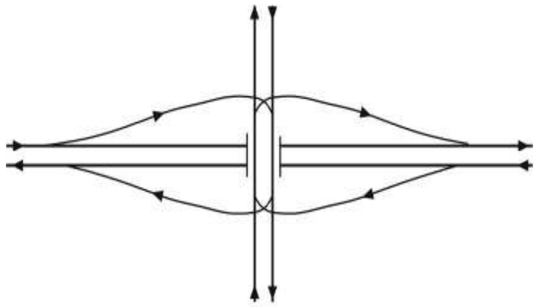
The Single Point Interchange brings the four ramps together at a point over the freeway. This interchange is required where space is at a premium or where the volume of right-turning traffic is very high. The principal operating difference between the Single Point and the Simple Diamond is that, in the former case, the right turns take place outside each other and in the latter they are “hooking” movements. The capacity of the Single Point Interchange is thus higher than that of the Simple Diamond. It does, however, require a three-phase signal plan and also presents pedestrians with wide unprotected crossings.

The various configurations of the Diamond Interchange are illustrated in Figure 9.9

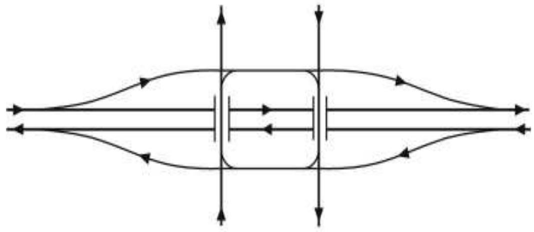
Par-Clo interchanges

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. In practice, they could perhaps be considered rather as a distorted form of Simple Diamond Interchange.

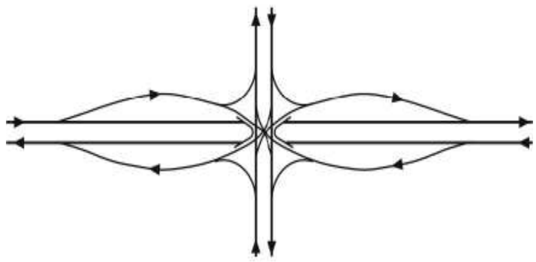
Three configurations of Par-Clo Interchange are possible: the Par-Clo A, the Par-Clo B and the Par-Clo AB. As in the case of the Trumpet Interchange, the letters have the significance of the loops being in advance of or beyond the structure. The Par-Clo AB configuration has the loop in advance of the 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 freeway. Both the Par-Clo A and the Par-Clo B have alternative configurations: the A2 and A4 and the B2 and B4.



Simple diamond



Split diamond



Single point diamond

Figure 9-9: Diamond Interchanges

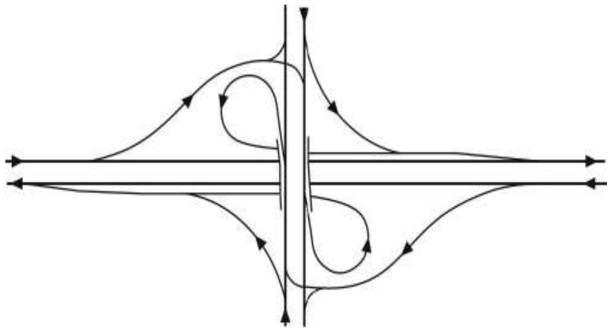
These configurations refer to two quadrants only being occupied or alternatively to all four quadrants having ramps.

The various layouts are illustrated in Figures 9.10, 9.11 and 9.12.

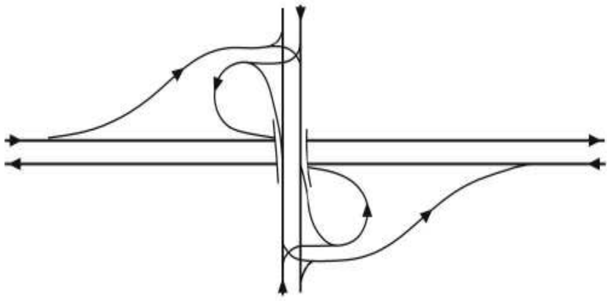
Internationally, the Par-Clo A4 is generally regarded as being the preferred option for an interchange between a freeway and a heavily trafficked arterial. In the first instance, the loops serve vehicles entering the freeway whereas, in the case of the Par-Clo B, the high-speed vehicles exiting the freeway 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 freeway 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.

The Rotary Interchange illustrated in Figure 9.12 has the benefit of eliminating intersections on the crossing road, and replacing them by short weaving sections. Traffic exiting from the freeway may experience difficulty in adjusting speed and merging with traffic on the rotary.



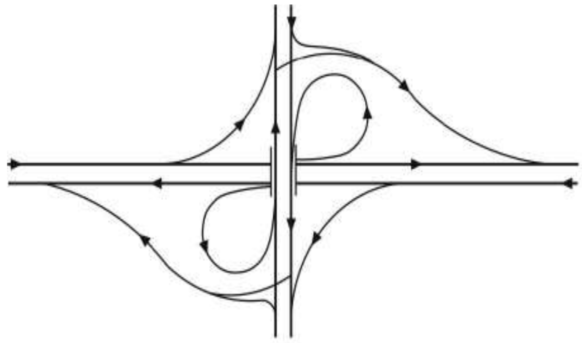
A4



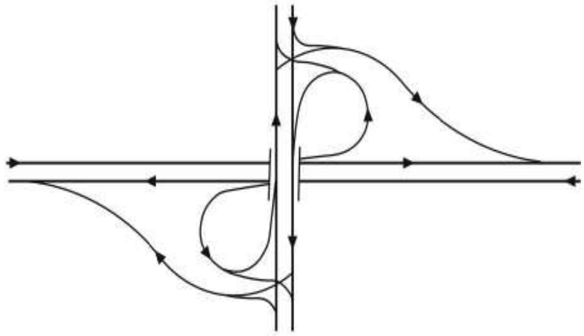
A2

Figure 9-10: Par-Clo A Interchange

Rotaries have also been used in the United Kingdom as systems interchanges. In this configuration, a two-level structure is employed. One freeway is located at ground level and the other freeway on the upper level of the structure with the rotary sandwiched between them. This is the so-called “Island in the Sky” concept.

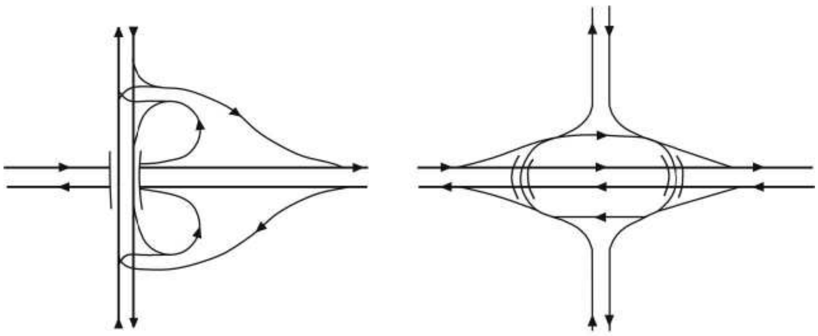


B4



B2

Figure 9.11: Par-Clo B interchanges



Parclo AB Rotary

Figure 9.12: Par-Clo AB interchanges and rotaries

9.7.4 Interchanges on non-freeway Roads

The application of interchanges where a non-freeway is a major route would arise where traffic flows are so heavy that a signalised intersection cannot provide sufficient capacity. In this case, the crossing road terminals would be provided on the road with the lower traffic volume. As a general rule, a simple and relatively low standard Simple Diamond or a Par-Clo Interchange should suffice.

An intersection with a particularly poor accident history may also require upgrading to an interchange. The accident history would provide some indication of the required type of interchange.

Where the need for the interchange derives purely from topographic restraints, i.e. where traffic volumes are low, a Jug Handle Interchange, illustrated in Figure 9.13, would be adequate. This layout, also known as a Quarter Link, provides a two-lane-two-way connection between the intersecting roads located in whatever quadrant entails the minimum construction and property acquisition cost.

Drivers would not expect to find an interchange on a two-lane two-way road and, in terms of driver expectancy, it may therefore be advisable to introduce a short section of dual carriageway at the site of the interchange.

9.8 Ramp Design

9.8.1 General

A ramp is defined as a roadway, usually one-way, connecting two grade-separated through roads. It comprises an entrance terminal, a midsection and an exit terminal.

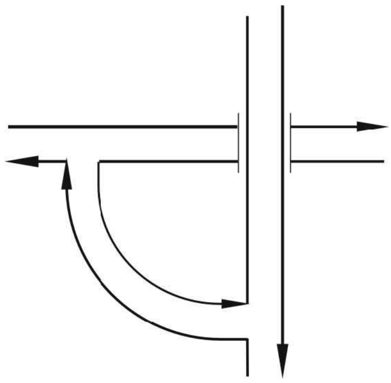


Figure 9.13: Jug Handle interchange

The general configuration of a ramp is determined prior to the interchange type being selected. The specifics of its configuration, being the horizontal and vertical alignment and cross-section, are influenced by a number of considerations such as traffic volume and composition, the geometric and operational characteristics of the roads which it connects, the local topography, traffic control devices and driver expectations.

A variety of ramp configurations can be used. These include:

- The outer connector, which serves the left turn and has free-flowing terminals at either end;
- The diamond ramp, serving both the left-and right-turns with a free-flowing terminal on the freeway and a stop-condition terminal on the cross-road;
- The Par-Clo ramp, which serves the right turn and has a free-flowing terminal on the freeway and a stop-condition terminal on the crossing road, with a 1800 loop between them;
- The loop ramp, serving the right turn and which has free-flowing terminals at both ends and a 2700 degree loop between them;
- The directional ramp also serving the right turn, with a curve only slightly in excess of 900 degrees and free-flowing terminals at either end, and,
- The collector-distributor road intended to remove the weaving manoeuvre from the freeway.

The express-collector system discussed later is a transfer roadway and is not an interchange ramp.

9.8.2 Design speed

Guideline values for ramp design speeds are given in Table 9.2. Strictly speaking, the design speed of a ramp could vary across its length from that of the freeway to that of the at-grade intersection, with the design speed at any point along the ramp matching the operating speed of the vehicles accelerating to or decelerating from the design speed of the freeway. The design speeds given in the table apply to the controlling curve on the mid-section of the ramp. The ramp design speed is shown as a design domain because of the wide variety of site conditions, terminal types and ramp shapes.

Table 9.2: Ramp Design Speed

Roadway Design Speed (km/h)	Ramp Design Speed Domain (km/h)
60	40-50
70	40-60
80	40-70
90	50-80
100	50-90
110	60-100
120	60-110
130	70-110

In the case of a directional ramp between freeways, vehicles must be able to operate safely at the higher end of the ranges of speeds shown in Table 9.2.

Factors demanding a reduction in design speed include site limitations, ramp configurations and economic factors. Provided the reduction is not excessive, drivers are prepared to reduce speed in negotiating a ramp so that the lower design speed is not in conflict with driver expectations.

For directional ramps and outer connectors, higher values in the speed domain are appropriate and, in general, ramp designs should be based on the upper limit of the domain. The constraints of the site, traffic mix and form of interchange, however, may force a lower design speed.

A Par-Clo or a loop ramp cannot be designed to a high design speed. A ramp design speed of 70 km/h, being the low end of the domain for a freeway design speed of 130 km/h, would require a radius of between 150 metres and 200 metres, depending on the rate of superelevation selected. It is not likely, particularly in an urban area, that the space required to accommodate a loop with this radius would be available. Even if the space were available, the additional travel distance imposed by the greater radius would nullify the advantages of the higher travel speed. The added length of roadway also adds the penalty of higher construction cost. This penalty also applies to the ramp outside the loop because it has to be longer to contain the larger loop. In general, loop ramps are designed for speeds of between 40 and 50 km/h. Because of the substantial difference between the freeway design speed and that of the loop ramp, it is advisable not to have a loop on an exit ramp if this can be avoided.

Safety problems on ramp curves are most likely for vehicles travelling faster than the design speed. This problem is more critical on curves with lower design speeds because drivers are more likely to exceed these design speeds than the higher ranges. Trucks can capsize when travelling at speeds only marginally higher than the design speed. To minimise the possibility of trucks exceeding the design speed it is suggested that the lower limits of design speeds shown in Table 9.2 should not be used for ramps carrying a substantial amount of truck traffic.

If site specific constraints preclude the use of design speeds that more-or-less match anticipated operating speeds, the designer should seek to incorporate effective speed controls, such as advisory speed signing, special pavement treatments, long deceleration lanes and the use of express-collector systems in the design.

9.8.3 Sight distance on ramps

It is necessary for the driver to be able to see the road markings defining the start of the taper on exit ramps and the end of the entrance taper. At the crossing road ramp terminal, lanes are often specifically allocated to the turning movements with these lanes being developed in advance of the terminal. The driver has to position the vehicle in the lane appropriate to the desired turn. It is therefore desirable that decision sight distance be provided on the approaches to the ramp as well as across its length.

Appropriate values of decision sight distance are given in Section 6.

9.8.4 Horizontal alignment

Minimum radii of horizontal curvature on ramps should comply with that given in Section 6 for various values of e_{max}. In general, the higher values of e_{max} are used in freeway design and the selected value should also be applied to the ramps.

Achieving the step down of radii from higher to lower design speeds on a loop may require the application of compound curves. In general, the ratio between successive radii should be 1,5 :1 and, as a further refinement, they could be connected by transition curves. The length of each arc is selected to allow for deceleration to the speed appropriate to the next radius at the entry to that arc. In the case of stepping up through successive radii, the same ratio applies but the design speed for the radius selected should match the desired speed at the far end of each arc.

It is recommended that the designer develops a speed profile for the loop and bases the selection of radii and arc lengths on this speed profile. As a rough rule of thumb, the length of each arc should be approximately a third of its radius.

If a crossover crown line, discussed below, is not used, the crossfall on the exit or entrance taper between the Yellow Line Break Point and the nose is controlled by that on the through lanes. Superelevation development can thus only commence at the nose. The distance required to achieve the appropriate superelevation thus determines the earliest possible location of the first curve on the ramp.

Ramps are relatively short and the radii of curves on ramps often approach the minimum for the selected design speed. Furthermore, if there is more than one curve on a ramp, the distance between the successive curves will be short. Under these restrictive conditions, transition curves should be considered.

Ramps are seldom, if ever, cambered and superelevation typically involves rotation around one of the lane edges. Drivers tend to position their vehicles relative to the inside edge of any curve being traversed, i.e. they steer towards the inside of the curve rather than away from the outside. For aesthetic reasons, the inside edge should thus present a smoothly flowing three-dimensional alignment with the outside edge rising and falling to provide the superelevation. Where a ramp has an S- or reverse curve alignment, it follows that first one edge and then the other will be the centre of rotation, with the changeover taking place at the point of zero crossfall.

In view of the restricted distances within which superelevation has to be developed, the crossover crown line is a useful device towards rapid development. A crossover crown is a line at which an instantaneous change of crossfall takes place and which runs diagonally across the lane. The crossover crown could, for example, be located along the yellow line defining the edge of the left lane of the freeway, thus enabling initiation of superelevation for the first curve on the ramp earlier than would otherwise be the case. The crossover crown should, however, be used with caution as it may pose a problem to the driver, particularly to the driver of a vehicle with a high load. This is because the vehicle will sway as it traverses the crossover crown and, in extreme cases, may prove difficult to control. The algebraic difference in slope across the crossover crown should thus not exceed four to five per cent.

9.8.5 Vertical Alignment

Gradients

The profile of a ramp typically comprises a midsection with an appreciable gradient coupled with terminals where the gradient is controlled by the adjacent road. If the crossing road is over the freeway, the positive gradient on the off-ramps will assist a rapid but comfortable deceleration and the negative gradient on the on-ramp will support acceleration to freeway speeds. In theory, thus, the higher the value of gradient, the better. Values of gradient up to eight per cent can be considered but, for preference, gradients should not exceed six per cent. Diamond ramps are usually fairly short, possibly having as little as 120 to 360 metres between the nose and the crossing road. The effect of the midsection gradient, while possibly helpful, is thus restricted. However, a steep gradient (8 per cent) in conjunction with a high value of superelevation (10 per cent) would have a resultant of 12,8 per cent at an angle of 530 to the centreline of the ramp. This would not contribute to drivers' sense of safety. In addition, the drivers of slow-moving trucks would have to steer outwards to a marked extent to maintain their path within the limits of the ramp width. This could create some difficulty for them. It is suggested that designers seek a combination of superelevation and gradient such that the gradient of the resultant is less than ten per cent. Table 9.3 provides an indication of the gradients of the resultants of combinations of superelevation and longitudinal gradient.

Table 9.3: Maximum Resultant Gradients

Superelevation (%)	Longitudinal Gradient (%)			
	2	4	6	8
4	4,47	5,65	7,21	8,94
6	6,31	7,20	8,49	10,00
8	8,25	8,94	10,00	11,31
10	10,19	10,73	11,63	12,80

The combinations of gradient and superelevation shown shaded in Table 9.3 should be avoided.

Curves

As suggested above, decision sight distance should be available at critical points on ramps. The K-values of crest curves required to meet this requirement are given in Table 9.4.

These values of K are based on the required sight distance being contained within the length of the vertical curve. It is, however, unlikely that, within the confined length of a ramp, it would be possible to accommodate the length of vertical curve required for this condition to materialise.

Table 9.4: K-Values of Crest Curvature for Decision Sight Distance

Design Speed (km/h)	K-Value
40	118
50	161
60	210
70	226
80	328
90	473
100	643
110	738
120	840
130	948

The designer should therefore have recourse to the following equation to calculate the K-value for the condition of the curve being shorter than the required sight distance.

$$K = \frac{2S}{A} - \frac{200(h_1^{0,5} + h_2^{0,5})^2}{A^2}$$

where:	K	=	Distance required for a 1%change of gradient (m)
	S	=	Stopping sight distance for selected design speed (m)
	h1	=	Driver eye height (m)
	h2	=	Object height (m)
	A	=	Algebraic difference in gradient between the approaching and departing grades (%)

On sag curves, achieving decision sight distance is not a problem so that the K-values for sag vertical curve given in Section 6 can be applied. If the interchange area is illuminated, as would typically be the case in an urban area, the K-values for comfort which are roughly half of those dictated by stopping sight distance can be used. Unilluminated, i.e. rural, ineterchanges would require the application of K-val-ues appropriate to headlight sight distance.

9.8.6 Cross-section

Horizontal radii on ramps are sharp, requiring widening of curve. The width of the ramps normally adopted is thus 4 metres. Because of the inconvenience of changing the lane width of comparatively short sections, this width is applied across the entire length of the ramp.

In the case of a loop ramp, the radius could be as low as 50 metres. At this radius, a semi trailer would require a lane width of 5.07 metres. It is necessary for the designer to consider the type of vehicle selected for design purposes and to check whether the four metre nominal width is adequate. If semi trailers are infrequent users of the ramp, encroachment on the shoulders could be considered.

Lane widening is described in detail in Section 6. As suggested above, regardless of the width required, it should be applied across the entire length of the ramp.

Ramp shoulders typically have a width of the order of 2 metres, with this width applying both to the inner and to the outer shoulder. In conjunction with the nominal lane width of 4 metres, the total roadway width is thus 8 metres. This width would allow comfortably for a truck to pass a broken-down truck. In addition, it would provide drivers with some sense of security in the cases where the ramp is on a high fill.

9.8.7 Terminals

The crossing road ramp terminals may be free-flowing, in which case their design is as discussed below. Crossing road ramp terminals that are at-grade intersections should be designed according to the recommendations contained in Section 8. It is worth noting that, from an operational point of view, what appears to be a four-legged intersection, e.g. the crossing road ramp terminals on a Diamond Interchange, is, in fact, two three-legged intersections back-to-back. Ideally, the crossing road ramp terminal should be channelised to reduce the possibility of wrong-way driving.

Vehicles entering or exiting from a freeway should be able to do so at approximately the operating speed of the freeway. Given the fact that the crossing road terminal is invariably a signalised or stop control at-grade intersection, the change in speed across the ramp is substantial. Provision should thus be made for acceleration and deceleration to take place clear of the freeway so as to minimise interference with the through traffic and reduce the potential for crashes. The auxiliary lanes provided to accommodate this are referred to as speed-change lanes or acceleration or deceleration lanes. These terms describe the area adjacent to the travelled way of the freeway, including that portion of the ramp taper where a merging vehicle is still clear of the through lane, and do not imply a definite lane of uniform width.

The speed change lane should have sufficient length to allow the necessary adjustment in speed to be made in a comfortable manner and, in the case of an acceleration lane, there should also be sufficient length for the driver to find and manoeuvre into a gap in the through traffic stream before reaching the end of the acceleration lane. The length of the speed change lane is based on:

- The design speed on the through lane, i.e. the speed at which vehicles enter or exit from the through lanes;
- The control speed of the ramp midsection, i.e. the design speed of the smallest radius curve on the ramp, and
- The tempo of acceleration or deceleration applied on the speed change lane.

Research has shown that deceleration rates applied to off-ramps are a function of the freeway design speed and the ramp control speed. As both speeds increase, so does the deceleration rate, which varies between 1.0 m/s2 and 2,0 m/s2. For convenience, the deceleration rate used to develop Table 9.5 has been set at 2.0 m/s2.

Deceleration should only commence once the exiting vehicle is clear of the through lane. Assuming that a vehicle with a width of 2.5 metres is correctly positioned relative to the ramp edge line to be centrally located within an ultimate ramp width of 4 metres, its tail end will clear the edge of the through lane at a distance, L, from the Yellow Line Break Point where

$$L = 3.2 / T$$

with T = Taper rate (as listed in Table 9.7)

The maximum length of design vehicles which is 21 metres and this should be added to the distance, L, to establish the distance from the Yellow Line Break Point, at which the deceleration can commence so that

$$L_T = 3.2/T + 21$$

Values of L_T, are listed in Table 9.5. From this table it follows that, in the case of a diamond ramp without curves, the distance from the Yellow Line Break Point to the crossing road ramp terminal should be not less than 436 metres.

The acceleration rate can, according to American literature, be taken as 0.7 m/s^2. The length of the acceleration lane is thus as shown in Table 9.6. An important feature of the acceleration lane is the gap acceptance length, which should be a minimum of 100 metres to 150 metres, depending on the nose width. The length of the entering vehicle is not relevant in determination of the taper length.

The lengths of the deceleration and acceleration lanes shown in Tables 9.5 and 9.6 apply to gradients of between - 3 per cent and + 3 per cent. Acceleration lanes will have to be longer on upgrades and may be made shorter on downgrades, with the reverse applying to deceleration lanes.

The actual entrance or exit points between the freeway and the ramp can take the form of a taper or a parallel lane. The parallel lane has the problem of forcing a reverse curve path, possibly followed after the nose by a further curve to the left, whereas the taper involves only a single change of direction. As such, drivers prefer the taper. In cases where an extended acceleration or deceleration distance is required, for example where the crossing road passes under the freeway resulting in the on-ramp being on an upgrade and the off-ramp on a downgrade, a straight taper would result in an inordinately long ramp. The parallel lane allows the speed change to take place in an auxiliary lane immediately adjacent to the through lanes thus reducing the spatial demands of the interchange. Furthermore, in the case of the on-ramp, the parallel configuration provides an extended distance to find a gap in the adjacent traffic stream. Typical configurations of on- and off-ramp terminals are illustrated in Figures 9.14 to 9.17.

Taper

Two dofferent criteria apply to the selection of the taper rate, depending on whether the ramp is an exit from or entrance to the freeway.

Table 9.5: Length of Deceleration Lanes (m)

Freeway design speed (km/h)	Ramp control speed (km/h)								
	L_T (m)	0	40	50	60	70	80	90	100
60	66	70	60	21					
70	75	95	85	45	25				
80	75	125	115	75	55	30			
90	85	155	150	110	85	60	30		
100	91	190	185	145	125	100	70	35	
112	101	235	225	185	165	140	110	75	40
120	107	280	270	230	210	180	155	120	85
130	111	325	320	275	225	230	200	170	135

Table 9.6: Length of Acceleration Lanes (m)

Freeway design speed (km/h)	Ramp controlling speed (km/h)								
	L_T (m)	0	40	50	60	70	80	90	100
60	45	200	110	60					
70	54	270	180	130	70				
80	54	350	265	215	155	80			
90	64	450	360	310	250	175	95		
100	70	550	460	415	350	280	200	105	
110	80	670	580	530	470	395	315	220	115
120	86	790	700	655	595	525	440	350	240
130	90	930	840	795	730	660	580	485	380

In the first case, the vehicle is merely required to achieve a change of direction from the freeway to the ramp. The taper rate should thus be sufficiently flat to ensure that the vehicle path can be accommodated within the lane width. In Table 9.7, the radii of curvature corresponding to a superelevation of 2.0 per cent for the various design speeds are listed as are the taper rates corresponding to the condition of the wheel path having an offset of 150 millimetres inside the Yellow Line Break Point.

This rate derives from the application of an “operating speed” previously assumed to be 85 per cent of the design speed and the further assumption that the wheel path could be allowed to pass over the Yellow Line Break Point. In practice, a 1: 15 rate requires that drivers accept a higher level of side friction in negotiating the change of direction. Passenger cars can negotiate tapers of this magnitude with ease so that, in the case of existing interchanges, it is not always necessary to incur the expenditure of upgrading to a flatter taper. Trucks do, however, sometimes roll over at the start of the ramp taper.

Table 9.7: Taper Rates for Exit Ramps

Design Speed (km/h)	Radius (m) for 2% superelevation	Taper rate 1:	Taper length, L_T (m)
60	1000	14	66
70	1500	17	75
80	1500	17	75
90	2000	20	85
100	2500	22	91
112	3000	25	101
120	3500	27	107
130	4000	28	111

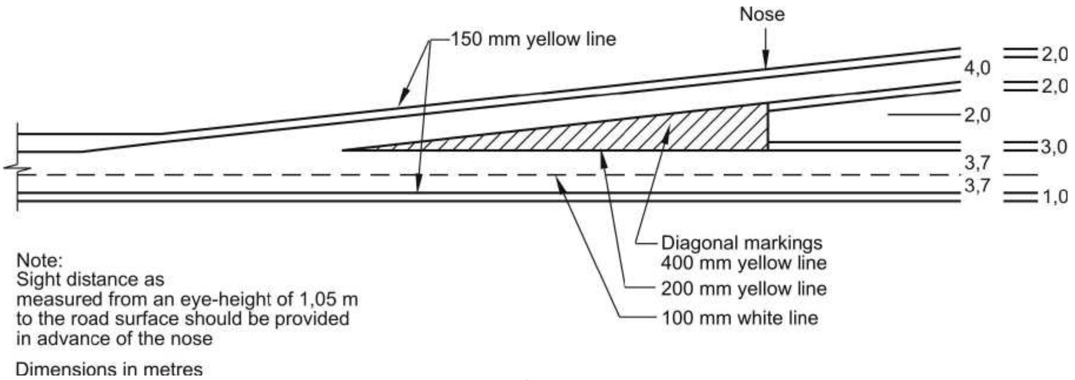


Figure 9-14: Single Lane Exit

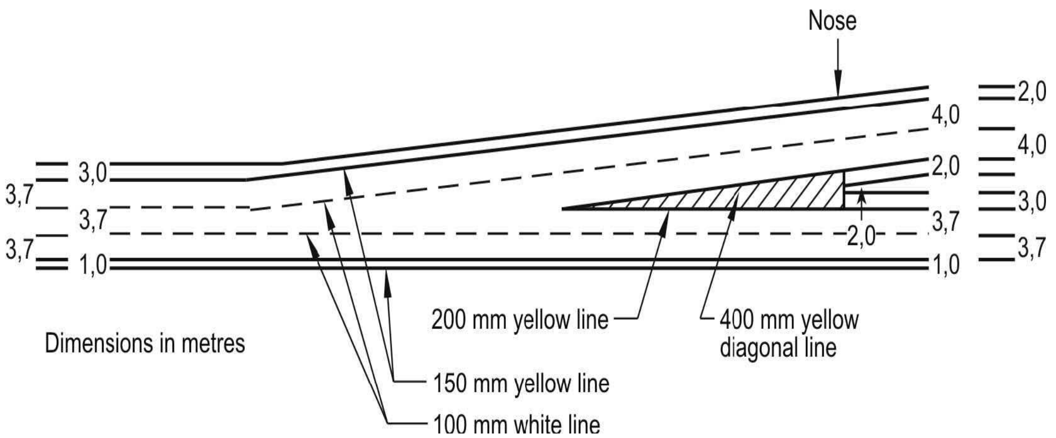
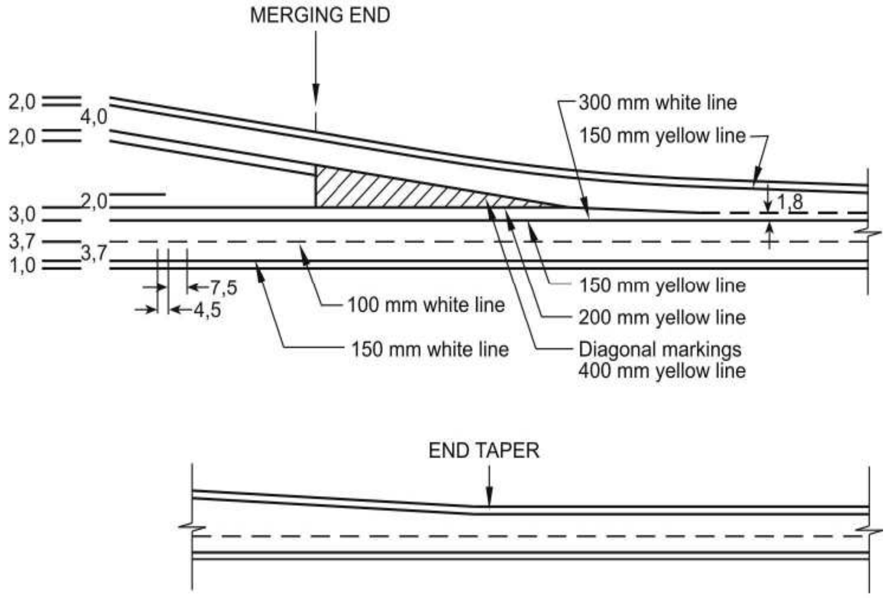


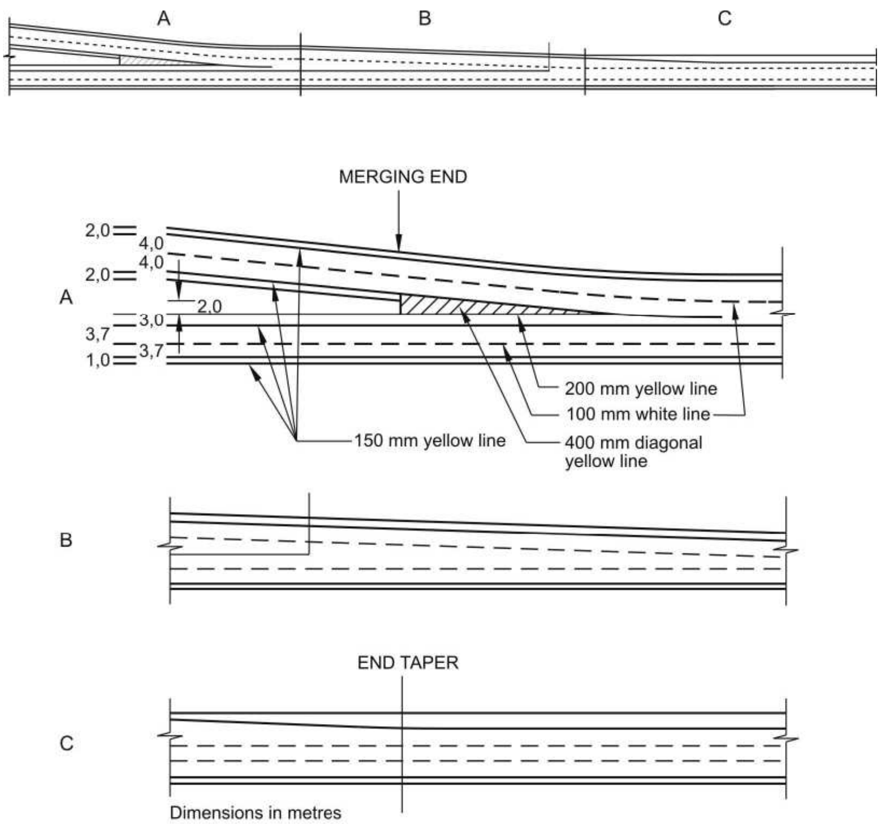
Figure 9-15: Two - Lane Exit

Note:
For 50 m advance of the merging end, the ramp should not be more than 0,75 m lower than the freeway



Dimensions in metres

Figure 9-16: One Lane Entrance



Note:
For 50 m in advance of the merging end, the ramp should not be more than 0,75 m lower than the freeway

Figure 9:17: Two-lane entrance

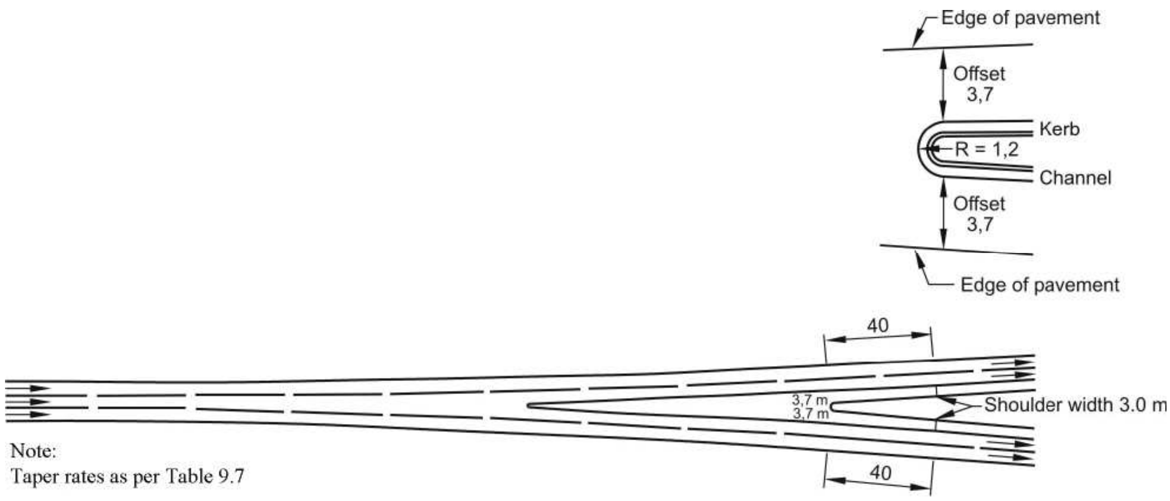


Figure 9-18: Major Fork

If there is a high percentage of truck traffic and if the incidence of roll-overs is unacceptably high, it may be necessary to consider upgrading to the tapers suggested in Table 9.7.

In the case of the entrance ramp, the driver of the merging vehicle should be afforded sufficient opportunity to locate a gap in the opposing traffic and position the vehicle correctly to merge into this gap at the speed of the through traffic. Taper rates flatter than those proposed in Table 9.7 should thus be applied to on-ramps. A taper of 1:50 provides a travel time of about 13 seconds between the nose and the point at which the wheel paths of merging and through vehicles would intersect. This has been found in practice to provide sufficient opportunity to prepare to merge into gaps in the through flow because drivers typically begin to locate usable gaps in the freeway flow prior to passing the nose.

Parallel

The parallel lane configuration is also initiated by means of a taper but, in this case, the taper rate does not have to be as flat as suggested above. As in the case of the climbing lane, a taper length of 100 metres would be adequate.

Parallel entrances and exits have a major advantage over straight tapers in respect of the selection of curves on the ramps. In the case of a design speed of 120 km/h, a straight taper provides a distance of just short of 300 metres between the Yellow Line Break Point and the nose. A portion of this distance should ideally be traversed at the design speed of the freeway with deceleration commencing only after the vehicle is clear of the outside freeway lane. About 220 metres is available for deceleration and this suggests that a vehicle exiting at 120 km/h is likely still to be travelling at a speed of about 95 km/h when passing the nose. The first curve on the ramp could, allowing for superelevation development to 10 per cent, be located 100 metres beyond the nose so that, at the start of the curve, the vehicle speed would be in the range of 80 to 90 km/h. The first curve should thus have a radius of not less than 250 metres. In the case of the parallel ramp, the radius could obviously be significantly shorter, depending on the length of the deceleration lane preceding the nose.

9.9 Collector - Distributor Roads

Collector-distributor roads are typically applied to the situation where weaving manoeuvres would be disruptive if allowed to occur on the freeway. Their most common application, therefore, is at Cloverleaf interchanges. The exit and entrance tapers are identical to those applied to any other ramps. The major difference between Cloverleaf interchange C-D roads and other ramps is that they involve two exits and two entrances in quick succession. The two exits are, firstly, from the freeway and, secondly, the split between vehicles turning to the left and those intending to turn to the right. The two entrances are, firstly, the merge between the two turning movements towards the freeway and, secondly, the merge with the freeway through traffic.

The distance between the successive exits should be based on signing requirements so as to afford drivers adequate time to establish whether they have to turn to the right or to the left to reach their destination. Nine seconds is generally considered adequate for this purpose and seeing that vehicles may be travelling at the design speed of the freeway as they pass the nose of the first exit, the distance between the noses should desirably be based on this speed.

The distance between successive entrances is based on the length required for the acceleration lane length quoted in Table 9.6.

9.10 Other Interchange Design Features

9.10.1 Ramp Metering

Ramp metering consists of traffic signals installed on entrance ramps in advance of the entrance terminal to control the number of vehicles entering the freeway. The traffic signals may be pre-timed or traffic-actuated to release the entering vehicles individually or in platoons. It is applied to restrict the number of vehicles that are allowed to enter a freeway in order to ensure an acceptable level of service on the freeway or to ensure that the capacity of the freeway is not exceeded. The need for ramp metering may arise owing to factors such as:

- Recurring congestion because traffic demand exceeds the provision of road infrastructure in an area;
- Sporadic congestion on isolated sections of a freeway because of short term traffic loads from special events, normally of a recreational nature;
- As part of an incident management system to assist in situations where a accident downstream of the entrance ramp causes a temporary drop in the capacity of the freeway; and,
- Optimising traffic flow on freeways.

Ramp metering also supports local transportation management objectives such as:

- Priority treatments with higher levels of service for High Occupancy Vehicles; and,
- Redistribution of access demand to other on-ramps.

It is important to realise that ramp metering should be considered a last resort rather than as a first option in securing an adequate level of service on the freeway. Prior to its implementation, all alternate means of improving the capacity of the freeway or its operating characteristics or reducing the traffic demand on the freeway should be explored. The application of ramp metering should be preceded by an engineering analysis of the physical and traffic conditions on the freeway facilities likely to be affected. These facilities include the ramps, the ramp terminals and the local streets likely to be affected by metering as well as the freeway section involved.

The “stopline” should be placed sufficiently in advance of the point, at which ramp traffic will enter the freeway, to allow vehicles to accelerate to approximately the operating speed of the freeway, as would normally be required for the design of ramps. It will also be necessary to ensure that the ramp has sufficient storage to accommodate the vehicles queuing upstream of the traffic signal.

The above requirement will almost certainly lead to a need for reconstruction of any ramp that is to be metered. The length of on-ramps is typically determined by the distance required to enable a vehicle to accelerate to freeway speeds. Without reconstruction, this could result in the ramp metering actually being installed at the crossing-road ramp terminal.

9.10.2 Express-collector systems

Express-collector systems are used where traffic volumes dictate a freeway width greater than four lanes in each direction. The purpose of the express-collector is to eliminate weaving on the mainline lanes by limiting the number of entrance and exit points while satisfying the demand for access to the freeway system.

An express-collector system could, for example, be started upstream of one interchange and run through it and the following, possibly closely spaced, interchange, terminating downstream of the second. The terminals at either end of the express-collector system would have the same standards as applied to conventional on- and off-ramps. The interchange ramps are connected to the express-collector system and not directly to the freeway mainline lanes.

Traffic volumes and speeds on the express-collector roads are typically much lower than those found on the mainline lanes, allowing for lower standards being applied to the ramp geometry of the intervening interchanges.

The minimum configuration for an express-collector system is to have a two-lane C-D road on either side of a freeway with two lanes in each direction. The usual configuration has more than two mainline lanes in each direction

A similar configuration is known as a dual-divided freeway, sometimes referred to as a dual-dual freeway. In this case, the C-D roads are taken over a considerable distance and may have more than two lanes. Furthermore, connections are provided at intervals between the outer and the core lanes along the length of the freeway. These, unfortunately, create the effect of right-side entrances and exits to and from the outer lanes and are thus contrary to drivers' expectations. The geometry of the situation is otherwise similar to that of the express-collector system.

SECTION 10: SPEED MANAGEMENT

10.1 Introduction

Major traffic safety problems arise when main roads pass through trading centres and towns. This is because of the mix of long-distance high-speed motor vehicles with local access traffic, parking and vulnerable road users. The safest solution and by far the most expensive is to build a by-pass. If this is not possible, a number of traffic safety measures must be implemented, such as:

- Speed limit and speed control measures;
- Separate vulnerable road users from the motor traffic by providing footways and cycleways; and,
- Separate the local traffic from the through traffic by providing service roads.

Through roads with heavy traffic can also be provided with a median to improve traffic safety. U-turns should then preferably be achieved by use of roundabouts, which maybe false, i.e. no connecting roads.

This section focuses on speed limits and speed control measures in built-up areas. However some of the measures described may have uses in other situations, such as in advance of hazardous bends or bridges. Advice on footways, cycleways and service roads is given in other sections.

10.2 Speed Management Principles

Through roads in trading centres and towns speed shall be limited to 50 km/h or less. On the busier sections these speed limits must be reinforced by speed control measures. The speed management principles for main roads through built-up areas are summarised in Figure 1.-1.

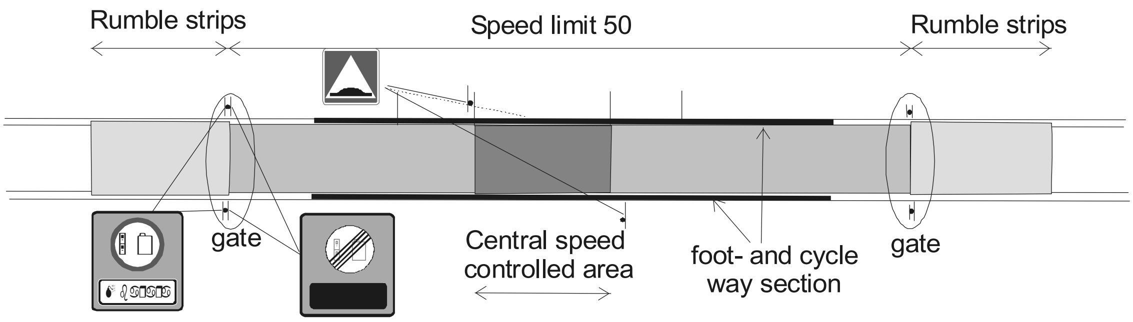


Figure 10-1: Speed management principles on 50km/h through road

The standard sequence is:
- rumble strips;
- gateway sign and gate;
- start of footways; and,
- speed controlled area – normally speed humps.

The gateway sign is double-sided and combines the speed limit sign with a panel showing the place name. The gateway sign should be preceded by rumble strips to alert drivers. Warning signs are not normally necessary. The rural speed limit should be taken down to 50 km/h by an intermediate speed limit.

The entrance to the built up area should be marked with a gate (see below). The cross-section within part of and sometimes all the 50 km/h area should normally have separate footways. Major trading centres and towns should also have service roads.

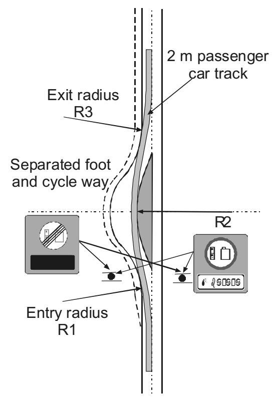
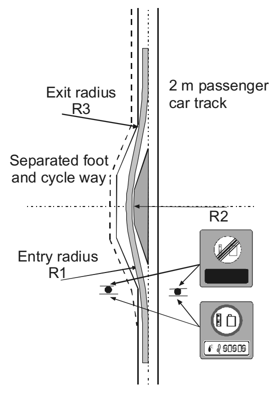
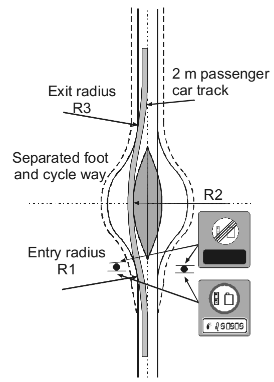
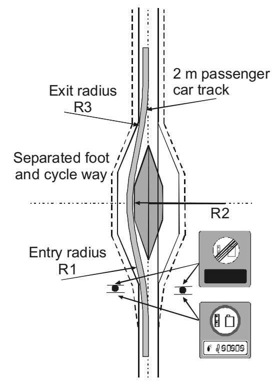
Details of the individual speed control measures are given below.

10.3 Speed Control Measures

10.3.1 Gates

It is necessary to emphasise the speed limit change at the entrance to the built-up area with a gate to signal very clearly to motor vehicle drivers that driving conditions are to change. The gate should preferably be designed according to speed control principles, i.e., the toughest vehicle path for a passenger car, through the gate or portal should have an entry radius R1 below 100 m for 50 km/h speed control and 50 m for 30 km/h speed control. Curves that follow (R2, R3) should have a radius greater than or equal to the entry radius. The gate could be one-sided with speed control only in the entry direction or two-sided with speed control also in the exit direction. The design can be tapered or smoothed with curves.

The narrowing of the carriageway through the gate can put pedestrians and cyclists at risk of being squeezed by motor vehicles. It is recommended that short footway / cycle by-passes be built around the gates.

entry speed controlled gate		entry and exit speed controlled gate	
smooth design	taper design	smooth design	taper design
			

10.3.2 Speed control zone

Principles

The next step is to use speed controlling measures within the busier part of the 50 km/h speed limit section.

The maximum intervals between speed control devices to achieve speeds in the range of:

- 30 km/h is preferably 50 m and not more than 125 m.; and,
- 50 km/h is preferably 125 m and not more than 175 m.

Speed control is most effectively achieved by humps. Speed control should preferably be located where judged reasonable for drivers. Sometimes formal pedestrian crossings could be implemented combined with humps, see Section 11.1.2. Well-designed roundabouts are also very effective speed control measures and are highly recommended. False roundabouts (i.e. where there is no intersection) are worth considering.

Humps

The most effective device to use for speed control is the speed hump. Two alternative designs have proved to be most effective. These are the circular hump and the plateau hump. The geometric designs are shown Figure 10-3 for length profile.

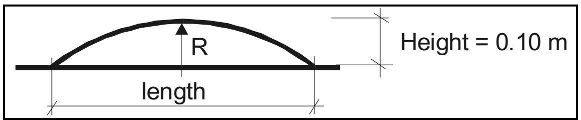
	Length profile
circular	
plateau	

Figure 10-3: Alternative design of humps

The circular hump is normally recommended. The plateau hump can be used in pedestrian and cycle crossings flush with connecting foot and cycle ways.

The height of the circular hump should be 100 mm. Various hump radii and chord lengths are given in Table 10.1. These are based on empirical studies into hump dimensions, speed, and driver / passenger discomfort. The 4 m long design giving car speeds in the range 30 km/h is recommended in residential areas. Main through roads with large ratios of trucks and buses should normally have the 6.0 m long design to ease discomfort for bus passengers – the chord is longer than the normal axle width.

Table 10-1: Detailed design of circular humps



car (truck) speed level	Radius (m)	Length (m)
20 (5)	11	3.0
25 (10)	15	3.5
30 (15)	20	4.0
35 (20)	31	5.0
40 (25)	53	6.5
45 (30)	80	8.0
50 (35)	113	9.5
(40)	180	12.0

The traffic level-of-service, especially for buses and trucks, can be improved if the hump entry and exit is smoothed with a fillet as shown below.

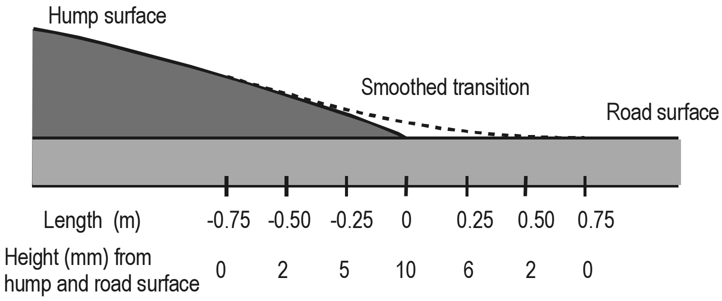
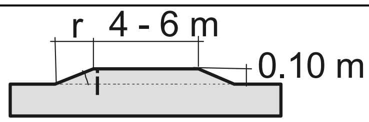


Figure 10-4: Detailed design of hump transition

The height of the plateau hump should also be 0.10 m. Table 10.2 gives recommended ramp grades and lengths based on empirical studies. The design with 1.0 m ramp length and 10% grade giving car speed levels around 30 km/h is recommended for residential roads. Main through roads with large ratios of trucks and buses should normally have the 6.0 m long design to ease discomfort for bus passengers.

Table 10-2: Detailed design of plateau humps



car (truck) speed level	ramp length r (m)	Grade i (%)
	0.7	14
25 (5)	0.8	12.5
30 (10)	1.0	10
35 (15)	1.3	7.5
40 (20)	1.7	6
45 (25)	2.0	5
50 (30)	2.5	4
(35)	3.3	3
(40)	4.0	2.5

Note:

- On a road with hard shoulders the hump must extend over the shoulder for a 1 m or so.
- Consider whether the hump will interfere with drainage. On roads with kerbed footways you may have to stop the hump 100 – 150 mm before the kerb to create a drain. This solution cannot be used at a flush pedestrian crossing.

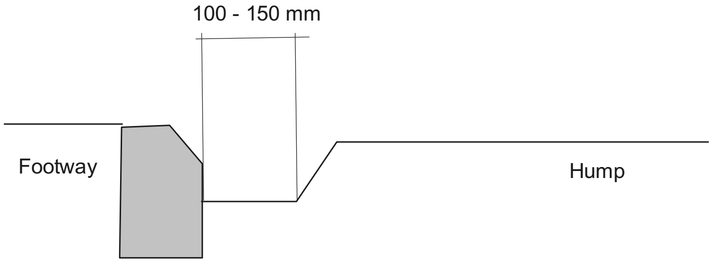


Figure 10-5:Example of hump drainage design at kerbed footway

Humps are only allowed on roads with speed limit 50 km/h or lower. They should always be clearly marked, as illustrated in Figure 10-6, with chequerboard markers (sign no. M35) and hump information signs (sign no. I02) in each direction of the road. Hump warning signs (sign no. W24) might also be needed.

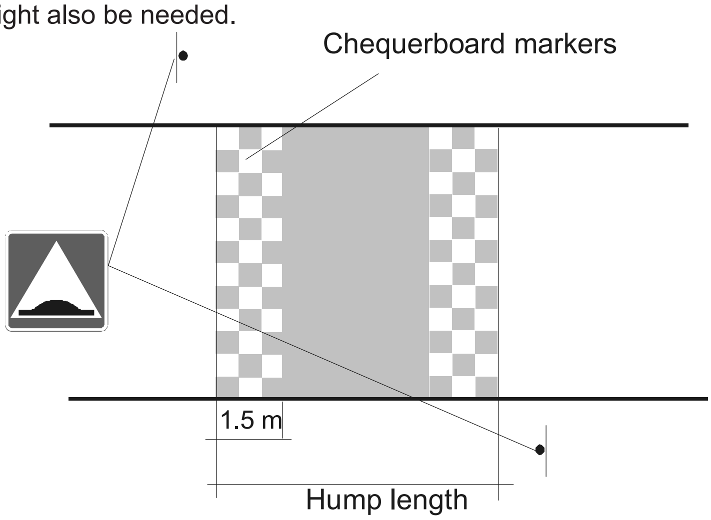


Figure 10-6: Markings and signing of humps

Rumble Strips

Rumble strips are transverse strips across the road used to alert and warn drivers with a vibratory and audible effect before a hazard such as a sharp bend, an intersection or a lower speed limit at the entry to a trading centre. Warning signs are not normally needed when the strips are built to the specifications given below.

Research in other countries indicates that speed reduction effects tend to be minor and also erode over time. Reliance should therefore not be placed on using rumble strips alone to reduce speeds.

Rumble strips can be used for example in the following situations:

- before a local speed limit;
- at an approach to a dangerous intersection;
- before a sharp bend; and,
- before a hump.

The following principles should be observed when using rumble strips:

- rumble strips should normally be in groups of 4 strips;
- the height of the strips shall be no more than 10 – 15 mm;
- the strip width should be 0.5 m;
- one set of rumble strips is usually enough within 50km/h sections;
- the last or only strip should be located 30 to 50 m before the hazard;
- pre-warning sets can, if used, be located 20 to 80 m before the hazard depending on speeds;
- rumble strips should preferably have yellow thermoplastic lines across the top for better visibility; and,
- strips should continue across the full width of the carriageway, including the shoulders but be terminated so that they do not interfere with drainage.

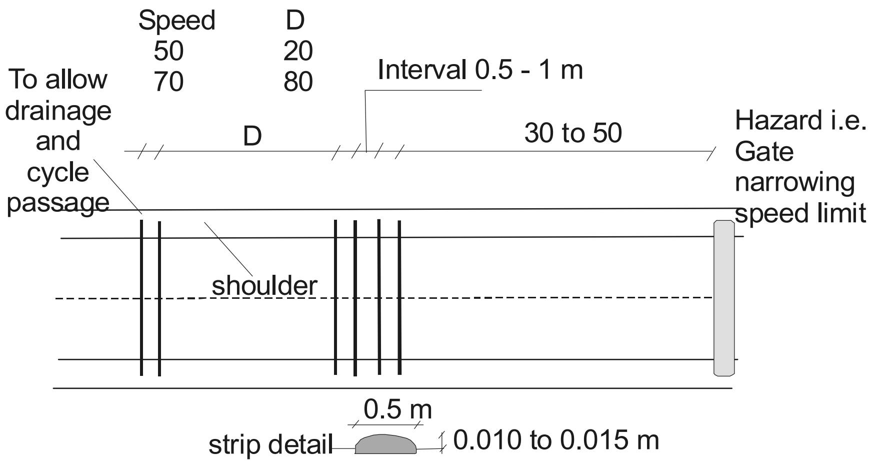


Figure 10-7: Design of rumble strips

Rumble strips create disturbing noises and can cause vibration problems on soft ground. Avoid installing them near houses, schools, hospitals, etc.

Narrowings and Chicanes

Road narrowings and chicanes can help control speeds, but they tend to be less effective than speed humps. The basic principles of one-sided and double-sided road narrowings are illustrated in Figure 10-8 below. Narrowings can put cyclists at risk of being squeezed by larger vehicles, so it is best to provide a short by-pass for them.

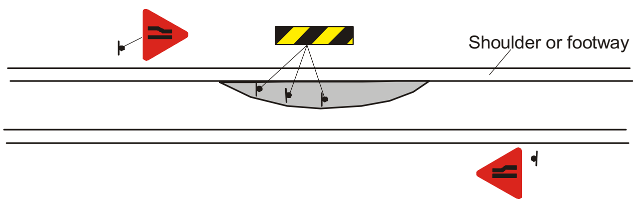
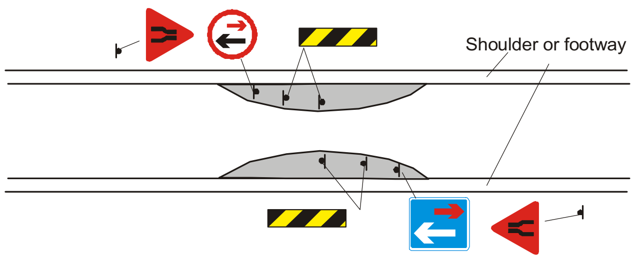
One sided narrowing	
Double sided narrowing	

Figure 10-8: Design of road narrowings

Table 10-3: can be used to judge the relationship between speeds and meeting situations. The conclusion is that a narrowing must be very tough to have some speed impact - 3.5 m is the recommended width between kerbs for one-way traffic. The narrowing should be designed with tapers at least 1:5.

Table 10-3: Road width for different speeds and meeting situations

Speed km/h	Meeting situation				
	Truck and bicycle	Two cars	Truck and car	Two trucks	
30	4.0 m	4.15 m	4.95 m		5.9 m
50	4.5 m	4.5 m	5.5 m	-	

Another alternative is to build a kerbed island (min. width 1.2 m and length 5 m) in the centre of the road, with 3.0 m – 3.5 m wide traffic lanes either side. This could also function as a pedestrian refuge, perhaps combined with a raised pedestrian crossing. The island must be well-signed to avoid it becoming a hazard. As with all narrowings, consider whether cyclists may be put at risk.

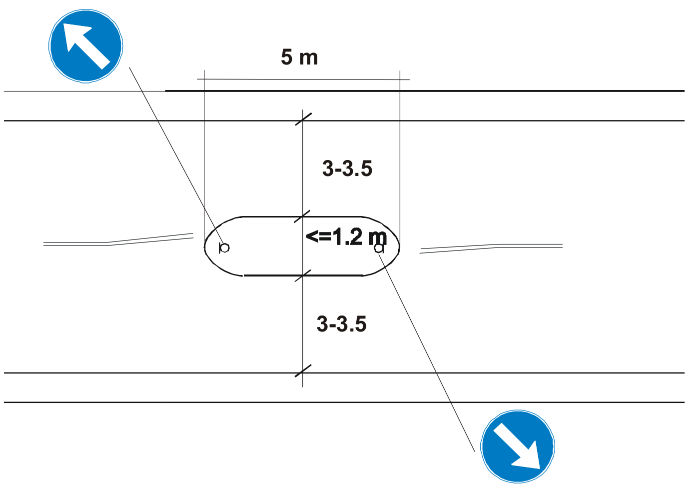


Figure 10-9: Example of road narrowing with central island

SECTION 11: OTHER ROAD FACILITIES

11.1 Pedestrian Facilities

In the past the needs of pedestrians were largely ignored, and this may be one reason why so many pedestrians are killed and injured on our roads. Pedestrians have as much right to use the road as motorists, and roads must be designed with their needs in mind. The first step is to identify major pedestrian generators (markets, shops, schools, etc.) and determine which are the most important pedestrian routes. The aim should be to develop a network of pedestrian routes and crossing facilities that is convenient to use and avoids conflicts with vehicular traffic.

11.1.1 Shoulders and Footways

The conventional view is that pedestrians in rural areas can walk on the road shoulders. The shoulder should be at least 1.5m wide, though 1m is just acceptable if there are constraints. The surface must be well drained and be as smooth as the traffic lanes – if not, pedestrians may prefer to walk in the traffic lane. The implication of this is that low-cost chip seal shoulders may not be a good investment. Letting pedestrians use the shoulders is not entirely satisfactory, as there is nothing to protect the pedestrian from speeding traffic. This is of particular concern on high-speed and / or high volume roads. In these situations it is preferable to provide a separate footway several metres beyond the edge of the shoulder – and separated from it by a grass strip (see also Section 7 – Cross-Section). Some criteria for the provision of footways are given in Table 11-1 below but these should be used with caution – in some circumstances footways can be justified at lower pedestrian flows.

Table 11-1: Criteria for provision of footways

Location of footway	Average daily vehicle traffic	Pedestrian flow per day	
		Speed limit of 60 – 80 km/h	Speed limit of 80 – 100km/h
One side only	400 to 1,400	300	200
	> 1,400	200	120
Both sides	700 to 1,400	1,000	600
	> 1,400	600	400

Standard footway widths are:

Absolute minimum: 1m (two persons cannot pass each other)

Desirable minimum: 1.8m (two persons can pass each other closely)

Light volume: 2.25m (two persons can pass each other comfortably)

Heavy volume: 3.5m + (space for three persons)

In urban areas the footways are normally raised, and edged with barrier kerbs. Barrier kerbs should normally be 100 – 150mm high. Higher kerbs (200mm) are sometimes used in order to deter vehicles from parking on the footway, but these are not recommended for general use, because they are too high for most pedestrians – who will prefer to walk in the traffic lane. The kerb should be lowered at all pedestrian crossings, and where private entrances, footpaths, and cycle tracks enter the carriageway. The “dropped kerb” (Figure 11-1) is particularly helpful to disabled persons.

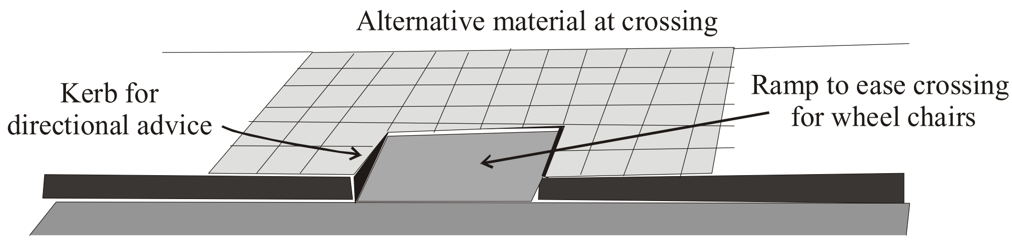


Figure 11-1: Dropped kerb

11.1.2 Pedestrian Crossing Facilities

It is difficult to set down criteria for the provision of pedestrian crossing facilities. Factors to take into account include:

- the volume of pedestrians crossing the road;
- the speed of the traffic;
- the width of the road;
- whether there are a lot of children crossing; and
- whether there are significant numbers of disabled pedestrians.

There are many things that can be done to help pedestrians cross the road, including:

- Formal crossings;
  i. uncontrolled (zebra) crossings; and
  ii. controlled (signal-controlled) crossings.
- Humped pedestrian crossings;
- Build-outs;
- Refuge islands;
- Medians;
- Footbridges (see next sub-section); and,
- Subways (see next sub-section).

The rules for the use of uncontrolled (zebra) and signal-controlled crossings are set out in the law. These formal crossings should be used where there are high volumes of pedestrians trying to cross wide and / or busy roads. If formal crossings (zebra crossings especially) are used in places where there is no obvious need, drivers will have even less respect for them than they have at present. The safety benefits of zebra crossings depend on the discipline amongst drivers. Where discipline exists and all drivers stopped for pedestrians, this can lead o severe congestion at busier crossings. Signal-controlled crossings, though more expensive to install and maintain, are likely to perform better, though there are not enough in Uganda to be sure of this. Refer to the Traffic Signs Manual for details of the layout of these crossings.

Experience in other countries suggests that zebra crossings can be made to work better if the crossing is marked on top of a plateau road hump (see Section 10), Speed Management). 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. Figure 11-2 shows the use of build-outs and refuge islands with a zebra crossing, but they can also be used on their own.

Build-outs are useful on wide roads where there is roadside parking, because they extend the footway further into the road thus improving intervisibility 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 R75, “Pass left side” signs and M53 traffic island markings. Alternatively, create refuges out of road markings (M53 traffic island marking) and put rumble strips within them. Providing refuge islands along the whole length of the road is the safest arrangement, and this will also help to discourage unsafe speeds and overtaking. Refuge islands should be 1.5 – 2.0 m wide, or 1.2 m absolute minimum. Make sure that there is sufficient width of carriageway left to enable traffic to flow freely. A width of 3.5 - 4 m is normally enough for one lane of traffic, but, if there are a lot of cyclists, this might need to be increased to 4.5 m.

When considering the provision of crossing facilities at a site always check the intervisibility between pedestrians and drivers. It should be at least equal to the stopping sight distance.

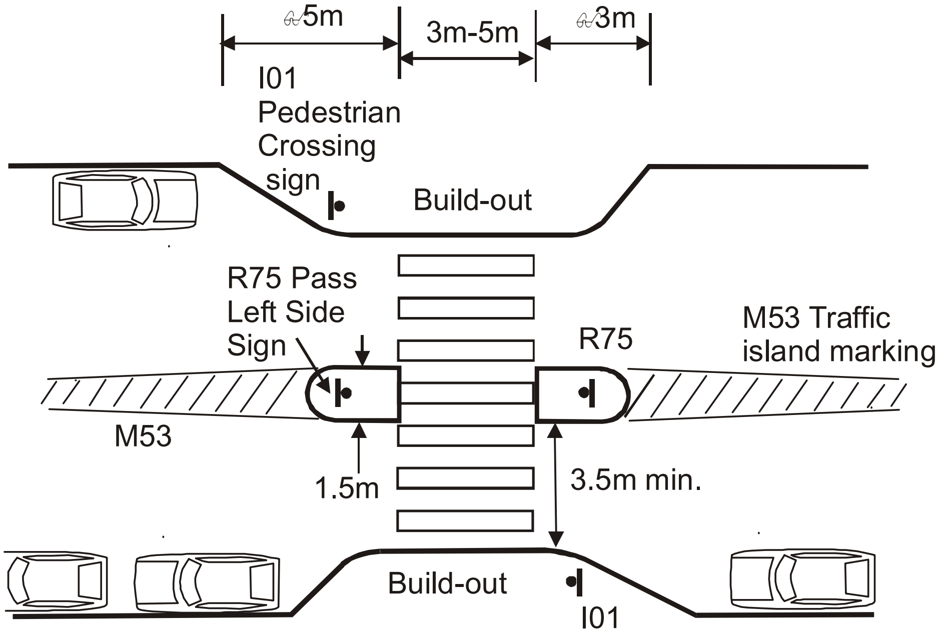


Figure 11-2: Pedestrian crossing facilities

Refer to Section 7 Cross-section for advice on pedestrian facilities on bridges.

11.1.3 Footbridges and Subways

Pedestrian footbridges and subways are not generally recommended, because they are inconvenient to use, and have a number of other problems, including crime, vandalism, and maintenance. However, they are very appropriate where the terrain is such that pedestrians can use the footbridge or subway without having to climb or descend. Subways are preferable to footbridges because the height difference is less, but they are much more costly, and the security and maintenance problems can be worse. Whenever possible the footbridge or subway should be in line with the normal path that pedestrians take when crossing the road. If pedestrians have to diverge from their direct route they will be discouraged from using the facility. Pedestrian barriers can be used to try and force pedestrians to use the facility, but local people will destroy them if they feel that the detour is unreasonable. When designing footbridges and subways make sure that they are easy to access (special attention to be made to women and disabled) and as pleasant and safe to use as possible. Subways 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. Recommended minimum dimensions for subways are given in Table 11-2.

Table 11-2: Minimum subway dimensions

Type of subway (m)	Width (m)	Height (m)
Narrow (short)	2.3	2.3
Standard	3.3	2.6
Wide (long)	5.0	2.6

Ideally, there should be a choice of stairs or ramps. Ramps should not normally be steeper than 5% (3% if used by cyclists) and should have a non-slip surface.

Recommended standards for stairs are:

- Flights of stairs (between landings) should be limited to 20 steps (9 steps where there are significant numbers of disabled persons);
- Stair landing should be a minimum of 1.8m deep;
- Step should be of equal height throughout the subway or footbridge;
- Optimum dimensions for stairs are 300mm tread (horizontal) and 130mm rise (vertical); and,
- 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 minimum clearance required for footbridges above the carriageway surface is 5.0m. This headroom must be maintained over the full width of the carriageway, including shoulders.

11.2 Bus Lay-bys

Bus stop lay-bys are recommended for new and upgraded roads, other than low-volume roads. They enable buses to slow down and stop outside the traffic lane, and this greatly reduces the risk of following traffic colliding with them or having to overtake in a panic. On busy roads bus drivers may be reluctant to use them, because of the difficulty of getting back into the traffic stream, but this problem can usually be overcome in liaison with the bus companies and the traffic police. At busy stops the waiting passengers sometimes stand in the lay-by and so prevent the buses from fully entering the lay-by. Pedestrian barrier can be installed to try and prevent this, but local publicity campaigns can also be effective.

To be fully effective, bus lay bays should incorporate:

a) a deceleration lane or tapper to permit easy entrance to the loading area. It must –be long enough to enable the bus to leave and enter the through traffic lanes at approximately the average running speed of the highway without undue inertial discomfort or sideway to passengers;

b) a standing space sufficiently long to accommodate the maximum number of buses expected to occupy the space at one time; and,

c) a merging lane to enable easy re-entry into the through-traffic lanes. The length of acceleration lanes from bus lay-bys, unlike deceleration lanes, should be well above the normal minimum values as the buses start from a standing position and the loaded bus has a lower acceleration capability than passenger cars.

The standard design for a bus lay-bay is shown in Figure 11-3, but the dimensions can be adjusted to suit local traffic situations.

Bus lay-bys should preferably be located on straight, level sections of road with good visibility (at least Stopping Sight Distance). They should be sited after intersections, to avoid stopped vehicles from interfering too much with the view of drivers who want to enter the main road from the minor road (see Figure 11-4). They should be sited after pedestrian crossings, for similar reasons. Do not site bus lay-bys opposite each other, because this can cause safety problems due to road blockage if both buses set off at exactly the same time. Stagger the lay-bys so that departing buses move away from each other (Figure 11-4).

When siting bus lay-bys bear in mind that the existing bus stops 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 passenger loading area with shelters to protect passengers waiting for bus against the sun and rain can help bus stop be more convenient and more attractive. Such shelters should as a minimum be sufficiently large to accommodate the volumes of offpeak waiting passengers and preferably be larger. Once the number of passengers that the shelter is to service has been known, the size of the shelter can be determined by multiplying a factor of 0.3 to 0.5 m2/passenger. The shelter should be provided with lighting, benches, route information, trash receptacles, public phone (where ever possible) and toilet specially for women and accompanied children. Ensure that pedestrian access is easy and convenient at bus lay-bys.

It is of major safety importance to study how passengers can reach the lay-by in a safe way.

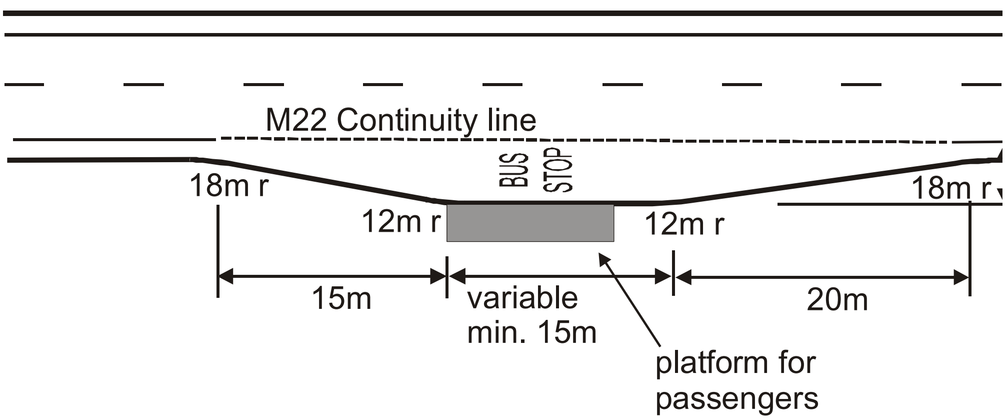


Figure 11-3: Standard bus lay-by

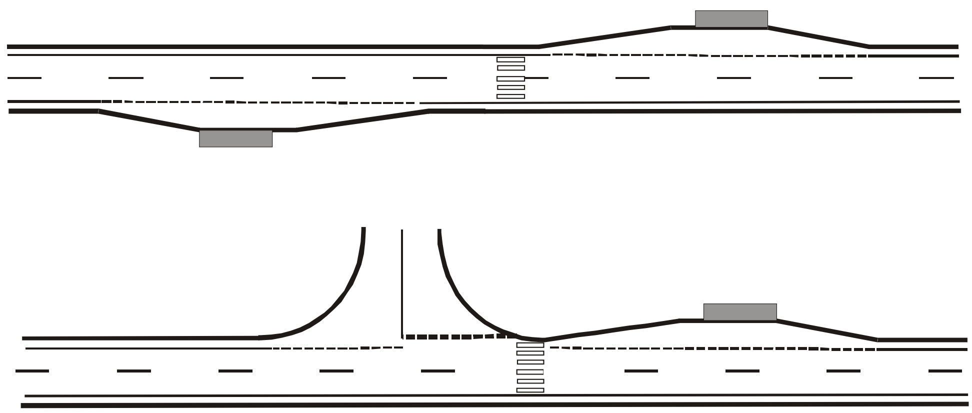


Figure 11-4: Recommended location for bus lay-bys

At heavily used bus lay-bys the dripping of oil and diesel onto the road surface may result in early failure of the pavement. In these situations it is preferable to use a concrete pavement. A loading area should provide 15 m of length for one bus, 30 m for two buses, 45 m for three buses and so on.

11.3 Cycle Facilities

Cycling has health, environment, economic and other benefits, and in rural areas bicycles may be the most common means of transport. Unlike the motor vehicle, bicycles are environmentally friendly. If adequately planed, designed and maintained, cycleways can plan an important role in the transportation system. Therefore, all highway, traffic management, safety and maintenance programmes should give consideration to the needs of cyclists. Although no research has been done, it is likely that, as in other countries, they have an above average risk of being involved in a road accident. It is important that more be done to reduce the danger of cycling. The first step when planning highway programmes or projects is to assess the demand and get some idea of journey patterns and volumes.

The conventional view is that cyclists in rural areas can use the shoulders, and this is acceptable provided that the combined volume of pedestrians and cyclists is low (<400 per day) and the shoulder is at least 1.5m wide. But, with heavier flows, and especially if there is high-speed traffic and / or a high proportion of heavy goods vehicles, it will be better to provide a separate cycleway or a combined cycleway and footway. Figure 11-5 shows the basic dimensions, and Table 11-3 gives recommended cycleway widths. Cycle ways need to have a smooth surface with good skid resistance.

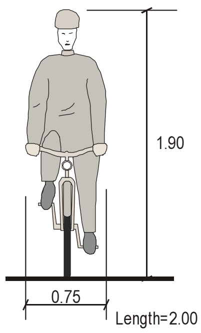


Figure 11-5: Cyclist dimensions

Table 11-3: Recommended widths for cycle facilities

Type	Minimum width (m)	Standard width (m)	Width for heavy usage (m)
Cycleway (separate from carriageway)	2.0	2.5	3.5
Combined cycleway and footway	2.0	3.0	4.5
Cycle lane (one way)	1.5	2.0	2.5

Table 11-4: Recommended Clearances

Type	Recommended Clearance [m]
Minimum overhead clearance	0.50
Clearance to wall, fence, barrier or other fixed object	0.50
Clearance to unfenced drop-off, e.g. embankment, river, wall	1.0
Minimum clearance to edge of traffic lane for speed limit of:	
50 km/h	0.50
80 km/h	1.00
100 km/h	1.50

Speed management measures (see Section 10) in trading centres and other built-up areas will help cyclists by reducing motor vehicle speeds, but the needs of cyclists must be considered from the beginning. Otherwise features such as gates and narrowings could increase the dangers to cyclists. If there are a lot of cyclists it may be worth providing a short by-pass (1m wide) which will enable cyclists to avoid the speed reduction measure.

In the larger urban areas there may be a case for developing a network of cycle facilities linking all the main travel generators. Complete segregation of cyclists and motorists is not necessary – they can mix safely at speeds up to 50 km/h providing there are not too many heavy goods vehicles. The network can be based on the existing road network, but with special facilities, such as cycleways and cycle lanes on difficult sections and at major intersections. The simplest way of assisting cyclists is to provide a wide (4.5m) nearside lane. Cycle lanes (created with road markings) should only be necessary on the busier roads. Providing for cyclists at major intersections can be difficult, but measures which control speeds, reduce conflicts, and improve visibility will be helpful for cyclists. All cycle facilities should be well signed for the benefit of both cyclists and other road users.

SECTION 12: ROAD FURNITURE, SAFETY AND MISCELLANEOUS DESIGN ITEMS

12.1 General

This section deals with road furniture and markings. It represents a collection of marginal elements intended to improve the driver's perception and comprehension of the continually changing appearance of the road. Elements addressed herein include traffic signs, road markings, marker posts, traffic signals, and lighting.

Traffic islands, kerbs and road markings delineate the pavement edges and thereby clarify the paths that vehicles are to follow. Safety fences prevent cars from leaving the road at locations where this would have the most severe consequences.

Fences and gates along the road reserve are a means of controlling access.

Marker posts assist in a timely perception of the alignment ahead and, when equipped with reflectors, provide good optical guidance at night.

Traffic signs provide essential information to drivers for their safe and efficient maneuvering on the road.

Traffic signs, road markings, and marker posts shall conform to the Ugandan Traffic Manual officially in use.

Lighting provides an effective means of reducing accidents especially at major junctions. The lighting poles will be considering as road furniture as are all other fixed objects along the carriageway which may form a traffic hazard.

12.2 Traffic Signs and Road Markings

The use of traffic signs and road markings, including road studs and delineators, is described in the Ministry's Traffic Signs Manual.

12.3 Safety Barriers

12.3.1 Principles

There are three main types of safety barrier:
- wire rope;
- guardrail, with beams and posts made of steel; and,
- concrete barrier, made of reinforced concrete.

They are used to prevent vehicles from hitting or falling into a hazard - such as falling down a steep slope, or falling into a river, hitting an obstruction near the edge of the road, or crossing a median into the path of traffic on the other carriageway. These events happen when a driver has lost control of the vehicle due to excessive speed, lack of concentration, tyre failure, collision, etc.

Ideally, the safety barrier will;

- prevent the vehicle from passing through the barrier (the vehicle will be contained);
- absorb (cushion) the impact of the vehicle without injuring the occupants (no severe deceleration);
- re-direct the vehicle along the road parallel to the other traffic; and,
- enable the driver to retain control of the vehicle (no spinning or overturning of the vehicle).

No safety barrier yet invented will perform to this standard in every impact. There is great variation in the impact circumstances (vehicle type, impact speed, impact angle, etc.,) so the design of a barrier is inevitably a compromise.

The specification, installation and maintenance of safety barrier is a highly technical subject, and this Manual can only give a brief introduction to the subject. Always seek advice from experts, as safety barrier can be useless and even dangerous if not properly designed and installed. Always purchase the components of safety barrier from a specialist manufacturer and obtain their advice. If possible, arrange for them to install it, or supervise the installation.

12.3.2 Performance

Conventional safety barrier installations are designed for impacts by cars travelling at 65 km/h hitting the barrier at a 25 degree angle. Barriers can be made that will cope with trucks and buses, but the high cost means that they can only be justified in exceptionally risky situations. Most barriers will not perform well when hit at a large angle - such as can happen when barrier is installed on the outside of a sharp bend.

Maximum permissible deflection is an important consideration. Concrete barrier does not deflect at all, so it is the best solution for shielding hazardous objects which are very close to the carriageway, or for stopping out-of-control vehicles from crossing a narrow median. With steel beam barriers the amount of deflection can be reduced to some extent by reducing the post spacing or using two beams nested one inside the other.

Table 12-1: Safety barrier characteristics

Category	Type	Deflection when hit	Comments
Flexible	Wire rope	2m +	Expensive; technically complicated; quick to repair
Semi-rigid	Steel beam weak post	1 - 2m	Performs well on high-speed roads when hit by cars; technically complicated
	Steel beam strong post	0.9m	Performs well in moderate-speed situations; most common type in developing countries
Rigid	Concrete barrier	No deflection	Expensive; high level of containment, but can cause severe deceleration when hit at a large angle; low maintenance costs

12.3.3 When to Use

When a roadside hazard is identified, the best solution is to remove the hazard. Where the hazard is a drop, it will be worth considering whether the slope can be flattened to make it less of hazard. If this cannot be done there may be a case for shielding the hazard with a barrier, but it is important to remember that installing a barrier is a second-best solution, as there is no certainty that it will perform successfully. Moreover, safety barrier is a hazard in itself. Collision with a barrier can cause death and serious injury - particularly to riders of two-wheelers. This means that safety barrier should only be installed when the consequences of an out-of-control vehicle hitting the unprotected hazard are likely to be more severe than those of impact with the safety barrier. However, this is very hard to judge. Vehicle occupants can sometimes die when the vehicle they are in collides with an insubstantial object, or drops a very short distance, yet clearly it is not economic to try and shield every hazard. The risk of a loss-of-control incident at a particular spot increases with traffic volume, traffic speed, and road curvature, but not in any predictable way.

On existing roads the main consideration will be the accident history. If collisions with the hazard are occurring often there may be justification for safety barrier – assuming the hazard cannot be removed. In most situations steel beam strong post guardrail will be the best option.

Cost-benefit analysis can help determine whether it is worthwhile installing barrier.

Steel beam strong post guardrail is also the preferred type for use on new roads. It is recommended that the following criteria be used:

For roads where cars travel in excess of 50 km/h:

• At >2 metre drops at sharp bends (defined as those where the safe speed to negotiate the bend is more than 15 km/h lower than the speed on the approach) where the side slope is steeper than 1 in 4;

• Where there is a risk that vehicles could fall into a body of water deeper than 1 m or onto a rail track;

• To shield any solid, substantial object within the clear zone;

• On medians less than 9 metres wide where the road has an ADT of >10,000; and,

• On straight sections or gentle bends on embankments as indicated in Figure 12-1 this level of provision may be unaffordable, in which case give priority to bends with a radius of less than 450 m and high-speed, high-volume roads.

These are crude guidelines and there is scope to deviate from them if a case can be made. The criteria should be re-assessed when more experience is gained.

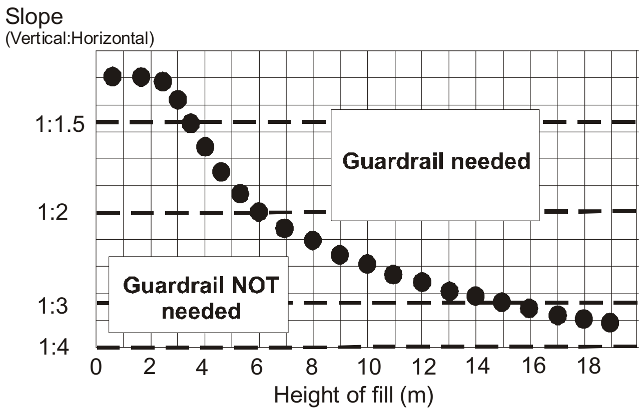
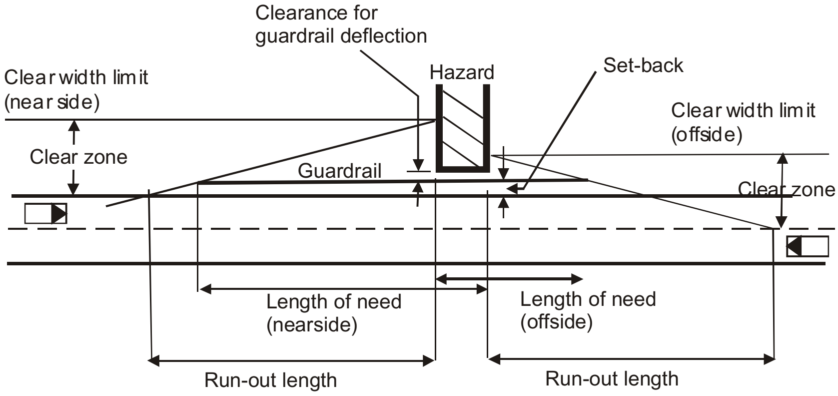


Figure 12-1: Guardrail – determination of need

12.3.4 Length of Need

Steel beam safety barrier is often installed in lengths that are too short to be effective. This is done to keep costs down, but the resulting installation may be completely useless. Generally you need at least 30 m of steel beam strong post guardrail for it to perform satisfactorily. Figure 12.2 gives guidance on determining the length of need. Note that on a two-way single carriageway road you have to consider both directions of travel - you cannot assume that vehicles will not hit the downstream end of a barrier. One of the common faults on hazardous bends is to stop the barrier at the point where the bend meets the tangent. Experience shows that some of the vehicles that fail to negotiate the bend will run off the road just beyond the tangent point. Always try and close off any gaps through which vehicles may fall.



Refer to Table 4.4 in Section 4.5 for details of clear zone widths

Note: This is a starting point for estimating length of need. Engineering judgement should be used to determine the precise length of guardrail to be installed.

Speed (km/h)	**Run-out length** (metres)
60	50 - 60
80	80 - 90
100	100 - 110
>110	120

Figure 12-2: Determining the length of need (adapted from Australian guidelines)

12.3.5 Steel Beam Strong Post Guardrail

Steel beam strong post guardrail is the most common type of safety barrier used in developing countries, and is available from many sources. The precise design varies in detail, but the basic characteristics are: (refer to Figure 12-3)

- steel beams with a W-shape (this is the part that comes into contact with the vehicle);
- the beams are 4130 m long;
- the beams are mounted on steel posts that are set either 1900 mm or 3800 mm apart;
- the beams are mounted so that the centre of the beam is 550 mm above the height of the road surface; and,
- there is a steel spacer block between the post and the beam to prevent the vehicle from hitting (“snagging”) on the post (“snagging” will usually cause the vehicle to spin round).

When an out-of-control vehicle hits the barrier the beam flattens, the posts are pushed backwards, and the tension in the beam builds up to slow the vehicle and redirect it back onto the road. That is if it performs successfully. The speed, mass and angle of the vehicle is critical to success. With heavy vehicles, high angles of impact and very high speeds the barrier may be torn apart or crushed. The containment capability can however be increased by using two beams, one mounted above the other.

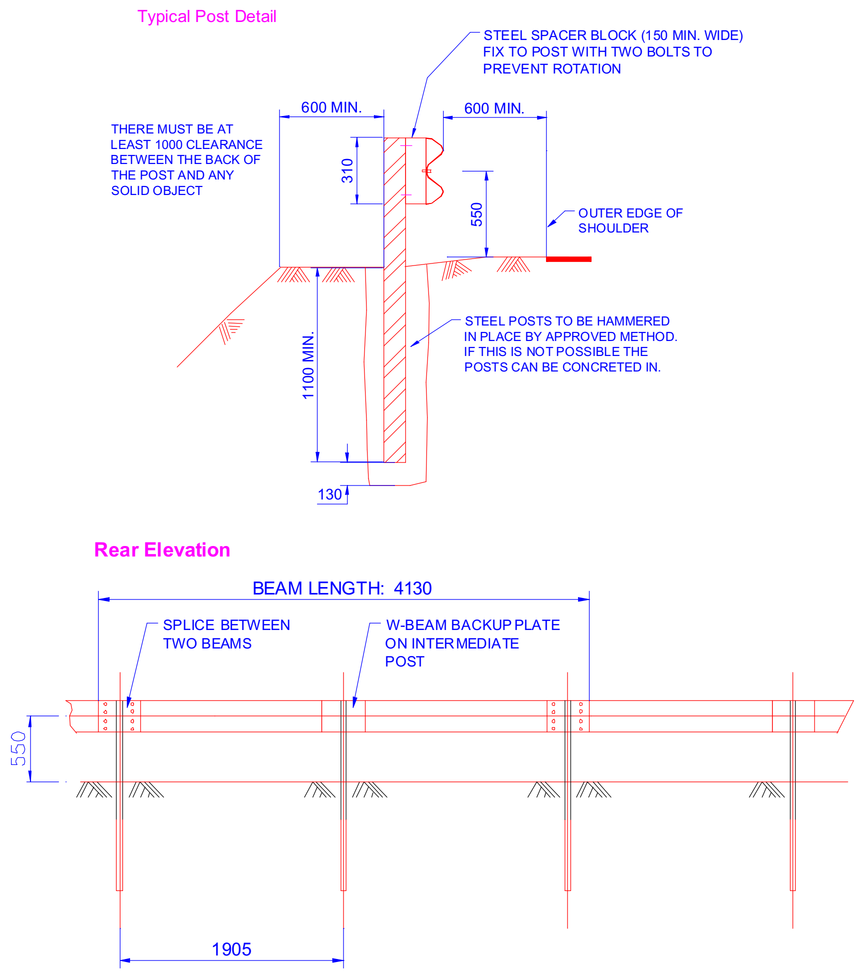


Figure 12-3: Steel beam strong post guardrail - typical details

12.3.6 Installation of Steel Beam Strong Post Guardrail

- The beams must be overlapped in the direction of travel, so that if they come apart in an impact there is not an end that can spear the vehicle;
- The beams must be bolted together with eight bolts and the whole structure must be rigid;
- The beam centre must be 550 mm +-5 mm above the adjacent road surface - if it is lower, vehicles may ride over it; if it is higher, vehicles may go under it;
- The spacer block must be fitted to the post with two bolts, otherwise it may rotate in a collision;
- There must be two layers of beam at each spacer block, so at the intermediate posts (i.e. those where there is no beam splice) insert a short section of beam between the main beam and the spacer block - this is often called a backup plate and it helps to prevent the beam hinging or tearing at this point;
- If the posts and spacers blocks are made of steel channel they must be installed so that the flat side faces the traffic - this reduces the risk of injury if they are hit by a person who has fallen from a vehicle;
- There must be a space of at least 1000 mm between the back of the post and any rigid obstacle - this can be reduced to 500 mm if the barrier is stiffened by putting in extra posts (at 952 mm centres), putting two beams together (one nested inside the other) and

using extra large concrete foundations;

- When installed on top of an embankment there must be at least 600 mm between the back of the post and the break of slope in order to have sufficient ground support for the post - where this is not possible, you must use much longer posts;
- Do not install the guardrail behind a kerb, because when a vehicle hits the kerb it will be pushed upwards and so will hit the guardrail too high - with a risk that the vehicle will go over the guardrail; and,
- Set the guardrail back from the shoulder edge (or carriageway edge if there is no shoulder) by at least 600 mm - putting it at the edge of the shoulder reduces the effective width of the shoulder and increases the risk of minor damage.

12.3.7 End Treatment for Guardrail

The end of a steel beam guardrail installation is a major hazard, as out-of-control vehicles can become impaled on it resulting in serious injuries for the occupants. Do not leave short (<80m) gaps in guardrail - instead make it continuous. There is no wholly safe way of terminating guardrail but the general advice is:

- stiffen the end section by installing the posts at 1905 mm spacing, AND
- flare the end section of the guardrail away from the edge of the shoulder until it is offset by at least 1m - use a flare rate of at least 1 in 10 - this reduces the risk of a direct impact; AND
- use a special impact-absorbing terminal piece or ramp the beam down sharply into the ground.

On a two-way road both the upstream and downstream ends of the guardrail will need to be terminated in the above way. One of the problems of ramped ends is that they can launch out-of-control vehicles into the air, with disastrous consequences. Try and avoid this by ramping the beam down sharply. Flaring is an effective way of reducing the risk of impact but this can be difficult to achieve in some situations, such as on narrow embankments.

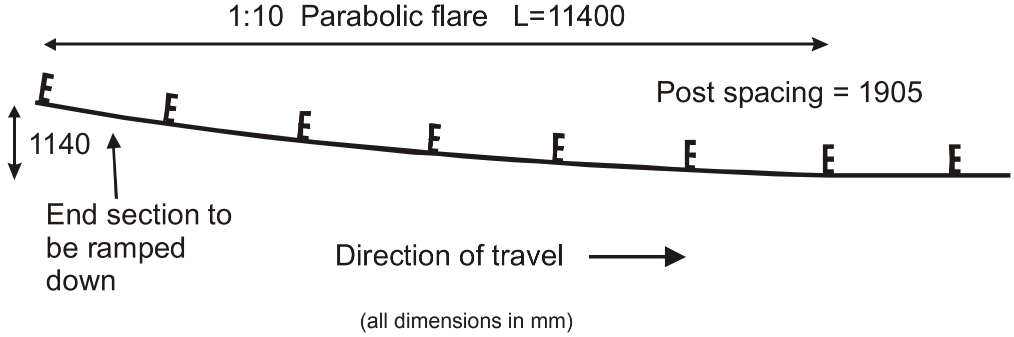


Figure 12-4: Flaring of the end of guardrail to reduce the risk of impact

12.3.8 Transition from Guardrail to Bridge Parapets and Concrete Barriers

Collisions with the ends of bridge parapets and concrete barriers are usually very severe. It is essential that these obstacles be shielded so that out-of-control vehicles will be redirected along the face of the parapet or concrete barrier. This is best done by installing a semi-rigid steel beam guardrail on the approach - normally at least 30 m long. It must line up with the face of the parapet / barrier and be strongly connected to it. The guardrail must be progressively stiffened so that deflection is reduced to zero as the parapet / barrier is reached. This is called a transition section. The stiffening is achieved by putting in extra posts, putting two beams together (one nested inside the other) and using extra large concrete foundations. See Figure 12-5. A steel connecting piece is used to bolt the end of the guardrail to the parapet or barrier

- the design of this will vary to suit the design of the parapet / barrier.

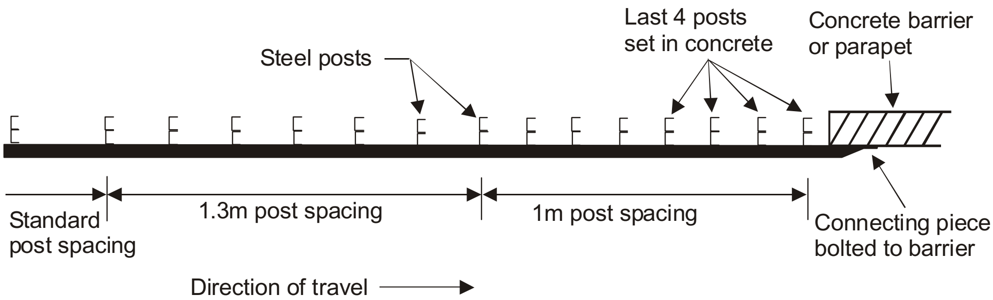


Figure 12-5: Typical transition (W-beam guardrail to rigid object)

12.3.9Median Barriers

High-speed dual carriageway roads with medians less than 9m wide may need to have median barriers to reduce the risk of cross-over accidents and/or to provide protection against collision with obstacles (e.g. lighting columns). Studies have shown that they may be cost-effective when the ADT exceeds 10,000. Median barriers should not normally be used on urban dual carriageways with speed limits of less than 80 km/h. If such roads have a cross-over problem it should be tackled through speed reduction measures.

Median barriers often take the form of two guardrail beams mounted back to back on one post – see Figure 12-6. They are not suitable where the median is narrower than 2.0 m because they deflect too much on impact. Rigid barriers made of reinforced concrete are preferred in this situation – see Figure 12-6. As always with safety barrier it is a problem to terminate it safely. If possible the barrier should be terminated at points where speeds are low, such as at roundabouts. Failing this, the guardrail beams should be flared and ramped down, or at least be capped with a protective end-piece (bull-nose terminal). Concrete barrier should be ramped down.

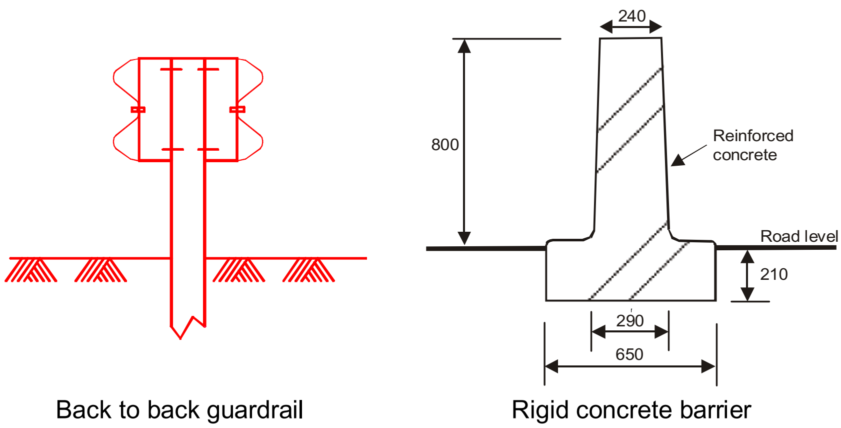


Figure 12-6: Median Barriers

12.3.10 Concrete Barrier

Concrete barriers are strong enough to stop most out-of-control vehicles, and being rigid there is no deflection on impact. This makes them suitable for use on narrow medians and where it is essential to keep vehicles on the road, such as at bridges. Small angle impacts usually result in little damage to the vehicle. However, large angle impacts tend to result in major damage to the vehicle, and severe injuries to the occupants. Research has shown that the conventional profile (commonly called New Jersey Barrier) tends to cause small vehicles to overturn, and the preferred shape is now a vertical or near-vertical wall (see Figure 12-6). Concrete barrier generally requires very little routine maintenance except after very severe impacts.

The ends of concrete barrier are very hazardous, so every effort should be made to terminate the barrier where speeds are low. The end of the barrier should be ramped down. If approach speeds are unavoidably high the end of the barrier should be protected by fitting a section (of at least 30 m) of semi-rigid guardrail.

12.4 Kerbs

12.4.1 Function

Kerbs have a number of useful functions:

- they define the edge of traffic lanes, traffic islands and footways – during both day and night (they reflect vehicle headlights);
- they support pavements and island structures so that edge break-up is avoided;
- they protect adjacent areas from encroachment by vehicles; and,
- they assist in drainage of the carriageway.

12.4.2 Types of Kerbs and their Application

The main types of kerbs and their applications are listed below: (see also Figure 12-7)

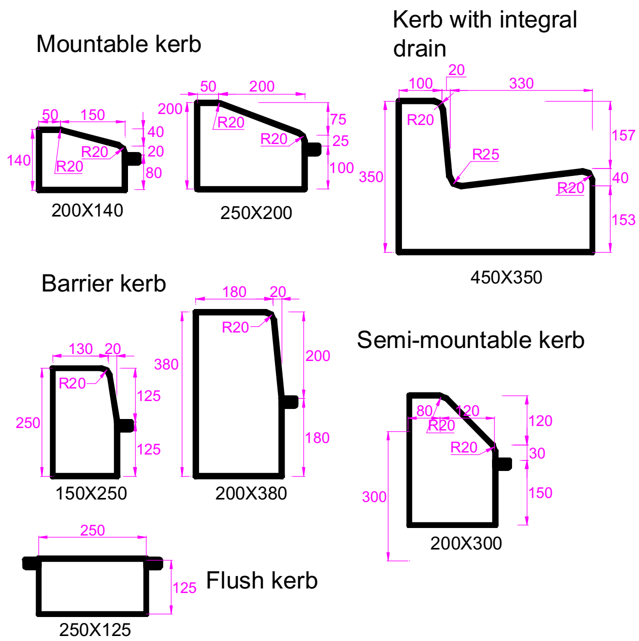


Figure 12-7: Types of kerbs

12.4.2.1 Barrier Kerbs

This kerb is used to provide protection to footways, traffic islands, pedestrian guardrail, traffic signs, etc. Kerbs on footways should have a height of no more than 125 – 150 mm above road level. If they are higher than this pedestrians may prefer to walk in the road. Barrier kerbs should not normally be used on roads with vehicle speeds in excess of 70 km/h.

12.4.2.2 Semi-mountable kerbs

These kerbs can be used in rural situations where high speeds would make the use of barrier kerbs risky. They are useful in defining and protecting the edges of the carriageway and traffic islands at intersections.

12.4.2.3 Mountable kerbs

These kerbs are used to define traffic islands and road edges in urban and rural situations where there is a high risk of the kerbs being hit by vehicles.

12.4.2.4 Flush kerbs

These kerbs are used to protect and define an edge which can be crossed by vehicles.

12.4.2.5 Kerbs with integral drain

This is a neat and effective way of providing drainage in urban areas, and it reduces the risk of water penetration into the edge of the pavement. Other types of kerbs can be designed with integral drains.

12.5 Bridge Parapets

A parapet is a protective fence or wall at the edge of a bridge or similar structure. There are two broad types:

Pedestrian parapets: These are designed to safeguard pedestrians but are not intended to contain vehicles. These are used where there is a safety barrier between the vehicle lanes and the footway (see Figure 12-8). Figure 12-8 shows a typical design for a lightweight steel parapet. There should not be any openings wider than 100mm, in order to prevent small children from squeezing through the parapet. Standard height is 1.0 m from the footway surface to the top of the parapet, but this should be increased to 1400 mm if cyclists are present.

Vehicle / pedestrian parapets: These are designed to contain vehicles and safeguard pedestrians. They are usually made either of metal (see Figure 12-8) or reinforced concrete. Vehicle / pedestrian parapets must be designed to contain out-of-control vehicles on the bridge and to deflect them back into the traffic lanes without severe deceleration or spinning. The design event is usually taken to be a 1.5 t vehicle travelling at 80 km/h and hitting the parapet at an angle of 200. Well-designed reinforced concrete parapets have the greatest containment capability and should be used where containment is essential. The basic design principles are:

- Parapets should present an uninterrupted continuous face to the traffic – i.e. no projections;
- Metal parapets should be designed with horizontal rails set in front of posts, so that vehicles brush against the rails and do not snag on the posts;
- As far as possible the horizontal rails on metal parapets should be continuous – if joints are unavoidable they must be strengthened so that there is no risk of them coming loose on impact and exposing free ends (like a spear);
- The ends of the bridge rails must be linked with a strong connecting piece so that there are no exposed ends;
- The standard height for parapets is 1m, but this should be increased to 1400 mm for cycleways, and 1500 mm for bridges over railways and other bridges where containment is essential;
- Metal parapets should have no openings wider than 100mm – if necessary the parapet should be faced with wire mesh panels;
- Parapets should be designed so that they are difficult to climb – i.e. no footholds or flat tops; and,
- Where kerbs are located in front of parapets, the height of the parapet should be increased

to take account of dynamic jump effects.

Reinforced concrete parapets should take the form of solid, continuous walls with no openings. A metal hand rail is often fitted along the top of the wall in order to improve the visual appearance.

In most cases there will need to be guardrail at both the approach and departure ends of the parapet in order to prevent out-of-control vehicles hitting the end of the parapet. This is particularly important with reinforced concrete parapets, because of their rigidity. The guardrail can also prevent out-of-control vehicles from going the wrong side of the parapet and dropping into the river / railway / etc below – this is a common incident at bridges where the approach is on a bend. The guardrail should be at least 30m long and should continue the line of the traffic face of the parapet. Refer to Figure 12-8 for the design of the transition section between guardrail and parapet. It is possible to design a metal parapet that incorporates w-beam guardrail, and this has the advantage that the guardrail can be extended off the bridge to protect vehicles on the approach sections. The containment capability can be increased by using two beams, one above the other.

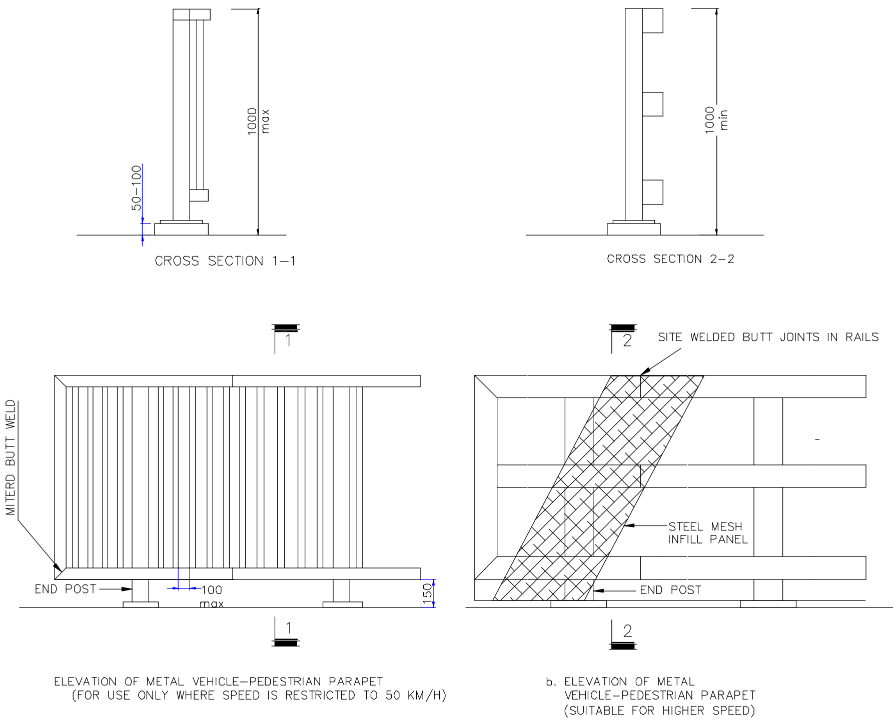


Figure 12-8: Vehicle / pedestrian parapet - typical details

12.6 Traffic Islands

12.6.1 Function

Traffic islands are a key element in the design of safe, efficient intersections. They can be used to:

- separate conflicting traffic streams;
- control the path of vehicles and reduce unnecessary areas of carriageway;
- provide segregated lanes for some vehicle types or some traffic movements;

- warn drivers that they are approaching an intersection;
- provide shelter to vehicles that are waiting to make a manoeuvre;
- slow vehicles down by deflecting them from a straight ahead path;
- assist pedestrians to cross the road; and,
- locate traffic signs and signals where they will be at least risk of being hit.

12.6.2 Design Requirements

Traffic islands must help drivers to recognise and follow a safe path through the intersection. This calls for care in location, alignment, sizing and construction details. The key requirements are:

- of sufficient size to be easily seen (min. 4.5m2);
- shape should take into consideration the wheel tracks of turning vehicles, the radii of left and right turns, island nose radii, etc (use vehicle turning circle templates);
- where islands are needed on high-speed rural roads consideration should be given to creating them with road markings – driver compliance can be encouraged by infilling them with rumble strips;
- where kerbed islands are to be provided on high-speed rural roads the kerbs should preferably be of the mountable or semi-mountable type;
- kerbed islands can be made more visible by painting the kerbs black and white (500mm sections);
- pedestrian refuges should normally have barrier kerbs and be 1.5 m wide (1.2 m absolute minimum);
- traffic signs (typically sign no. R75 “Pass Left Side Only”) should be used on kerbed islands, and they should be positioned so that there is at least 300 mm clearance between the edge of the sign and the traffic face of the kerb;
- the nose at the approach end of an island should have a minimum radius of 0.6 m – at other corners the radius can be as small as 300 mm;
- on high-speed roads it is advisable to offset the edges of islands from the edge of the through traffic lane by 0.6 m – 0.9 m, to reduce the risk of collisions; and,
- road markings (typically sign no. M53 “Traffic Island Marking”) should be used to guide drivers safely past the island.

12.7 Pedestrian Barrier

12.7.1 Function

Uncontrolled pedestrian movements are a significant factor in urban traffic and safety problems. Pedestrian barrier can bring big improvements by segregating pedestrians from vehicular traffic and channelling them to safe crossing points. At intersections, barrier can:

- reduce conflicts by channelling pedestrians to crossing points on the approaches;
- discourage buses, minibuses and cyclists from stopping and parking within the intersection;
- discourage delivery vehicles from loading or unloading within the intersection; and,
- discourage roadside vendors from occupying the road space in the intersection

Other applications include:

- Schools – barrier can be used to prevent children from running into the road from the school gate;

- Bus parks, cinemas, stadiums, etc – barrier can channel pedestrian flows at areas of heavy pedestrian movement;
- Pedestrian crossings, subways', footbridges – barrier helps channel pedestrians to the crossing facility; and,
- Medians – barrier can be used to deter pedestrians from using the median to cross the road, though barriers on the footways are more likely to be effective.

Nobody likes walking further than they have to, so, although pedestrians can be guided to a certain extent, do not try and force them to make unreasonably long detours. People will smash the barrier if it is seen as too much of an obstacle.

12.7.2 Design

Ideally, pedestrian barrier should:

- be effective;
- be strong and easily maintained;
- cause minimum damage to vehicles and the occupants when hit;
- not be hazardous to pedestrians, including the disabled;
- not interfere with visibility; and,
- look acceptable.

The choice is between steel railings and brick or concrete walls. Steel railings should be designed with a minimum number of horizontal elements, because these are potentially hazardous in a vehicle collision. The design principles are similar to those for steel bridge parapets – see Figure 12-8 – with the added requirement for them to be see-through. The railings should be about 1.0 m high and be rustproof. Brick or concrete walls are likely to be cheaper and easier to maintain but they take up more space. All barriers should be set back (normally 300 mm) from the traffic face of the kerb to give adequate clearance for passing vehicles. Ends of pedestrian barrier may need to be fitted with reflectors to make them less of a hazard at night.

12.8 Lighting

Lighting contributes to road safety and improves personal security. Lighting is provided to improve the safety of a road. Priority in the provision of lighting should be given to areas with a high proportion of night-time pedestrian accidents, such as bus parks, pedestrian crossings and entertainment centers. Lighting should be provided on all main roads in urban areas. The provision of lighting improves safety at isolated major intersections in rural areas. Statistics indicate that the night-time accident rate is higher than during daylight hours, which, to a large degree, may be attributed to impaired visibility. In urban areas, where there are concentrations of pedestrians and junctions, fixed source lighting tends to reduce accidents. However, lighting of rural highways is seldom justified except at junctions, intersections, and railway level crossings, narrow or long bridges, tunnels, sharp curves, and areas where there is activity adjacent to the road (e.g. markets).

To minimize the effect of glare and to provide the most economical lighting installation, luminaries should be mounted at a height of at least 9 meters. High mounted luminaries provide greater uniformity of lighting and mounting heights of 10 to 15 meters are frequently used. High mast lighting (special luminaries on masts of 30 meters) is used to illuminate large areas such as intersections. This type of lighting gives a uniform distribution of light over the whole area and thus illuminates the layout of the intersection.

Lighting columns can be a hazard to out-of-control vehicles. The lighting scheme should aim to minimize the number of lighting columns and ensure that the poles are not located in vulnerable positions. Standards for lateral clearance, clear zones, etc. must be respected. Lighting columns (poles) should be placed behind vertical kerbs whenever practical. The appropriate distance is 0.5m behind the kerb for roads with a design speed of 50 km/h or less, and 1.2m or greater for roads with a design speed of 80 km/h or greater. Where poles are located within the clear zone, regardless of distances from the edge of the carriageway, they should be designed to include a frangible impact attenuation feature. However, these types of poles should not be used on roads in densely populated areas, particularly with footways. When struck, these poles may collapse and cause injury to pedestrians or damage adjacent property. Because of lower speeds and parked vehicles on urban roads, there is much less chance of injuries to vehicle occupants from striking fixed poles as compared to higher speed roads.

On dual carriageways, lighting may be located either in the median or on the other side of each carriageway. However, with median installation, the cost is generally lower and illumination is higher on the high-speed outer lanes. On median installations, dual mast arms should be used, for which 12-15 meter mounting heights are favored.

These should be protected with a suitable safety barrier. On narrow medians, it is preferable to place the lighting poles so that they are integral with the median barrier.

When it is intended to install road lighting in the future, providing the necessary conduits/ducts as part of the initial road construction can bring considerable savings.

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