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MINISTRY OF ROADS AND TRANSPORT

ROAD DESIGN MANUAL

Volume 1 – Geometric Design

Part 3 – Geometric Design of Highways, Rural Roads, and Urban Roads

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Foreword

This manual provides a well-considered basis for designing, constructing, and rehabilitating or upgrading roads in Kenya for many years ahead. An intensive review and update of old manuals was undertaken to ensure that this and other manuals are fit for purpose. Deliberate efforts were made to ensure its relevance to the needs of our local practitioners whilst taking cognisance of futuristic trends in the region and elsewhere.

The Ministry involved key stakeholders in the review process, as well as the general public. A series of stakeholder workshops ensured that all possible different opinions were taken into account, from both the public and the private sector.

On behalf of the Government of Kenya, I would like to thank the African Development Bank for its support to the process of preparing this Manual. I would also like to thank the National Steering Committee, the Technical Task Force, the Technical Administrators, and the KeNHA Project Coordination Team for the sterling work done. I also thank the Consultant, TRL Limited for their role in providing technical expertise that was essential for the success of the exercise.

My Ministry commits itself to ensuring the maximum use of this manual by all departments and road agencies.

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Hon. Onesmus Kipchumba Murkomen

Cabinet Secretary, Ministry of Roads and Transport

Preface

Designing a new road or upgrading an existing road requires many skills and effective documents to provide advice, instructions, and guidance, not only at the design stage but from conception, planning, through to maintenance, and eventual rehabilitation or upgrading.

A road is designed to provide good service for many years and therefore good planning and good-long term management are required. These activities rely on data and information.

The procedures for the geometric design of roads presented in this manual will assist in achieving the above and are applicable to rural, inter-urban and urban roads.

The manual adopts and encourages context sensitive design, a concept that seeks to produce a design that combines good engineering practice in harmony with the natural and built environment whilst meeting the required constraints and parameters surrounding each and every project.

It further addresses the needs of pedestrians, bicyclists, motor cyclists and non-motorised traffic.

Users of the manual are expected to follow the standards set there-in and seek approval of the Ministry should any departures be warranted.

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Eng. Joseph M. Mbugua

Principal Secretary, Ministry of Roads and Transport

Document Management

Document status:

This document has the status of a Manual. Users shall apply the contents there-in to fully satisfy the requirements set out.

The content of the manual is based on current practice in Kenya and latest practices in the road sector – both regionally and internationally.

Sources of the document:

Copies of the document can be obtained from:

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Type of request:

General – G; Editorial – E; Technical - T

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A **National Steering Committee** consisting of the following members had an oversight on the drafting process and their efforts are hereby acknowledged.

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Two highly experienced and able Technical Administrators were charged with ensuring managing the entirety of the manuals review and update process on behalf of the Ministry. These were **Eng. J. M. Kung'u**, Chief Engineer, Roads Division and **Eng. S. K. Kogi,** OGW, Chief Engineer, Materials Testing & Research Division. Their contribution to the success of the project is invaluable.

The technical work was ably undertaken under the guidance of a **Technical Task Force** and its various sub-committees that was constituted as follows:

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The Consultant for the review and updating of the manuals and specification for road and Bridge Construction was TRL Limited (UK), assisted by Consulting Engineers Group, India and Norken International Limited, Kenya. The Consultant's staff are listed in the table below.

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Lastly, but not least, the entire manuals review and updating process would not have happened were it not the efficient coordination by the KeNHA team led by **Eng. J. N. Gatitu**, Director for Policy, Strategy and Compliance.

Abbreviations

AADT	Average Annual Daily Traffic
ADT	Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
BC	Beginning of Curve. Also designated called PC or TC
BVC	Beginning of the Vertical Curve
CADD	Computer-Aided Design and Drafting
CEF	Car Equivalence Factor
CS	Circular Curve to Spiral Transition point
CSIR	Council for Scientific and Industrial Research
DFL	Design Flood Level
DHV	Daily High Volume or Design High Volume
EC	End of Curve
EF	Equivalence Factor
EOD	Environmental Optimised Design
EP	Edge of Pavement
ESA	Equivalent Standard Axles
EVC	End of the Vertical Curve
FH	Free Haul
GDP	Gross Domestic Product
HDM	Highway Development and Management Model
HMA	Hot Mixed Asphalt
IMT	Intermediate Forms of Transport
LVR	Low Volume Road
MESA	Million Equivalent Single Axle
NC	Normal Crossfall or Camber
NMT	Non-motorised Traffic
PC	Point of Curvature. Point on the Tangent where the Circular Curve begins. Also designated BC or TC
PCU	Passengers Car Units
PI	Point of Intersection - where the line of two Tangents meet
PIARC	World Road Association
PRC	Point of Reverse Curvature

PSD	Passing Sight Distance
PT	Point of Tangency. Where a Circular Curve ends. Also designated EC
RC	Reverse Camber
SANRAL	South African National Roads Agency Limited
SATCC	Southern Africa Transport and Communications Commission
SC	Spiral to Circular Curve Transition point
TC	Telecommunications Corporations
TC	Tangent to Curve. Also designated BC or PC
TRL	Transport Research Laboratory
TS	Tangent to Spiral Transition point
VPI	Vertical Point of Intersection
vph	Vehicles per Hour

Glossary of Terms

Acceleration lane	An auxiliary lane to enable a vehicle to increase its speed so that it can merge safely with through traffic.
Access Control	The condition where the road agency controls the right of landowners to direct access to and from a public highway.
Arterial	A highway designed to move relatively large volumes of traffic at high speeds over long distances. Arterials offer little or no direct access to abutting properties.
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)	The total yearly traffic volume in both directions divided by the number of days in the year.
Average Daily Traffic (ADT)	The total traffic volume during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period.
Average Running Speed	The total distance travelled by all the vehicles divided by the running time of all the vehicles; also referred to as the space mean speed. [The time mean speed is the average of all recorded speeds.
Axis of Rotation	The line about which the pavement is rotated to super-elevate the roadway. This line normally maintains the highway profile.
Broken-back Curve	Two curves in the same direction connected by a tangent shorter than 500 m.
Bus Lay-bys	Lay-by reserved for public service vehicles.
Camber	The convex shape given to the curved cross-section of a roadway.
Capacity	The maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions; usually expressed as vehicles per hour, passenger cars per hour, or persons per hour.
Carriageway	Portion of the roadway including the various physically contiguous traffic lanes and auxiliary lanes, serving one or both directions of traffic, and not including shoulders.
Catchwater Drain	Located above a cut face to ensure that storm water does not flow down the cut face causing erosion and deposition of silt on the roadway.
CEF	Car Equivalence Factors. To convert all non-motorised traffic and motorcycles to a common unit for judging road width requirement for safety purposes. Note that they are not the same as PCU values that are used for capacity and congestion estimates for heavily trafficked roads.

Channelisation	The use of pavement markings or islands to direct traffic through an intersection.
Circular Curve	Usual curve configuration for horizontal curves.
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.
Clover-leaf interchange	An interchange with four loop ramps and four diagonal ramps, with no traffic control on either crossing roadway.
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	A standard of road that is characterised by an approximately even distribution of access and mobility functions.
Collector-Distributor	A road used at an interchange to remove weaving from the through lanes and to reduce the number of entrances to and exits from the through lanes.
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.
Connector	A collective term for interchange links, link roads, ramps and loops.
Crest	Peak formed by the junction of two gradients.
Crest Curve	Convex vertical curve.
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 often considered "unreasonable".
Critical Slope	Side slope on which a vehicle is likely to overturn. Slopes with inclinations greater than 1V:3H are considered critical.
Crossfall	The transverse slope with respect to the horizon, measured in % transversely across the surface of the roadway.
Culvert	A structure, usually for conveying water under a roadway but can also be used as a pedestrian or stock crossing, with a clear span of less than six metres.
Cycle Lane	A portion of the roadway which has been designated by road markings, striping and signing as being exclusively for the use of cyclists.
Cycle Path	Also known as a bike way. A path physically separated from motorised traffic by an open space or barrier and located either within the road reserve or an independent reserve.
Deceleration Lane	An auxiliary lane to enable a vehicle leaving the through traffic stream to reduce speed without interfering with other traffic.
Decision Sight Distance	Allows for circumstances where complex decisions are required by a driver or unusual manoeuvres have to be carried out. As such, it is significantly longer than Stopping Sight Distance.

Deflection Angle	Successive angles from a tangent subtending a chord and used in setting out curves.
Depressed Median	A median lower in elevation than the travelled way and designed to carry a portion of the storm water falling on the road.
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 Period of a Pavement	The period of time that an initially constructed or rehabilitated pavement structure will perform before reaching a level of deterioration requiring more than routine or periodic maintenance.
Design Hour	The hour in which the condition being designed for, typically the anticipated flow is expected to occur. This is often the thirtieth highest hour of flow in the design year.
Design Speed	An index which links road function, traffic flow and terrain to the design parameters of sight distance and curvature to ensure that a driver is presented with a reasonably consistent speed environment. It is now defined as the 85^{th} percentile speed of passenger cars travelling in free flow conditions. 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 that pass over a given section of a lane or roadway during a given time period.
Design Vehicle	Vehicle whose physical characteristics and proportions are used in setting geometric design.
Design Year	The last year of the design life of the road or any other facility, often taken as twenty years although, for costly structures such as major bridges, a longer period is usually adopted.
Directional Distribution	The percentages of the total flow moving in opposing directions, e.g. 50:50, 70:30, with the direction of interest being quoted first.
Diverging	The opposite of merging. When a traffic stream splits into two or more streams.
Divided Highway	A highway with separate carriageways for traffic moving in opposite directions.
Eighty-fifth percentile speed	The speed below which 85 % of the vehicles travel on a given road or highway.
Equivalent Standard Axles (ESA)	A measure of the potential damage to a pavement caused by a vehicle axle load expressed as the number of equivalent 80 kN single axle loads that would cause the same amount of damage. The ESA values of all the traffic are combined to determine the total design traffic for the design period.
Equivalency Factors	Used to convert traffic volumes into cumulative equivalent standard axle loads.
Eye Height	Assumed height of a driver's eyes above the surface of the roadway used for the purpose of determining sight distances.
Footway	The rural equivalent of the urban sidewalk.

Free Haul	Maximum distance through which excavated material may be transported without added cost above the unit bid price.
Freeway	A multilane, divided highway with a minimum of two lanes for the exclusive use of traffic in each direction and full control of access without traffic interruption.
Gap	The space or time between two vehicles, measured from the rear bumper of the front vehicle to the front bumper of the second vehicle.
Ghost Island	An area of the carriageway suitably marked to separate lanes of traffic travelling in the same direction on both merge and diverge layouts.
Grade Line	The line describing the vertical alignment of the road or highway.
Grade Separation	A crossing of two highways or roads, or a road and a railway, at different levels.
Gradient	Rate of rise or fall on any length or road, with respect to the horizontal. It is typically expressed as a percentage or as the vertical rise or fall in m/100 m. In the direction of increasing stake value, upgrades are taken as positive and downgrades as negative.
Hairpin Curve	A bend in a road with a very acute inner angle at or near minimum radius, making it necessary for a vehicle to turn sharply almost 180°. Sometimes also called switchback curves.
Hairpin Stack	Sequence of hairpin curves employed to traverse a mountainous or escarpment terrain section.
High Occupancy Vehicle (HOV) Lane	A special lane open only to vehicles carrying two or more passengers.
High Speed	Speeds above 80km/h
Horizontal Clearance	Lateral clearance between the edge of shoulder and obstructions.
Horizontal sight distance	The sight distance determined by lateral obstructions alongside the road and measured at the centre of the inside lane.
Interchange	A system of interconnecting roads (referred to as ramps) in conjunction with one or more grade separations providing for the movement of traffic between two or more roadways which are at different levels at their crossing point.
Intersection sight distance	The sight distance required within the quadrants of an intersection to safely allow turning and crossing movements.
Jug Handle	A ramp where a right turn is made at an at-grade intersection by taking traffic off to the left.
Kerb	Concrete, often precast, element adjacent to the travelled way and used for drainage control, delineation of the pavement edge or protection of the edge of surfacing. Usually applied only in urban areas.
Kerb Ramp	The treatment at intersections for gradually lowering the elevation of sidewalks to the elevation of the street surface.

K value	The distance over which a one % change in gradient takes place.
Lane Gain	A layout where a merging connector road becomes a lane or lanes of the downstream main carriageway
Lane Drop	A layout where a lane or lanes of the upstream carriageway becomes the diverging connector road.
Left Hand Lane	On a dual roadway, the traffic lane nearest to the verge or shoulder (in countries where traffic moves on the left).
Left Turn Lane	An auxiliary lane to accommodate deceleration and storage of left-turning vehicles at junctions (in countries where traffic moves on the left).
Level of Service	A qualitative concept, from LoS A to LoS F, which characterises acceptable degrees of congestion as perceived by drivers. Capacity is defined as being at LoS E.
Link Road	In the context of junctions, a one way connector road adjacent to but separate from the mainline carriageway carrying traffic in the same direction, which is used to connect the mainline carriageway to the local highway network.
Loop	A ramp requiring vehicles to execute a left turn by turning left, accomplishing a 90-degree right turn by making a 270-degree left turn
Mainline	The primary through roadway as distinct from ramps, auxiliary lanes, and collector-distributor roads.
Median	The area in the middle of a roadway separating the opposing traffic flows. The median thus includes the inner shoulders.
Meeting Sight Distance	Distance required to enable the drivers of two vehicles travelling 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.
Merging	Movement of a vehicle or vehicles from one or more lanes into a traffic stream.
Non-recoverable Slope	Transverse side slope whose gradient is sufficiently steep that a motorist running onto it from the main road is unable to make a sufficiently controlled manoeuvre.
Normal Crown (NC)	The typical cross-section on a tangent section of a two-lane road or four-lane undivided road.
Normal Traffic	Traffic which would pass along the existing road or track even if no new pavement were provided.
Nose	A paved area, approximately triangular in shape, between a connector road (ramp) and the mainline at a merge or diverge, suitably marked to discourage drivers from crossing it.
Object Height	Assumed height of a notional object on the surface of the roadway used for the purpose of determining sight distance.
Operational mistake	The first unintended action within a chain of driving actions which may result in a driving mistake. It may be caused by the

	interaction of the characteristics of the road and the reactions of the driver.
Operating Speed	Highest overall speed at which a driver can travel on a given road under favourable 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, neither exceeding at any time the speed limit.
Outer separator	Similar to the median but located between the travelled way of the major road and the travelled way of parallel lanes serving a local function if these lanes are contained within the reserve of the major road. If they fall outside this reserve, reference is to a frontage road.
Overpass	Grade separation where the subject road passes over an intersecting road or railway.
Parking Bay	Area provided for taxis and other vehicles to stop outside of the roadway.
Partial Clover Leaf Interchange	An interchange with loop ramps in one, two or three (but usually only two) quadrants. A Par-Clo A Interchange has the loops in advance of the structure and Par-Clo B Interchange has the loops beyond the structure. A Par-Clo AB Interchange has its loops on the same side of the crossing road.
Passing Bay	Widened section of an otherwise single lane road where a vehicle may move over to enable another vehicle to pass.
Passenger car equivalents (units) (PCE or PCU).	The number of passenger cars that will result in the same operational conditions as a single heavy vehicle of a particular type under specified roadway, traffic, and control conditions. In studies involving capacity and LS at interceptions it might be necessary to extend the PCU concept to motorcycles.
Passing Lane	A lane added to improve passing opportunities in one direction of travel on a conventional two-lane highway. Situated on the high speed side of a carriageway into which high speed traffic can divert so that slow vehicles can remain in lane and can be easily overtaken. The alternative is a climbing (or crawler) lane added to the slow side of the carriage into which slow traffic must divert so that fast traffic can continue in the fast lane without deviation.
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 travelling at the design speed, should it come into view after the overtaking manoeuvre is started.
Pedestrian Refuge	Raised platform or a guarded area so sited in the roadway as to divide the streams of traffic and to provide a safe area for pedestrians.

Point of Curvature (PC)	Beginning of a horizontal curve, often referred to as BC (Beginning of Curve).
Point of Intersection (PI)	Point of intersection of two tangents.
Point of Reverse Curvature (PRC)	Point where a curve in one direction is immediately followed by a curve in the opposite direction. Typically applied only to kerb lines.
Point of Tangency (PT)	End of horizontal curve, often referred to as EC (End of Curve).
Point of Vertical Curvature (PVC)	The point at which a grade ends and the vertical curve begins, often also referred to as BVC (Beginning of Vertical Curve).
Protected Turn	A turn at an intersection that is controlled by traffic signals such that there are no conflicts with any other vehicles (i.e. no other vehicles can cross the path of the turning vehicles.)
Vertical Point of Intersection (VPI)	The point where the extension of two grades intersect.
Point of Vertical Tangency (PVT)	The point at which the vertical curve ends and the grade begins. Also referred to as EVC (End of Vertical Curve).
Quarter Link	An interchange with at-grade intersections on both highways or roads and two ramps (which could be a two-lane, two-way road) located in one quadrant. Because of its appearance, also known as a Jug Handle Interchange.
Ramp	A one-way, often single-lane, road providing a link between two roads that cross each other at different levels.
Recoverable Slope	Side slope of limited grade such that a motorist can generally return to the roadway. (Slopes < 1:4)
Relative Gradient	The slope of the edge of the travelled way relative to the grade line.
Reverse Camber (RC)	A super-elevated section of roadway sloped across the entire travelled way at a rate equal to the normal camber.
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 central reserve.
Right Turn Lane	Auxiliary lane to accommodate deceleration and storage of right-turning vehicles at junctions.
Roadside	A general term denoting the area beyond the shoulder breakpoints
Roadbed	The extent of the road between shoulder breakpoints.
Road Prism	The lateral extent of the earthworks.
Road Reserve	Strip of land legally awarded to the Roads Authority, specifically for the provision of public right of way, in which the road is, or will be, situated and where no other work or construction may take place without permission from the Roads Authority.
Road Safety Audit	A formal systematic road safety assessment of a road scheme carried out by an independent, qualified auditor who reports on the project's accident potential for all kind of road users.

Roadway	The area normally travelled by vehicles and consisting of traffic lanes, including auxiliary lanes and shoulders
Rural Road or Highway	Characterised by low volume high-speed flows over extended distances. Usually without significant daily peaking but could display heavy seasonal peak flows.
Safety Rest Area	Roadside area with parking facilities for the motorist to stop and rest.
Sag Curve	Concave vertical curve
Shoulder	Part of the road outside the carriageway, but at substantially the same level, for accommodation of stopped vehicles for emergency use, for lateral support of the carriageway and for use by pedestrians and cyclists when no other facility has been provided.
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 Friction	The resistance to centripetal force keeping a vehicle in a circular path. The designated maximum side friction represents a threshold of driver discomfort and not the point of an impending skid.
Side Drain	Open longitudinal drain situated adjacent to and at the bottom of cut or fill slopes.
Side Slope	Area between the outer edge of shoulder or hinge point and the ditch bottom.
Sidewalk	The portion of the cross-section reserved for the use of pedestrians
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.
Sight Triangle	The area in the quadrants of an intersection that must be kept clear to ensure adequate sight distance between the opposing legs of the intersection.
Simple Curve	A curve of constant radius without entering or exiting transitions.
Speed Hump (Bump)	Device for controlling the speed of vehicles, consisting of a raised area or recess on the roadway.
Speed Profile	The graphical representation of the 85th percentile speed achieved along the length of the highway segment by the design vehicle.
Spiral Curve	Transition curves between straight (tangent) sections of road and a circular curve.
Standard	Design value that may not be transgressed, e.g. an irreducible minimum or an absolute maximum, except in unusual conditions and with Ministry's approval. For geometric design, a 'standard' is not to be understood as an indicator of quality, i.e. an ideal to be strived for.

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

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

Super-elevation Run-off The super-elevation transition section consists of the super-elevation runoff and 'tangent runout' sections. The super-elevation runoff section consists of the length of roadway needed to accomplish a change in outside lane cross slope from zero (flat) to full super-elevation, or vice versa. See also 'tangent runout'.

Systems Interchange Interchange connecting two freeways, i.e. a node in the freeway system.

Tangent The straight portion of a highway between two horizontal curves.

Tangent Runout The tangent runout section consists of the length of roadway needed to accomplish a change in outside-lane cross slope from the normal cross slope rate to zero (flat), or vice versa.

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

Terrain classification

Flat	0-10	5m contours crossed per km
Rolling	11-25	5m contours crossed per km
Mountainous	>25	5m contours crossed per km

Traffic 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 travelled way 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.

Transition Curve Curve whose radius changes continuously along its length, used to connect a tangent with a circular arc or two circular areas of different radii.

Transition Length Length of the transition curve.

Travelled Way The lanes of the cross-section used for the movement of vehicles. The travelled way excludes the shoulders, auxiliary lanes, bus-bays, etc.

Trumpet Interchange	A three-legged interchange containing a loop ramp and a directional ramp, creating between them the appearance of the bell of a trumpet.
Trunk Road	International Trunk Road linking centres of international importance and crossing international boundaries or terminating at international ports.
Turning Lanes	Lanes which separate turning vehicles from the through traffic lanes.
Turning Roadway	Channelized turning lane at an at-grade intersection. Sometimes this includes interchange ramps and intersection curves for left turning vehicles.
Turning Template	A graphic representation of a design vehicle's turning path for various angles of turn. If the template includes the paths of the outer front and inner rear points of the vehicle, reference is to the swept path of the vehicle.
Underpass	A grade separation where the subject highway passes under an intersecting highway.
Urban Road or Highway	Characterised by high traffic volumes moving at relatively low speeds and pronounced peak or tidal flows. Usually within an urban area but may also be a link traversing an unbuilt up area between two adjacent urban areas, hence displaying urban operational characteristics.
Value Engineering	A management technique in which intensive study of a project seeks to achieve the best functional balance between cost, reliability and performance.
Verge	The area between the edge of the road prism and the reserve boundary.
Vertical Alignment	Direction of the centreline of a road in side profile.
Vertical Curve	Curve on the longitudinal profile of a road, normally parabolic.
Warrant	A guideline value indicating whether or not a facility should be provided. Once the warranting threshold has been met, this is an indication that the design treatment should be considered and evaluated and *not* that the design treatment is automatically required.
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.

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1 Introduction

1.1 General

This manual was prepared by the Ministry as part of a series of manuals that cover the entire project cycle. The series incorporate best practices, climate change considerations, and recent technologies thereby enabling the provision of road infrastructure that is safe, secure, and efficient.

The Kenya road manual series is as follows:

Project Cycle Stage	Manual: Volume or Part/Chapter	Code	Status
A. General	**Procedures and Standards Manual**	**PSM**	
	1. General		Not developed
	2. Policies		Not developed
	3. Procedures Guidance		Not developed
	4. Codes of Practice		Not developed
	5. Guidelines		Not developed
	6. Product/Testing Standards		Partially developed
B. Planning	**Network and Project Planning Manual**	**NPM**	
	1. Road Classification		Partially developed
	2. Route/Corridor Planning		Not developed
	3. Roadside Development and Control		Not developed
	4. Highway Capacity		Not developed
	5. Project Planning		Not developed
C. Appraisal	**Project Appraisal Manual**	**PAM**	
	1. Environmental Impact Assessment and Audit		Not developed
	2. Social Impact Assessment		Not developed
	3. Traffic Impact Assessment		Not developed
	4. Road Safety Audits		Partially developed
	5. Project Appraisal		Partially developed
	6. Feasibility Studies		Partially developed
D. Design	**Road Design Manual**	**RDM**	
	1. Geometric Design		Partially developed
	2. Hydrology and Drainage Design		Partially developed

Project Cycle Stage	Manual: Volume or Part/Chapter	Code	Status
	3. Materials and Pavement Design for New Roads		Partially developed
	4. Bridges and Retaining Structures Design		Partially developed
	5. Pavement Maintenance, Rehabilitation and Overlay Design		Partially developed
	6. Traffic Control Facilities and Communication Systems Design		Partially developed
	7. Road Lighting Design		Not developed
E. Contracts	**Works and Services Contracts Manual**	**WSCM**	
	1. Forms of Contracts		Partially developed
	2. Standard Specification for Road and Bridge Construction		Partially developed
	3. Bills of Quantities		Partially developed
	4. Standard/Typical Drawings		Partially developed
F. Construction	**Road Construction Manual**	**RCM**	
	1. Construction Management		Not developed
	2. Project Management		Partially developed
	3. Site Supervision		Not developed
	4. Quality Assurance		Not developed
	5. Quality Control		Not developed
G. Maintenance	**Road Asset Management Manual**	**RAMM**	
	1. Maintenance Management		Partially developed
	2. General Maintenance		Partially developed
	3. Pavement Maintenance		Partially developed
	4. Bridges and Structures Maintenance		Partially developed
H. Operations	**Road Operation Manual**	**ROM**	
	1. Traffic Management		Partially developed
	2. Vehicle Load Control		Not developed
	3. Emergency Services		Not developed
	4. Tolling		Not developed

Project Cycle Stage	Manual: Volume or Part/Chapter	Code	Status
I. Monitoring and Evaluation	**Monitoring and Evaluation Manual**	**MEM**	
	1. Performance Monitoring Manual		Not developed
	2. Technical Audits		Not developed
	3. Poverty, Gender Equality and Social Inclusion Monitoring		Not developed

This manual, as indicated below, is Volume 1 – Geometric Design Part 3: Geometric Design for Highways, Rural and Urban Roads and is part of the Road Design Manual (RDM) whose volumes, parts and coding system are as follows:

Vol	Manual Title	Part Name	Part/Code
1	Road Design Manual: Vol. 1 - Geometric Design	Part 1 – Topographic Survey	RDM 1.1
		Part 2 – Traffic Surveys	RDM 1.2
		Part 3 – Geometric Design of Highways, Rural Roads and Urban Roads	RDM 1.3
2	Road Design Manual: Vol. 2 - Hydrology and Drainage Design	Part 1 – Hydrological Surveys	RDM 2.1
		Part 2 – Drainage Design	RDM 2.2
3	Road Design Manual: Vol. 3 - Materials and Pavement Design for New Roads	Part 1 – Alignment Survey and Materials Prospecting	RDM 3.1
		Part 2 – Materials Field and Laboratory Testing	RDM 3.2
		Part 3 – Pavement Foundation and Materials Design	RDM 3.3
		Part 4 – Flexible Pavement Design	RDM 3.4
		Part 5 – Rigid Pavement Design	RDM 3.5
4	Road Design Manual: Vol. 4 - Bridges and Retaining Structures Design	Part 1 – Bridge and Culvert Design	RDM 4.1
		Part 2 – Retaining Structures Design	RDM 4.2
		Part 3 – Bridge Condition Survey	RDM 4.3
		Part 4 – Bridge Maintenance Design	RDM 4.4
5	Road Design Manual: Vol. 5 – Pavement Maintenance, Rehabilitation and Overlay Design	Part 1 – Pavement Condition Survey	RDM 5.1
		Part 2 – Pavement Maintenance, Rehabilitation and Overlay Design	RDM 5.2
6	Road Design Manual: Vol. 6 - Traffic Control Facilities and Communication Systems Design	Part 1 – Road Marking	RDM 6.1
		Part 2 – Traffic Signs	RDM 6.2
		Part 3 – Traffic Signals and Communication System	RDM 6.3
		Part 4 – Other Traffic Control Devices	RDM 6.4

Vol	Manual Title	Part Name	Part/Code
7	Road Design Manual: Vol. 7 - Road Lighting Design	Part 1 – Grid-connected Road Lighting	RDM 7.1
		Part 2 – Solar Road Lighting	RDM 7.2

1.2 Objectives of this Part

Previously geometric design was defined as the process whereby the layout of the road in the terrain is designed to meet the needs of the road user, but this definition is no longer adequate or even correct unless the road user is defined more appropriately and more inclusively to include not only those travelling on the road but also those living and working in and around roads. It is not only urban areas that this extended definition is required; in rural areas, pedestrians and others who do not drive a road motor vehicle also interact extensively with vehicular traffic, hence the entire road environment is important.

Thus, the geometric design standards are intended to meet three important objectives.

1) To provide satisfactory levels of service, safety, and comfort for travellers;
2) To provide adequate road and street infrastructure for the needs of the entire community;
3) To optimise or minimise earthworks volumes to reduce construction costs.

1.3 Scope of this Part

The geometric design standards and characteristics of a road depend on several principal defining factors namely:

1) The function of the road;
2) The traffic that it is designed to carry; (this could be classified as a component of its function, but it is more convenient to keep it separate);
3) The location of the road and especially the topography of the alignment of the road,
4) The surroundings of the road (urban or rural) and
5) The climate.

Designing a new road or upgrading an existing road requires many skills and effective documents to provide advice, instructions, and guidance, not only at the design stage but from conception, planning, through to maintenance, and eventual rehabilitation or upgrading. A road is designed to provide good service for many years and therefore good planning and good-long term management are required. These activities rely on data and information.

This manual provides guidelines for design of roadways, for motorized and non-motorized traffic, in both urban and rural areas to enable designers to balance the needs of all road users and the environment traversed by the road. It includes guidelines for design of:

i) Cross sections and head-rooms
ii) At-grade junctions
iii) Grade separated junctions.

iv) Road safety facilities

v) Roadside amenities

vi) Arboriculture

1.4 Organisation of this Part

The manual is divided into 17 chapters as follows:

Chapter 1 is dealing with general issues such as introduction, objectives and scope of the manual part, scope of geometric design, the design process, data requirements and organisation of the manual.

Chapter 2 addresses road classification. This includes both administrative and functional classes. It also focuses on dimensions and selection of appropriate road design class.

Chapter 3 is dedicated to design controls and criteria affecting the selection of the geometric design values. These include design vehicles, driver performance, traffic characteristics, capacity, level of service etc.

Chapter 4 is about 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.

Chapter 5 covers the design of the horizontal alignment.

Chapter 6 is specific to the design of the vertical alignment, in general.

Chapter 7 attends the aspect of combining the horizontal and vertical aspects of the alignment to ensure that the phasing results in enhancement of road safety.

Chapter 8 presents geometric design standards for the various classes of rural, inter-urban and urban roads. The standards are meant as a guide for the designer.

Chapter 9 discusses at-grade intersections, including design requirements and procedure, selection of intersection type, T-junctions, cross junctions; and intersection elements including turning lanes and traffic islands.

Chapter 10 is specific to roundabouts, which essentially are in important type of at-grade intersections.

Chapter 11 is about grade-separated intersections or interchanges.

Chapter 12 gives a comprehensive treatise of the design of road safety systems.

Chapter 13 covers provision of roadside facilities and amenities.

Chapter 14 is on the modelling of the design and calculation of earthworks and pavement quantities.

Chapter 15 outlines the aspects to be considered to assure quality designs and drawings.

Chapter 16 provides guidance on requesting departures from standards.

Chapter 17 is a bibliography of references.

1.5 Design Process

The design process considers three general types of projects:

1) New construction: new construction projects are those that construct roads on new alignment where no existing roadway is present. Some new construction, particularly in rural areas, is accomplished at sites with no existing development; other new construction projects—in rural areas and particularly in urban areas—may involve removing existing structures.
2) Reconstruction: reconstruction projects are projects that utilize an existing roadway alignment (or make only minor changes to an existing alignment) but involve a change in the basic roadway type. Changes in the basic roadway type include widening a road to provide additional through lanes or adding a raised or depressed median where none currently exists, and where these changes cannot be accomplished within the existing roadway width (including shoulders).
3) Improvement: improvement projects on existing roads are those that keep the existing roadway alignment (except for minor changes) and do not change the basic roadway type. Such projects are classified for design purposes by the primary reason the project is being undertaken or the specific need being addressed. The typical project needs addressed by road and street improvement projects on existing roads include repair infrastructure condition; reduce current or anticipated traffic operational congestion; and reduce current or anticipated crash patterns.

The development of any of the three type of projects above will typically follow a process similar to that shown in Figure 1-1. There are likely to be several iterations before achieving a solution that optimises the design criteria, some of which may be conflicting. Regardless of the option chosen, all geometric designs should be subject to design and safety reviews at appropriate phases in the design process.

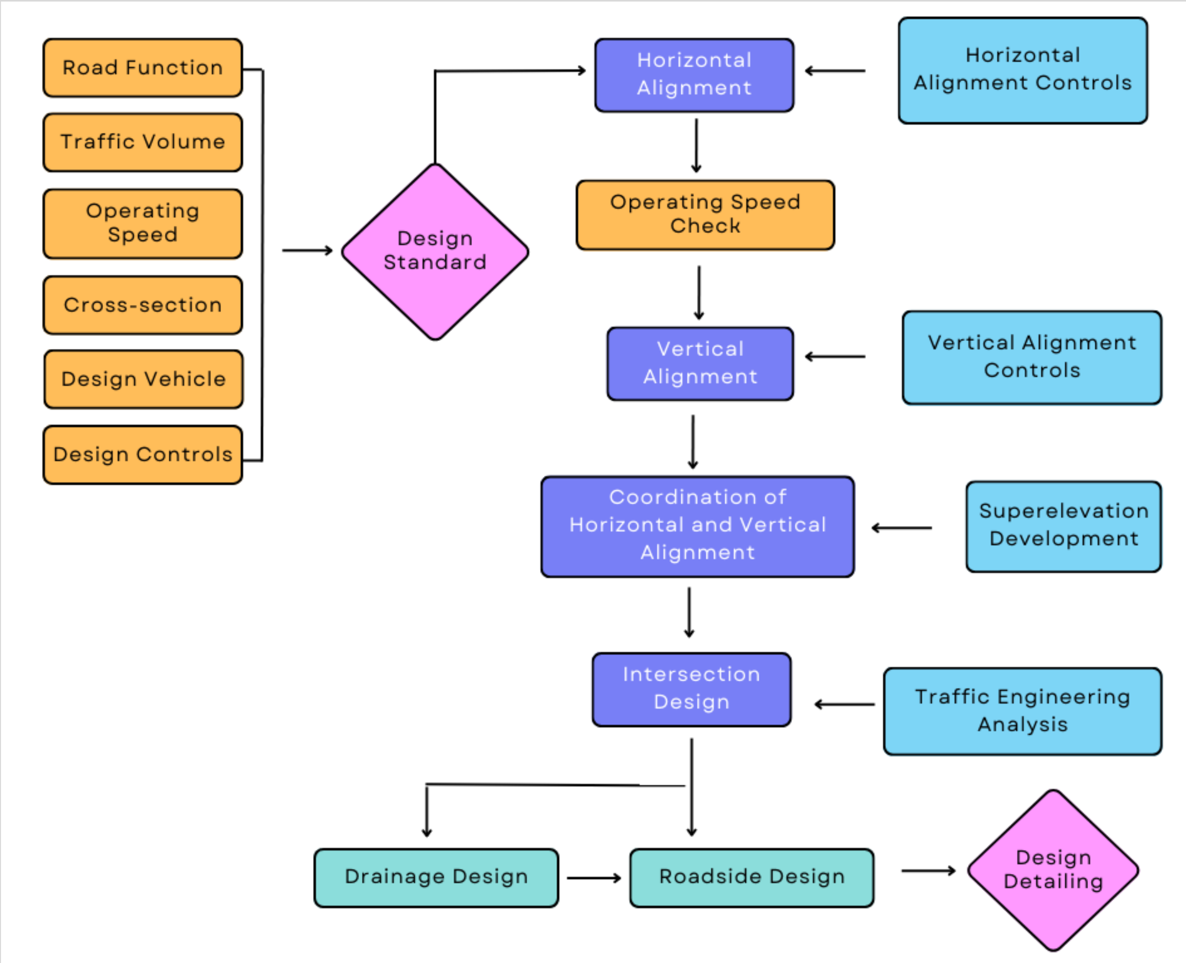


Figure 1-1: Design process

1.6 Data Requirements

1.6.1 Site investigation, traffic assessment and route selection

Topographic information is required for many aspects of highway engineering including the planning of highway investment, for selecting the precise route of the road and for locating sources of road building materials. Information about vehicle types, the number of vehicles and their loading is also required for forecasting the traffic that will use the road and therefore for designing both its geometric and structural features.

Data will be available from various sources, but new data will probably be required, and existing data may need to be updated depending on its date of acquisition. Topographic surveys and site investigations are described in detail in **RDM Volume 1 Part 1: Topographic Surveys**, and traffic surveys and assessment are described in **RDM Volume 1 Part 2: Traffic Surveys.**

1.6.2 Topographic surveys

The road cuts, embankment heights and fills are determined from the topography along the alignment. These data are also required for the design of horizontal and vertical curves. Therefore, topographical surveys are important for the entire geometric design. These will include conventional longitudinal and transverse level surveys along the entire length of the road at appropriate intervals – usually 20 m in the longitudinal direction on the proposed centreline and at 2 m interval offsets on each side of the centreline up to the edge of the right-of-way. The survey should use a GPS-based total-station with readings taken on an automatic theodolite. The data should preferably be reported in a format that can be inserted directly into any geometric design software being used for the project. During this survey, temporary benchmarks must be fixed and referenced.

Topography data obtained by photogrammetry can be completer and more accurate, with features such as fences, walls, utility poles, and existing improvements, in addition to elevation contours and spot elevations. These data will be of great use to the designer in determining road reserve restrictions, swamp or rock locations, and the need for relocating utility lines or other physical features that affect the design.

The topographic data should be checked for completeness against other maps available to the designer. Topography data obtained by field ground surveys will generally be collected electronically for downloading to appropriate computer-aided design software.

1.6.3 Traffic assessment

Traffic assessment covers the calculation of design traffic for new trunk roads, or rehabilitation or widening of existing roads. There are several situations, and each requires the traffic to be estimated in a slightly different way.

1) The most common situation is upgrading an existing road to a road class where traffic congestion is not expected. Since the road already exists it is likely that traffic information is already available. If not, the survey techniques described in **RDM Volume 1 Part 2: Traffic Surveys** must be used to count the traffic and to forecast traffic growth.
2) Upgrading an old track or relatively low class of road in poor condition is also a common situation. The existing traffic may not be a reliable guide to future traffic. Large increases in traffic often occur quite quickly after a new road is opened, exceeding normal growth patterns because of additional generated traffic, and diverted traffic. Such growth needs to be estimated as part of the traffic survey.
3) Upgrading an existing road to one of the highest classes where traffic congestion is a possibility over the design period. In this situation a normal two-lane road may not be sufficient, and a 4-lane dual carriageway may be required to prevent serious traffic congestion. More detailed traffic flow information is required because hourly flow rates are critical.
4) Designing and building an entirely new road. This is the most difficult situation for estimating traffic because little or no data are likely to exist.

The traffic volume on a specific road section can vary significantly between peak hours, daily traffic, and seasonal flows and therefore the peak flows can be high enough to control the road design. The extent of the variation depends on factors such as the road function, the traffic conditions, location, and environment. Localised information on seasonal variations should be obtained by collection of appropriate traffic data and by local knowledge where no traffic studies, statistics or comparative counts are available. Reference should also be made to the **RDM Volume 1 Part 2: Traffic Surveys.**

2 The Road System

2.1 General

Functional classification of roads is an approach to ensure that there is a feasible and logical road network throughout the country. Kenya's road classification reflects the ways in which the community links to the wider world. The system of classification directs motorists towards the most suitable routes for reaching their destination. It achieves this by identifying roads that are best suited for the traffic.

For rural roads the major function is to provide mobility, i.e., to cater for through and long-distance traffic where high and uniform speeds and uninterrupted traffic flows are desirable. For urban roads the major function is to provide land access mobility, hence high speeds are unnecessary and, for safety reasons, undesirable.

While the functional classes provide general information for the designer about the character of the roadway at any given location, roadway conditions may vary widely within these classes. Thus, merely identifying the functional and context classification for the roadway is not a sufficient basis for design. More specific guidance on the design of individual roadways within given functional and context classes is provided by:

1) A modal classification system so that the needs of all transportation modes are considered in the design of each roadway or project;
2) A design approach that describes roads by both their functional class and the design standard of their geometry. There is, of course, usually a strong correlation between the two.
3) A flexible design approach to finding the appropriate balance for each project to meeting the needs of all users and transportation modes.

Functional classification defines the role of each roadway in serving motor-vehicle movements within the overall transportation system. The functional classification of a roadway suggests its position within the transportation network and its general role in serving personal vehicles, truck, and transit vehicles. However, it is important to understand that functional classification, by itself, does not address explicitly fitting the roadway into the community or the needs of other transportation modes particularly non-motorized transport.

2.2 Hierarchy of Functional Classification

Motor vehicle travel involves a series of distinct travel movements. The six recognizable stages in most trips include main movement, transition, distribution, collection, access, and termination. For example, Figure 2-1 shows a hypothetical trip using a freeway, where the main movement of vehicles is uninterrupted, high-speed flow. When approaching destinations from the international highways, vehicles reduce speed on ramps, which act as transition roadways. The vehicles then enter moderate-speed national highways (distributor facilities) that bring them nearer to the vicinity of their destination neighbourhoods. They next enter inter-county and sub-county roads that penetrate neighbourhoods. The vehicles finally enter local access roads that provide direct approaches to individual residences or other terminations. At their destinations, the vehicles are parked at an appropriate terminal facility.

Each of the six stages of this hypothetical trip is handled by a separate facility designed specifically for its function. Because the movement hierarchy is based on the total amount of traffic volume, travel on international highways is generally the highest level in the movement hierarchy, and lowest in the movement hierarchy is travel on local roads and streets.

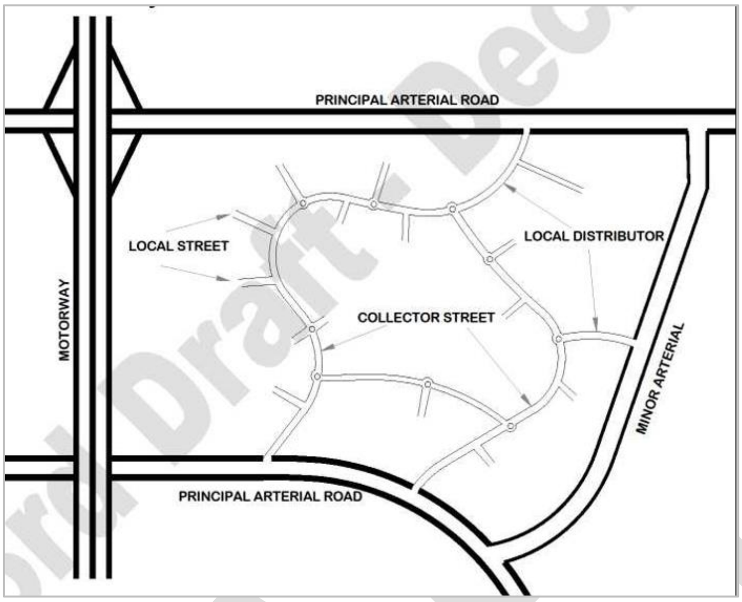


Figure 2-1: Hierarchy essentials

Despite the hypothetical representation in travel, in practice, not intermediate facilities are required. This hypothetical approach mostly applies to conditions of low-density peri-urban development. For instance, terminal facilities may be present on high-speed urban arterials, local roads from estates may also directly connect to such high-speed roads. Some large traffic generators may be connected directly to a ramp to an international highway, so that traffic does not need to use the primary distributor system.

However, this absence of intermediate facilities for some trips does not eliminate the need for all functional classes of roads and streets in the transportation network. It rather highlights the importance for designer to be cognizant of the road users expected and typical road behaviour in such situations. If a designer fails to recognize and accommodate each of the different trip levels of the movement hierarchy, conflicts and congestion are prone to occur at interfaces between public highways and private traffic-generating facilities.

2.3 Kenya Road Classification

2.3.1 Functional classification and administrative categories

The roads in Kenya are classified based on their functional and administrative categories as shown in Table 2-1

Table 2-1: Kenya road classification system

Road class	Link role	Functional category	Proposed description	Constitution administrative class/ Road authority
Interurban Roads				

Road class	Link role	Functional category	Proposed description	Constitution administrative class/ Road authority
A	International Highways	Major arterial roads through urban & rural areas	Roads forming the Trans-Africa and the Eastern Africa Highways routes linking centres of international importance, crossing international boundaries at designated border posts, or terminating at international ports.	National Government (KeNHA)
B	National Highways	Major arterial roads through urban & rural areas	Roads forming the national highways routes linking centres of national importance to each other or to Class A roads.	National Government (KeNHA)
C	Primary Roads/ Inter County Roads			
Rural Roads				
D	Secondary Roads/ Inter Sub-County	Major collector roads in rural areas	Roads located within a county linking major centres in at least two sub-counties/districts or two higher class roads	County Roads/ County Government
E	Minor Roads/ Sub-County Roads	Minor collector roads in rural areas	Roads located within a sub-county linking rural centres (markets/local centres) to each other or two higher class roads	County Roads/ County Government
F	Forest Roads	Local rural road	Roads in Government gazetted forests	National Government (KFS)/ County Government
G	Government Institutions Access Roads	Local rural road	Access road to Government facilities in rural and remote areas	County Government/ National Government
K	Coffee Roads	Local rural road	Roads accessing coffee (Kahawa) growing areas	County Government
L	Land Access Roads	Local rural road	Roads accessing settlement scheme areas	County Government
P	Game Park and Game Reserve Roads	Local rural road	Roads in National Parks and Game Reserves	National Government (KWS)
R	Rural Access Roads	Local rural road	Roads accessing rural communities or villages	County Government
S	Sugar Roads	Local rural road	Roads accessing sugar growing areas	County Government

Road class	Link role	Functional category	Proposed description	Constitution administrative class/ Road authority
T	Tea Roads	Local rural road	Roads accessing tea growing areas	County Government
U	Unclassified Rural Roads	Local rural road	Unclassified roads including minor roads	County Government
W	Wheat Roads	Local rural road	Roads accessing wheat growing areas	County Government
Urban Roads				
UA	Urban Arterial Roads	Arterial roads within municipalities and cities	Major and minor arterial roads located within municipalities and cities	National Government (KURA)
UC	Urban Collector Roads	Urban collector road within municipalities and cities	Major and minor collector roads including primary and secondary distributors located within cities, municipalities, and other urban centres	County Government
UL	Local Urban Streets	Local urban streets in urban areas	Local urban streets in residential, industrial, and commercial areas	County Government

2.3.2 Route numbering system

Numbers are allocated to roads to aid road users when navigating the network. To avoid confusion, it is important that numbers are used in a consistent fashion. To ensure this, the Chief Engineer, Ministry of Roads will maintain a central register of all road numbers in in Kenya. This will avoid duplications and ensure that road numbers are not reused so quickly as to cause confusion.

A road number should apply to a single route. This route can be composed of a number of different physical roads and can change direction at junctions. Where two roads temporarily merge together, a number can reemerge at a later point. Road agencies should avoid situations where a number 'forks' onto two distinct roads, other than at junctions, slip roads or one-way systems. In all cases, the overriding aim must be to avoid confusion for the motorist.

If an agencies or authority wishes to create a newly numbered road, they will need to contact the Chief engineer-Roads to obtain an unused number. It is recommended that the agency must first consider whether a particular number would fit with existing numbers in the surrounding area.

The numbering system for the Kenya road network is as in Table 2-2.

Table 2-2: Kenya route numbering system

Category	Road Class	Numbering System
Inter-urban roads	Trans Africa Highways, Class A	1) **Main route**: Designated with alphanumeric number having letter "A" and three-digit number i.e. A104 and A109. 2) **Bypasses or connecting routes and full or partial circumferential beltways around and within urban areas**: Designated with the number of the main route and an even number suffix preceded by dash e.g. A104-2, with suffixes for each county assigned progressively from West to East. 3) **Supplemental radial and spar routes connecting with the main route at one end**: Designated with the number of the main route and an odd number suffix preceded by dash e.g. A104-1, with suffixes for each county assigned progressively from West to East. 4) **Express Ways**: Designated with the number of the main route and letter E suffix preceded by dash e.g. A104-E2, with digit assigned progressively from West to East.
	Eastern Africa Regional Highways, Class A	1) **Main route:** Designated with alphanumeric number having letter "A" and one to two-digit number i.e. A1 and A23. 2) **Bypasses or connecting routes and full or partial circumferential beltways around and within urban areas**: Designated with the number of the main route and an even number suffix preceded by dash e.g. A23-2, with suffixes for each county assigned progressively from West to East. 3) **Supplemental radial and spar routes connecting with the main route at one end**: Designated with the number of the main route and an odd number suffix preceded by dash e.g. A23-1, with suffixes for each county assigned progressively from West to East. 4) **Express Ways**: Designated with the number of the main route and letter E suffix preceded by dash e.g. A23-E, with digit assigned progressively from West to East.
	National Highways, Class B	1) **Main routes**: Designated with alphanumeric number having letter "B" and one to two-digit number ranging from 1 to 99 progressively from West to East. 2) **Bypasses or connecting routes and full or partial circumferential beltways around and within urban areas**: Designated with the number of the main route and an even number suffix preceded by dash e.g., B1-2. To avoid duplication, progressive prefixes are used from South to North for North bound routes and West to East for East bound routes. 3) **Supplemental radial & spur routes connecting with the main route at one end**: Designated with the number of

Category	Road Class	Numbering System
		the main route and an odd number suffix preceded by dash e.g., B1-1. To avoid duplication, progressive prefixes are used from South to North for North bound routes and West to East for East bound routes. 4) **Express Ways**: Designated with the number of the main route and letter E suffix preceded by dash e.g., B1-E2, with a digit assigned progressively from West to East.
	Primary Roads or Inter County Roads, Class C	1) Designated with alphanumeric number having letter “C” and a number from 100 to 899 progressively from West to East e.g., C100.
	Secondary Roads or Inter Sub-County Roads, Class D	1) Designated with alphanumeric number having letter “D”, County number in accordance with the First Schedule of the Constitution of Kenya 2010 and one to two-digit number suffix preceded by a dash ranging from 1 to 19 progressively from West to East e.g., D16-19.
Rural roads	Sub-County Roads, Class E	1) Designated with alphanumeric number having letter “E”, County number in accordance with the First Schedule of the Constitution of Kenya 2010 and two-digit number suffix preceded by a dash ranging from 20 to 99 progressively from West to East e.g., E16-99 for a road in Machakos County
	Rural Access Roads, Class R	1) Designated with alphanumeric number having letter “R”, County number in accordance with the First Schedule of the Constitution of Kenya 2010 and three-digit number suffix preceded by a dash ranging from 20 to 99 e.g., E16-99 for a road in Machakos County.
	Urban Arterials, Class UA	1) Designated with road name and an alphanumeric number having letter “UA”, Municipality or City Code in accordance with international postal union code and one to two-digit number suffix preceded by a dash ranging from 1 to 19 progressively from West to East e.g., UA01-19.
Urban roads	Urban Arterials, Class UC	1) Designated with road name and an alphanumeric number having letter “UC”, municipality, or City Code in accordance with international postal union code and two-digit number suffix preceded by a dash ranging from 20 to 99 progressively from West to East e.g., UC01-99.
	Urban Streets, Class UL	1) Road Name and alphanumeric code prefix having county number, municipality (city) number and two-digit number suffix preceded by a dash ranging from 100 to 999 progressively from West to East e.g., UC01-99 e.g..

2.4 Control of Access

Uncontrolled access to roadside development along roads whose major function is to provide mobility will result into an increased safety risk, 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 Highway Authority.

Although control of access is one of the most important means of preserving the efficiency and road safety of major roads, roads without 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 private 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 connection with public roads, there may be (some) private access connections.
3) Unrestricted access- means that preference is given to local traffic, with road serving the adjoining areas through direct connections. However, the detailed location and layout of the accesses should be subject to approval by the Highway authority in order to ensure adequate standards of visibility, surfacing, drainage etc.

Road function determines the level of access control needed. Motorways (expressways) should always have full control of access. For all purpose roads, the general guidelines for the level of access control in relation to the functional road classification are given in Table 2-3 and Table 2-4.

Table 2-3: Level of access for interurban and rural roads

Functional class	**Level of access control**	
	Desirable	**Reduced**
A	Full	Partial
B	Full	Partial
C	Full or Partial	Partial
D	Partial	Unrestricted
E	Partial or Unrestricted	Unrestricted
F, K, G, T, S, W, U	Unrestricted	Unrestricted

Table 2-4: Level of access for urban roads

Functional class	**Level of access control**	
	Desirable	Reduced
UA	Full	Partial

Functional class	Level of access control	
	Desirable	Reduced
UC	Full	Partial
UL	Full or Partial	Partial

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

Control of access is accomplished either by a 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 lanes and junctions should be carefully considered at the design stage and included in the final design for the project.

2.5 Road Reserve

Road reserves are provided in order to accommodate future road connections or changes in alignment, road width or junction layout for existing roads and to enhance the safety, operation, and appearance of the roads. The road reserve should always be determined and shown on the final design plans for road projects. The applicable road reserve width for interurban and rural roads are shown in Table 2-5 and for urban roads in Table 2-6Table 2-6.

Table 2-5: Road reserve interurban and rural

Functional class	Desirable	Reduced
A	60	40
B	60	40
C	40	40
D	25	25
E	20	20

Table 2-6: Road reserve urban

Functional class	Level of access control	
	Desirable	**Reduced**
UA	60	40
UC	60	40
UL	25	25

The reduced widths should be adopted only when this is 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.

For dual carriageway roads it may be necessary to increase the road reserve width above the given values. As a general rule the road reserve boundary should be at a distance from the centreline of the nearest carriageway equal to half the road reserve width for single carriageway roads.

2.6 Functional Classification and Design Standards

The function of a particular road in the national, regional, and local road network clearly has a significant impact on the design criteria to be used, and the design engineer must consider 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 and expected performance.
2) Selection of the level of access control compatible with the function of the road.
3) Consideration of modal classification to cater for the needs of all transportation modes anticipated on the road.
4) Selection of geometric design standards compatible with function and level of access control and maintaining a flexible design approach aimed at finding the appropriate balance for each project that meets the needs of all users and transportation modes.

When the functional classification and level of access control are given, 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 junctions, frequency of accesses, standards of alignment and grades, traffic controls and road reserve widths.

The interurban and rural roads are divided into the five major functional classes according to their major function that depends primarily on the traffic level plus several agricultural and specialist minor road classes and the urban roads divided into three main classes (Table 2-7). Each functional class can be built in most terrains with the design speed appropriate to the terrain and their location (urban or rural). Design speed is the most important factor that determines the geometric characteristics of the road therefore there are a considerable number of design standards that are possible although many of them are not practicable or required.

The derivation of the design standards listed in Table 2-7, Table 2-8 and Table 2-9 below is described in the following chapters and summarised in Chapter 8 for convenience and ease of reference, showing, for each functional class, the different design speeds, and the main characteristics of each design.

Table 2-7: Design standards and typical functional classes

Design Standard	Description of function	Typical functional class
DR1, DU1	The highest geometric design standard for rural or urban areas. Roads of this standard usually service long trips with high speed of travelling, comfort, and safety. This standard always includes divided carriageways with full access control.	A, UA
DR2, DU2	High geometric standards suitable for long to intermediate trip lengths with high to medium travelling speeds. The road is usually designed with partial access control.	A, B, UA
DR3, DU3	Medium geometric standard road suitable for intermediate trip lengths with medium travelling speeds and partial access control.	B, C, UC
DR4, DU4	Low geometric standards serving mainly local traffic. Partial or no access control.	B, C, D, UC
DR5, DU5	The lowest geometric standard road with two-way flow. This standard is used for roads accommodating local traffic with low volumes of commercial traffic.	C, D, UL
DR6	Very low geometric standards and is applied to very low traffic volume. It is used for urban areas where two-way traffic is not required.	D, E, UL
DR7	Provides for a road with low geometric standards with a gravel surfacing carrying low traffic volumes. The need for provision for two-way traffic is very low	E, F, G, S, W, T

Notes:

a) *An A or AU road will generally be among the widest, most direct roads in an area, and will be of the greatest significance to through traffic.*

b) *A B or UC road will still be of significance to traffic (including through traffic), but less so than an A or AU road respectively.*

c) *C, D or UL road will be of lower significance and be of primarily local importance but will perform a more important function than local roads.*

d) *A local road such as class F, W, S etc will generally have very low significance to traffic and be of only very local importance.*

Table 2-8: Design speeds for interurban and rural design standards

Design standards	Maximum design speed (km/h)				Application to functional classes
	Flat	Rolling	Mount-ainous	Escarpment	
DR1	120	100	80	-	A
DR2	110	90	70	50	A, B
DR3	100	90	70	50	B, C
DR4	90	85	60	50	B, C, D
DR5	80	65	50	40	C, D
DR6	70	65	50	40	D, E
DR7	50	40	30	30	E, Minor roads

Table 2-9: Design speeds for urban design standards

Design standards	Design speed (km/h)	Functional class
DU1	100	UA
DU2	80	UA, UC
DU3	70	UC
DU4	50	UC, UL
DU5	30	UL

3 Design Controls and Criteria

3.1 General

Road projects are developed to meet increasing travel demand, address crash problems, improve streetscapes, create walkable and accessible communities, rehabilitate existing infrastructure or a combination of any of these reasons.

The objective of the design (see Table 3-1) influences the controls and criteria to be adopted. With a clear understanding of the intended project objectives, designers can exercise professional judgment and flexibility to implement solutions in financially or physically constrained environments. This balanced approach can improve operational efficiency, road safety and public amenity whilst minimising the environmental impacts of noise, vibration, pollution, and visual intrusion.

Table 3-1: Objectives of design

Aspect	Objective
Safety	Optimise safety by providing a road and roadside that is designed to minimise killed and serious injuries (KSIs) for all road users.
Traffic	Optimise operational efficiency by providing a road that can carry the required volume of traffic at a speed that is consistent with the functional class of the road and road safety objectives
Uniformity of design	Maintain uniformity of design parameters along a route and/or within a network to provide a consistent and operationally effective travel experience relative to the functional class of the road.
Economic viability	Development of economically efficient designs to optimise the benefit of limited funds available for road construction and maintenance.
Future development	Adequately provide for the future requirements of the road network by considering the ultimate road layout required to serve general traffic growth and adjacent development in the vicinity of the works.
Universal design	Optimise opportunities to cater for the needs of all road user groups including all types of vehicles expected to use the road.
Environmental impacts	Minimise adverse environmental impacts (during construction and operation) and enhance the environment where possible both in the immediate vicinity of the road and over a wider area.
Public participation	Where practicable, provide a design that takes account of the views of the community including local residents, businesses, community groups and road users.
Active travel	Optimise the opportunity for physical activity through transport by creating safe and attractive walking and cycling environments within communities.

The selection of basic design controls and criteria occurs very early in the project development process and should consider the needs of all modes of transportation as well as the community and context in which the project is located. As the project progresses through preliminary and detailed design, early assumptions may be revised as more information becomes available. Appropriate standards (Chapter 8) and combinations of these elements should be determined on the basis of the following controls and criteria:

1) Terrain, land use and physical features.
2) Environmental and social considerations.
3) Safety considerations.
4) Road function and access control.
5) Traffic characteristics.
6) Road capacity and level of service.
7) Speed.
8) Design vehicle.
9) Driver factor.
10) Other road users.
11) Sight distance.
12) Road surface.
13) Economic and financial considerations.

All these controls and criteria should be considered in order to arrive at a final design which is in balance with the physical and social environment, which meets future traffic requirements, and which encourages consistency and uniformity of operation. In this way it is possible to eliminate at the design stage any environmental and operational problems which would otherwise increase accident potential and other detrimental effects and create disruption and additional costs for remedial measures in the future.

Alternative construction technologies incorporating labour intensive methods have a bearing on design criteria for lower class (D and E) roads; these cases require special consideration and sometimes a relaxation of the standards.

3.2 Safety Considerations

The Safe System approach to road safety seeks to have a road system that is designed to reduce the severity and incidence of crashes. The key principles to the Safe System approach recognizes:

1) Users of the road system will make mistakes, and the design must accommodate these.
2) That there are known physical limits to the amount of force the human body can withstand before serious injury or death occur, and the infrastructure design should ensure that the forces in collisions do not exceed these limits.

3) That there is a shared responsibility between providers, designers, operators, and users of the system for safety (with the responsibility of users often defined as their being compliant with road rules).

Safe design is one of the main objectives of design and it is embedded into the principles, criteria and values for the various design elements given in this manual. These various aspects are introduced throughout the manual in the appropriate chapters and some of them include:

1) Designing for an appropriate design speed for different situations and design features that reduce speed differentials between vehicles, e.g., flat grades and speed change lanes.
2) Provision of a balanced design, i.e., compatibility between the various design elements.
3) Provision of design elements compatible with traffic volumes and type of traffic (long-distance, through, local, etc.).
4) Provision of physical separation and separate facilities between motor vehicles and non-motorized traffic.
5) Adequate provision for other non-motorized travellers (cyclist, pedestrians), vulnerable road users (three-wheelers and motorcycles)
6) Avoidance of surprise elements for the drivers, i.e., no abrupt changes in standard, adequate visibility conditions and proper phasing of horizontal and vertical alignment.
7) Avoidance of situations where drivers must make more than one decision at a time.
8) Proper location and design of junctions with particular emphasis on sufficient sight distances, minimal conflict points, and clearly defined and controlled traffic movements.
9) Proper design, application and location of traffic signs, road markings and other traffic control devices.
10) Provision of proper drainage of the road surface.
11) Provision of safety barriers.
12) Provision of better road surfaces that provide higher levels of friction, thereby reducing the number and severity of skidding crashes.

Some crashes will happen even on roads designed to high safety standards because of the human, vehicle element and other environmental conditions involved. A basic consideration in road design is, therefore, to minimise severity of injuries and damage when crashes do occur. Important issues are:

1) Determination of the Network Roadside Risk Intervention Threshold (NRRIT) as the initial step by the road agency. NRRIT defines the threshold beyond which designers need to intervene to treat roadside hazards (**Austroads, 2022: Guide to Road Design Part 6: Roadside Design Safety and Barriers**).

2) Roadside slopes should be made as flat as possible whenever feasible, desirably 1:4 or flatter, and the roadside area should be well-rounded where slope planes intersect.

3) Road sign and lighting supports, and other utility poles should be located far enough from the carriageway to make them unlikely to be struck by an out- of-control vehicle, or they should have breakaway capability (frangible) and/or guardrail.
4) All drainage structures should be designed so that out-of-control vehicles can either pass safely over them or be safely deflected.
5) Barrier systems should be considered only when fill slopes of 1:4 or flatter are not feasible, and the damage caused by hitting a safety barrier would be less serious than damage from leaving the carriageway.
6) Roadside barriers should be provided at dangerous obstacles which cannot be removed, and which would cause serious damage if hit by an out-of-control vehicle (e.g., bridge piers, aggressive cliff faces and abutments).

Road safety considerations and features are built into the principles, criteria and values for the various design elements given in this manual. However, this does not necessarily ensure that the completed road will be of a safe design unless the designer is fully aware of, and considers, road safety aspects throughout all phases of the design work.

Where upgrading is required for rural and urban roads, crash data should be obtained and evaluated for considerations with respect to the new design. Such data should include crash type, cause, and severity. This will help in identifying the measures of crashes prevention that must be considered in the road design.

Due to the migratory nature of crashes on a network, it is desirable that a proactive approach of road safety assessments should be employed during upgrading of existing tracks or on new designs.

To improve road safety, the geometric design should consider the road environment, road characteristics and human factors which are explained under various chapters. This holistic approach is aimed at reducing the probability of 'failure' to the lowest possible level and should minimise the adverse consequences should failure occur.

To ensure road safety at various project stages, a road safety audit should be considered as detailed in the **Project Appraisal Manual Part 4: Road Safety Audits.**

3.3 Driver Performance and Human Factors

3.3.1 Human factors

Human factors in this context are defined as the 'contribution of the stable physiological and psychological limits of humans to the development of a technical dysfunction or failure in handling machines and vehicles'. This is not a new idea but was first introduced over 80 years ago. It excludes temporary mental or physical conditions. It is concerned with the general and stable reactions of common road users. The subject deals with identifying road characteristics that are not compatible with normal human threshold limit values and, therefore, can potentially trigger crashes.

It is therefore important that human factors are considered in many aspects of design, as indicated in the details of design in this manual, but the subject is of such importance that the fundamental principles are summarised here.

Drivers learn through experience that some events are likely to happen or are unlikely to happen. Thus, a roadway should confirm what drivers expect based on previous experience and should present clear clues as to what is expected of them. If these expectations are violated, problems are likely to occur which, in the most severe cases, may lead to crashes.

To avoid surprises, and their possibly dangerous consequences, it is essential to provide drivers with a continuous flow of information on the state of the road ahead, including information on:

1) The road alignment,
2) Approaching decision points, and
3) Other traffic, vehicular or pedestrian activity which may affect, or be in conflict, with them.

This advice should be mostly visual and provided by means such as road layout, signposting, traffic signs, and pavement markings. The receipt and subsequent treatment of this information depends largely on each driver's visual ability, reaction time and decision-making skills.

Designers should apply the following criteria:

1) Unexpected, unusual and inconsistent design should be avoided or minimised so that complex decisions by the driver are *not* required.
2) Predictable behaviour is encouraged through familiarity. For example, similar junction designs should be used in similar situations and the range of possible designs should be minimised.
3) Consistency of design should also be maintained from element to element along the road. This corresponds to relating the design speed to actual driver behaviour as expressed by the 85th percentile speed of cars under free-flow conditions. The difference between the 85th percentile and the design speed on an element such as a horizontal curve should be less than 20km/h.
4) To avoid information overload, information provided to the driver should be presented in sequence to avoid presenting several alternatives at the same time.
5) Clear sight lines and sight distances must be sufficient to allow time for good decision making.
6) Where possible, margins are allowed for recovery in case of error.

3.3.2 The 6 second rule

A user-friendly road will give a driver enough time to assess a situation and to modify driving behaviour accordingly. Therefore, design must account for driver’s reaction times by providing sufficient sight distances.

It is not enough to provide the driver with a section of road that allows only a reaction time of 2-3 seconds. The design should also provide an anticipation section with a minimum of 2-3 seconds more to identify an unexpected or unusual situation which may require more complex decision demands and to adjust driving accordingly. Sight distances for various situations are described in detail in Section 3.7 but are also so fundamental to geometric design that they are discussed throughout this manual.

In situations that are more complex or involve higher speeds, it is recommended that there should also be an advance warning section with proper signing and instructions.

Thus, it is necessary to arrange transition zones, remove visibility restrictions, and make junctions perceptible at least 6 seconds before any critical location (e.g., junctions, curves, railway crossings, bus stops, bicycle paths, entrances of villages and towns, end of newly upgraded road sections and changes in road hierarchy).

3.3.3 The field of view rule

The field of view can either stabilise or destabilise drivers, it can tire or stimulate them. It can also result in either increased or reduced speed. Speed, lane-keeping, and reliability of direction are functions of the quality of the field of view.

A good-quality field of view safeguards the driver and keeps him from drifting to the edge of the lane or even leaving it. Misleading and eye-catching objects in the periphery of the field of view activate unconscious changes in direction.

A user friendly, self-explaining road will give drivers a well-designed field of view and will give good optical guidance. A self-explaining road will avoid optical illusions or misleading eye-catching objects that destabilise drivers and negatively impact their driving, especially in conditions of adverse visibility.

3.3.4 The logic rule.

Drivers follow the road with an expectation logic based on their experience and recent perceptions. Unexpected abnormalities disturb a mostly automatic chain of actions and may cause a driver to ‘stumble’ (to use an apt analogy). Several critical seconds can pass before the disturbance can be processed. Designers should introduce inevitable changes as early and clearly as possible and avoid sudden changes that would confuse the driver.

3.4 Road Function, Environment and Access Control

Kenya has an established functional hierarchy for the road network. This hierarchy enables each road authority or agency to systematically plan and develop their network to meet the needs for local access, urban/city travel, inter-county, and international travel.

The basic premise of whether a road is located in an urban or rural area will to a certain extent impact on the attributes for which it is designed. Rural roads generally carry lower traffic volumes and are not subject to as many constraints as urban roads. Public expectation also differs in relation to operating speeds, abutting access, geometry, and cross-section.

Further details on the aspects of road function and environmental context have been provided in Chapter 2.

Depending on the function of a road, various levels of access control should be imposed. All points of access should be carefully considered and planned at the design stage. Access should not be allowed at locations where entering and leaving vehicles will create a hazard, particularly where sight distances are restricted or at points too close to other junctions. The proper location and design of access points may in some cases necessitate adjustments to the initial alignment.

3.5 Land Use, Physical Features and Terrain

3.5.1 General

Road design is an exercise in three-dimensional planning the success of which will be measured not only by the efficiency of the road but by its appearance and impact upon the adjoining areas.

Information regarding topography, land use and physical features are essential and should be obtained in the early stages of planning and design. In this respect it is necessary to consult with the physical planning authorities (Physical Planning Department and the Provincial Physical Planning Officers.) in order to co-ordinate the project with existing and proposed land uses and to protect the selected route from conflicting development.

Man-made features such as agricultural, industrial, commercial, residential and recreational developments are important controls for the route location and final design. Care should be taken to avoid unnecessary destruction, demolition or severance of valuable properties.

3.5.2 Route location

The first fundamental consideration in the design process is route location. An appropriate route shall be located that takes into account the intended functional classification, public opinion, environment, cost and benefits. The objective of route selection should be to choose a route that has both the minimum effect on landform and requires the smallest quantity of large earthworks.

A fundamental consideration in route location and final design is to fit the road sympathetically into the landscape, with a broad awareness of the character and features of the area through which it passes. This is required not only to obtain an aesthetically pleasing alignment, but in general is also necessary in order to obtain the most economic solution and the best possible service to the traversed area with the least detrimental effects.

3.5.3 Land use integration

It is important to consider routes that use the existing landform and minimise the land use.

In urban areas, an existing and future development land use study is essential for the proposed project. This will establish zones for residential or commercial use and open space. It should also include size of population and other related information.

3.5.4 Geology and geomorphology

Geological, soil, climatic and drainage conditions also affect the location and geometries of a road.

This section of the manual concerns the main geomorphological, geological and geotechnical features and drainage characteristics that shall be considered in the corridor where the different route alternatives are located. These existing physical conditions affect the location and geometries of the road and a general study of the area shall be conducted using available data such as exiting topographical maps, geological maps, climatic data and previous studies.

3.5.5 Erosion

Of particular importance in road design is the prevention of soil erosion. Areas should be identified where there are possible occurrences of landslides, slips, earth flows, and rock falls. These areas are to be avoided if possible in identifying alignment alternatives. Similarly cuts on steep slopes in volcanic rock should be avoided as these may result in collapse of the hillside. Areas of unstable soil and marked erosion should also be avoided, and in all cases where the foregoing are unavoidable a detailed geotechnical study of slope stability should be undertaken

3.5.6 Terrain

3.5.6.1 General

Terrain has the greatest effect on road costs and, therefore, it is not economical to use the same standards in all terrains. Inevitably, terrain therefore has a strong influence on the level of service that can be provided (Section 3.6).

The terrain class is a characteristic of the landscape where the road is to be built and is established before the road is designed. It is therefore independent of the alignment that is finally selected for the road. It is determined by counting the number of 5 metre contours crossed by a straight line connecting the two ends of the road section in question according to the following definitions in Table 3-2**.**

Table 3-2: Terrain classification

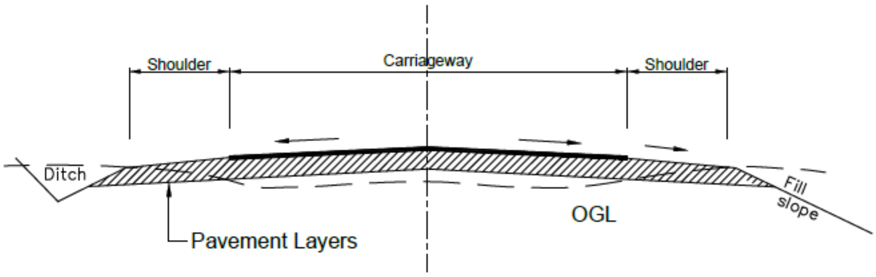
Terrain	Definition
Flat	The transverse ground slopes perpendicular to the ground contours are generally below 3 %.
Rolling	The transverse ground slopes perpendicular to the ground contours are generally between 4 % and 25%.
Mountainous	The transverse ground slopes perpendicular to the ground contours are generally above 26 % and 50%.
Escarpment	Escarpments are geological features that require special geometric standards because of the engineering problems involved. The transverse ground slopes perpendicular to the ground contours are generally greater than 50%.

In mountainous areas, the geometric standard takes account of the constraints imposed by the difficulty and stability of the terrain. The design standard may need to be reduced locally to cope with exceptionally difficult terrain conditions, but every effort should be made to design the road alignment so that the maximum gradient does not exceed the standards defined in Chapter 6.

It is difficult to provide adequate compaction on gradients greater than 10%, but, where higher gradients cannot be avoided, they should be restricted in length. Gradients greater than 10% should not be longer than 250 m. Horizontal curve radii of as little as 13 m may also sometimes be unavoidable, even though a minimum of 15 m is specified.

3.5.6.2 *Flat terrain*

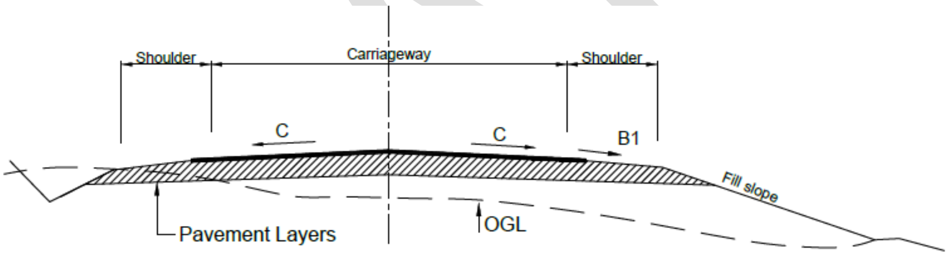
The topographical condition where highway sight distances, as governed by both horizontal and vertical restrictions are generally long or could be made to be so without construction difficulty or expense. The natural ground cross slopes perpendicular to natural ground contours in a flat terrain are generally below 3 %.



Typical Cross Section in Flat Terrain

3.5.6.3 *Rolling terrain*

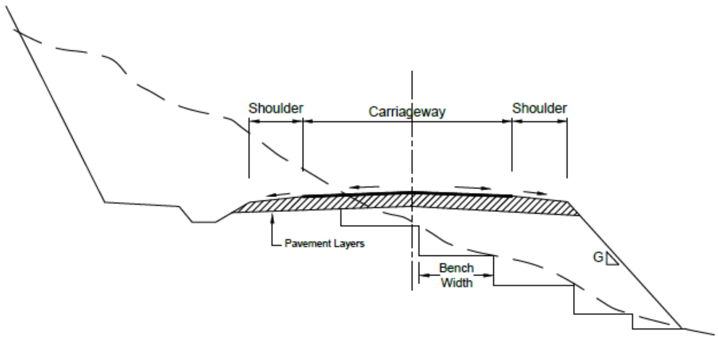
The topographical condition where the natural slopes consistently rise above and fall below the road or street grade and where occasional steep slopes offer some restrictions to normal horizontal and vertical roadway alignment. The natural ground cross slopes perpendicular to contours in rolling terrain are generally between 3-25 %.



Typical Cross Section in Rolling Terrain

3.5.6.4 *Mountainous terrain*

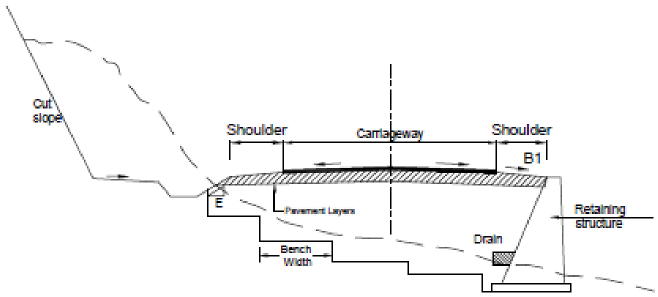
In mountainous terrain the route location and certain design features may be almost entirely governed by the terrain. The longitudinal and transverse changes in the elevation of the ground with respect to the road or street are abrupt and benching and side hill excavation are frequently required to obtain acceptable horizontal and vertical alignment. The natural ground cross slopes perpendicular to contours in mountainous terrain are generally above between 25-50 %.



Typical Cross Section in Mountainous Terrain

3.5.6.5 Escarpment terrain

The natural ground cross slopes perpendicular to contours in mountainous terrain are generally above 50 %.



Typical Cross Section in Escarpment Terrain

3.6 Speed and Speed Controls

3.6.1 General

Speed is a primary factor in all modes of transportation and geometric design of roads. The operating speed of vehicles on a roadway depends, in addition to capabilities of the drivers and their vehicles, upon general conditions such as the physical characteristics of the roadway, traffic conditions, the weather, driver behaviour, other types of vehicles using the road, roadside activities and the legal speed limitations. The actual speed on roadway usually reflects a combination of these factors.

A fundamental aim of road design is to provide a road system and environment that contributes to the prevention of vehicle crashes, particularly those involving fatalities and serious injuries, as well as satisfying the expectation of and need for efficient transportation. It should be noted that speed is a contributing factor to the severity of the crash. Consequently, a basic requirement of geometric road design is to provide geometry that is suitable for the speeds at which drivers choose to operate vehicles on roads or sections of roads. Designers should also consider vehicle speeds on the approaches to intersections, and it is important that speeds are managed to reduce the impacts of a crash, should one occur.

3.6.2 Speed classification

The term "speed" is often used very loosely when describing the rate of movement of road traffic. Road design recognizes various definitions or classifications of speed, all of which are interrelated. The sub-divisions are:

Desired Speed - the speed at which a driver wishes to travel, determined by a combination of motivation and comfort.

Design Speed - the speed selected as a safe basis to establish appropriate geometric design elements for a particular section of road and which should be a logical one with respect to topography, anticipated operating speed, the adjacent land use and the functional classification of the road.

Operating Speed - observed speeds during free flow conditions. For an individual driver, operating speed is generally lower than desired speed since operating conditions are not usually ideal.

Running Speed - the average speed maintained over a given route while a vehicle is in motion. The running time is the length of the road section divided by the time required for the vehicle to travel through the section. Thus, in determining the running speed, the times en-route when the vehicle is at rest are not taken into account in the calculations. Running speeds are generally used in road planning and capacity and service level analyses. The difference between running speed and design speed is strongly affected by traffic volumes.

Posted Speed - is a speed limitation set for reasons of safe traffic operations rather than for geometric design considerations and is aimed at encouraging drivers to travel at appropriate speeds for all prevailing conditions.

3.6.3 Design speed

The design speed is an index which links road function, traffic flow and terrain to the design parameters of sight distance and curvature to ensure that a driver is presented with a reasonably consistent speed environment**. *It is defined as the 85th percentile speed of passenger cars travelling in free flow conditions. Thus, it is not the average speed or the maximum speed that drivers are expected to use in practice***. This is an important distinction because it also means that the maximum speed would be inherently less safe but also most roads will only be constrained to minimum parameter values over short sections or on specific geometric elements.

The vital importance of road safety and the critical role of design speed and sight distances have been previously emphasised. This chapter introduces the specifications for design speed and the calculations of sight distance recommended for different driving situations.

The design speed should be logical with respect to the topography, the adjacent land use, the functional classification of the highway and the anticipated operating speed. However, it should be remembered that higher design speeds mean that sight distances will be longer. On the one hand this should make a road safer unless it encourages drivers to increase speed hence robust efforts to enforce speed limits should also be adopted. In view of the mixed traffic and the cost benefit of lower design speeds, it is prudent to use values of design speed towards the lower end of the internationally acceptable ranges.

Historically, a single design speed was used as the basic parameter for each road, or at least a significant length of road. Roads designed in this way have a consistent minimum design standard. The most common location where problems occur on roads designed with a single speed, is at the end of straights where vehicle operating speeds often exceed the design speed of the curve.

Changes in terrain and other physical controls may dictate a change in the design speed on certain sections. If a different design speed is introduced the change should not be abrupt but there should be a transition section of at least 1 km to permit drivers to change speed gradually before reaching the section of' the road with a different design speed. The transition sections are sections with intermediate design speeds and, where the magnitude of the change in the design speeds are large, more than one transition section will be required. Some of the factors that influence the choice of design speed include:

85th percentile operating speed	The 85th percentile operating speed should be the basis for setting the design speed. This accounts for the impact of intersections and traffic control devices; crash prevention factors (radius tracking, curve width); and several unresolved areas of driver behaviour (excess speed, increasing speed in curves, acceleration factors in straights, effect of enforcement). 85th percentile operating speed approach has the provision of a design margin that is typically 6 to 10 km/h over the speed limit on the less constrained sections of the road.
Posted speed limit	For all projects, the selected design speed should equal or exceed the anticipated posted speed limit of the completed facility. This requirement recognizes the relationship between likely operating speeds and roadway design, and that the posted speed limit creates a driver expectation of safe operating speed.
Balance	The design speed should be a reasonable balance between topography, urban and rural character, and the functional class of the roadway. A roadway in level terrain may justify a higher design speed than one in rolling terrain, and a roadway in a rural setting may justify a higher design speed than one in an urban area.
Traffic volume	Traffic volumes may impact the recommended design speed. A roadway carrying a large volume of traffic may justify a higher design speed than a similar facility with lower traffic volumes. However, a low design speed should not be automatically assumed for a low traffic volume road where the topography is such that drivers are likely to travel at high speeds. Drivers do not adjust their speeds to the functional classification of the roadway, but rather to the physical limitations and to traffic using the facility.

Design speed is significant only when the physical road characteristics limit the speed of travel. However, the design speed concept alone does not ensure sufficient coordination among individual geometric features to ensure consistency. It controls only the *minimum* value of the maximum speeds for the individual features along an alignment.

Example:

A road with an 80 km/h design speed may have only one curve with a design speed of 80 km/h and all other features may have design speeds of 120 km/h or greater. As a result, operating speeds approaching the critical curve are likely to exceed the 80 km/h design speed. Such an alignment would comply with an 80 km/h design speed, but it would violate a driver's expectancy and result in an undesirable alignment.

Thus, the various design elements must be combined in a balanced way, avoiding the application of minimum values for one, or a few, of the elements at a particular location when the other elements are considerably above the minimum requirements. Thus, the radii of curves within a section should be consistent, not merely greater than the minimum value.

When a substantial length of road is being designed, the designer should aim for a constant design speed to maintain consistency. In practice the speed of motorised vehicles on many roads in flat and rolling terrain will only be constrained by the road geometry over relatively short sections *but it is important that the level of constraint is consistent for each road class and set of conditions.*

The design speeds for the rural road standards are shown in Table 3-3 and in Table 3-4 for urban roads.

Table 3-3: Design speeds for rural roads

Functional class	Terrain							
	Flat		Rolling		Mountainous		Escarpment	
	Max	Min.	Max	Min.	Max	Min.	Max	Min.
A	120	110	100	90	80	70	-	-
B	110	90	90	85	70	60	50	50
C	100	80	90	65	70	50	50	50
D	90	70	85	65	60	50	50	40
E	70	50	65	40	50	30	30	30

Table 3-4: Design speeds for urban roads

Functional class	Maximum	Reduced [a]
UA	100	80
UC	70	50
UL	50	30

Notes:

a. *Reduced design speed should be considered in tandem with the operating speeds and spot speeds. It is recommended that spot speed measurements be undertaken to establish the 85th percentile speeds. The posted speed limit must not be higher than the selected design speed within any part of the road section.*

3.7 Sight Distances

3.7.1 General

Throughout the length of any road sight distances must be provided that are sufficient to enable drivers to absorb all relevant features of the road and the traffic conditions ahead and take the necessary actions to avoid hazards and proceed in a safe, efficient and orderly way. The following sight distance concepts are applicable to geometric design:

1) Stopping Sight Distance.
2) Meeting Sight Distance.
3) Passing Sight Distance.
4) Visibility Splays.

Stopping Sight Distance is applicable to all types of road. Meeting Sight Distance is applicable to two-way single carriageway roads with insufficient width for passing. Passing Sight Distance is applicable to two-way, 2-lane single carriageway roads. Visibility splays are required at junctions.

The minimum values for sight distances are generally determined by the design speed. However, on road sections where it has to be expected that actual vehicle speeds will be considerably above the design speed, sight distances should be determined by this expected speed rather than the design speed in order· to ensure safe operation of vehicles.

Determination of each of the site distances depends on the initial speed of the vehicle and the factors listed in Table 3-5.

Table 3-5: Parameters values used for calculating sight distances.

Characteristic	Value
Car driver's eye height	1.05 m
Truck drivers eye height	1.8 m
Height for Stopping Sight Distance of general object in the road	0.2 m
Height for Stopping Sight Distance for flat objects in the road (e.g., potholes, wash-out)	0.0 m
Height for Stopping Sight Distance for a vehicle in the road	0.6 m
Object height for Passing Sight Distance (e.g., roof of car)	1.3 m
Object height for Decision Sight Distance	0.0 m
Driver’s reaction time	2.5 s
The maximum deceleration rate for cars	3.0 m/s^2
The maximum deceleration rate for trucks	1.5 m/s^2
Friction between tyres and road surface	Table 3-6
The efficiency of the brakes of the vehicle	
Gradient of the road	*varies*

3.7.2 Friction between tyres and roadway

The roadway surface should provide a level of skid resistance that will accommodate the braking and steering manoeuvres that can reasonably be expected for the particular site. Lack of skid resistance is a contributory factor in many crashes and is often a trigger for resurfacing maintenance if the skid resistance is lower than a specified amount.

The coefficient of friction is the ratio of the frictional force on the vehicle and the component of the weight of the vehicle perpendicular to the frictional force. Longitudinal friction coefficients depend on:

1) Vehicle speed;
2) Type, condition, and texture of the roadway surface;
3) Weather conditions; and
4) Type and condition of tyres.

Although the value decreases as speed increases, there are considerable differences in research findings especially at lower speeds because of the wide range of conditions that are encountered, thus it is difficult to select representative values. For example, worn tires are common and climate varies from wet to arid with time of the year.

The longitudinal coefficients of friction, as determined by various authors, are shown in Table 3-6 using the lowest results of friction tests. The values allow a reasonable safety factor to cater for the wide range of conditions. Gravel roads, particularly, can have low friction characteristics. Hence pragmatic engineering judgement was required to select working values based on a systematic reduction in the values used for paved roads.

Side friction coefficients are also dependent on vehicle speed, type, condition and texture of roadway surface, weather conditions, and type and condition of tyres.

Table 3-6: Friction factors

Friction type	Road type	Design speed (km/h)									
		30	40	50	60	70	80	90	100	110	120
Longitudinal friction factors	Paved	0.40	0.37	0.35	0.33	0.32	0.305	0.295	0.285	0.29	0.28
	Unpaved	0.32	0.30	0.28	0.26	0.25	0.24	0.235	0.23	0.23	0.23
Side friction factors	Paved	0.21	0.19	0.17	0.16	0.14	0.13	0.12	0.10	0.10	0.095
	Unpaved	0.165	0.15	0.135	0.125	0.12	0.11	0.10	0.095	0.09	0.09

3.7.3 Stopping sight distance

Stopping Sight Distance is the distance required by a driver of a vehicle travelling at a given speed to bring the vehicle safely to a stop before reaching an object that becomes visible on the carriageway ahead. It includes the distance travelled during the perception and reaction times and the vehicle braking distance.

Stopping Sight Distance is the minimum sight distance requirement for all types of road and must be provided at every point along the road.

It is the sum of two distances:

1) The braking distance ($V^2/254(f+g)$)
2) Brake reaction distance (0.278 x prt x V), where prt is perception reaction time in seconds.

The stopping sight distance is therefore given by the following formula:

Equation 3-1: Stopping sight distance.

$$d = (\mathbf{0.278})(\mathbf{t})(\mathbf{V}) + \frac{\mathbf{V^2}}{(\mathbf{254(f+g/100)})}$$

Where:

d	=	*stopping distance (m)*
prt	=	*perception reaction time*
V	=	*initial speed (km/h)*
f	=	*longitudinal coefficient of friction between tyres and roadway.*
g	=	*gradient of road as a percentage (downhill is negative).*

Values for desirable and minimum Stopping Sight Distance for various design speeds and gradients are given in Table 3-7 and Table 3-8 respectively.

Table 3-7: Desirable stopping sight Distance

Design speed (km/h)	**Desirable stopping sight distance (m) For gradients (Up)**				**(Down)**		
	9%	**6%**	**3%**	**0**	**-3%**	**-6%**	**-9%**
40	40	40	45	45	45	45	50
50	55	50	60	60	60	65	65
60	-	75	50	80	80	85	90
70	-	35	100	100	105	110	120
80	-	120	125	130	135	145	155
90	-	145	155	165	170	185	200
100	-	-	185	200	215	235	255
110	-	-	230	250	270	295	-
120	-	-	285	310	340	-	-

Table 3-8: Minimum stopping sight distance

Design Speed (km/h)	**Minimum stopping sight distance (m) For gradients (Up)**				**(Down)**		
	9%	**6%**	**3%**	**0**	**-3%**	**-6%**	**-9%**
40	40	40	40	40	40	45	45
50	50	55	55	55	60	60	65
60	-	65	70	70	70	75	80
70	-	80	85	85	90	95	100
80	-	95	100	100	105	110	120

Design Speed (km/h)	Minimum stopping sight distance (m) For gradients (Up)					(Down)	
	9%	6%	3%	0	-3%	-6%	-9%
90	-	110	115	120	125	135	145
100	-	-	135	140	145	155	170
110	-	-	155	165	170	185	-
120	-	-	175	190	20O	-	-

3.7.4 Meeting sight distance

Meeting Sight Distance is the distance required to enable drivers of two vehicles travelling 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 as given in Table 3-7 and Table 3-8 plus a 10m safety distance.

Meeting Sight Distance is the minimum sight distance requirement for two-way, 1-lane single carriageway roads and should generally be provided for all roads with carriageway widths less than 5.0m. It is measured for an object height of 1.3 m (i.e., the height of an approaching passenger car) and an eye height of 1.05 m.

This distance is normally set at twice the stopping sight distance for a vehicle that is stopping to avoid a stationary object in the road. An extra safety margin of 20-30 m is sometimes provided. Although a vehicle is a much larger object than is usually considered when calculating stopping distances, these added safety margins are used partly because of the very severe consequences of a head-on collision and partly because it is difficult to judge the speed of an approaching vehicle, which could be considerably greater than the design speed.

It is particularly important to check this on existing roads that have a poor vertical alignment that may contain hidden dips that restrict sight distances. However, since single lane roads have a relatively low design speed, meeting sight distances should not be too difficult to achieve.

3.7.5 Intersection sight distance

Intersection Sight Distance is similar to stopping sight distance except that the object being viewed is another vehicle that may be entering the road from a side road or crossing the road at an intersection. On straight sections of road, many vehicles will exceed the road’s design speed but being straight, sight distances should be adequate for vehicles that are travelling straight through the junction on the major road. The situation is quite different for vehicles that may need to slow down or stop at the junction. This is because the time required to accelerate again and then to cross or turn at the junction is now much greater, hence longer sight distances are required.

Table 3-9: Intersection sight distances (m)

Design speed of major road (Km/h)	120	100	85	70	60	50
Distance (m)	295	215	160	120	90	70

3.7.6 Decision sight distance

Stopping sight distances are usually sufficient to allow reasonably competent drivers to stop under ordinary circumstances, but these distances are often inadequate when drivers need to make complex decisions or when unusual or unexpected manoeuvres are required. The driving task is constrained or limited by the human factors involved (Section 3.3).

Decision Sight Distance, sometimes termed 'anticipatory sight distance', is the distance required for a driver to:

1) Detect an unexpected or otherwise 'difficult-to-perceive' information source or hazard in a roadway environment that may be visually cluttered;
2) Recognize the hazard or its potential threat;
3) Select an appropriate speed and path; and
4) Complete the required manoeuvre safely and efficiently.

Critical locations where errors are likely to occur and where it is desirable to provide decision sight distance include:

1) Areas of concentrated demand where sources of information such as roadway elements, opposing traffic, traffic control devices, advertising signs and construction zones, compete for attention (i.e., visual noise);
2) Approaches to interchanges and intersections;
3) Railway crossings, bus stops, bicycle paths, entrances of villages and towns;
4) Newly upgraded road sections or a change of road hierarchy;
5) Changes in cross-section such as at toll plazas and lane drops;
6) Design speed reductions.

The minimum decision sight distances that should be provided for specific situations are shown in Table 3-10. If it is not feasible to provide these distances because of horizontal or vertical curvature, or if relocation is not possible, special attention should be given to the use of suitable traffic control devices for advance warning.

Although a sight distance is shown in the table for the right side (off-side) exit, exiting from the right side, except on LVRs, is undesirable because, to be safe, crossing a fast-moving traffic stream requires traffic control; the efficiency of the junction is thus severely reduced. Furthermore, a right-side exit is also in conflict with the expectancy of most drivers, and this compromises safety. The reason for providing this value is to allow for the possibility that an off-side (right side) exit might be necessary sometimes, usually with traffic control.

In measuring decision sight distances, the 1.05 m eye height and 0 mm object height have been adopted.

Table 3-10: Decision sight distances for various situations (m)

Design speed (km/h)	Situations				
	Interchanges		Lane drop, merge	Lane shift	Intersections
	Sight distance to nose (m)		Sight distance to taper area(m)	Sight distance to beginning of shift (m)	Sight distance to turn lane (m)
	Near (left)-side exit	Off (right)-side exit			
50	NA	NA	150	85	150
60	200	275	200	100	200
80	250	340	250	150	250
100	350	430	350	200	350
120	400	500	400	250	400

Source: SANRAL. Geometric Design Guidelines.

3.7.7 Passing sight distance

Passing Sight Distance is the minimum sight distance on two-way single carriageway roads that must be available to enable the driver of one vehicle to pass another vehicle safely without interfering with the speed of an oncoming vehicle travelling at the design speed, should it come into view after the overtaking manoeuvre is started. The manoeuvre is one of the most complex but important driving tasks. It is also relatively difficult to quantify for design purposes because of the various stages involved, the large number of relative speeds of vehicles that are possible, and the lengthy section of road needed to complete the manoeuvre. Figure 3-1 shows a schematic representation of normal and reduced passing sight distances.

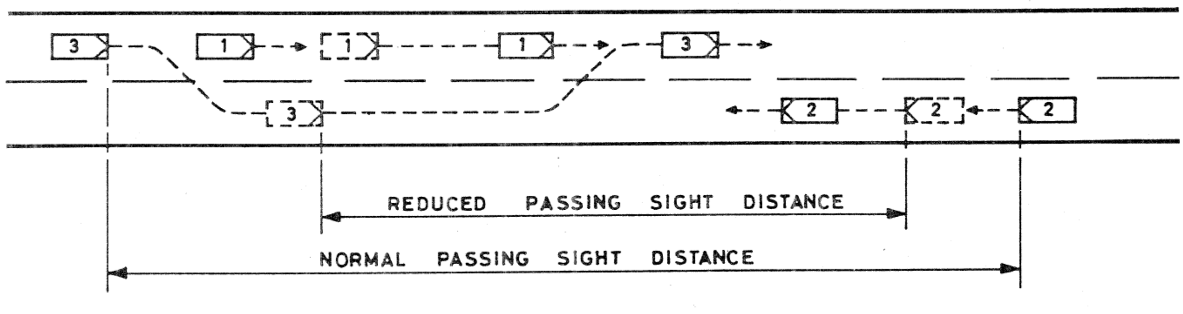


Figure 3-1: Provision of safe passing sight distance

Values for minimum Passing Sight Distance at various design speeds are given in Table 3-11. In general it is sufficient to provide the Reduced Passing Sight Distance as given in the table.

Table 3-12 gives guide values for the extent to which passing sight distance should be provided.

required by a vehicle to overtake or pass another vehicle safely on a two-lane single carriageway road is the distance which will enable the overtaking driver to pass a slower vehicle without causing an oncoming vehicle to slow below the normal speed.

A driver finding that he has insufficient distance after initiating the passing manoeuvre can choose to abort the manoeuvre. The Minimum Passing Sight Distance is then the sight distance required on a two-lane road to enable the passing manoeuvre to be aborted.

Table 3-11: Passing sight distances (m)

Design speed (Km/hr)	Normal passing sight distance (m)	Reduced passing sight distance (m)
50	250	175
60	325	225
70	400	275
80	475	325
90	525	350
100	575	375
110	625	400
120	700	450
140	775	500

Table 3-12: Guide values for provision of minimum passing sight distances (m)

Design speed (Km/hr)	Minimum proportion of road with reduced passing sight distance		
	ADT < 1000pcu	ADT = 1000-3000pcu	ADT > 3000pcu
50	1/5	1/5	1/5
60	1/5	1/4	1/4
70-80	1/5	1/4	1/3
90-120	1/5	1/3	1/2
120-140	1/3	1/2	2/3

3.7.8 Headlight sight distance

Headlight Sight Distance is used to design the rate of change of gradient for sag vertical curves. Where the only source of illumination is the headlamps of the vehicle, the illuminated area depends on the height of the headlights above the road and the divergence angle of the headlight beam relative to the grade line of the road at the position of the vehicle on the curve. For cars, a headlight height of 0.6 m and a beam divergence of 1 degree is usually used for calculation purposes. At speeds above 80 km/h, only large, light-coloured objects can be perceived at the generally accepted stopping sight distances.

3.7.9 Control of sight distance

Available sight distances should be checked throughout the road length in the early stages of the design of the alignment, and any necessary adjustments to the line should be made to meet the minimum requirements for sight distance. Details of crest and sag curve design are provided in Chapter 6.

The following guidelines control of sight distances apply:

1) Available sight distance should be checked separately for each type of sight distance and for each direction of travel.
2) The following values should be used for determination of sight lines:
    a) Drivers eye height 1.10m
    b) Object height for Stopping Sight Distance - 0.1m
    c) Object height for Meeting and Passing Sight Distance – 1.10m
3) In horizontal curves it may be necessary to remove obstructions or widen cuttings on the insides of the curves to obtain required sight distance. Required sight areas for various radii and sight distances are given in Figure 3-3 and Figure 3-3. Within the sight area the terrain should be the same level as the carriageway, and other obstructions should be removed. In cases where the provision of the sight area requires extensive earth- works or costly removal of obstructions, it may be necessary to adjust the alignment. The distance labelled M in the diagram must be clear of obstruction to allow a clear view along the sight line shown.
4) Sudden reductions of available sight distance should be avoided. Where reductions are necessary, they should be logical in relation to the physical surroundings.

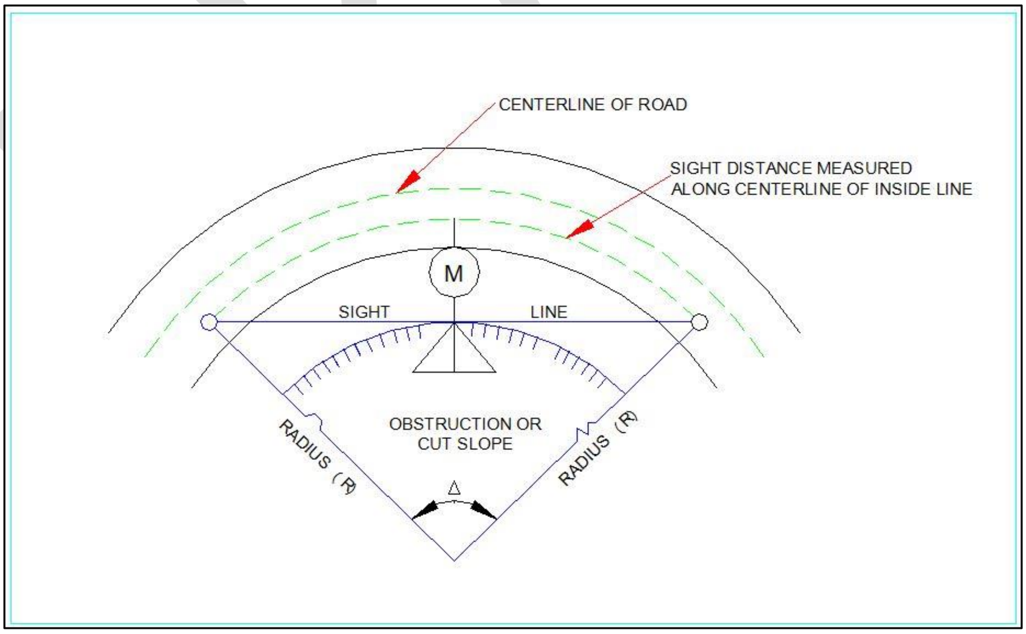


Figure 3-2: Sight Distance for horizontal curves

Relevant formulae for determining the length of the middle ordinate M are as follows:

Length of Sight Line (S) = $2R\sin\left(\frac{\Delta}{2}\right)$

Length of Middle Ordinate (M)= = $R(1 - \cos(\frac{\Delta}{2})$

where Δ = Deflection angle (°)

Example:

Radius = 1000 m, $\Delta = 20^0$;

S = 2R sin(Δ/2)	M = R (1 – cos(Δ/2)
= 2(1000) (sin(10°)	= 1000(1- cos(10°)
= 347 m	= 15.2 m

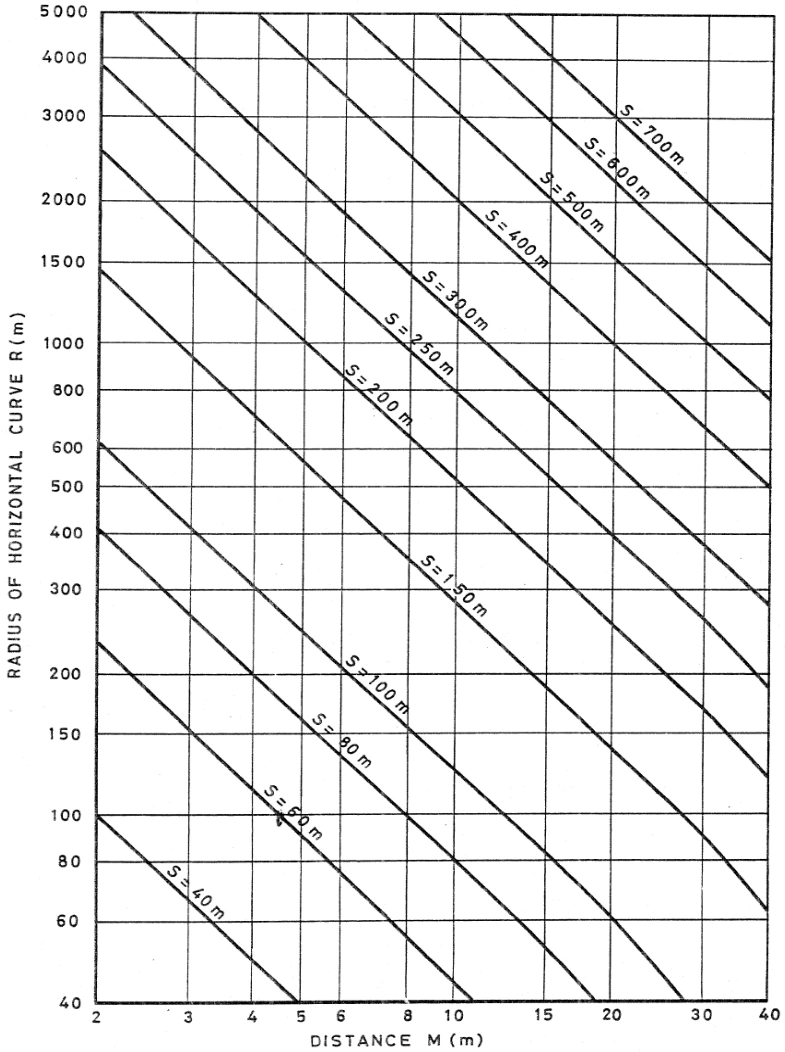


Figure 3-3: Sight distances on horizontal curves

3.8 Traffic Volume

3.8.1 General

The design of a roadway and its features should explicitly consider traffic volumes, operational performance, and user characteristics for all transportation modes. All information should be considered jointly. Financing, quality of foundations, availability of materials, cost of right-of-way, and other factors all have important bearing on the design; however, the traffic volume and modal mix can indicate the need for the improvement and directly influence the selection of geometric design features, such as number of lanes, widths, alignments, and grades.

Traffic data for a road or section of road are generally available or can be obtained from field studies in accordance with the **Road Design Manual Volume 1 Part 2: Traffic Surveys**. The data collected include traffic volumes for days of the year and time of the day, as well as the distribution of vehicles by type and weight. The data also include information on trends that the designer may use to estimate the traffic to be expected in the future.

The traffic volume on a specific road section can vary significantly between peak hours, daily traffic and seasonal flows. The extent of the variation depends on factors such as the road function, the traffic conditions, location and environment. Localized information on seasonal variations should be obtained by collection of appropriate traffic data and also by local knowledge where no traffic studies, statistics or comparative counts are available.

The general measures of vehicular traffic on a road are:

1) Average Annual Daily Traffic (AADT) – The total traffic volume for the year divided by 365.
2) Average Daily Traffic (ADT) – The total traffic volume during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period.
3) Hourly Traffic -Traffic volume that can pass over a given section of a lane/carriageway in one direction (or in both directions for a two-lane highway) during a given hour.

3.8.2 Design volume - low traffic volume roads

The geometric design control of a roadway is based on the **Design Volume,** which is the estimated traffic volume at a certain future year, the “Design Year”, usually 10 (ten) – 20 (twenty) years after the year of opening of the new road.

The most adequate design control for low volume roads is AADT in year 10 after opening, estimated from historical AADT data and the envisaged social economic development pattern. For roads with large seasonal variation but still moderate traffic volumes, it may be sufficient to determine the design volume in year 10 after opening as ADT during the peak months of the year.

3.8.3 Design volume - high traffic volume roads

On major roads carrying relatively heavy traffic volumes throughout the year (current AADT > 1000) **hourly traffic** has to be used for the determination of the Design Volume. However, it would obviously be wasteful to design the road for the maximum peak hour traffic in the design year, since this traffic volume would occur only during one or a very few hours of the year. As a general rule, heavily trafficked roads should be designed to accommodate the 30th to 50th highest hourly volume in year ten after opening (DHV = Design Hourly Volume), depending on economic considerations.

In urban areas, an appropriate DHV may be determined from the study of traffic during the normal daily peak periods. Because of the recurring morning and afternoon peak traffic flow, there is usually little difference between the 30th and the 200th highest hourly volume. For typical urban area conditions, the highest hourly volume is found during the afternoon workto-home travel peak. One approach for determining a suitable DHV is to select the highest afternoon peak traffic flow for each week and then average these values for the 52 weeks of the year.

If the morning peak-hour volumes for each week of the year are all less than the afternoon peak volumes, the average of the 52 weekly afternoon peak-hour volumes would have about the same value as the 26th highest hourly volume of the year. If the morning peaks are equal to the afternoon peaks, the average of the afternoon peaks would be about equal to the 50th highest hourly volume. The volumes represented by the 26th and 50th highest hours of the year are not sufficiently different from the 30 HV value to affect design.

Therefore, in urban area design, the 30th highest hourly volume can be a reasonable representation of daily peak hours during the year. Flexibility may be appropriate in those areas or locations where recreational or other travel is concentrated during particular seasons. In such locations, a distribution of traffic volume where the hourly volumes are much greater than the 30 HV may result; the 30 HV in such cases may be inappropriate as the DHV and a higher value should be considered in design. Specific measurements of traffic volumes should be made and evaluated to determine the appropriate DHV. In the usual case, future travel demand is determined from the urban area transportation planning process in terms of total daily trips that are assigned to the transportation system. Consideration of the split between public and private transportation is also incorporated into this process. These assigned trips constitute the traffic volumes on links of the future street and road network. In some instances, these volumes (ADT) are provided directly to roadway designers. In others, they are converted by the operational transportation study staff to directional volumes for the design hour. From a practical standpoint, the latter approach may be the more desirable because the transportation study staff is often in a better position to evaluate the effects that the assumptions inherent in the planning process have on the resulting design volumes. Two-way DHVs (i.e., the 30 HV, or its equivalent) may be determined by applying a representative percentage (usually 8 to 12 percent in urban areas) to the ADT. In many cases this percentage, based on data obtained in a traffic count program, is developed and applied system-wide; in other cases, factors may be developed for different facility classes or different areas of an urban area, or both.

3.8.4 Conversion of AADT to DHV

In design, peak-hour volumes are sometimes estimated from projections of the AADT. Traffic forecasts are most often cast in terms of AADTs based on documented trends and/or forecasting models, because daily volume, projections can be more confidently made using them. AADTs are converted to a peak-hour volume in the peak direction of flow. This is referred to as the “directional design hour volume” (DDHV), and is found using the following relationship:

$$DDHV = AADT * K * D \qquad (3\text{-}1)$$

Where: K = proportion of daily traffic occurring during the peak hour

D = proportion of peak hour traffic traveling in the peak direction of flow

On two-lane two-way roads, the Design Hourly Volume (DHV) is usually the total volume for both directions. For roads with more than two lanes or where a two-lane road is to be widened at a later date, the volume in each direction must be known. In peak hours on multi-lane roads, the volume in the peak direction can vary from 55 to 70 percent of the total flow depending on the origins and destinations of the traffic. The directional split may be greater on a highly recreational route. The design must therefore consider the proportion of traffic in one direction to ensure an adequate design is undertaken.

For design, the K factor often presents the proportion of AADT occurring during the 30th peak hour flow. This means if the 365 peak hour volumes of the year at a given location are listed in descending order, the 30th peak hour is the 30th on the list and represents a volume that exceeded 29 hours of the year. The design hour should be one that is “not exceeded very often or by much” (AASHTO, 2001).

For rural roads, the 30th peak hour may have a significantly lower volume than the worst hour of the year, as critical peaks may occur only in frequently. In such cases it is not economically feasible to invest large amounts of capital in providing additional capacity or higher road class that will be used in only 29 hours of the year.

In urban cases, where traffic is frequently at capacity levels during the daily commuter peaks, the 30th peak hour is often not substantially different from the highest peak hour of the year.

Factors K and D are based upon local or regional characteristics at existing locations. A general range of these factors is given in Table 3-13. The values are illustrative, and specific data on these characteristics should be available for the project road from the local highway authorities.

Table 3-13: General ranges for K and D factors

Facility Type	**Normal Range of Values**	
	K-Factor	**D-Factor**
Rural	0.15 – 0.25	0.65 – 0.80
Semi-urban	0.12 – 0.15	0.55 – 0.65
Urban	0.07 – 0.12	0.50 – 0.60

(Source: Traffic Engineering, by Roess, Prassas & MacShane)

The most adequate design control for low-volume roads is AADT in year 10 after opening, estimated from historical AADT data and the envisaged socio-economic development pattern. For routes with large seasonal variations but still moderate traffic volumes, it may be sufficient to determine the Design Volume in year 10 after opening as ADT during the peak months of the year.

3.8.5 Projection of future traffic

For most design purposes, an estimate of the traffic in the design year is required. The nominated design year is used to define the design life of a traffic system. Design parameters are based on traffic forecasts for the design year. Although this is the case, even with a developed economy and stable economic conditions, traffic forecasting is an uncertain process. When forecasting traffic growth, it is usual to separate the traffic into the following three categories:

- **Normal traffic:** Traffic which would travel along the same road even if no improvement were provided.
- **Diverted traffic**: Traffic that changes from another route (or mode of transport) to the project road because of the improved facility, but still travels between the same origin and destination.
- **Generated traffic**: Additional traffic that occurs in response to the provision or improvement of the road.

3.9 Road Capacity and Level of Service

3.9.1 General

The term capacity is used to express the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or a carriageway during a given period of time under prevailing carriageway and traffic conditions.

Highway capacity information serves three general purposes:

1) It is used in transportation planning studies to assess the adequacy or sufficiency of the road network to service existing traffic and to estimate the time when traffic growth may overtake the capacity of the network or result in unacceptable congestion.
2) Highway capacity information is of vital importance in the design of roads and road networks. Knowledge of highway capacity is essential to the proper fitting of a planned road to the requirements of estimated traffic, both in the selection of the road type and the dimensions of lanes and weaving sections.
3) Capacity information is utilized in the analysis of traffic operations (either existing or planned) for many purposes, but particularly for the identification of bottlenecks and assessing the benefits to be accrued from spot improvements to the geometry of the road.

3.9.2 Passenger Car Units

Vehicles of different types require different amounts of road space because of variations in size and performance. In order to allow for this in capacity measurements for roads and junctions, traffic volumes are expressed in terms of passenger car units (pcu's). The basic unit is the car (taxis, mini vans, vans, pickups and three-wheeler vehicles also count as one unit).

The vehicles definitions applying to the different vehicle types are given in Table 3-14.

Table 3-14: Passenger car units

Vehicle category	Vehicle class	Code	Vehicle description	Class by axle configuration
Passenger Vehicles	Pedal Cycle	PC	Non-motorised bicycle or tricycle.	
	Motorcycle	MC	Self-propelled vehicle with less than 3 wheels.	
	A motorized rickshaw (Tuk-tuk)	MR	Self-propelled vehicle with three wheels	
	Cars, Jeeps, SUV, Pick-up	C	Passenger motor vehicle with seating capacity of not more than nine persons including the driver.	2-Axle Rigid
	Microbus	MCB	Two axle rigid chassis passenger motor vehicle with seating capacity of 10 to 14 persons including the driver.	2-Axle Rigid
	Minibus	MB	Two axle rigid chassis passenger motor vehicle with seating capacity of 15 to 25 persons including the driver.	2-Axle Rigid
	Bus	B	Two axle rigid chassis passenger motor vehicle with seating capacity of 26 to 53 persons including the driver.	2-Axle Rigid
	Omnibus	OB	Three or four axle passenger motor vehicles with seating capacity of more than 53 persons including driver	3 or 4-Axle rigid or articulated
Goods Vehicles	Light Goods Vehicle	LGV	Two axle rigid chassis goods vehicle of gross vehicle weight not exceeding 3,500 kg.	2-Axle Rigid
	Medium Goods Vehicles	MGV	Two axle rigid chassis goods vehicle or tractor with gross vehicle weight of 3,500 kg to 8,500 kg	2-Axle Rigid
	Heavy Goods Vehicle	HGV	3 or 4 axle rigid chassis goods vehicle or tractor with gross vehicle weight greater than 8,500 kg.	3 or 4-axle rigid
	Articulated Heavy Goods Vehicle	AHGV	Articulated goods vehicle having 3 or more axles of gross vehicle weight exceeding 8,500 kg.	3 or more-Axle articulated

As different types of vehicles affect the capacity of rural roads, urban roads, roundabouts and traffic signals in varying degrees, the weighting for each class of vehicle has to be varied to suit the purpose for which it is to be used. For example, a heavy goods vehicle on a rural road is rated as equivalent to 3 cars, but on an urban road to only 2, and at traffic signals to 1.75. The appropriate values for different types of vehicles under different conditions are shown in Table 3-15.

Table 3-15: Passenger car units

Vehicle class	Rural standard	Urban standard	Roundabout design	Traffic signal design
Pedal Cycle (PC)	0.3	0.3	0.3	0.2
Animal/ hand cart	0.7	0.7	0.7	0.7
Motorcycle (MC)	0.5	0.5	0.5	0.33
Three wheelers and Tuk-tuk	0.7	0.7	0.7	0.5
Cars, Jeeps, SUV, Pick-up (C)	1.0	1.0	1.0	1.0
Microbus (MCB)	1.5	1.5	1.5	1.2
Minibus (MB)	2.0	2.0	2.0	2.0
Bus (B)	2.5	2.5	2.3	2.0
Omnibus (OB)	3.0	3.0	2.8	2.25
Light Goods Vehicle (LGV)	1.5	1.5	1.5	1.2
Medium Goods Vehicle (MGV)	2.5	2.5	2.8	1.75
Heavy Goods Vehicle (HGV)	3.0	3.0	2.8	2.25
Articulated Heavy Goods Vehicle (AHGV)	3.5	3.5	3.5	3.0

The values quoted are guide values only. The values depend on many other variables such as road gradient, traffic speed, traffic mix, and degree of congestion. This topic is addressed comprehensively in the Highway Capacity Manual (2010).

These values differ in rolling and mountainous terrain as shown in Table 3-16Table 3-16 but precise values cannot be calculated so a degree of interpolation and engineering judgement is required.

Table 3-16: Variation in PCU values

	Terrain		
Vehicle class	Flat	Rolling	Mountainous/ escarpment
Pedal Cycle (PC)	0.3	0.3	NA
Animal/ hand cart	0.7	1.5	2.0

Vehicle class	Terrain		
	Flat	Rolling	Mountainous/ escarpment
Motorcycle (MC)	0.5	0.5	0.7
Three wheelers and Tuk-tuk	0.7	0.7	1.0
Cars, Jeeps, SUV, Pick-up (C)	1.0	1.0	1.5
Microbus (MCB)	1.5	1.5	2.0
Minibus (MB)	2.0	2.0	3.0
Bus (B)	2.5	3.5	6.0
Omnibus (OB)	3.0	4.5	8.0
Light Goods Vehicle (LGV)	1.5	1.5	3.0
Medium Goods Vehicle (MGV)	2.5	5.0	10
Heavy Goods Vehicle (HGV)	3.0	6.0	12
Articulated Heavy Goods Vehicle (AHGV)	3.5	8.0	20

3.9.3 Traffic for design

The design of main traffic routes in built-up areas should be based on peak-hour demands and not, as in rural areas, on the average daily traffic. Due allowance should be made, especially in intersection design, for tidal flows during the morning and evening peaks and for any other peaks during the day as, for example, at lunch time.

Typical capacities for two-way rural roads are shown in Table 3-17, for two-way urban roads in Table 3-18 and for one-way urban roads in Table 3-19.

Table 3-17: Practical capacities of 2-lane single carriageway interurban and rural roads

Basic capacity (pcu/h	Operating speed (km/h)	Design speed (km/h)	Proportion of road with passing sight distance > than min. PSD	Design capacity (pcu/h)
2000	95	110-120	100% 80% 60%	400 360 300
2000	80	90-100	100% 80% 60% 40%	800 700 600 480

Basic capacity (pcu/h	Operating speed (km/h)	Design speed (km/h)	Proportion of road with passing sight distance > than min. PSD	Design capacity (pcu/h)
2000	65	80	100% 80% 60% 40% 20%	1120 1060 940 760 560
2000	55	70	100% 80% 60% 40% 20%	1340 1240 1140 1040 880

Table 3-18: Practical capacities of two-way urban roads

Effective width of carriageway	2-lane		3-Lane	4-lane	6-lane
	6- 7.0 m	7.3m	10-11m	14-14.5m	20m
Description	**Capacity in pcu's per hour for both directions of flow**			**Capacity in pcu's per hour for one directions of flow**	
All-purpose roads with no frontage access, no parked vehicles and negligible cross traffic	1350	1500	2200	2200	3300
All-purpose streets with high-capacity junctions and 'no waiting' restrictions	1000	1200	1800	1350	2250
All-purpose streets with capacity restricted by parked vehicles and junctions	450-600	600-750	1100-1300	900-1000	1500-2000

Table 3-19: Practical capacities for one-way urban roads

Effective width of carriageway excluding refuges, or central; reservation	**2-lane**		**3-lane**	**4-lane**	**Comments**
	5.5m	**7.0m 7.3m**	**10m 11m**	**14m 14.5m**	
Description	**Capacity in pcus per hour**				
Urban motorways with grade separation and no frontage access.		3000	4500	6000	Applicable to the highest category of distributor
All-purpose roads with no frontage access, no standing vehicles and negligible cross traffic.	1650	2200 2400	3300	4400	Applicable to all-purpose distributors
All-purpose streets with high-capacity junctions and ‘no waiting’ allowed.	1100	1450 1600	2400	3350	Applicable to distributors and access roads but capacity restricted by junction
All-purpose streets with capacity restricted by parked vehicles.	720	950 1100	1800	2800	Applicable to roads with waiting vehicles and with heavy cross traffic limiting capacity

3.9.4 Level of Service (LOS)

The quality of service provided by a specific road section under specific conditions is described as the Level of Service (LoS). The Level of Service describes the ability of the driver to drive at a speed of his choice, to overtake or change lanes. It thus provides an indication of travel times, traffic interruptions, and comfort. The LoS therefore depends on characteristics that cannot be easily measured, and which can be very subjective compared with many other road characteristics, nevertheless it is a useful parameter. Table 3-20 summarises the operating conditions applying to each LoS.

Conditions are defined as either ‘uninterrupted’ flow conditions or ‘interrupted’ flow conditions. Uninterrupted flow facilities are provided for high mobility corridors that have minimal disruption to the traffic stream from elements external to the traffic stream such as accesses and intersections.

Interrupted flow facilities provide a high degree of controlled and uncontrolled access to the road through the provision of traffic signals, stop signs, yield signs and other controls that disrupt or significantly vary the speed of travel on any given section of road irrespective of volume of traffic.

Six levels of service are defined varying from level A which is the free flow condition characterised by low traffic volume where drivers can maintain a high speed (if desired) to level E where the traffic is approaching saturation with drivers travelling at low speeds due to the high volume of traffic or congestion. The traffic volume at level of service E is defined as the capacity of the facility. Level of service F is the forced flow condition where the traffic density is maximum with drivers subjected to frequent stops and queues.

To determine the LoS of a section of road, a set of standard conditions are defined which are termed 'base conditions' at which the free flow speed is known (100 km/h in flat terrain, at least 7.3 m carriageway, no obstructions within 1.8 m from the edge of carriageway, and no passing sight distance restrictions). Under base conditions, the maximum service volume which a two–lane road can carry, are shown in Table 3-18.

Table 3-20: Level of service for base conditions and uninterrupted flow

Level of service	Two-lane rural road without access control	Multi-lane rural road without access control
A	Average travel speed of ≥ 90km/h. Most passing manoeuvres can be made with little or no delay. Service flow rate is a total of 490 PCU/h for both directions and about 15% of capacity can be achieved. Maximum AADT is 2,800[(1)].	Average travel speed ≥ 95 km/h. Under ideal conditions, flow rate is limited to 720 PCU/lane/h or 33% of capacity.
B	Average travel speed of ≥ 80km/h. Flow rates may reach 27% of capacity with continuous passing sight distance. Flow rate of 780 PCU/h total for both directions. Maximum AADT is about 5,200[(1)].	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/h average travel speed under ideal condition.
C	Flow still stable. Average travel speed of ≥ 70km/h. Flow rates under ideal condition equal to 43% of capacity with continuous passing sight distance or 1,190 PCU/h total for both directions. Maximum AADT is about 7,900[(1)].	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/h average travel speed.

Level of service	Two-lane rural road without access control	Multi-lane rural road without access control
D	Approaching unstable flow. Average travel speed of ≥ 60km/h. Flow rates, two directions, at 64% of capacity with continuous passing opportunity, or a total of 1,830 PCU/h for both directions. Maximum AADT is about 12,000(1).	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 (capacity)	Average travel speeds in neighbourhood of 60 km/h. Flow rates under ideal conditions, total two way, equal to 3200-2800 PCU/h. Maximum AADT is about 18,000(1) 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/h under ideal conditions. Average travel speeds about 88 km/h.
F	Forced congested flow with unpredictable characteristics. Operating speeds less than 72 km/h.	Forced flow congested condition with widely varying volume characteristics. Average travel speed of less than 50 km/h.

Note 1 Based on peak hour flow of 15% of AADT.
Source: Highway Capacity Manual Chapter 12

The speed is then adjusted according to the conditions that differ from the base conditions for the road being evaluated. These are characterised as roadway, traffic, or control conditions.

Roadway conditions include the number of lanes, lane width, no passing lengths, and design speed; the latter controlling the vertical and horizontal alignment. Traffic conditions include vehicle types and directional distribution.

For interrupted flow, traffic control conditions are important and analysis quite complex because of the range of conditions that can affect capacity. However, for uninterrupted flow conditions the analysis of service level is relatively straightforward.

Vehicle capacity is the maximum number of vehicles that can pass a given point during a specified period under the prevailing roadway, traffic, and control conditions.

For a two-lane, two-way road in flat terrain, capacity is reached when the traffic level (sum of both directions) approaches 2,800 PCU per hour (Harwood et al.; 1999). This peak traffic (per hour) is usually between 12 and 18% of the AADT and a value of 15% is a reasonable average. The capacity also depends on directional split as shown in Table 3-21.

Table 3-21: Dependence of capacity on directional Flow

Directional split	Total capacity (PCU/h)
50:50	2800
60:40	2630
70:30	2490
80:20	2320
90:10	2100

Capacity is reduced if the physical features of the road are deficient in some way:

1) Lane widths of 3.65 m are the minimum necessary for heavy volumes of mixed traffic, i.e., before capacity of the lane is reduced.
2) Narrow shoulders cause vehicles to travel closer to the centre of the carriageway, and vehicles making emergency stops must park on the carriageway. This causes a substantial reduction in the effective width of the road, thereby reducing capacity.
3) Side obstructions such as poles, bridge abutments, retaining walls or parked cars that are located within about 1.5 m of the edge of the carriageway contribute towards a reduction in the effective width of the outside traffic lane.
4) Imperfect horizontal or vertical curvature.
5) Long and/or steep hills and sharp bends result in restricted sight distances. Drivers have reduced opportunities to pass and so the capacity of the road is reduced.

The capacities of some rural roads and the great majority of urban roads are controlled by the layouts of intersections. Table 3-22 provides guidance for a realistic example. Based on different types of terrain and with the following typical conditions or assumptions, the Table illustrates that for more severe terrain and for the lower traffic classes, the expected speeds, traffic flows and resulting LoS are controlled by geometric design factors and not by capacity or traffic flow:

1) Traffic mix is 14% trucks;
2) Directional split is 60/40;
3) No-passing zones,
   - i) Level terrain 20%
   - ii) Rolling terrain 40%
   - iii) Mountainous terrain 60%
   - iv) Ratio of Highest Hourly Volume to AADT = 0.15

Care is required if a particular level of service must be achieved because the traffic levels are such that the capacity of the road for a particular service level is likely to be exceeded before the end of the design period.

Table 3-22: Traffic flow for two-lane rural roads

Level of Service	Maximum (PCU/h/l) and AADT					
	Flat Terrain		Rolling Terrain		Mountainous/ Escarpment Terrain	
	PCU/h	AADT	PCU/h	AADT	PCU/h	AADT
A	240	1,600	110	700	50	300
B	480	3,200	280	1,800	130	900
C	790	5,300	520	3,500	240	1,600
D	1,350	9,000	800	5,300	370	2,500
E	2,290	15,200	1,480	9,900	810	5,400

Source: Harwood et al. Capacity and Quality of Service of Two-lane Highways. NCHRP Project 3-55(3), Geometric Design Guidelines (2003), South African National Roads Agency Limited

The Figures in Table 3-20 show typical values for the terrain based on assumed proportion of the road with passing sight distances that meet minimum requirements which depends on road width and operating speed hence the values shown are for guidance only.

Constructing extra capacity in the future by adding an additional lane is often difficult and very costly. A whole life cost analysis might prove useful to justify the costs but, in general, if capacity is expected to be exceeded towards the end of the design period it is usually better to design for it at the beginning. Thus, where computations indicate that a two-lane road is not adequate for existing or projected demands, various multi-lane options must be considered and analysed.

When the volume of traffic is high, the road space occupied by different types of vehicles is an important element in designing for capacity, namely the highest traffic flow per hour that the road can carry. As traffic increases, traffic interaction increases until the traffic level exceeds the capacity of the road.

The values quoted are guide values only. The values depend on many other variables such as road gradient, traffic speed, traffic mix, and degree of congestion. This topic is addressed comprehensively in the Highway Capacity Manual, 6th Edition (2016).

3.9.5 Car equivalent factors (CEF) for NMTs

Various non-motorised traffic categories are combined to determine a total car equivalency factor (CEF) to identify when additional safety features for NMT need to be included in the design. The individual CEF values are shown in Table 3-23. These should not be confused with Passenger Car Units (PCU) that are used for capacity and congestion estimates for heavily trafficked roads.

Table 3-23: Car equivalency factor

Vehicle	CEF value
Pedestrian	0.15

Bicycle	0.2
Bicycle with trailer	0.35
Small animal-drawn cart	0.7
All based on a passenger car = 1.0	

3.10 Other Road Users

3.10.1 Pedestrians

In general, pedestrian safety is enhanced by the provision of median refuge islands of sufficient width at road junctions, and separate pedestrian footpaths (sidewalks) where pedestrian traffic warrants it, for example, on approaches to villages. Indeed, the design of rural roads and highways through urban centres should encompasses a range of safety features including traffic calming and well-designed village centres permitting trading in relative safety, as described in Chapter 12. In metropolitan areas pedestrian facilities need to be incorporated as a traffic stream in all facilities.

The age of pedestrians is an important factor that may explain behaviour that leads to collisions. It is recommended that older pedestrians are accommodated by using simple designs that minimize crossing widths and assume lower walking speeds. Pedestrian safety is further enhanced by the provision of:

1) Lighting at locations that demand multiple information gathering and processing;
2) Safe crossing locations with provision of speed management measures;
3) Pedestrian overpasses across fully access controlled roads;
4) Well-lit pedestrian underpasses.

3.10.2 Pedal cyclists

Improvements of the following can considerably enhance the safety of a street or highway and provide for bicycle traffic:

1) Paved shoulders;
2) Bicycle-safe drainage grates;
3) Adjusting manhole covers (if present) to the grade; and,
4) Maintaining a smooth and clean riding surface

3.10.3 Motorcyclists

Motorcyclists, like pedestrians, are equally vulnerable in the case of road casualty. The poor protective nature of motorcyclists warrants serious considerations be given to the safety of the motorcycle rider/pillion. The risk of crash must be kept at the lowest level as motorcyclists are very susceptible to serious injury or death. Hence there must be some limit to the maximum speed of travel along the motorcycle lane. At an impact of 60 kph, a motorcyclist, has little chance of surviving a crash.

The operation of the motorcycle depends on adequate continuous traction and on consistency in the available friction between the tyre and road surface. The inherent instability of the vehicle (two-wheeler) can lead to loss of control at points of sudden change, particularly where surface irregularities occur, with hazardous consequences for the rider. Motorcyclist’ risk is heightened at bends, intersections and at roundabouts/ traffic circles.

These consequences can be serious if solid objects are in the path of the motorcycle because it can easily slide out of control. Care is therefore required when providing roadside and road surface furniture, which may be a positive safety feature for other road users.

The following should be provided to benefit motorcyclists:

1) Uniformity of road design.
2) Familiar and standard treatment with no surprises.
3) Use of optimum values of design parameters rather than minimum.
4) A high standard of workmanship and maintenance practices.
5) Advance information, warning, and good delineation of the road.
6) Sufficient clear zone; as the motorcycle envelope can extend beyond the edge of the traffic lane such as leaning into a curve; hence road furniture must be placed outside the potential line of this encroachment.
7) Consistency of road surface condition; since the motorcyclist must lean into the curve, or into the wind on a straight, any change in traction with the road due to a change in surface conditions should be avoided.
8) Motorcycle friendly safety barriers at locations of high risk to motorcyclists.

3.11 Design Vehicle

The design vehicle is the vehicle whose physical characteristics and proportions are used in setting geometric design standards. Adequate geometric design must cater for the characteristics of the vehicles that are to use the road and the largest vehicle must be able to pass a similar vehicle safely and to negotiate all aspects of the horizontal and vertical alignment. The vehicle characteristics and dimensions affecting design are:

1) Wheelbase
2) Vehicle height and width
3) Minimum turning radius,
4) Travel path during a turn, and
5) Power-to-weight ratio,

The principal road elements that are affected are:

1) Maximum gradient,
2) Lane width,
3) Widening of horizontal curves and ,
4) Junction design.

In general, buses and heavy vehicles should be used as the design vehicle for cross section elements, with the car as the design vehicle for the horizontal and vertical alignment. For most major intersections along arterial roads or within commercial areas, it is common practice to accommodate rigid trucks and vehicle combinations. The occasional larger vehicle may encroach on adjacent lanes while turning but not on the footpath.

The vehicle fleet changes slowly over time, but it is not possible to alter the geometric design of the current network at the same rate hence some element of prediction is required. The present vehicle fleet includes a high number of 4-wheel drive utility vehicles. Fortunately, such vehicles do not pose any enhanced requirements for geometric design. On the other hand, larger trucks could do so and could be banned from using some existing roads. Future roads should be designed to cater for changes in the truck fleet.

In view of the requirements of international travel and the available port facilities in the region, recommendation for harmonization of the design vehicles for EAC Partner States has been adopted for this Manual. The design vehicles indicated in Table 3-24 should be used in the control of geometric design until a major change in the vehicle fleet is observed.

In view of the low density of roads (and, hence, lack of alternative routes) together with the limited choice of vehicle for many transporters, it is prudent to be conservative in choosing the design vehicle for each class of road so that the maximum number of vehicle types can use them. Table 3-24 shows typical design vehicles that are often used, but, for high volume roads in Kenya, the design vehicle should be a truck and trailer except for very severe escarpment terrain.

Table 3-24: Design vehicle characteristics

Design vehicle	Code	Height (m)	Width (m)	Length (m)	Front overhang (m)	Rear overhang (m)	Wheelbase (m)	Minimum turning radius (m)
4x4 utility	DV1	1.3	2.1	5.8	0.9	1.5	3.4	7.3
Single unit truck	DV2	4.3	2.6	11.0	1.5	3.0	6.5	12.8
Single unit bus	DV3	4.3	2.6	12.1	2.1	2.6	7.6	12.8
Truck + semi-trailer	DV4	4.3	2.6	15.2	1.2	1.8	4.8+8.4 = 13.2	13.7
Typical 5- axle truck	DV5	4.6	2.6	18.5	1.2	1.8	5.5+10.0 = 15.5	13.7
Rigid truck and drawbar trailer combination	DV6	4.6	2.6	22	1.2	1.8	6.1+12.9 = 19	12.0
Interlink (with a short truck tractor)	DV7	4.6	2.6	22	1.2	1.8	4.1+6.52+7.94 = 18.9	12.0
Semi-trailer with long trailer	DV8	4.6	2.6	22	1.2	1.8	6.1+12.8 = 18.9	13.7

Figures below illustrated the turning path of the various design vehicles:

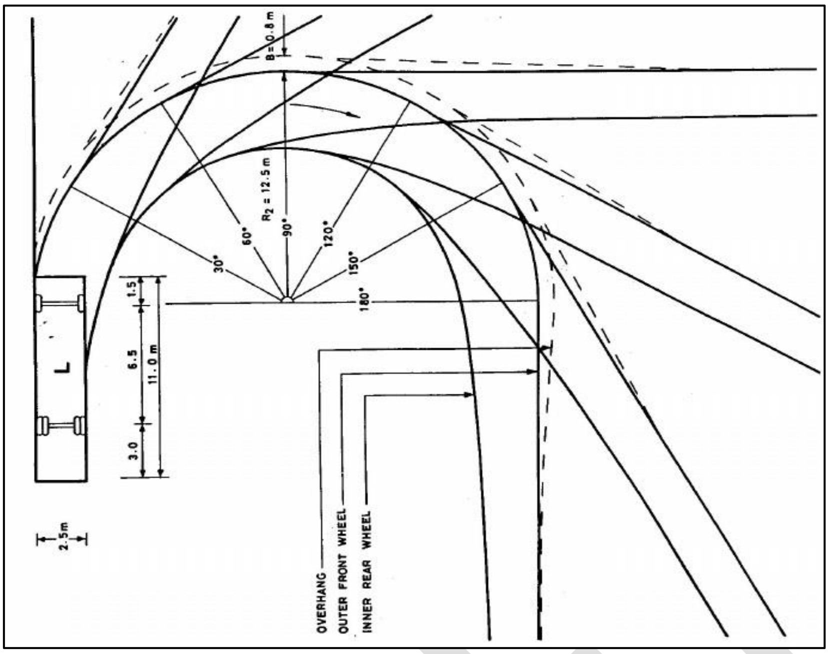


Figure 3-4: Dimensions and Turning Radius Path for Single Unit Truck (DV2)

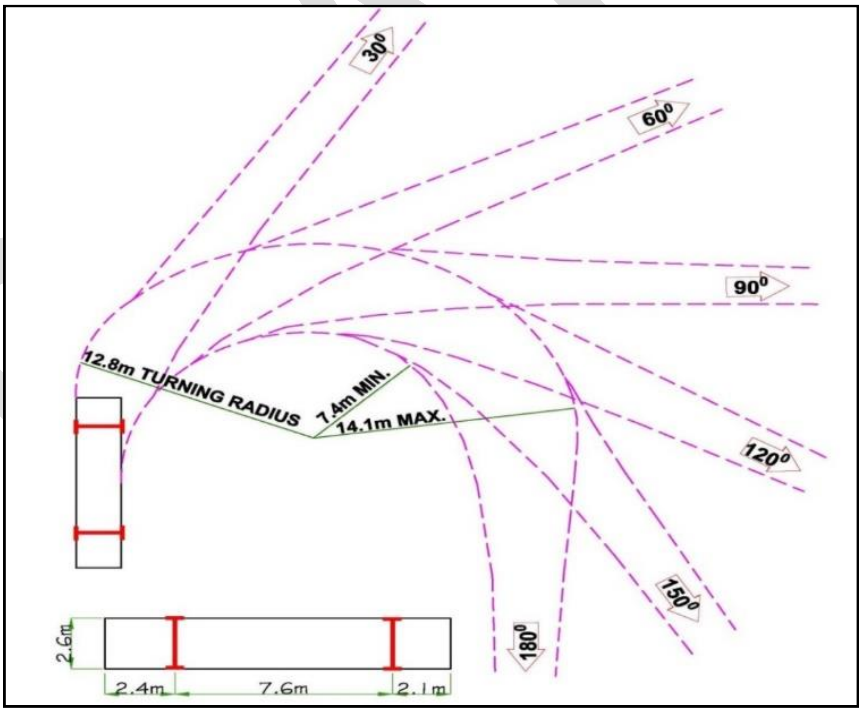


Figure 3-5: Dimensions and Turning Radius Path for Single Unit Bus (DV3)

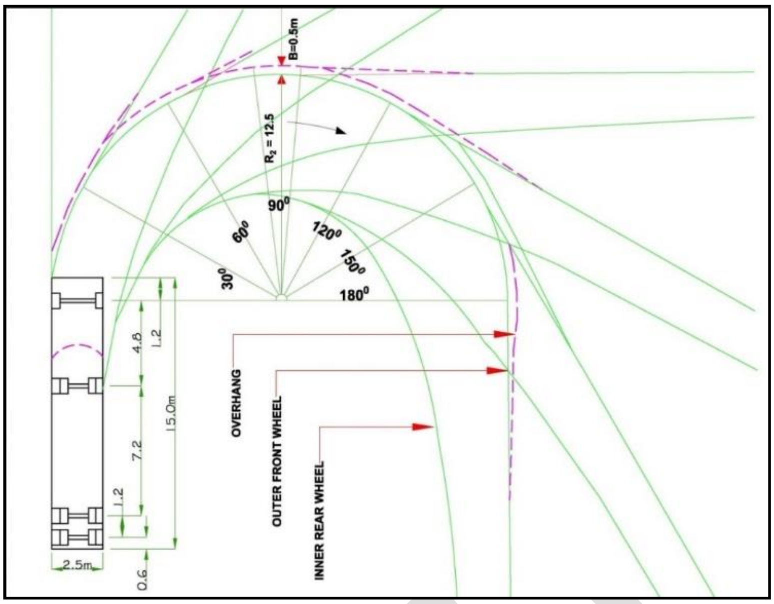


Figure 3-6: Dimensions and Turning Radius for a Semi-Trailer Combination

(15m overall; also applicable for Truck (Tandem) plus Trailer DV4)

3.12 Environmental and Social Considerations

No road project is without both positive and negative effects on the environment. The location and design of a road should aim at maximising the favourable effects of the project, such as providing or removing undesirable traffic from environmentally vulnerable areas, while at the same time minimising the adverse effects of the project as much as possible.

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

1) The preservation of the natural beauty of the countryside.
2) The preservation of areas and land used for specific purpose, including:
    i) National parks and other recreational areas,
    ii) Wildlife and bird sanctuaries,
    iii) Forests and other important natural resources,
    iv) Land of high agricultural value or potential,
    v) Other land of great economic value of importance in a local context,
    vi) Forests, wetlands, and other important natural resources,
    vii) Historic, archaeological, and cultural sites, cemeteries, and other man-made features of outstanding value.
3) The prevention of soil erosion and sedimentation,
4) The prevention of health hazards by ponding of water leading to the formation of swamps,
5) The avoidance or reduction of visual intrusion,
6) The prevention of undesirable roadside development.

Other considerations are mainly related to the operation of the road as a facility for moving traffic and include the following detrimental effects:

1) Noise pollution,
2) Air pollution,
3) Vibration,
4) Severance of areas (barrier effect).

These operational effects are mainly a problem of urban roads and traffic, but in some cases are also relevant to the design of roads in rural areas.

Some of the adverse environmental effects are easy to quantify (e.g., noise levels and air quality), whilst others are more difficult (e.g., visual impact). In many cases it is necessary to employ the services of other professions to reach a proper evaluation of the problems and establish adequate remedial measures. These will apply mostly to new roads.

3.12.1 Indirect effects

Indirect factors (also known as secondary or tertiary, factors) that are usually linked closely with the project and may have more long-term consequences on the environment than direct impacts. Indirect impacts are more difficult to measure but can ultimately be more important. Over time they can affect larger geographical areas of the environment than anticipated. Some of the indirect factors to consider include:

1) Degradation of surface water quality by the erosion of land cleared because of a new road.
2) Potential for spontaneous urban growth near a new road.
3) Increased deforestation of an area, stemming from easier (more profitable) transportation of timber or charcoal to market.
4) Potential influx of settlers into an undeveloped area.

3.13 Economic and Financial Considerations

3.13.1 General

The standard of a road and associated level of service increases with the level of traffic. This is entirely consistent with economic principles in that the basic whole life costs and lost benefits from a poor road network subject to congestion, poor surface condition and so on can be calculated with a tolerable degree of precision. Vehicle operating costs, in particular, are very dependent on road condition and travel time is also a major cost that is greatly reduced when traffic can travel speedily.

Although many of the costs and benefits of modifications and improvements in a geometric design can be calculated there remain several issues that cannot be quantified in monetary terms. This includes many environmental issues (e.g., benefits of not damaging a wetlands area), preservation of cultural issues (e.g., not using a burial area as part of a new road), the long-term costs of not re-instating quarries properly, the cost of road crashes and the reduction in accident costs when significant safety features are incorporated into the design; and many more.

Thus, although economic analysis is vital for optimising some of the major costs, there will always be a need to include other issues that are difficult, if not impossible, to quantify and therefore consultation, compromise, and flexibility are essential, and the services of an experienced Transport Economist should be obtained. Computer models such as "HDM-4", software for Highway Development, and other maintenance Management Systems which estimates the costs of different investment strategies for rural and urban roads can be used to analyse the cost and benefits of projects. HDM-4 is a decision-making tool for checking the Engineering and Economic viability of investments in road projects which has been developed by The World Bank for global use.

3.13.2 Design Life

Roads are expected to last for many years and always longer than it is prudent to forecast future developments in most subjects from technology, human needs and requirements, climate and so on. Consequently, the most accepted period for defining 'design life' is 40 years. Nevertheless, for pavement structural purposes the design is usually based on the 'mid-life' traffic primarily based on financial considerations of the cost of rehabilitation thus resulting in a pavement that is essentially over-designed for the early part of its life but under-designed for the later part. While this might not be the ideal based on whole life costing (WLC) principles, it has been is pragmatic and generally acceptable because of the uncertainties involved. However, the relatively recent discovery that so-called long-life pavements (LLPs) can be constructed that do not require substantial structural improvement but merely new surfacing during a long design life has changed this and designing for a longer period has now become a better option in many cases.

This is fortunate because for geometric design slightly different considerations apply for two reasons. First of all, the volume of traffic will continue to grow but rehabilitation if traffic exceeds the design level would require a new carriageway or new road entirely so, although the costs will be in the future and therefore discounted in the WLC calculations, the optimum method is to base the geometric design on the traffic expected at the end of the design life period and to leave a space for widening, if necessary, at that time.

For the purposes of geometric design In Kenya a design life of **20 years** shall be adopted.

4 Cross Section

4.1 General

The major geometric design elements constituting the cross-section are the carriageway, the shoulders and the ditches and, for dual carriageway roads, the central reserve. The carriageway includes the travelled way, any auxiliary lanes such as acceleration and deceleration lanes, climbing lanes, passing and bus bays and lay-bys.

Also related to the cross-section are cycle tracks and footpaths.

Many roads in Kenya, particularly those providing access as their major function, carry a considerable number of pedestrians and cyclists who make use of the shoulders and carriageway edges because separate facilities for them are not provided. From a traffic safety point of view this is an undesirable situation and cycle tracks and/or footpaths should be included in the cross-section where appropriate (at the cost of the width of the shoulders).

The selection of standards for the cross-section is dependent on the controls and criteria described in Chapter 3. Lane and shoulder widths, (ditch) slopes etc. should be adjusted to traffic requirements (traffic volume, traffic composition, vehicle speeds) and characteristics of the terrain. This means that the cross-section may vary over a particular route because the controlling factors are varying. The b sic requirements are, however, that changes in cross-section standards shall not be made unnecessarily, that the cross-section standards shall be uniform within each sub-section of the route and that any changes of the cross-section shall be effected gradually and logically over a transition length. Abrupt or isolated changes in cross-section standards lead to increased hazards and reduced traffic capacity and complicate construction operations.

In certain cases, however, it may be necessary to accept isolated reductions in cross-section standards, for example when an existing narrow structure has to be retained because it is not economically feasible to replace it. In such cases a proper application of traffic signs and road markings is required to warn motorists of the discontinuity in the road.

In specific cases it may be economic to select a stage-construction, i.e. to construct a road to a gravel standard in a first stage and improve the road to a bitumen standard when warranted by increased traffic. The conversion from gravel to bitumen has to be taken into account in the cross-section.

In order to simplify the selection and design of the cross-section elements and promote uniformity in standards, a set of standard cross- sections has been laid down, and guidelines are presented for the selection of the appropriate cross-section.

This chapter is primarily concerned with the components that comprise the interaction with the users of the road namely the carriageway and pavement itself for the vehicles, and footpaths for pedestrians.

The cross-section consists of the carriageway, shoulders and kerbs, drainage features, and earthwork profiles. These components can be designed for a range of conditions and a range of users and therefore there is a variety of designs that can be tailored for specific purposes. The type of cross-section to be used in the development of any road project depends on:

1) Urban or rural location

2) Terrain
3) The functions of the road, for example, through route or local access
4) New road or treatment of an existing road
5) Traffic volume and mixture
6) Number and type of trucks
7) Provision for public transport
8) Walking and cycling needs
9) Creating accessible environments for all
10) Place function and associated space requirements (for roadside vending and other roadside amenities)
11) Environmental constraints, for example, topography, existing public utility services, existing road reserve widths, significant vegetation, geology
12) Space provision for on-street parking.
13) Lane widening on low radius curves and junctions;
14) Provision of space in the median for bridge piers to support overpass structures;
15) Accommodation of barrier systems and their associated dynamic deflection or working width requirements;
16) Accommodation of traffic movements at intersections, U-turn facilities, interchanges, collector-distributor roads, and local roads;

The desirable widths of each element of the cross section may be selected using the information in this chapter. In most cases, the preferred width of cross-section elements will exceed the available width and therefore, an iterative process should be followed to optimise function, safety, environmental impact, economy, and aesthetics.

Through this iterative process, the selection of the various elements should not be done in isolation but rather an optimized allocation of space to provide the most appropriate toad safety, amenity, and operational outcomes.

A key consideration in determining the appropriate cross-section of a road is to understand the type and mix of people and traffic expected to use the road. Designers must be aware of the externalities of transport including crashes, pollution, congestion, and endeavour to provide road space for all types of transport, including:

1) Pedestrians (particular attention to children, persons with disabilities, the elderly, and women)
2) Personal mobility devices
3) Bicycles and handcarts
4) Motorcycles
5) Public transport
6) Cars

7) Trucks

In most urban areas, due to limited space, use of time restricted exclusive bus lanes, bicycle lanes in clearways etc. during peak hours can provide priority to more efficient modes of transport to encourage their use and assist in combating congestion.

4.2 Cross-section Components Nomenclature

The basic nomenclature of cross-section components of a road in a rural setting is given in Figure 4-1 and largely applies to the urban setting as well.

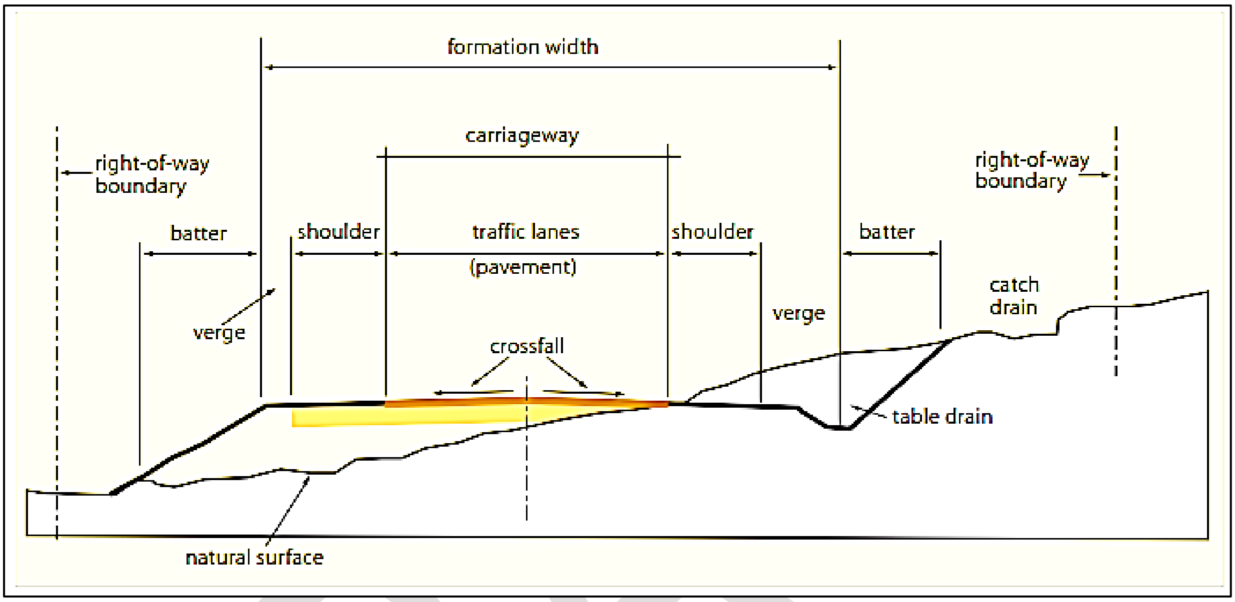


Figure 4-1: Basic cross section components nomenclature

4.3 Clear Zone

Clear zone should not be confused with road reserve (see Section 2.5). For adequate safety, it is desirable to have an area free of obstacles as wide as practicable. The necessary width of clear zones depends primarily on the design speed on a specific roadway section. It is measured from the edge of the travelled way and, ideally, also depends on the traffic level and whether the road is in cut or fill. The values recommended are shown in Table 4-1.

Table 4-1: Recommended clear zones (m)

Design speed (km/h)	Minimum values for medium to low traffic		Minimum values for low traffic	
	Fill slopes	Cut slopes	Fill slopes	Cut slopes
<60	3	3	2.5	2.5
60-80	5.5	4	4	2.5
80-95	8.5	5.5	5	3.0
95-110	10.5	6.5	7.5	4
>110	13	8.5	8.0	5

Source: Derived from SANRAL, Geometric Design Guidelines

The zone should extend beyond the toe of the slope. Lateral clearances between roadside objects and obstructions and the edge of the carriageway should normally be not less than 1.5 m. At existing pipe culverts, box culverts and bridges the clearance cannot be less than the carriageway width; if this clearance is not met, the structure must be widened. New pipe and box culvert installations, and extensions to them, must be designed with a 1.5 m clearance from the edge of the shoulder.

For the horizontal clearance to road signs, marker posts, etc. an absolute minimum of 0.5 m from the edge of the carriageway is legally required.

Elements such as side slopes, fixed objects and drainage features are items that a vehicle might encounter if it leaves the roadway. The safety measures that can be taken depend on the probability of a crash occurring, the likely severity, and the available resources. In order of priority, these measures are:

1) Removal.
2) Relocation.
3) Reduction of impact severity (using breakaway features – e.g., supports of large vertical signs - or making it traversable – e.g., culvert ends).
4) Shielding with a road restraint system.

It is recommended that a safety assessment of the network should be undertaken as a planning tool to assist in evaluating the safety of individual road segments.

4.4 Staged Construction

When planning policies and economic considerations recommend staged construction, the designer should ensure the available road reserve can accommodate full development of the facility in future.

The initial stage cross-section should allow for the maximum future reuse of pavements and underground drainage and facilitate construction to the ultimate width. Both the initial and ultimate cross-section should provide for the safe and efficient movement of all relevant road users including pedestrians and cyclists.

4.5 Crossfall

Normal crossfall (or camber) of 2.5% should be sufficient to provide adequate surface drainage whilst not being so great as to make steering difficult. Auxiliary lanes shall have a crossfall of the same direction and rate as the adjacent lane. The ability of a surface to shed water varies with its smoothness and integrity. On unpaved roads the minimum acceptable value of crossfall should be related to the need to carry surface water away from the pavement structure effectively, with a maximum value above which erosion of material starts to become a problem.

4.6 Traffic Lanes

There are many types of lanes that may exist in a cross section, and each has its own purpose and sizing needs. General-purpose traffic lanes need to accommodate a variety of vehicle types including buses, freight vehicles, personal automotive vehicles, and bicycles. The target speed, modal priority, balance of performance needs, and transportation context are all considerations when determining size, type, and number of lanes.

For paved roads, it is important to consider the required width for road marking. Road marking will affect lane width and allowance should be included when defining lane width. An additional lane width of 0.15- 0.20 m will provide the necessary allowance for road markings as well as sufficient passing clearance between large commercial vehicles (refer to **RDM Volume 6: Part 1 – Road Marking**).

On every occasion where standards need to differ from those that drivers expect, transition zones and adequate warning signs must be provided to promote the adaptation of the driver's reactions (6-Seconds Rule, Section 3.3.2).

4.6.1 Through lanes

Through lanes are the most common lane type. All roadways have at least one lane in each direction to provide unimpeded traffic flow from Point A to Point B.

Established or common practice in Kenya can be useful in determining and documenting lane width selection where a range of width values is provided. Table 4-2 gives ranges of widths for the road classes gazetted in Kenya.

Table 4-2: Through lanes width ranges (m)

Road category	Most common functional class and type	Surface type	Lane width range
Inter-urban roads	A (International highways)	Paved	Dual 2* 7.3
	B (national highways)	Paved	7.3
	C (primary roads/ inter county Roads)	Paved	7.0
Rural roads	D (secondary roads/ inter sub-county).	Paved	7.0
	E (minor roads/ sub-county roads)	Paved or unpaved	6.5- 7.0[1]
	F, G, P, S, W, T, U (local roads)	Paved or unpaved	5.5- 7.0[1]
Urban roads	UA (urban arterial roads)	Paved	Dual 2* 7.3
	UC (urban collector roads)	Paved	7.3
	UL (local urban streets)	Paved	6.0- 7.0

Notes:

***1**. 3.75m on urban arterials and urban collectors provide increased benefit on high speed, free-flowing arterials. On interurban roads, 3.75 m lanes provide increased benefit where there are higher truck volumes.*

***2.** On UC (urban collector) multilane facilities with width constraints, narrower inside lanes of 3.5 m may be permitted to allow for wider outside lanes for bicycles, pedestrians, and transit. Narrower lanes also allow shorter pedestrian crossing times because of reduced crossing distances.*

***3.** On urban streets, through lanes of 3.0 m may be used as a speed reduction strategy and they also allow shorter crossing times for pedestrians. If there is considerable truck and bus volume traffic, width of 3.5 is recommended.*

4.6.2 Auxiliary lanes

An auxiliary lane is primarily for the acceleration or deceleration of vehicles entering or leaving the through travelled way. They are normally provided for at-grade intersections on multilane divided highways with access control. Where roadside conditions and right of way allow, auxiliary lanes may be provided on other through roadways.

Justification for an auxiliary lane depends on many factors, including speed; traffic volumes; capacity; type of highway; design and frequency of intersections and crash history.

When either deceleration or acceleration lanes are to be used, design them in accordance with Section 9.11.14.

A dedicated deceleration lane is advantageous because it removes slowing vehicles from the through lane. An acceleration lane on the other hand may not be as advantageous as entering drivers can wait for an opportunity to merge without disrupting through traffic. However, acceleration lanes for right-turning vehicles provide a benefit by allowing the turn to be made in two movements.

A lane width of 3.5 m should be provided.

4.7 Shoulders

4.7.1 General

The shoulder is the portion of the roadway next to and immediately connected to the carriageway. In rural areas it provides:

1) Lateral support for the pavement layer connected to it;
2) Accommodation for stopped vehicles;
3) Space for traditional and intermediate non-motorised traffic, animals, and pedestrians.
4) Space for the recovery of errant vehicles and for any emergency use;

In addition:

1) It enables non-motorised traffic (pedestrians and cyclists) in rural areas to travel with minimum encroachment on the carriageway.
2) Sight distance is improved in cut sections, thereby improving safety;
3) Lateral clearance is provided for signs and guardrails.
4) It acts as a barrier for moisture seepage to the carriageway.

For shoulders to function effectively, they must be durable to support occasional vehicle loads in all kinds of weather without failure. Paved or stabilized shoulders are required to minimize the risk of such structural failure. Paved shoulders of the same strength and standard as the pavement should be considered for the higher road standards.

4.7.2 Shoulder widths

Shoulder width is measured from the outer edge of the traffic lane to the edge of usable carriageway and excludes any berm, verge, rounding, or extra width provided to accommodate guideposts and guard fencing.

Table 4-3 provides values for shoulder widths for both urban and rural roads based on functional classification. These widths allow a vehicle to stop, or a maintenance vehicle to operate, with only partial or no obstruction of the traffic lanes.

Table 4-3: Recommended widths of shoulder

Road category	Most common functional class and type	Shoulder width
Inter-urban roads	A (International highways)	2.7
	B (National highways)	2.7
	C (Primary Roads/ Inter County Roads)	2.0
Rural roads	D (Secondary Roads/ Inter sub-county).	1.5
	E (Minor Roads/ Sub-County Roads)	1.0
	F, G, P, S, W, T, U (Local roads)	1.0
Urban roads	UA (Urban arterial roads)	1.0- 1.5
	UC (Urban collector roads)	1.0- 1.5
	UL (Local urban streets)	1.0- 1.5

A width of 2.7 m recommended for class A and B roads is adequate to allow a passenger vehicle to stop clear of the traffic lanes. On urban roads, a shoulder may be omitted where kerb and channel has been constructed. On class AU and AC roads, shoulders may be provided to perform a similar function to those on rural roads, particularly for drainage, on-road cyclists and in case a vehicle breakdown occurs.

4.7.3 Shoulder crossfall

All shoulders should be sloped sufficiently to drain carriageway surface water rapidly. A cross fall of 4% is recommended. Where the shoulder consists of full depth pavement and is sealed, its slope may be the same as the adjacent pavement to facilitate construction. However, the designer must seek permission from the Chief Engineer through the Departure from Standards Form (see Chapter 16).

On superelevated sections of roads, the shoulder on the high side and low side must maintain the same crossfall of 4%.

Table 4-4: Shoulder crossfall

Material in shoulder	Crossfall	Remarks
Earth	4%	
Gravel or crushed rock	4%	
Full depth pavement with bitumen seal or asphalt wearing course	4%	Can match traffic lane*
Concrete	4%	Can match traffic lane*

**Must be applicable through the Departure form Standards form.*

4.7.4 Sealing of shoulders

Shoulders should normally be sealed. Sealing of shoulders is frequently done to reduce maintenance costs and to improve moisture conditions under pavements, especially under the outer wheel path. However, a most important benefit of sealed shoulders is that they reduce crash rates, particularly with respect to run-off-road crashes.

For structural purposes, preventing the ingress of water through the surface of the shoulders provides a large benefit in terms of greatly improved road pavement performance and subsequent whole life costs. Sealed shoulders also provide other benefits and should be used if any of the following conditions apply:

(i) For motorways;

(ii) Pedestrian use is significant and a segregated footpath cannot be provided;

(iii) Non-motorised traffic is significant;

(iv) The shoulder material is erodible;,

(v) There is likely to be a shortage of material for maintenance of the shoulders;

(vi) Where heavy vehicles tend to use the shoulder as an auxiliary lane;

(vii) In front of vehicle restraint systems;

(viii) Where the total resultant gradient exceeds 6%.

Full surfacing implies continuous surfacing along the length of the road but not necessarily across the full width of the shoulder, although the former is the desirable option. The designer should consider the economic merits of a relatively narrow surfaced shoulder compared with a wide unsurfaced shoulder.

In severe terrain, the width of an existing roadway is usually narrow and, where the shoulder width can only be maintained through an excessive volume of earthwork, e.g., in escarpment conditions, standards can be reduced further through the Departure from Standard process described in Chapter 16.

4.7.5 Shoulder drop-off

The vertical height difference between the paved surface and the unpaved shoulder is the pavement shoulder drop-off. Horizontal curves are particularly prone to drop-offs, especially when vehicles stray on to the edge of the travel lane.

A drop-off as small as 7.5 cm can create an unsafe condition when the vertical angle is 90 degrees. Shoulder edges should be finished to the same level as the edge of the carriageway and should be rounded or tapered.

4.8 Footpaths

4.8.1 General

Footpaths are reserved for use by pedestrians, people in wheelchairs, and personal mobility devices. Hand carts for ferrying goods or passengers should not be on the footpath.

On urban streets, it is necessary to provide footpaths for pedestrian traffic. In commercial areas or areas where the road reserve width is restricted, footpaths may extend from the kerb to the road reserve boundary.

The decision as to whether a footpath is included in the cross section on both sides of the road or only one side will depend on local guidance requirements and connectivity to the wider pedestrian network.

These requirements should be considered early in the design process to avoid compromised designs where pinch points become necessary or require costly retrofit. Innovations and better use of existing road space can contribute to maintaining and enhancing the place function of street design.

The width of a footpath for pedestrians is dependent on its location, purpose, and the anticipated demand on the facility. Minimum widths are as tabulated in Table 4-5 . If the footpath is immediately adjacent to the kerbing, the minimum width should be increased by 0.6m. This is to make provision for street lighting and other road furniture. It also allows for the proximity of moving vehicles and the opening of car doors.

The width of the footpath may need to be greater than the recommended minimum in the following situations:

1) At a pedestrian crossing point to allow people to pass those waiting;
2) At a bus stop;
3) Where service lines and roadside furniture or structures restrict the width;
4) Where higher pedestrian volumes are anticipated (e.g., near shops) .

Footpaths must be continuous, not too sinuous and be unimpeded by obstructions.

Kerbs raised medians and channelising islands can be major obstructions to the elderly and people with disabilities, particularly those in wheelchairs. The most common method for minimising the impact of these obstacles is to provide ramps, also referred to as kerb cuts or dropped kerbs. Ramps should have a slope of not more than about 6 %. A kerb height of 150 mm would thus require a ramp length of 2.5 m. Ideally there should be a clear footpath width of 1.5 m beyond the top end of the ramp so that, where a ramp is provided, the overall footpath width should be not less than 4 m. Wheelchairs may be 0.75 m wide so that two would require a ramp width of 2 to 2.5 m. If it is not possible to provide this width, a width of not less than 1.5 m should be considered.

4.8.2 *Footpath crossfall*

The crossfall of a paved footpath may vary from flat (for example with slotted drain tops) to 2.5%. Footpaths that cross accesses and driveways may need a steeper cross-slope to match the gradient of the driveways but should not exceed a cross slope of 5%. When developing the cross-section detail the designer must be aware that excessive crossfall may cause problems for people with a visual or mobility impairment. Designing for users with disabilities is likely to result in a more accessible and walkable environment for all people including younger and older pedestrians.

4.8.3 *Footpath zones*

There are three distinct footpath zones as shown in Figure 4-2 within the area between the edge of the road and the frontage of adjacent property, and it is important to distinguish between the total width and the width of the zone likely to be used by pedestrians who are walking through this zone.

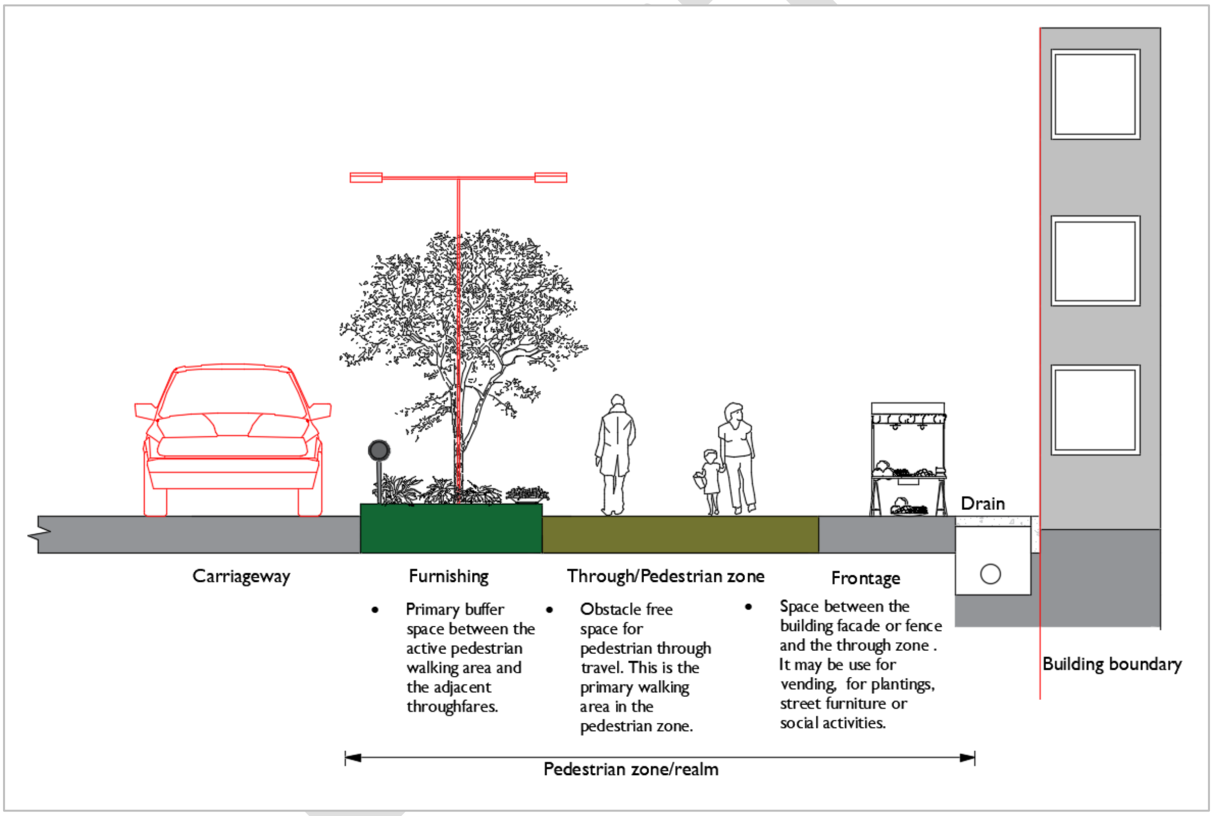


Figure 4-2: Distinct zones of a footpath

4.8.4 *Operating space for pedestrians*

Design of pedestrian facility adopts the body shape of a typical pedestrian, represented as a simplified body eclipse of 0.50 by 0.60m with a total area of 0.30 sqm. In evaluating a pedestrian facility, an area of 0.75 sqm is used a s a buffer for each pedestrian. This also allows for pedestrians with young children, carrying load, personal articles, natural psychological preferences to avoid bodily contact with others and body sway.

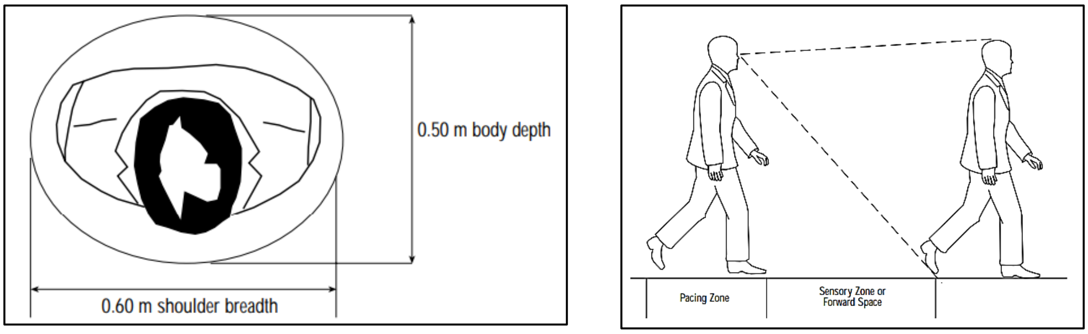


Figure 4-3: Pedestrian eclipse and pedestrian walking space requirement

4.8.5 Width of footpaths

The widths provided in this section are for a clear width on a path. Intrusions in or over a path, such as vegetation, signs, poles, fences, or seats may become obstacles or hazards to path users, reducing the width of the clear path and should be removed wherever practicable. At locations where the intrusion is unable to be removed, pedestrians should be alerted with sufficient time to enable the obstacle or hazard to be avoided. The footpath width required depends on the envelope (i.e., space) occupied by pedestrians together with appropriate clearances. The clearances are required between path users travelling in the same direction or opposite directions, and also between path users and the edge of the path.

	Width of pedestrian with ambulant disabilities, elderly, expectant women, or a pedestrian carrying load on head/back	1.0m
	Pedestrian using a wheelchair or a pedestrian holding a child	1.2m
	Pedestrian on a wheelchair and pedestrian ambulant disabilities, elderly, expectant women, or a pedestrian carrying load on head/back to pass each other freely.	1.5m
	Two pedestrians on a wheelchair to pass each other freely	1.8m

Figure 4-4: Pedestrian path requirements

To avoid interference when two pedestrians pass each other, each pedestrian should have at least 0.8m of walkway width. Minimum requirement for other pedestrians is as shown in Figure 4-4.

Table 4-5: Recommended footpath widths

Situation	Suggested minimum width (m)	Comments
General low volume	1.5	General minimum widths for most roads and streets. Clear width required for one wheelchair with 0.15m allowance both sides. Not adequate for commercial or shopping environments.
High pedestrian volumes	2.4 (or higher based on volume)	Generally commercial and shopping areas.

Adapted from Austroads 2007: Guide to Road Design Part 6A.

Notes:

a) *While the minimum width may be used where volume is low it is generally desirable to provide a path that will accommodate two pedestrians side by side.*

b) *Wider than the minimum width (e.g., up to 5 m) may also be necessary at locations where pedestrian flows are high or where pedestrians gather such as in the vicinity of schools and associated road crossings, at recreation facilities and at important bus stops.*

c) *Where volume is significant it may be necessary to provide adequate congregation areas clear of the path required for through movement of pedestrians.*

The lane concept is important in understanding how many people can walk abreast, and in the determination of minimum footpath width that say can allow two pedestrians to pass each other freely. However, to analyse pedestrian flow and space requirements, studies have shown that pedestrians do not walk in organized lanes. Pedestrian flow relationships should be used to determine the LoS criteria for pedestrian flow. This detailed analysis has not been covered under this manual; however, the designer can refer to external references such as: **HCM 2010** and **Austroads Guide to Road Design Part 6A.**

4.8.6 Footpaths in rural areas

Footpaths are not normally provided in rural areas. Table 4-6 indicates conditions where paved footpaths are recommended in rural areas. Footpath width can be constrained to 1.0 m, but a width of 1.8 m allows two people to walk side by side.

The safest location for footways is at the edge of the road reserve. In rolling or mountainous terrain through cuts and fills, such a footway is not comfortable for walking and so pedestrians often prefer to walk on the more level surface of the shoulder. In level terrain, the footpaths should, if possible, be situated at least 3.0 m away from the travelled way. In the case of a high-volume high-speed road this unfortunately corresponds to a location immediately outside the edge of the usable shoulder.

In cases where footpaths are not warranted but where many pedestrians walk alongside the road, the road shoulder should be upgraded to cater for them. The width of these shoulders should be at least 2.0 m. If the shoulders are not surfaced, they should be bladed and compacted regularly to provide pedestrians with a hard surface to walk on. In high rainfall areas, a portion at least of the shoulder should be paved, with this paved area being at least 1.5 m wide.

Table 4-6: Recommendations for paved footpaths in rural areas

Footpath	ADT (vehicles/day)	Pedestrian flow per day	
		Design speed < 80km/h	Design speed > 80km/h
On one side of the road	300-1500	300	200
	>1500	200	120
On both sides of the road	700 - 1500	1000	600
	>1500	600	400

4.9 Cycle Lanes

4.9.1 General

A separate cycle lane should be provided on urban roads where pedal cycle traffic is high and to encourage bicycle use in traffic congested areas. On roads of classification UL, separated facilities for cyclists may not be necessary, as the lower speed of motor traffic should enable cyclists to safely share the road with other users. On UA and UC roads, it is usually necessary to ensure that adequate space exists for cyclists to share the road safely and comfortably, particularly when the road forms part of a principal or regional bicycle network. It may be possible to reduce the widths of other lanes to allocate additional space for use by cyclists.

4.9.2 Horizontal alignment

The horizontal alignment standard adopted on roads to serve the needs of motor traffic will normally be satisfactory for bicycle traffic, provided the operational aspects of cycling are understood by designers.

4.9.3 Grades

Excessive gradients on hills can be unpleasant to cyclists and act as a deterrent to bicycle riding, designers should strive to minimize gradients on all new developments. It may be possible to achieve flatter grades on important collector roads for little additional cost.

In situations where a steep gradient is unavoidable, additional pavement width of minimum 0.25 m should be provided to allow for operating characteristic of cyclists on steep gradients.

4.9.4 Widths of cycle lanes

When the volumes of cyclists are not able to be determined, the widths shown in Table 4-7 provide acceptable ranges for bicycle paths. Designers should provide a greater width where it is needed (e.g., very high demand that may also result in overtaking in both directions).

Table 4-7: Recommended cycle lane widths

Situation	Suggested minimum width (m)
General low volume	1.2
High cyclist volumes	2.0 (or higher based on volume)

Alternatively, where a shared path for cyclists and pedestrians is provided, it is suggested that the path widths be as Table 4-8

Table 4-8: Recommended widths of shared paths

Situation	Suggested minimum width (m)		
	UL road class	UA and UC road class	Recreational paths
Shared path	2.0	3.0	3.0

Figure 4-5: Shared paths for cyclists and pedestrians

4.9.5 Cyclists and parking lanes

The presence of parked cars puts cyclists under additional stress, as they must constantly search for car occupants to assess whether a door is likely to be opened into their path. Collisions between cyclists and opening doors of parked cars are a significant concern to cyclists. Such incidents should be an equal concern for car occupants in view of their duty of care obligations.

When on-street parking is present adjacent to a cycle lane, a raised kerb should be provided as a strip to protect cyclists. The raised separator generally requires breaks in the kerb to maintain the free drainage of the road or otherwise a specific drainage system needs to be installed.

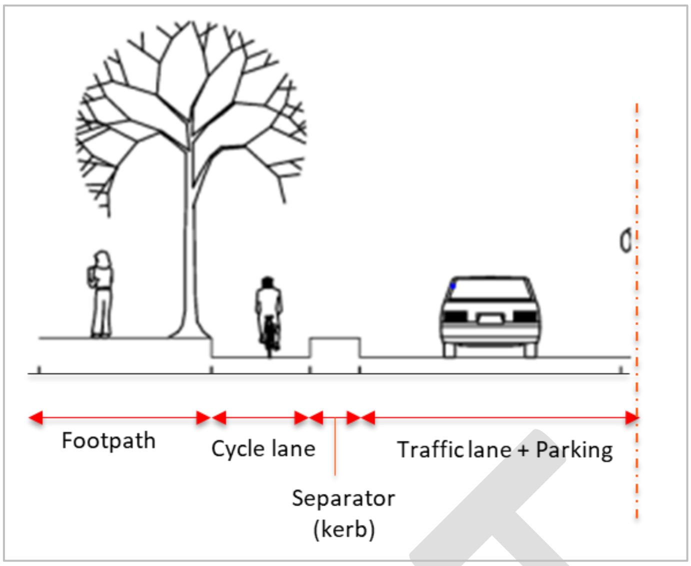


Figure 4-6: Cross-section of a protected bicycle lane

4.9.6 Cycle lane crossfall

Water ponding on paths has a significant impact on the level of service provided to cyclists as spray leads to grit on both bicycle and rider and pedestrians, who may have to travel off the path to avoid the ponded water.

On straight sections crowning of the pavement is preferable as it results in less accumulation of debris. On sealed surfaces a crossfall of 2.5% should be adequate to effectively dispose of surface water whereas unsealed surfaces may require 4% to prevent puddles of water from developing. Where it is a shared-use path, the needs of other path users (e.g., mobility impaired pedestrians) should be considered.

Where open drainage ditches are provided adjacent to the main carriageway, it may be more relevant to provide the non-motorized traffic facilities beyond the drainage ditch and away from the carriageway.

4.10 Motorcycle Lanes

In Kenya, the share of the motorcycle in the total road traffic has increased compared to other modes of road transportation. It is considered a cheap mode of transport compared to other modes, provides door-to-door connectivity, has a small size, and has high manoeuvrability.

Generally, motorcycles share the carriageway with other motorised vehicles. Provision of separate lanes where there are large numbers of motorcycles can reduce the potential for conflicts with larger vehicles. Motorcycle lanes if required should be installed on the outside lane of the main carriageway for each direction of traffic flow. Motorcycle lanes may be separated from the rest of the road by painted lines or physical barriers.

Warrant analysis for motorcycle lanes on road class UA and UC should be:

1) High ADT volumes of more 15,000 vehicles per day;
2) More than 30% of the traffic in mainstream comprises motorcycles; and
3) Crash frequency of more than 5 KSI crashes involving motorcyclists.

The geometric design of motorcycle lane is affected by the physical characteristics and the proportion of various sizes of motorcycle. The recommended design vehicle for motorcycle lane for typical Kenya traffic is of capacity less than 250 cc.

Length	Width	Height	Turning radius
2.6 m	1.0 m	1.64 m (inclusive of rider)	3.0 m

4.11 Medians

4.11.1 General

Medians are provided on all dual carriageway roads such as interurban roads and high speed urban arterial roads. The median is the total area between the outer edges of the outer traffic lanes of a divided road and includes the outer shoulders and central islands. Medians separate opposing streams of traffic but have other very significant advantages. They:

1) Significantly reduce the risk of collisions with opposing traffic.
2) Improve capacity by restricting access to property and minor side streets.
3) Provide a safety refuge for pedestrians making it easier and safer to cross busy roads.
4) Prevent irregular U-turn movements.
5) Provide a space to collect run-off water and carry it to the drainage system.
6) Provide headlight glare screening with shrubs planted in the central area. A maximum stem thickness of 100 mm is recommended.
7) Provide space for public transport (HGV) lanes and bus stops, and light rail tracks and platforms.
8) Provide space for road furniture, parking in slow speed urban areas and space for future additional traffic.

4.11.2 Median treatments

Medians are either restrictive or non-restrictive. Restrictive medians physically limit motor vehicle encroachment, using raised curb, median barrier, fixed delineators, vegetative strips, or vegetative depressions. Non-restrictive medians limit motor vehicle encroachment legally and use pavement markings to define locations where turns are permissible.

Depressed median

Raised median

Painted median

Wide centerline median

Figure 4-7: Typical median treatments

Painted median.	Painted medians, also known as flush medians, are a low-cost option that address head-on crashes by improving lateral separation of vehicles and discouraging overtaking.
Wide centreline	Wide centrelines are a type of non-restrictive median treatment, typically provides a 1 m wide narrow median, increasing the separation of vehicles, but with negligible effect on vehicle travel speeds. Minimum width of 1 m is recommended.
Depressed median	Depressed median is not common on urban roads. It acts both as a physical separator and also facilitates drainage.
Raised median	Typical on urban roadways. They are restrictive medians preferred for their conspicuity and physical deterrent effect in preventing cross-median manoeuvres. They can also accommodate road furniture and can be landscaped for aesthetics and to restrict headlight glare. For a raised median, kerbing offers a modest degree of protection to pedestrians who may be on the median while crossing the road. In addition, kerbing can, to a very limited extent, redirect errant vehicles back into their own lanes.

4.11.3 Median widths

The width of a median is measured from edge of travelled way to edge of travelled way and includes shoulders. Median widths can vary greatly based on the functional use of the median, the functional use of the shoulders, target speed, and context (see Table 4-9).

In areas where land is expensive, an economic comparison of wide medians to narrow medians with barrier should be carried out. Considerations of right of way, construction, maintenance, and safety performance are important.

The widths of medians need not be uniform, the designer can provide for a transition between median widths as long as practical. Independently aligned and graded carriageways may be beneficial, provided that the opposing carriageway is not out of sight for extended periods.

When staged construction is applied (see Section 4.4), the usual practice is to widen the carriageways into the median. As such, it is typical to construct a wide median in the initial stages of a project as this removes the need to modify interchange ramps and other intersection treatments with side roads or longitudinal drainage systems.

At locations where the median will be used to allow vehicles to make a U-turn, the designer should provide the widths in accordance with the design vehicle. Where feasible, medians should be widened at intersections on interurban roads and to ensure sufficient width to store right turning vehicles.

On urban roads with a design speed of more than 70 km/h, a lateral clearance of 0.5 m should be provided from the edge of the travelled lane. Design speeds of 70km/h or less, no lateral clearance is required.

Medians wider than 10 m allow for effective planting and landscaping, however, they have the disadvantage of reducing the capacity of signalised intersections due to the increased clearance times for crossing traffic. The plant species selected for median landscaping should be carefully considered for their impact on safety. Paving of narrow (or residual) medians (< 2 m wide) should be considered to minimise maintenance costs, and exposure of maintenance personnel to improve workplace safety.

Table 4-9: Recommended minimum widths of medians

Median function	Recommended minimum widths
Access control- restrictive	Width of the median feature (also include provision for shoulder and allowances for shy line effect).
Access control- non restrictive	Minimum 1 m. Less widths of up to 0.75m when the traffic flows and operating speeds are low.
Raised median/cut-through island for a pedestrian and/or bicyclist refuge that allows crossing in two stages	2.5 m
Vehicle storage space for crossing at intersections (i.e., includes a 3.5 m wide adjacent traffic lane)	6.0 m
Median U-turn or Median crossover	In accordance with the design vehicle
Recovery area	20 m

4.11.4 Median slopes

Two different conditions dictate the steepness of the slope across the median, namely drainage and safety. The normal profile of a median is a negative camber, i.e., sloping towards a central low point, to facilitate drainage.

The flattest slope that is recommended for depressed medians is 10% as shown on Table 4-10. Slopes flatter than this may lead to ponding and to water flowing from the median to the carriageway. Slopes steeper than 1:6 make control of an errant vehicle difficult, leading to a greater possibility of cross-median crashes. If surface drainage requires a median slope steeper than 1:6, this aspect of road safety might justify replacing the type of stormwater drainage.

When considering the slope to be constructed on depressed medians, designers shall consider the following needs:

1) Drainage: the median should be wide enough to be driveable, yet deep enough to construct a drain that will carry the design volume of storm water whilst providing freeboard to the road pavement. Narrow medians (less than 10 m wide) result in shallow drains, which require regular outlets to manage the volume of storm water.
2) Safety barriers: designers should consider the appropriate median slopes to be used in conjunction with safety barriers, for traffic on both carriageways.
3) Intersections – where an at-grade intersection is provided across a median (formal intersection or an emergency crossing point), the median should be sloped accordingly to provide a smooth transition between the depressed median drain and the crossing roadway. Drainage facilities (e.g., culvert end walls etc.) should be constructed to avoid the need for safety barriers as these restrict sight distance for vehicles at the intersection.

Table 4-10: Recommended median slopes.

Median type	Minimum	Maximum	Comments
Depressed	1:10	1:6	
Raised	1:40	1:6	The minimum allows for stability of wheelchair users on the refuge.
At median opening	1:25	1:20	Minimum should be provided if the roadway has significant truck traffic

4.11.5 Median slope treatments

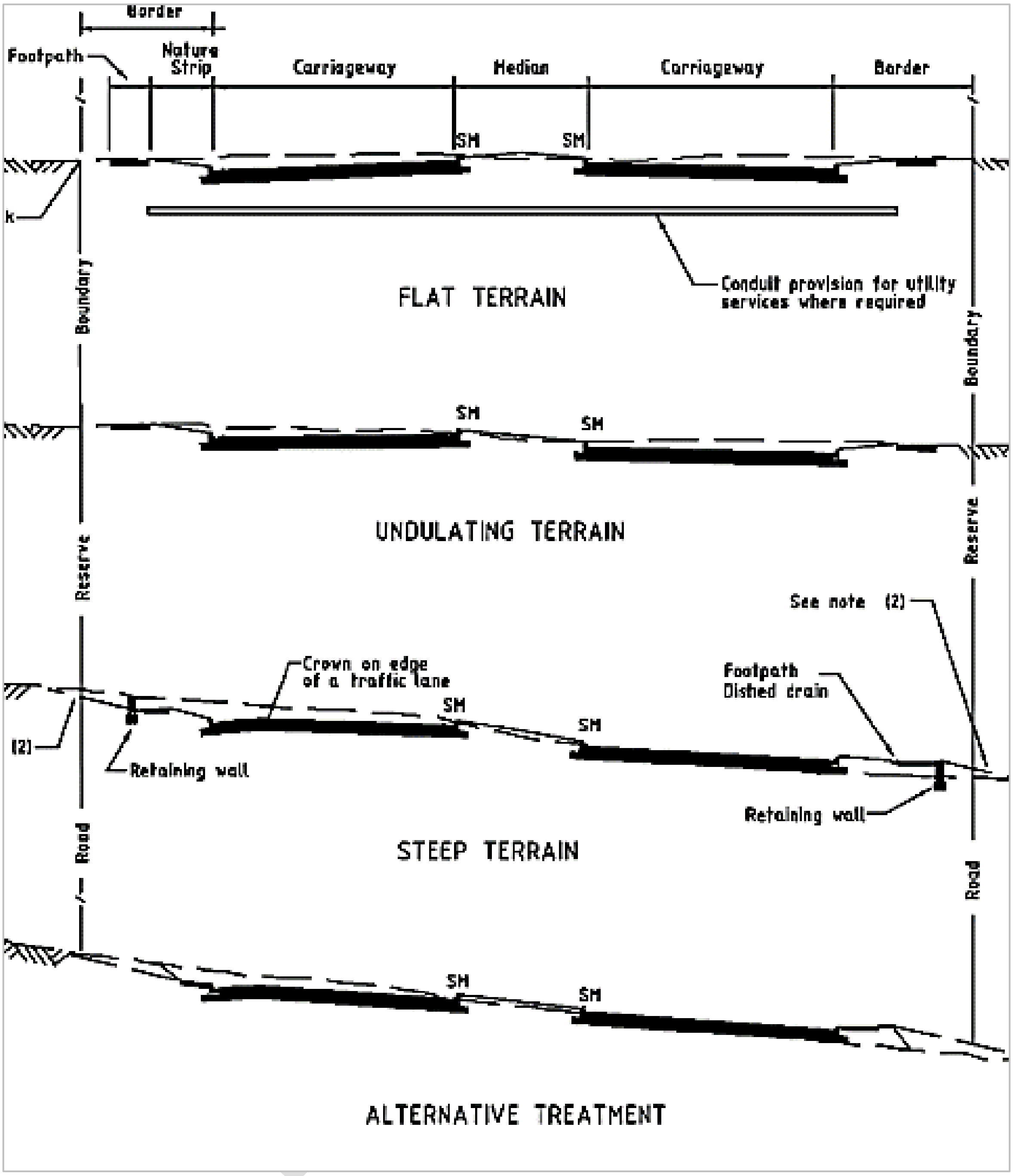


Figure 4-8: Median slope treatment (30-50m corridor)

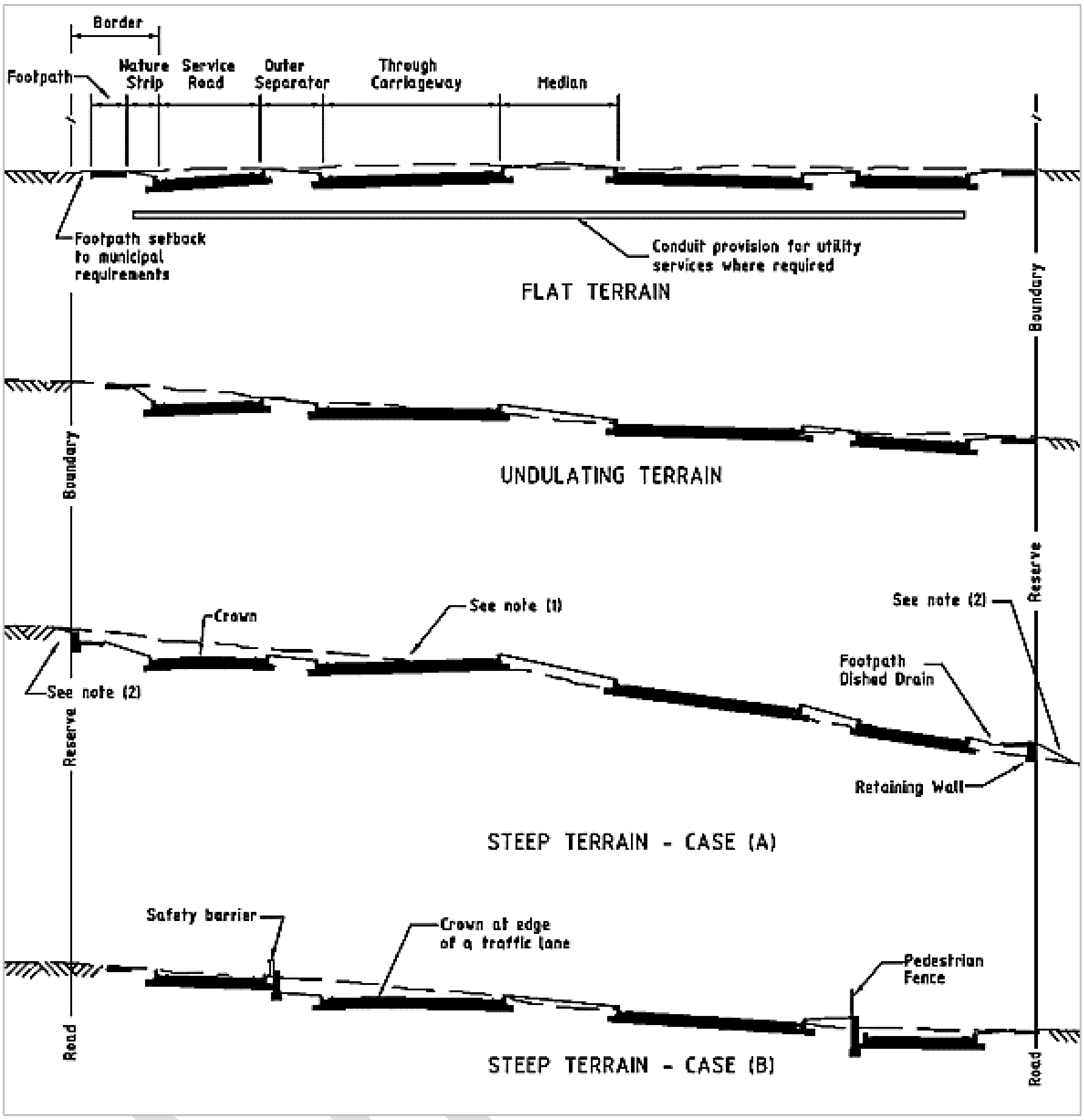


Figure 4-9: Median slope treatment >50m corridor

4.11.6 Median transitions

Where the road needs to transition from a divided to an undivided facility, appropriate transitions are required to safely merge and diverge vehicles. This is discussed in Section 9.11.6

4.12 Outer Separators

The outer separator is the area between the edges of the travelled way of a major road and an adjacent parallel service road or street. It comprises the left shoulder of the major road, an island and the right shoulder of the adjacent road or street. Its purpose is to separate streams of traffic flowing in the same direction but at different speeds and to modify weaving manoeuvres. It is a buffer between through traffic and local traffic on a frontage or service road. It is typically applied where the corridor must serve the two functions of long-distance travel and local accessibility. An arterial passing through a local shopping area is an example of this application.

In general, the standards applied to medians are equally appropriate to outer separators.

4.13 Service Roads

Service roads are roads that are constructed between the principal carriageway and the property line. They may be continuous or discontinuous and are usually restricted to one way traffic. They provide property access, and commercial access to link detached local roads, and may provide through traffic function if continuous.

The number of lanes for service roads depends on expected traffic volumes and demand for on-street parking. The width of the separator between the service road and the adjacent arterial road should be evaluated to allow the placement of necessary roadside furniture such as streetlights, access control fencing, bus bays, and planting. The traffic flow on the access road and the adjacent through traffic carriageway shall usually be in the same directions, to avoid driver confusion and headlight glare. Typical minimum lane widths for service roads are shown in Table 4-11.

Table 4-11: Recommended widths for service roads

Lane type	Road width (m) for a service road that primarily provides			
	Residential access		Commercial/ industrial access	
	Parking one side	Parking both sides	Parking one side	Parking both sides
One-way single lane	5.5 m	7[a] m	6.0 m	8.5 m
Two-way two lanes	6.5 m	8.0 m	8.0 m	10.5 m

Notes:

a. Width staggered parking, the width can be reduced to 5.5 m.

In most cases, the operating speed on service roads will be similar to that of local collector or local urban streets and they should be designed accordingly. With high traffic volumes, the service road may perform a significant traffic function and the operating speeds could be higher. In these cases, it might not be appropriate to allow parking on the service road. Such service roads will usually be confined to rural or peri-urban areas.

In rural areas, it is preferable that the local road network provides the service road function. This will avoid the service road encouraging ribbon development along the major road. It also probably represents the best use of resources, avoiding the construction of additional road infrastructure.

4.14 Transit Lanes

4.14.1 General

When designing the cross section including public transit, it is important that designers appreciate that public transport is not simply another set of vehicles operating independently on the road network. Public transport is a comprehensive service system that involves vehicles, infrastructure, systematically planned strategic routes and schedules, operational systems and most importantly, passengers.

When developing new roads, designers should establish whether there are any public transport services proposed for the route. If so, the road alignment should be designed to provide acceptable ride quality for passengers and minimal delay. This applies equally to horizontal and vertical alignments, intersection layouts, and to mid-block curves and gradients.

By common practice in Kenya, buses and matatus share mixed traffic lane with other vehicles. Due to the need for boarding and alighting, they tend to ply the left lane of divided and undivided roadways. A typical bus is 2.5 m (bus body), but modern buses can occupy a wider space, up to 0.3 m on each side. Preferred lane widths for buses should therefore be a minimum of 3.5 m to allow for two buses to safely pass each other. On urban collector roads with considerable bus traffic, a traffic lane of 3.0 m is not recommended.

On a multilane corridor, a wider outside lane of 4.5 m lane width can be considered as an acceptable wide outer lane that can be used jointly by buses, matatus, motorcyclists and bicyclists.

4.14.2 Dedicated transit lanes

Dedicated roadway for the exclusive use of public transit includes high standard stations that are highly accessible to all passengers including persons who have a disability that may involve impairment to mobility, vision, or hearing.

The dedicate lanes offer unimpeded, relatively high-speed environment where mass transit vehicle delays are minimised, and schedule adherence is enhanced. They are usually constructed as a median facility in a corridor or on an offset or a new or alignment through a greenfield area.

Factors that influence geometric design of the dedicated transit lines and its interface with the normal road system may include:

1) Access is controlled through interchanges or signalised intersections with measures in place to provide a high level of priority for public transport services.
2) Development of the infrastructure may be staged to allow construction in new housing or commercial areas when development of these areas is well advanced and passenger demand is at a sufficient level.
3) Transit stations are often located near large commercial precincts where bus terminals provide efficient interchange between feeder services and the mainline services.
4) Transit lines may be designed to allow for future upgrading to other mass transit technologies and this would impose some specific requirements with respect to vertical and horizontal geometry (grades and curves) and vertical and lateral clearances.

Cross section requirements vary depending on the transit service and has not been covered under this Manual. Reference should be made to specific manuals and guidelines for detailed design of the cross-sectional elements.

4.15 Bus Stops, Lay-bys, and Parking Bays

Lay-bys clear of the lanes for through traffic considerably reduce the interference between buses, taxis, and other traffic. Bus lay-bys serve to remove buses from the traffic lanes and parking bays are spaces provided for taxis and other vehicles to stop outside the roadway.

Pedestrian crashes at bus stops are common but can be reduced significantly by good design of the bus stop area. Ideally a bus stop or a lay-by should be designed as a short auxiliary lane with adequate entry and exit tapers and separated from the travelled way by means of a separator from the through lanes. A further safeguard is the use of pedestrian guardrails to prevent passengers from crossing the road until they are well clear of the bus and have a clear vision of the road.

The location and design of lay-bys should provide ready access in the safest and most efficient manner possible. To be fully effective, lay-bys should incorporate:

1) A deceleration lane or taper to permit easy entrance to the loading area.
2) A standing space sufficiently long to accommodate the maximum number of vehicles expected to occupy the space at one time.
3) A merging lane to enable easy re-entry into the through-traffic lanes.

The deceleration lane should be tapered at an angle flat enough to encourage the bus or taxi operator to pull completely clear of the through lane as in Figure 4.12. A taper of 10:1, longitudinal to transverse, is a desirable minimum.

A loading area should provide 15 m of length for each bus. The width should be at least 3.5 m and preferably 4.0 m. The merging or re-entry taper may be somewhat more abrupt than the deceleration taper but, preferably, should not be sharper than 6:1.

The total length of lay-bys for a two-bus loading area of minimum design should be as shown in Figure 4.12 and in the Standard Detail Drawings. These lengths of lay-bys expedite bus manoeuvres, encourage full compliance on the part of bus and taxi drivers, and lessen interference with through traffic. Sufficient footpaths should also be provided at bus lay-bys.

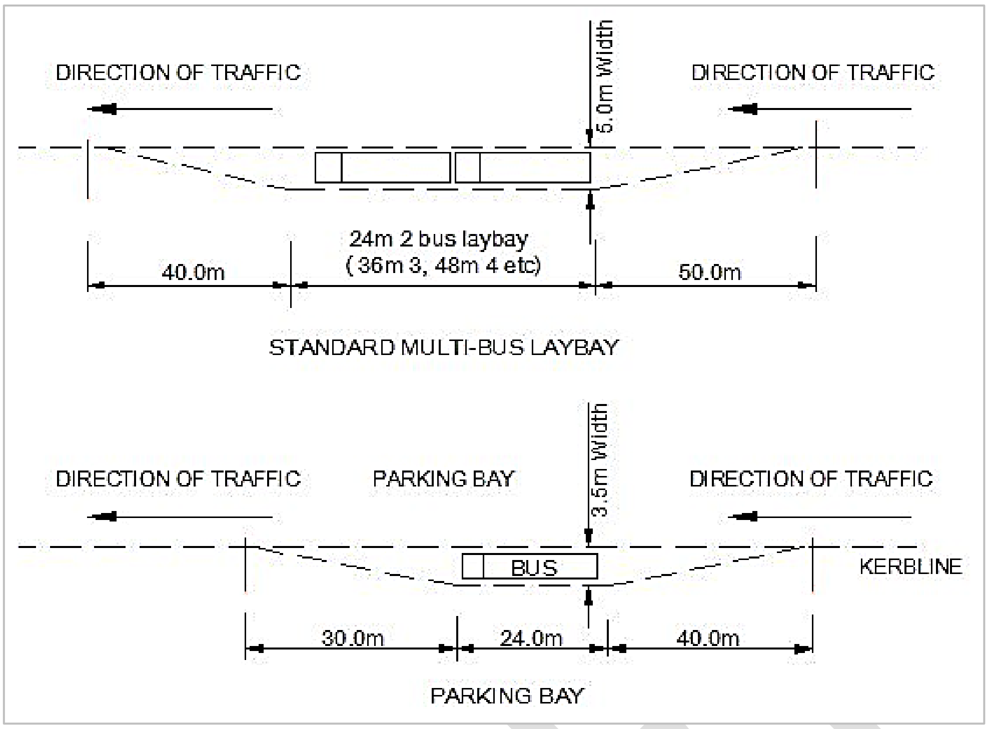


Figure 4.12: Bus lay-bys and parking bays

Bus stops should not be located immediately in advance of an intersection because of the restriction of sight distance that this would impose on drivers approaching the intersection. On the other hand, they should not be too far away because many passengers may want access to the roads forming the intersection. Ideally the bus stop should, except near roundabouts, be located after the intersection but not more than 50 m from it.

4.16 Refuge Islands

Refuge islands can be used to help pedestrians to cross particularly wide or busy roads. They allow pedestrians to cross one direction of traffic flow at a time without seriously affecting traffic capacity, especially where the traffic through a junction is controlled by traffic signals.

In mountainous areas or in rolling topography, restricted sight distance does not always allow pedestrians enough time to cross the road safely. In such areas, if the minimum sight distances for pedestrians crossing rural roads shown in Table 4-12 cannot be attained, refuge islands should be provided. In problem areas, properly designed refuge islands are considered a safe alternative. Where these are used, pedestrian risk is reduced by 50 %.

Refuge islands should be at least 1.5 m wide (preferably 2.0 m) and may take the form either of raised islands or of marked refuges with oblique parallel lines. If raised, the sides should be semi-mountable. In addition, the approaches to the refuge island should be tapered and clearly demarcated with the necessary road signs and markings. The road markings together with retroreflective road studs should channelise vehicular traffic away from the refuge island. A ‘pass this side’ (left) should also be displayed prominently to safeguard drivers.

Table 4-12: Pedestrian sight distances (m)

Design speed (km/h)	Cross section		
	2-Lane	3-Lane	4-Lane
60	85	130	170
70	100	150	200
80	115	170	230
90	130	190	255
100	140	215	285
110	155	235	310
120	170	255	340

4.17 Pedestrian Overbridge

The minimum clear width of a pedestrian bridge should be 1.8m. This width is adequate for the passage of up to 300 people per hour and allows two wheelchairs to pass.

For shared bicycle/pedestrian bridges, the minimum width is 3.0m. Where the volumes of pedestrians and/or cyclists is high, the two functions should be segregated and the appropriate width for each function shall be allowed.

Care is needed in the design process to ensure that the pedestrian overbridge provides a more direct and attractive route than attempting to cross at grade. Even the provision of pedestrian barrier in the central reserve will not prevent pedestrians crossing at grade if it is perceived that the at-grade route is more direct.

4.18 On-street Parking

On-street parking is typically provided in urban and peri-urban areas but is not necessarily required. On-street parking can help visually narrow the street in places to assist in conveying the surrounding context for the segment.

On-street parking can be either parallel or angled.

Parallel parking offers the least impediment to the orderly and regular flow of traffic along the road, and it requires a lesser width of roadway. While it limits the number of vehicles parked along the kerb (compared with angled parking), it has the advantage of minimising crashes associated with parking and unparking manoeuvres.

Entering and leaving parking spaces from a through traffic lane introduces slow-moving and reversing movements that may conflict with the traffic flow. Hence, there should desirably be 0.5 m clearance from the nearest moving traffic lane.

Angled parking provides more capacity; however, it presents a greater hazard to road users than parallel parking. Change from parallel to angled parking through redesign must be justified on grounds of optimised use of road space and no increase in crash risk.

Provide for vehicle overhang within the furnishing zone for all angled parking locations.

When designing parking locations for freight loading areas, it is important to consider both the delivery vehicle size and how the vehicle loading/unloading is done.

Motorcycle parking zones are normally provided in groups according to demand. Conversion of parking spaces can provide the required facilities. Use of irregular spaces and undersize remnants should also be considered. Where cars occupy motorcycle spaces, installation of kerbing may be required. The minimum size of a motorcycle space is 2.5 m x 1.2 m.

Table 4-13: On-street parking space dimensions

Typical parking	Space requirement	
	Width	Length
Parallel parking	2.4m	6m
Angled parking at 45 degrees [a]	2.4m to 2.7m	5.4m

Notes:

Parallel parking not ideal for disabled parking. On road section with parallel parking, considerations should be made to provide dedicated spots of angled parking for the disabled. The width of parking space should be increased by 0.40 m for easy manoeuvrability.

4.19 Widening

4.19.1 Widening on curves and embankments

The use of long curves of low radii should be avoided where possible because drivers following the design speed will find it difficult to remain in the traffic lane. However, widening of the carriageway where the horizontal curve is tight is usually necessary.

This is required because:

1) A vehicle travelling on a curve occupies a greater width of pavement than it does on a straight. At low speeds the rear wheels track inside the front wheels, and the front overhang reduces the clearance between passing and overtaking vehicles. (At high speeds the rear wheels track outside the front.).
2) Vehicles deviate more from the centerline of a lane on a curve than on a straight.

Widening ensures that the rear wheels of the largest vehicles remain on the road when negotiating the curve and, for two-lane roads, it ensures that the front overhang of the vehicle does not encroach on the opposite lane. Widening is also important for safety reasons.

The degree of curve widening required depends on the radius of the curve, the width of the lane on the straight road, the length of the vehicle plus other factors such as overhang of the front of the vehicle, and wheelbase and track width. However, the design clearly cannot be tailored to a particular vehicle and the same considerations should be used as for the design of the class of road being designed. Curve widening is required on all standards of roads and should be sufficient to cater for the design vehicle. Table 4-14 shows the values to be used.

Table 4-14: Widening on curves and high fills.

Radius of curve (m)	Curve widening single Lane (m)	Curve widening: two lanes (m)[1]	Fill widening	
			Height of fill (m)[4]	Amount (m)
>250	none	none	0.0-3.0	none
120- 250	none	0.6	3.0- 6.0	0.3
60-120	none	0.9	6.0 - 9.0	0.6
40-60	0.6	1.2	Over 9.0	0.9
20-40	0.6	1.5	Over 9.0	0.9
<20	See Section 5.12 on hairpin curves.			

Notes:

i. *Curve widening shall generally be applied to both sides of the roadway. It should start at the beginning of the transition curve and be fully widened at the start of the circular curve.*

ii. *Curve widening is generally not applied to curves with a radius greater than 250 m regardless of the design speed or the lane width.*

iii. *Vehicles need to remain centred in their lane to reduce the likelihood of colliding with an oncoming vehicle or driving on the shoulder.*

iv. *Sight distances should be maintained as discussed above.*

v. *Widening on high embankments is recommended for design classes A, B through to C. The steep drops from high embankments unnerve some drivers and the widening is primarily for psychological comfort although it also has a positive effect on safety. Widening for curvature and for high embankments should be added where both situations apply.*

vi. *Widening should transition gradually on the approaches to the curve so that the full additional width is available at the start of the curve.*

vii. *Although a long transition is desirable to ensure that the whole of the travelled way is fully usable, for the improvement of existing roads this results in narrow pavement slivers that are difficult, and correspondingly expensive, to construct in existing roads. In practice curve widening is thus applied over no more than the length of the super-elevation runoff preceding the curve.*

viii. *For ease of construction, the widening is normally applied only on one side of the road. This is usually on the inside of the curve to match the tendency for drivers to cut the inside edge of the travelled way.*

[1] The height of fill is measured from the edge of the shoulder to the toe of the slope.

ix. *Widening is provided to make driving on a curve comparable with that on a tangent. On older roads with narrow cross-sections and low design speeds and hence sharp curves, it is possible that widening may not always have been provided, due to the inconvenience to widening the surfacing of a lane. Where the road has to be rehabilitated and it is not possible to increase the radius of curvature, the designer should consider the need for curve widening.*

x. *In urban areas curve widening is required to cater for buses and trucks (Table* 4-15*). For larger vehicles, there are several swept path analysis programs that are* available.

Table 4-15: Widening for buses and trucks.

Radius of curve (m)	Single unit bus or truck, widening. (m)	19m semi-trailer (m)
40	1.03	
50	0.82	
60	0.71	1.27
70	0.59	1.03
80	0.52	0.91
100	0.41	0.71
120	0.36	0.63
140	0.32	0.56
160	0.28	0.49
180	0.24	0.42
200	0.24	0.35

4.19.2 Fill widening

If fill widening is required, benching should be applied (Figure 4-10) which modifies the road width independent of travelled road width.

Fill widening should be considered:

1) To maintain proper compaction of the road in cases where there are high fills and where edge compaction cannot be achieved to the required specification.
2) To give sufficient space for roadside furniture such as guardrails, traffic signs and guideposts.

In such cases, the 0.6 m of fill widening for fill height greater than two metres is recommended. Fill height in excess of 6.0 m should be avoided, and if unavoidable potential stability problems should be investigated.

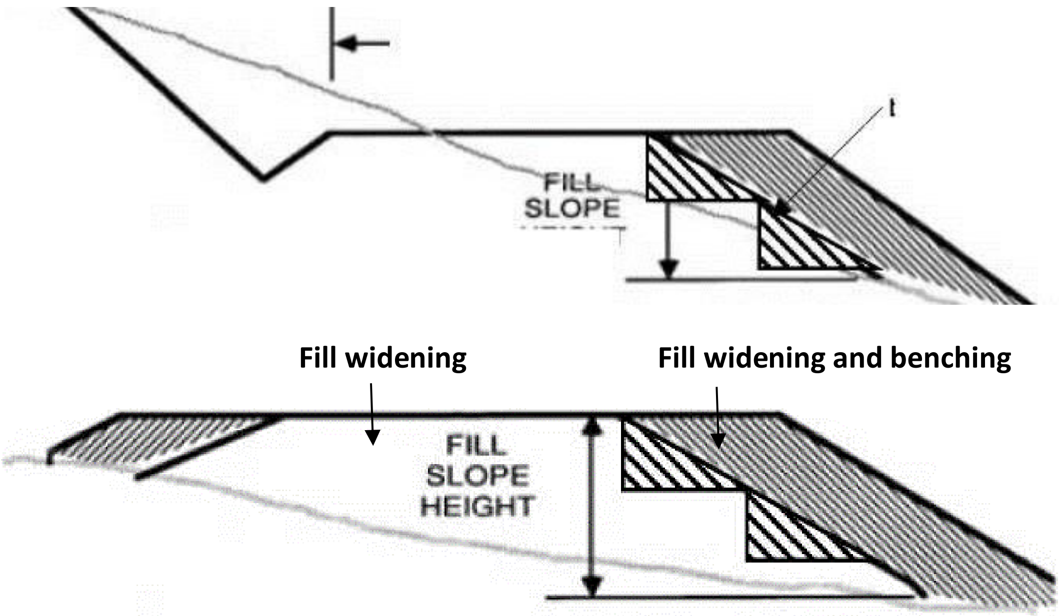


Figure 4-10: Fill widening and benching.

Cut slopes are inherently more stable than fill slopes. The designer should try to minimize fill slope length by "pushing" the alignment into the hill side to minimize erosion. If economical, this will result in longer cut slopes and add slight to moderate cuts at the centreline. The result will be a moderate fill slope as shown in Figure 4-10 with no additional fill widening required.

4.19.3 Widening for vehicle restraint systems (VRS)

The introduction of a safety barrier adjacent to the carriageway shall only be considered where the elimination of all hazards w*i*thin the clear zone is not reasonably practicable in terms of engineering, economic, environmental or sustainability considerations (see Section 4.2)

The ideal position of a VRS in relation to the edge of the road will depend, inter alia, on the type of device being considered and, on the type, and location of hazards being protected. In general, the designer should provide the maximum width of level verge or central reserve in front of the system as possible to optimise the opportunity for an errant vehicle to regain control without striking the VRS.

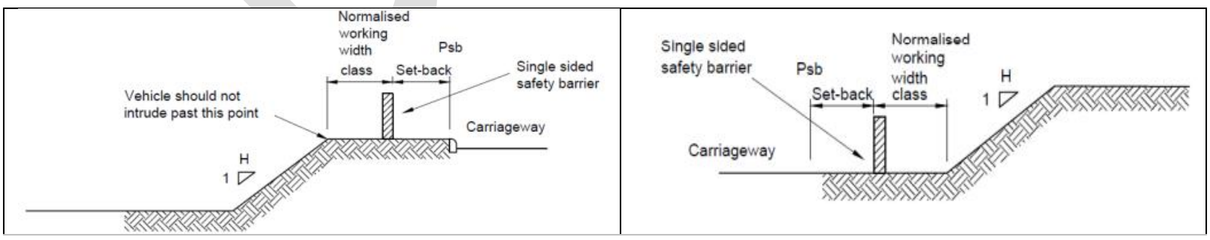


Figure 4-11: Set back and working widths for VRS (a) on fill (b) in cut.

Widening should allow for both the setback width and working width of the VRS.

4.19.3.1 Setback width

Physical objects such as VRS immediately adjacent to the edge of the carriageway can result in drivers reducing speed and positioning their vehicles away from the obstruction. The purpose of the setback is to provide a lateral distance between the VRS and the carriageway which reduces the effect of the safety barrier on driver behaviour.

Table 4-16: Setback widths for VRS installation

Location	Desirable minimum set back (m)
Verges with no adjacent sealed or hard shoulder	1.2[a]
Verges with adjacent sealed or hard shoulder	0.6[b]
Median (central reserves)	1.2[a]

Notes:

a. *For design speeds of 80km/h and less, the width can be relaxed to 600m. Further, on locations with physical restraints such as a structure, setback width can be 1.0 m.*

b. *Can be relaxed to 0.4 m where it is considered necessary to position the VRS away from the edge of an existing embankment in order to provide support to the foundation.*

Set-back greater than the desirable minimum values should be provided in the following circumstances:

1) At verges for roads where continuous or near continuous VRS is proposed to prevent a driver from mounting the verge in an emergency;
2) Where use of the minimum set-back in central reserves can result in the paved width being closer than 600mm to the VRS; and
3) To achieve a smooth alignment with a parapet.

On central reserves where there are no obstructions and there is only one double sided deformable safety barrier between carriageways, the minimum set-back on both sides of the safety barrier shall be as stipulated in Table 4-16 but also no less than the working width of the safety barrier minus the actual width of the safety barrier.

4.19.3.2 Working width

Working width is defined as the distance between the barrier side facing the traffic before impact of the road restraint system and the maximum dynamic lateral position of any major part of the VRS. It is therefore a function of the deflection behaviour of the safety barrier under impact. Rigid barriers do not allow for deflection. For semi-flexible barriers, a working width of minimum 1.0 m should provide. This distance should be measured from the post.

4.19.4 Additional width for mixed traffic

For the lower road standards, modifications to the standards are made for high volumes of non-motorised vehicles, motorcycles, pedestrians (and other forms of intermediate transport). CEF are defined for this purpose as shown in Section 3.9.5 and the modifications to specifications are summarised in Table 4.5 where it should be noticed that the increases in width are sometimes for the shoulders and not for the carriageways.

The modifications are not possible on escarpments. In mountainous terrain, they are only possible along relatively flat sections. In these circumstances the CEF values are only likely to be high where the population is high, and this is likely to be defined as a populated area where widening is justified for that reason alone.

Table 4.5: Road Width Adjustments for CEF greater than 300 AADT

Standard	AADT	Modification
Paved	>10,000	None
Paved	>10,000	None
Paved	1,000 – 3,000	Shoulder width increased to 2.5 m each side
Unpaved	1,000 – 3,000	Increase width to 11.0m
Paved	300-1,000	Shoulder width increased to 2.0 m each side
Unpaved	300-1,000	Increase width to 10.0m

4.19.5 4.7.6 Additional Width Based on Surrounding Land Use

The more populated areas in village centres are not normally defined as ‘urban’, but in any area having a reasonable sized population, or where markets and other business activities take place, the geometric design of the road needs to be modified to ensure good access and to enhance safety. This is done by using:

1) A wider cross section.
2) Specifically designed lay-bys for passenger vehicles to pick up or deposit passengers.
3) Roadside parking areas.

The additional width depends on the status of the populated area that the road is passing through. If the road is passing through a town or a larger populated area, an extra carriageway of 3.5 m width is provided in each direction for parking and for passenger pick-up and a 2.5 m pedestrian footpath is also specified. The latter is essentially the shoulder. In addition, the main running surface is paved and is increased to at least 7.0 m wide if the AADT is 1000 - 3000. However, complete village design for road safety is essential and is described in detail in Chapter **Error! Reference source not found.**.

When passing through a village, a 2.5 m paved shoulder is specified but no additional footpath although one could easily be provided if required.

4.19.6 4.7.8 Edge Marking

At night and during inclement weather it is important that the driver should be able to distinguish clearly between the shoulder and the lane. Edge marking is a convenient method of indicating the boundary between the lane and the shoulder. Rumble strips (Refer to RDM Volume 6 Part 4 Other Traffic Control Facilities) can also be used and have been shown to reduce the rate of run-off-road incidents by up to 25%.

4.20 Side Slopes and Back Slopes

Three regions Figure 4-12) of the roadside are important when evaluating safety: -

1) The hinge point. Rounding at the hinge point at the top of the slope can significantly reduce the hazard potential. Similarly, rounding at the toe of the slope is also beneficial;

2) Embankment or fill slopes. Slopes parallel to the flow of traffic may be defined as recoverable, non-recoverable, or critical depending on their slope, Table 4-17.
3) Toe of the slope (intersection of the fore slope with level ground or with a back slope, forming a ditch).

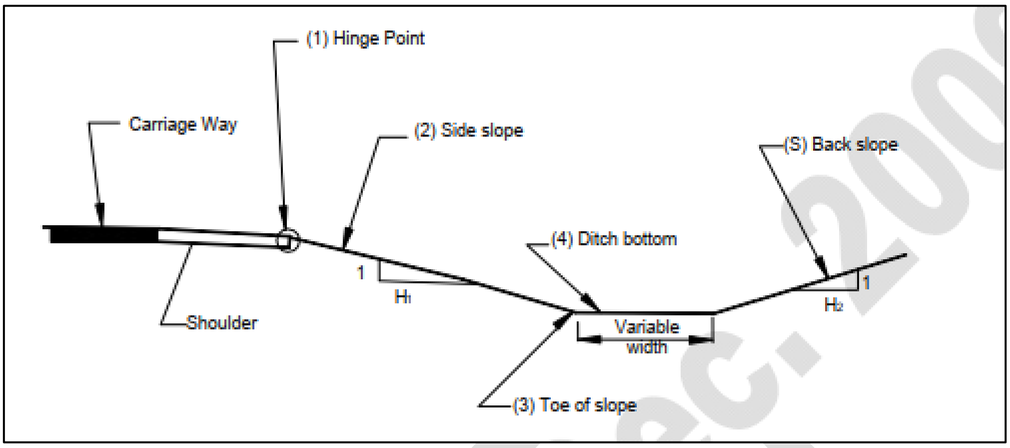


Figure 4-12: Details of the road edge

The selection of a side slope and back slope is dependent on safety considerations, height of cut or fill, and economic considerations. The guidance in this chapter is mainly applicable to new construction or major reconstruction. On maintenance and rehabilitation projects, the primary emphasis is placed on the roadway itself. Because of environmental impacts or limited road reserve it may not be cost-effective or practical to bring these projects into full compliance with these side slope recommendations.

The slopes of the sides of the road prism should be shallow for reasons of safety; slopes of 1:4 are considered the steepest acceptable. If steeper slopes are necessary, then vehicle restraint systems might be needed, and the design of the slope will need to take account of the geotechnical properties of the material (**Road Design Manual Volume 4 Bridges and Retaining Structures Design: Part 4.2 – Retaining Structures Design)**.

Table 4-17: Recoverable and non-recoverable slopes

Type of slope	Definition
Recoverable slopes	Recoverable slopes include all embankment slopes of 1:4 or flatter. Motorists who encroach on recoverable slopes can generally stop their vehicles or slow them enough to return to the roadway safely. For higher traffic volumes, side slopes should be designed with a 1:6 ratio. Although the influence of back-slopes is generally less than that of fore-slopes, a ratio of 1:3 or flatter is recommended. Fixed obstacles such as culvert head walls should not extend above the embankment within the clear zone distance.
Non-recoverable slopes	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

Type of slope	Definition
	roadway easily. Vehicles on such slopes 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 should extend beyond the slope, and a clear run-out area at the base is desirable.
Critical slope	A critical slope is one on which a vehicle is likely to overturn. Slopes steeper than 1:3 generally fall into this category.

Table 4-18 indicates the side slope ratios recommended for use in the design according to the height of fill and cut, the material, and the practical experience of the costs of construction. It will be noted that, with the single exception of roads in areas of black cotton soils, the recommended slopes are often too steep to meet the recommendations for adequate safety. Achieving a good safety design is clearly a function of overall cost and is only likely to be viable for the highest classes of road.

Table 4-18: Slope ratio table – vertical to horizontal

Material	Height of Slope (m)	Side Slope (V:H)		Back Slope
		Fill	Cut	V:H
Earth Soil	0.0 – 1.0	1:4		1:3
	– 2.0	1:3		1:2
	>2.0	2:3		2:3
Compacted laterite gravel	0-1m	1:3		1:2
	1-3m	1:2		2:3
	>3m	2:3		1:1
Strong Rock	0.0 – 2.0	4:5	1:2	2:1
	>2.0	1:1		4:1
Weathered Rock	0.0 – 2.0	2:3		2:1
	>2.0	1:1		3:1
Decomposed Rock	0.0 – 1.0	1:3		1:3
	– 2.0	1:2		1:2
	>2.0	2:3		2:3
Black Cotton Soil (expansive clays)	0.0 – 2.0	1:6	-	-
	>2.0	1:4		

This table should be used as a guide only, particularly because applicable standards in rock cuts are highly dependent on costs. Also, certain soils that may be present at subgrade level may be unstable at 1:2 side slopes and therefore a higher standard will need to be applied for these soils. The detailed design stage should include a geotechnical analysis which will indicate the steepest batters appropriate for slopes using in-situ material. Slope configuration and treatments in areas with identified slope stability problems should be addressed as a final design issue.

4.21 Roadside Drains

The choice of side drain cross-section depends on the required hydraulic capacity, arrangements for maintenance, space restrictions, traffic safety and any drainage requirements relating to the height between the crown of the pavement and the drain invert.

Side drains should be avoided in areas with expansive clay soils such as black cotton soils. Where this is not possible, they should be kept at a minimum distance of 4-6 m from the toe of the embankment, dependent on functional classification.

For design of side drains and drains on urban roads, reference should be made to **Section 6.4** and **Section 7.3** of the **Road Design Manual: Volume 2 - Hydrology and Drainage Design - Part 2: Drainage Design of Highways, Rural and Urban Roads.**

4.22 Typical Cross-sections

4.22.1 Typical Cross-sections for inter-urban and rural roads

Typical cross-section applicable to inter-urban, international and rural dual carriageway roads is shown in Figure 4-13 below. Chapter 8 provides the designer with geometric design details applicable to the design standards set for inter-urban and rural roads in Chapter 2 Section 2.6.

Each road should be designed in accordance with its specific requirements and no designs will be exactly the same. Thus, instead of prescribing exact design parameters, guidelines are given on ranges within which acceptable designs can normally be accommodated. This provides the required flexibility to adapt the design to the different traffic volumes and compositions and terrains as well as the provision of access with limited budgets.

The design standards given in Chapter 8 must be adopted as guidance.

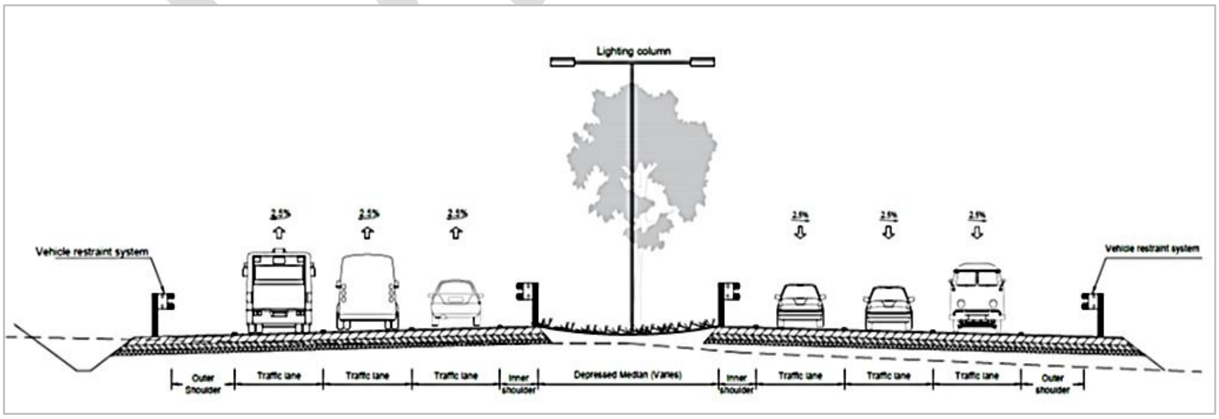


Figure 4-13: Basic cross section components – interurban international road

Figure 4-14, on the other hand, shows the typical cross-sectional components of a rural single carriageway road.

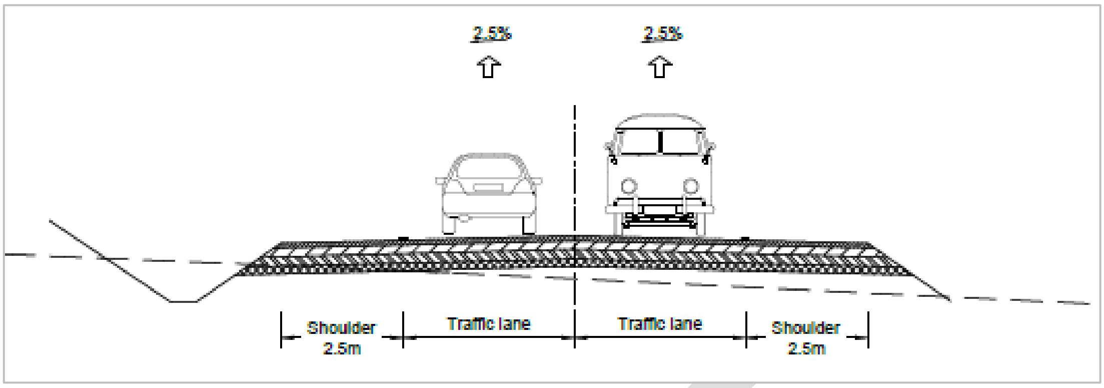


Figure 4-14: Basic cross section components – rural single carriageway

4.22.2 Urban Roads

Typical cross-section applicable to urban roads are given in is shown in Figure 4-15, Figure 4-16 and Figure 4-17 below, for arterial, collector and local roads. Chapter 8 provides the designer with geometric design details applicable to the design standards set for urban arterial, collector and local roads in Chapter 2 Section 2.6. These standards are to be used as guide by the designer.

In any typical urban setting there are many options available to the designer in selecting the cross-section to apply. There will be no single right answer to cross section selection and the designer must aim to meet the demands of functionality and economy. Experience shows that careful consideration of functional classification and context sensitive design in terms of the complete street approach, will determine the selection of the elements required in a particular cross section.

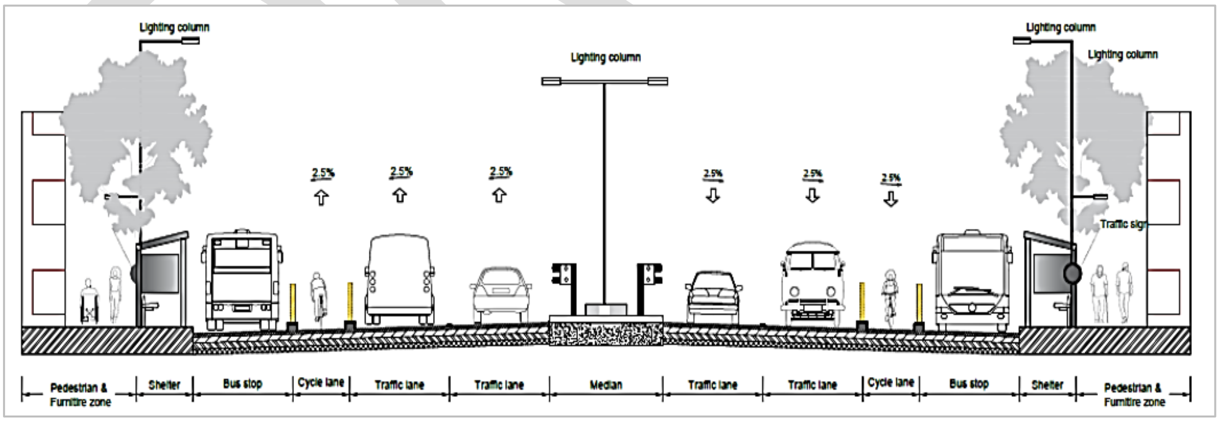


Figure 4-15: Basic cross section components – urban arterial road

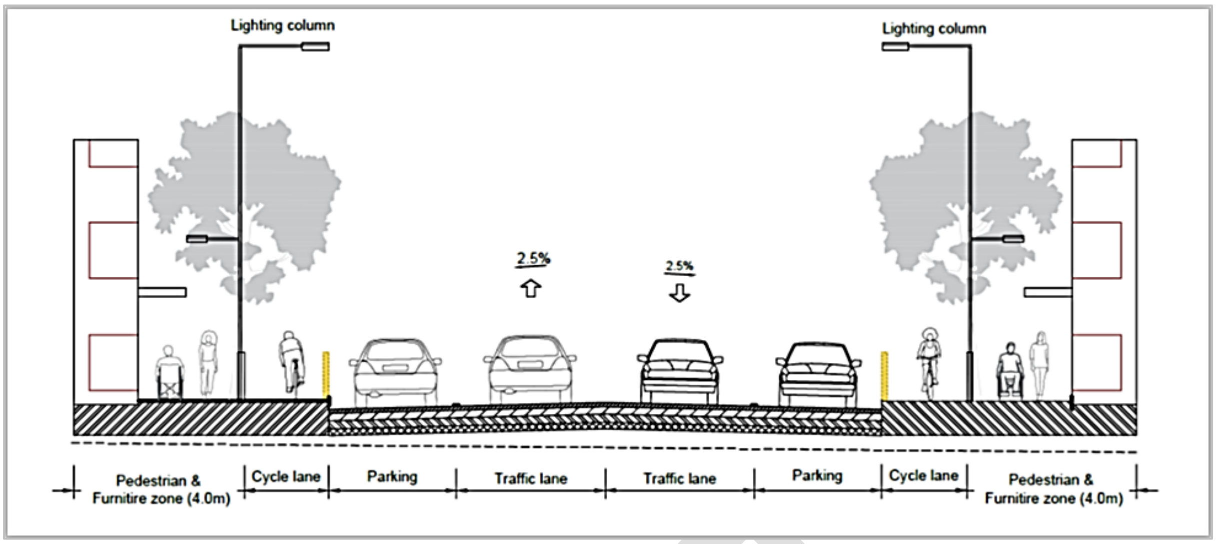


Figure 4-16: Basic cross section components – urban collector road

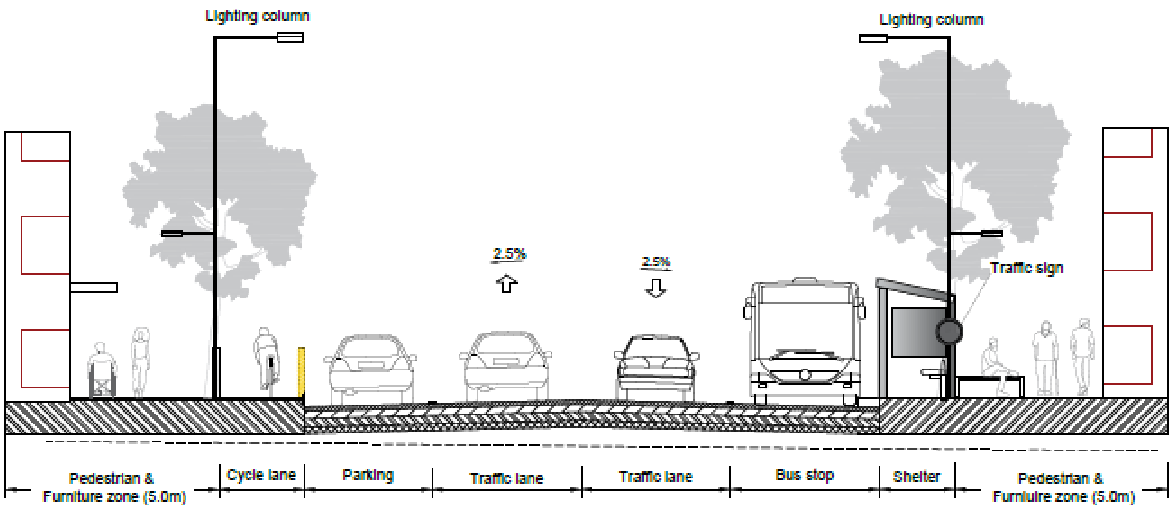


Figure 4-17: Basic cross section components – urban local road

5 Design of Horizontal Alignment

5.1 General

The objective of the design of the alignment is to provide a safe road which can be driven at a reasonably constant speed. It consists of a series of straight sections (tangents), circular curves and transition curves (spirals) between the tangents and the circular curves. All need to be designed to fit together 'harmoniously and smoothly'; sharp changes in the geometric characteristics of both horizontal and vertical alignments must be avoided.

On all roads except those with the lowest design speed (i.e., the lowest classes) a vehicle negotiating a horizontal curve requires an inward radial force to provide the necessary centripetal acceleration to counteract the centrifugal force and prevent sliding. This radial force is partly provided by the sideways friction between the tyres and the road surface, but this is usually insufficient. An additional force is provided from the component of the vehicle's weight that acts towards the centre of the curve when the vehicle is tilted by means of super-elevation. The force depends on the speed of the vehicle, the radius of the horizontal curve and the degree of tilting, or super-elevation.

A transition curve, whose radius changes continuously between a straight section of road and the neighbouring connected circular curve, is used to reduce the abrupt introduction of centripetal acceleration as vehicles travel between the superelevated ends of each straight section of the road.

Transition curves are not normally required when the radius of the horizontal curve is large, and they are not normally used on the lower classes of road.

The factors having an impact on the design of the horizontal alignment include;

1) Physical constraints such as the general shape of the topography, including the presence of watercourses, land use, and man-made features. Geophysical conditions such as expansive clays and so on should also be considered.
2) The effect the road may have on the environment such as its effect on ecologically sensitive areas, adjacent land use and community impacts.
3) Cost of land acquisition, construction, and maintenance.
4) Road user costs.
5) Safety on the basis of human factor considerations, context sensitive design and consistency of alignment.
6) Highway classification and design policies.

5.2 Elements of a Circular Curve

Basic equation for a circular curve is:

Equation 5-1: Horizontal curve formulae

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

Where:

R = radius (m)
V = speed (km/h)
e = superelevation rate (decimal)
f = side friction factor (decimal) (Section 3.3)
The elements of a circular curve are illustrated in Figure 5-1.

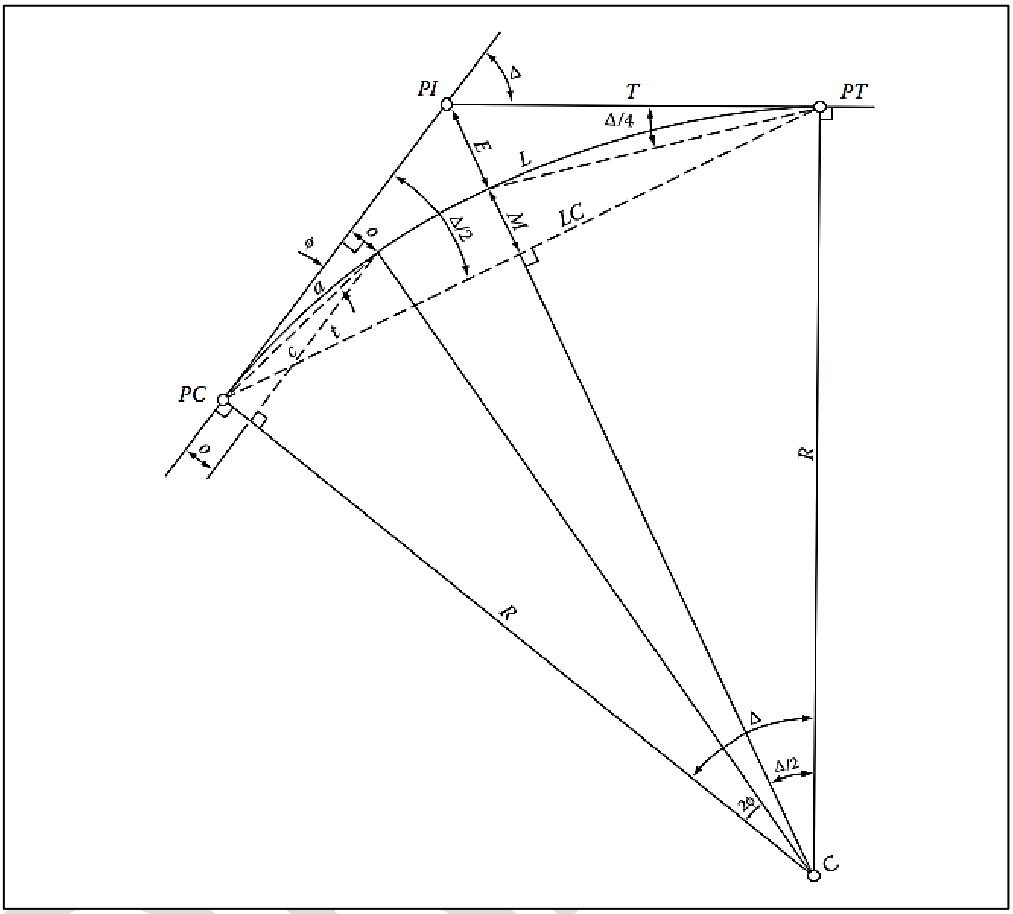


Figure 5-1: Elements of a curve

Where:

Δ = deflection angle (degrees)

T = tangent distance (m) = distance from point of intersection (PI) to the beginning of the curve (BC), that is, point of curvature (PC) × tangent curve (TC) or to the end of curve (EC), that is, point of tangency (PT) × curve tangent (CT)

L = length of curve = distance along curve from BC to EC

R = radius of curvature (m)

m = external distance (m) = PI to midpoint of curve

C = centre of curve

LC = long chord (m) = BC to EC

M = middle ordinate (m) = midpoint of arc to midpoint of long chord

a = length of arc from BC to any point on the curve (m)

c = length of chord from BC to any on the curve (m)

ϕ = deflection angle from BC to any point on curve (degrees)

t = distance along the tangent from BC to any point on the curve (m)

o = tangent offset to any point on curve

$$T = R\tan(\Delta/2) \qquad c = 2R\left(\sin\frac{90a}{\pi R}\right)$$

$$L = \frac{\Delta}{360}2\pi \qquad \cos\phi = \frac{R-o}{2R}$$

$$m = \frac{R}{\cos\left(\frac{\Delta}{2}\right)} - R = T\tan\Delta/4 \qquad t = R\sin 2\phi$$

$$LC = 2R\left(\sin\left(\frac{\Delta}{2}\right)\right) = 2T\left(\cos\left(\frac{\Delta}{2}\right)\right) \qquad o = c\sin\phi$$

$$M = R\left(1-\cos\left(\frac{\Delta}{2}\right)\right) = E\cos\left(\frac{\Delta}{2}\right) \qquad o = R - \sqrt{R^2 - t^2}$$

$$a = \frac{\Delta\pi R}{90} \qquad o = R(1-\cos 2\phi)$$

Figure 5-2: Curve formulae

5.3 Minimum Horizontal Radius of Curvature

The minimum horizontal radius of curvature, R_{min}, for a particular speed is:

Equation 5-2: Minimum horizontal radius of curve

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

Where,

V_D = vehicle speed(km/h)

e = maximum super-elevation (%/100)

f = side friction coefficient (Section 3.3.)

The minimum radii of curvature for different speeds can be calculated using Equation 5-1 but recall that the 'design speed ''used to calculate the design curvature, amongst other things, is normally higher than the speed that most drivers would normally use. It is a conventional design index used for calculating stopping distances and thereby the required sight distances for example, hence the values calculated could be viewed as veering on the side of safety. When used for this calculation of R_{min} and using the standard values of super-elevations and pragmatic coefficients of friction, the results shown in Table 5-1 for paved roads are obtained and Table 5-2 for Class A roads and Table 5-3 for unpaved roads.

As the radius increases, the crash rate decreases hence the minimum values should be used only under the most critical conditions and the deviation angle of each curve should be as small as the physical conditions permit.

For small changes of direction, it is often desirable to use a large radius of curvature. This avoids the appearance of a kink and reduces the tendency for drivers to cut corners. In addition, it reduces the length of the road segment and therefore the cost of the road provided that no extra cut or fill is required.

For unpaved roads the friction is usually considerably less than on paved roads. In these calculations it has been assumed that it is 80% of the value for paved roads but this is dependent on a tightly knit and dry surface of good quality gravel with no loose stones; in other words, a surface on which the speed limit could be maintained. A poorly bound surface with many loose particles has a very low value of friction and it must be assumed that on such a surface, a vehicle will be driven at a speed that is much lower than the normal speed or at a speed dictated by the sight distances and radii of curvature.

Table 5-1: Minimum radii for horizontal curves for paved roads (m)

Design speed (km/h)	**120**	**110**	**100**	**90**	**85**	**80**	**70**	**65**	**60**	**50**	**40**	**30**
Side friction factor (f)	**0.09**	**0.1**	**0.11**	**0.12**	**0.13**	**0.13**	**0.15**	**0.16**	**0.16**	**0.17**	**0.19**	**0.21**
Super-elevation = 4%	NA	NA	525	400	335	300	205	170	140	95	55	30
Super-elevation = 6%	755	595	465	355	300	265	185	155	130	85	50	26
Super-elevation = 8%	665	530	415	320	270	240	170	140	120	80	45	24

Table 5-2: Desirable radii for horizontal curves for rural arterial roads (m)

Limit	**Preferred**
R =1000m	R >3780m
It is preferable to use horizontal alignments that do not require super-elevation	

Table 5-3: Minimum radii for horizontal curves for unpaved roads (m)

Design speed (km/h)	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**85**	**90**	**100**
Side Friction Factor	0.19	0.165	0.15	0.14	0.12	0.12	0.10	0.10	0.10	0.09

Super-elevation=4%	15	35	65	115	175	255	355	415	475	610

5.4 Isolated Curves

The horizontal curvatures over a particular road section should be as consistent as possible. Long tangent roadway segments joined by an isolated curve designed at or near the minimum radius are unsafe. Long straight sections encourage drivers to drive at speeds more than the normally expected speed, hence sudden and unexpected sharp curves are dangerous. Good design practice is to avoid the use of minimum standards in such conditions.

For isolated curves, the minimum horizontal curve radii shown in the tables in Section 5.3 should be increased by 50%. This will usually result in the ability to negotiate the curve at a speed approximately 10 km/h higher than the average travel speed.

5.5 Overall Consistency of Horizontal Curves

Under normal circumstances sections of road will contain many curves whose radii are larger than the minimum radii specified in the design standards. For reasons of safety and driver comfort it is not advisable for two consecutive curves to differ in radius by a large amount even though both radii are greater than the minimum. Figure 5-3 shows the required ratio of radii for consecutive curves. Consecutive horizontal curves are defined as curves where the distance between the end of one and the beginning of the next is less than the radius of the larger curve. The best result will be achieved when the two radii are similar (labelled ‘very good’ in the diagram). If the ratio of radii falls outside the ‘good’ category but inside the ‘useable’ category some discomfort or inconvenience will be felt because of the increase in centripetal force when entering the tighter curve.

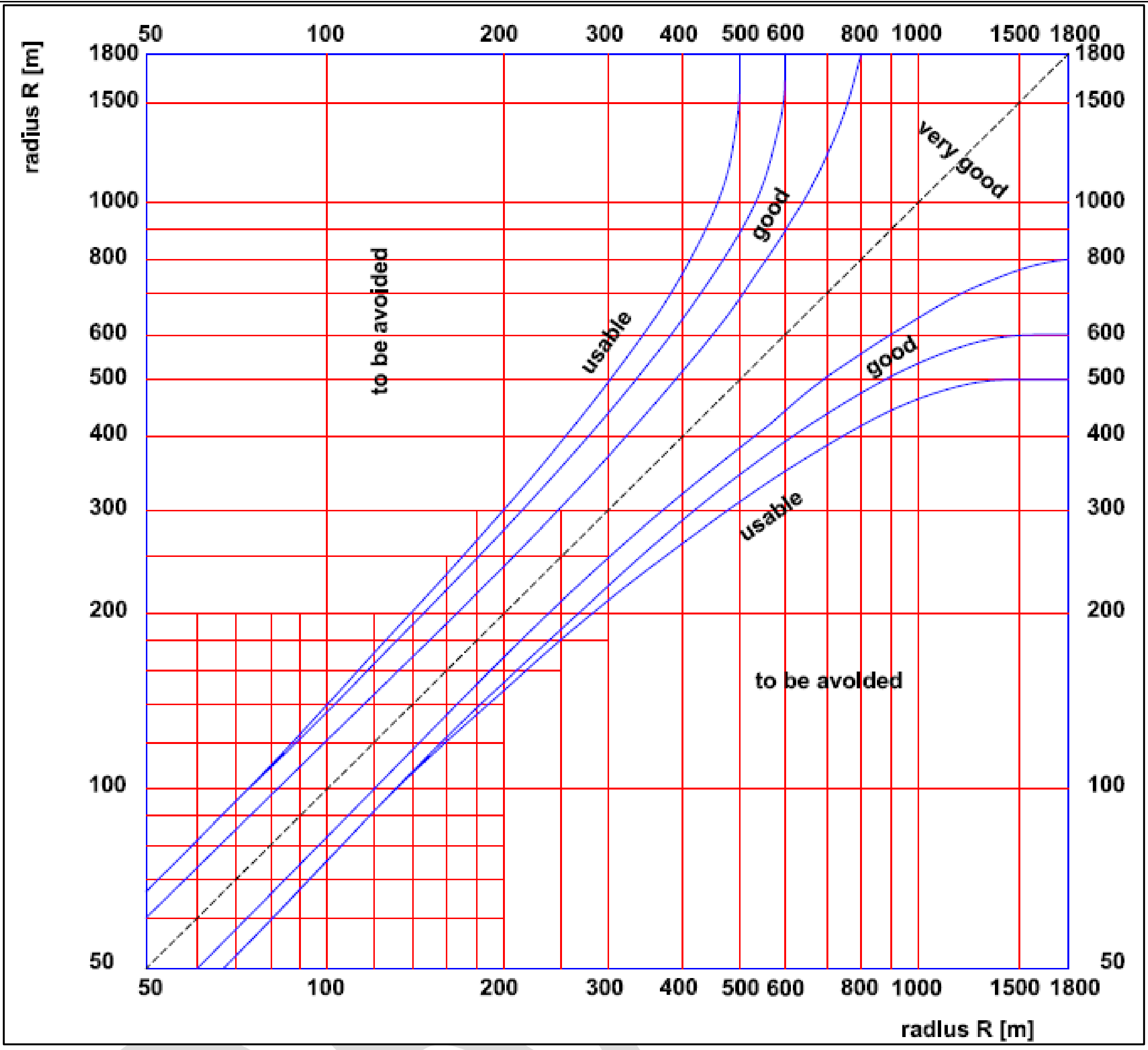


Figure 5-3: Comparison of Radii of Consecutive Horizontal Curves

(Source: German Road and Transportation Research Association, Cologne, Germany (1973). *Guidelines for the design of rural roads (RAL), Part II*.

5.6 Length of Tangent Sections

5.6.1 Design speeds greater than 120km/hr

Long tangents of twenty kilometres or more have crash rates similar to those on minimum length tangents. The lowest crash rate occurs in a range of lengths from 8 to 12 kilometres. This range is recommended for the maximum length of tangent on any route where the design speed is 120 km/h or more. However, a long tangent can cause serious problems of dazzle from approaching headlights which can be extremely dangerous and therefore, if there are large volumes of night traffic, tangent lengths shorter than this should be used. In some cases, it may even be necessary to consider including a median in the cross-section and planting shrubs in it or providing some other means of reducing glare and dazzle. Light from headlights strikes at a very flat angle, therefore a conventional fence is effective in reducing glare.

An alternative is sometimes recommended for the maximum length of tangents in which a winding alignment is used with tangents deflecting 5 to 10 degrees alternately from right to left. However, such 'flowing' curves restrict the view of drivers on the inside carriageway and reduce safe overtaking opportunities, therefore such a winding alignment should only be adopted where the straight sections are very long. In practice this only occurs in very flat terrain. The main problem is to ensure that there are sufficient opportunities for safe overtaking and therefore, if the straight sections are long enough, a semi-flowing alignment can be adopted at the same time. If overtaking opportunities are infrequent, maximising the length of the straight sections is the best option.

5.6.2 Design speeds less than 120 km/hr

At lower design speeds, maximum lengths considerably shorter than 8 km should be used. Drivers should be encouraged to maintain a normal speed which is closer to the design speed to reduce the possibility of an error of judgment leading to a crash. A maximum tangent length, measured in metres, of 15-20 times the design speed in km/hr, achieves this effect. For example, a design speed of 100 km/hr suggests that tangents should, ideally, not be longer than 1.5 – 2.0 km.

If the achievable maximum length of tangent across the length of the route is regularly greater than this guideline value, a higher design speed should be considered.

5.6.3 Minimum length of tangent

If the tangent is too short it will not be possible, in going from the shorter to the longer radius, for the driver to accelerate to the operating speed of the following curve within the length of the tangent; $T \leq T_{min}$ metres. Thus, tangent lengths are based on the following considerations.

1) The minimum length of tangent must allow for the run-off of the super-elevation of the preceding curve followed by the development of that for the following curve.
2) Minimum length of ($6xV_D$) m should also be adopted between circular curves following the same direction.
3) This distance should be calculated during detailed design, but a tangent length of less than 200 m is likely to prove inadequate.

Table 5-4: Minimum and maximum lengths of tangents

Design speed (km/h)	Minimum length of tangent (m)	Maximum length of tangent (m)
120	720	2400
110	660	2200
100	600	2000
90	540	1800
85	510	1700
80	480	1600
70	420	1400
65	390	1300
60	360	1200
50	300	1000
40	240	800
30	180	600

Note:

1. *The minimum and maximum lengths of tangents apply when a circular curve is followed by the development of the following curve.*

5.7 Length of Circular Curves

5.7.1 Minimum length of circular curves

For small changes of direction, it is often desirable to use a large radius of curvature. This avoids the appearance of a kink and reduces the tendency for drivers to cut corners. In addition, it reduces the length of the road segment and therefore the cost of the road provided that no extra cut or fill is required.

Minor roads: On minor roads, a minimum length of 300 m is a suitable criterion. If space is limited, this length may be reduced to 150 m but for deflection angles of less than 5°, the minimum length of the curve should be increased from 150 m by 30 m for each one degree decrease in the deflection angle.

Major roads: On major roads the minimum curve length in metres should be three times the design speed in km/h. The increase in length for decreasing deflection angle also applies to these roads. In the case of a circular curve without transitions, the length in question is the total length of the arc and, where transitions are applied, the length is that of the circular curve plus half the total length of the transitions.

For aesthetic reasons, on high-speed controlled access facilities, the desirable minimum length for curves should be double the minimum length described above or six times the design speed in km/h.

Table 5-5: Minimum length of circular curves

Design speed (km/h)	Minimum length of curve (m)[3]
110	330 [1]
100	300 [1]
90	300 [2]
80	300 [2]
70	300 [2]
60	300 [2]
50	300 [2]

<u>Notes</u>

1. *This value or the length of the circular curve plus half the total length of the transitions, whichever is the longest.*
2. *If space is restricted, this can be reduced by 30m for every degree less than 50 that the curve deflects.*
3. *The maximum length of circular curve is 800 – 1000 m.*

5.7.2 Maximum length of circular curves

The main problem introduced by a long curve is its effect on passing opportunities. On a left-hand curve, an overtaking manoeuvre would have to commence at a considerable distance behind the leading vehicle if the driver is to be sure that there are no approaching vehicles in the opposite lane that are close enough to threaten safe overtaking. Furthermore, the distance required for overtaking is greater than that on a right-hand curve. The length of a curve should not exceed 1,000 m, the preferred maximum length being 800 m.

5.8 Minimum Turning Radii

Buses, trucks, trucks with trailers and 4x4 utility vehicles require minimum design turning radii of 12.8 m, 13.7 m and 7.3 m respectively as shown in Section 3.11.

It is not possible to exclude any of these vehicle categories from the lower standard roads and, as a certain amount of tolerance is required for safe operations, the absolute minimum horizontal curve radius of 15 m is specified for all design standards.

For reasons of safety and ease of driving, curves near the minimum radius for the design speed should not be used at the following locations.

1) On high fills, because the lack of surrounding features reduces a driver’s perception of the alignment.
2) At or near vertical curves (tops and bottoms of hills) because the unexpected bend can be extremely dangerous, especially at night.
3) At the end of long tangents or a series of gentle curves because actual speeds will exceed design speeds.
4) At or near intersections and approaches to bridges or other water crossing structures.

5.9 Super-elevation

On all roads except those with the lowest design speed (i.e., the lowest classes), a vehicle negotiating a horizontal curve at or near to the design speed requires the additional force provided by the component of the vehicle’s weight that acts towards the centre of the curve when the vehicle is tilted by means of super-elevation. The required force depends on the speed of the vehicle, the radius of the horizontal curve and the degree of tilting, or super-elevation.

At any design speed, the degree of super-elevation that is necessary for curves of radii greater than the minimum is less than that required for the minimum radius. Thus, higher values than strictly necessary can be used ranging up to the maximum value (i.e., that value required for the minimum radius).

A tighter curve can be designed if higher values of super-elevation are used, but high values of super-elevation are not recommended especially if the friction is low, such as in locations where mud is likely to contaminate the road surface regularly. Also, high values are not recommended where mixed traffic and/or roadside development severely limit the speed of vehicles.

In the current design practice in Kenya, superelevation exceeding 6% is rarely used. In urban areas an upper limit of 4 % should be used except on a high-speed urban road where 6 % is acceptable. On loop ramps at interchanges, superelevation up to 8% may be adopted but this should be guided by considerations of driver expectation and comfort, and stability of typical vehicles within the section. Adoption of superelevation above 6% should be with approval of the Chief Engineer.

Either a low maximum rate of super-elevation or no super-elevation at all should be used within important intersection areas or where there is a tendency to drive slowly because of turning and crossing movements, warning devices, and signals. Super-elevation is, however, a requirement for all standards of roads and, whatever value is selected as the maximum, it should be applied consistently on a regional basis.

Superelevation deficiency (inadequate super elevation compared to what is specified) has potential impacts to safety as shown in Table 5-6. Inadequate superelevation can cause vehicles to skid as they travel through a curve, potentially resulting in a run-off-road crashes. Trucks and other large vehicles with high centres of mass are more likely to roll over at curves with inadequate superelevation.

Table 5-6: Potential safety impacts of superelevation deficiency

Safety & operational issues	Inter-urban	Rural	Urban
Run-off-road crashes	✓	✓	
Cross-median crashes	✓		
Cross-centre line crashes		✓	
Skidding	✓	✓	✓
Large vehicle rollover crashes	✓	✓	

Tables of super-elevation rates based on the radius of the curve; the design speed of the road are shown at the end of this chapter.

5.9.1 Super-elevation development

Superelevation development has two components: tangent runout and superelevation runoff. Tangent runout involves the rotation of the outside lane(s) of the cross-section from the normal camber, usually 2.5 per cent, to a zero crossfall. Superelevation runoff then continues this rotation until a crossfall equal to the slope of the normal camber across the full width of the travelled way is achieved. From this point further, the entire width of the travelled way is rotated until the full superelevation appropriate to the design speed and radius of curvature is achieved. The process is illustrated in Figure 5-4 for the case of rotation around the centreline.

The axis of rotation can, in fact, be located anywhere across the cross-section or even outside it. Selection of its location is dependent on the constraints under which the super elevation has to be developed. This is particularly so in the case of superelevation development in urban areas. These constraints could involve issues of drainage, aesthetics or fitting the cross-section to the topography. The problem to be solved is largely one of the location of the road edges relative to the ground line. In the case of a two-lane road, the axis of rotation would typically be located on the centreline. Other standard locations are the inside and outside edges of the travelled way.

The rotation of dual carriageways often takes place around the outer edge of the median island so that the median shoulders rotate in concert with the travelled lanes.

However, as in the case of the two-way road, no hard and fast rules can be laid down concerning the selection of the location of the axis of rotation. It could be at the centreline of the median, at the edge of the median or even having the entire cross-section rotating as a unit around one of other of the outer edges of the cross-section. Each case would have to be considered on its own merits.

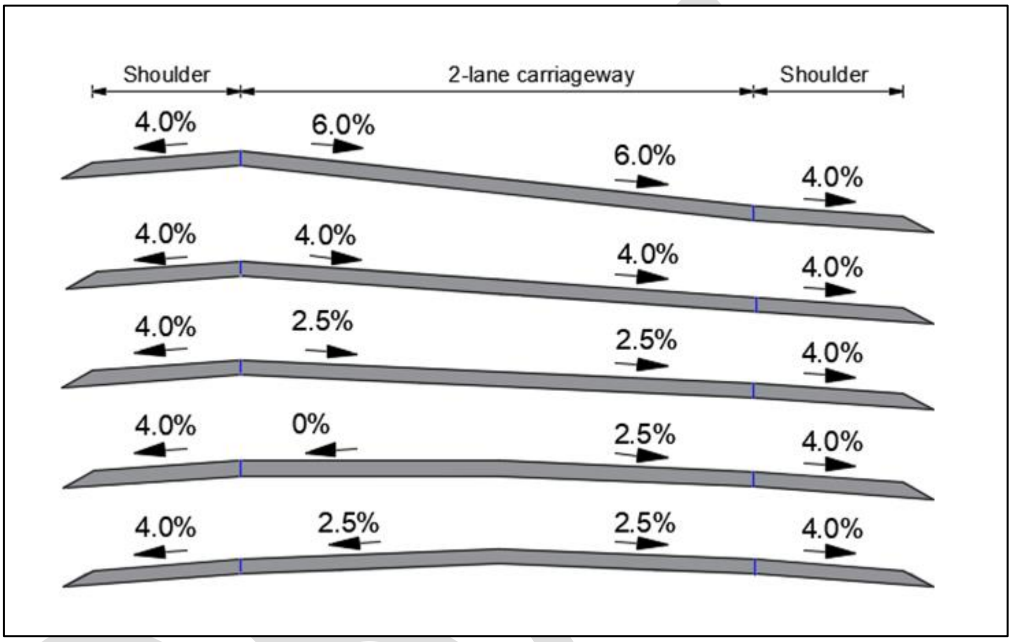


Figure 5-4: Superelevation of a two-lane carriageway about the centreline

5.9.2 Tangent runout

The tangent to spiral transition point (TS) is where the camber has been reduced to zero on the outside half of the carriageway. (Figure 5-5). The length of the tangent up to this point is called the tangent runout.

The length of tangent runout is determined by the amount of adverse cross slope and the rate at which it is removed. This rate of removal should preferably be the same as the rate used to effect the super-elevation runoff. Between the TS and SC (the super-elevation runoff) the travelled way is rotated to reach the full super-elevation at the SC.

5.9.3 Super-elevation runoff

The super-elevation runoff is the length of road needed to accomplish the change in cross slope from the first section in which the adverse crown was removed to the fully super-elevated section and is effected over the whole length of the spiral transition curve.

Its end point is the beginning of the circular curve itself which is denoted by SC (the *Spiral to Curve* transition point) or, alternatively called PC (the *Point of Curvature* i.e., the point where the circular curve begins*)*.

The length of runoff, shown in Table 5-7 and illustrated in Figure 5-4 and Figure 5-5, is the spiral length with the tangent-to-spiral point (TS) at the beginning and the spiral-to-curve point (SC) at the end. The length of the transition curve is proportional to the total super-elevation and should not be less than the values shown in Table 5-7. A simple practical rule is that it must not be less than the distance travelled in 2 seconds at the design speed.

Table 5-7 Minimum length of super-elevation run-off for two-lane roads

Design speed (km/hr)	Run off (m)
120	70
110	65
100	60
90	50
80	45
70	40
60	35
50	30
40	30
30	20

5.10 Transition Curves

All the components of the horizontal alignment are shown in Figure 5-5**.** Unless the circular curve has a large radius or the design speed is low, a spiral transition is required between the tangent (which is straight) and the circular curve itself.

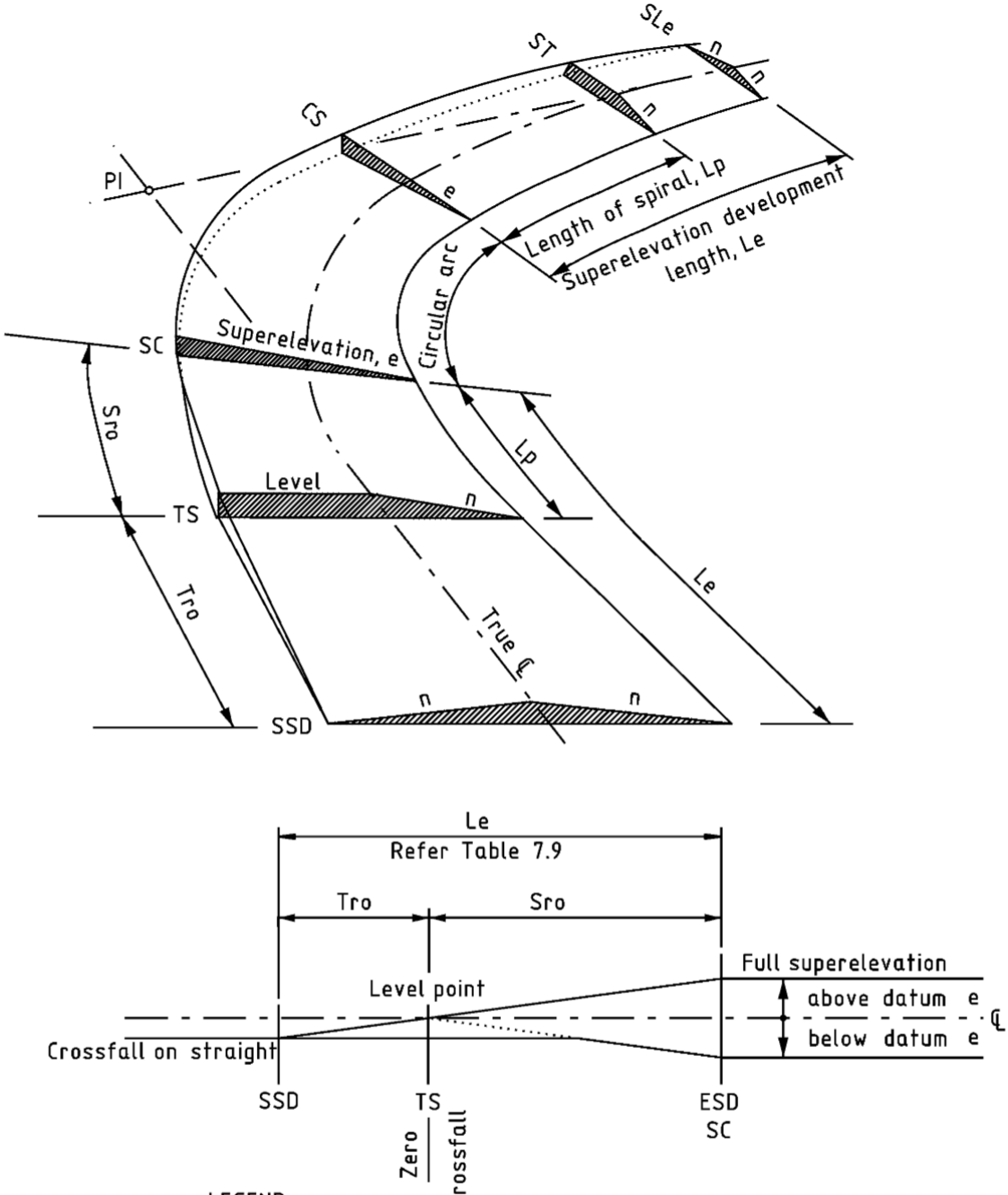
The spiral transition curve, whose radius changes continuously between a straight section of road and the neighbouring connected circular curve, is used to reduce the abrupt introduction of centripetal acceleration as a vehicle travel between the superelevated ends of each straight section of the road.

The end of the preceding tangent (which is the beginning of the superelevated section) is where gradual removal of the camber on the outside lane (or lanes) begins. If this is not removed, it would become adverse camber on the curve and would have the opposite effect to the one required.

Not all circular curves require a spiral transition. Transitions curves are not necessarily required:

1) For large radius horizontal curves (defined in Table 5-8)
2) Where the operating speed is less than 70 km/h
3) Where the associated shift in circular arc (for the necessary transition length) is less than 0.25 to 0.3 m, because drivers have sufficient room to make the transition path without encroaching into an adjoining lane.

Current design practice is to place approximately two-thirds of the runoff on the tangent approach and one-third on the curve.



LEGEND

1	PI	Point of Intersection of the main tangents
2	TS	Tangent to Spiral – common point of tangent and spiral
3	SSD	start of superelevation development
4	ESD	end of superelevation development
5	SC	Spiral to Curve – common point of spiral and circular curve
6	Lp	Length of spiral, TS to SC (m)
7	Le	Length of Superelevation Development (m)
8	n	Normal pavement crossfall (%)
9	e	Pavement superelevation (%)
10	Tro	Tangent Runout
11	Sro	Superelevation Runoff

Figure 5-5: Elements of Super-elevation and transition curve

The purpose of introducing transition curves is to:

1) provide a length over which super-elevation run-off and/or transition for widening is applied;
2) provide a length over which smooth steering adjustments can be made especially between reverse curves.
3) improve the appearance such as on a bridge where a rigid handrail follows the exact geometry of the lane;
4) improve aesthetics of the circular curves that are visible at the end of a long straight.

For the various design speeds, a radius corresponding to a specified centripetal acceleration can be calculated. Thus, a changing radius at a specific speed corresponds to a specific rate of change of centripetal acceleration. For comfort, the range varies between 0.4 and 1.3 m/s^2. If the radius of the circular curve is less than the values shown in Table 5-8, then transition curves are required to achieve this degree of comfort. For curves of large radius, the rate of change of lateral acceleration is small and transition curves are not normally required. Transition curves are also unnecessary for roads of low design speeds or low classification**.**

Table 5-8: Transition curve requirements (m)

Design speed (km/hr)	Transition required if radius of curve is less than:
70	290
80	380
85	428
90	480
100	590
110	720
120	850

Source: AASHTO. *A Policy on Geometric Design of Highways and Streets, (2011*).

Generally, the Euler spiral, which is also known as the clothoid, is used in the design of transition curves.

The radius of curvature at a point on a curve is the radius of the circle that fits the curve at that point. It is calculated from:

Equation 5-3: Radius of curvature at a point on a curve

$$\boldsymbol{R} = {[\mathbf{1} + (\frac{\boldsymbol{dy}}{\boldsymbol{dx}})^2]^{3/2}} \Big/ {\frac{\boldsymbol{d^2y}}{\boldsymbol{dx^2}}}$$

The equation for the applicable clothoid transition curve is:

Equation 5-4: Applicable clothoid transition curve

$$A^2 = R.L$$

Where,

A = the clothoid parameter,

R = the radius at the end of the clothoid,

L = the length of the clothoid.

The radius (R) varies from infinity at the straight end of the spiral to the radius of the circular arc at the end of the transition.

The clothoid parameter, A, expresses the rate of change of curvature along the clothoid. Large values of A represent slow rates of change of curvature while small values of A represent rapid rates of change of curvature.

The most rapid rate of change of curvature is represented by the minimum permissible value of A_{min}. The determination of A_{min} is based upon considerations of the rate of change of centrifugal acceleration, super-elevation run-off, aesthetics, and the ratio of the radii of consecutive curves of a compound curve as described below: -

By selecting 0.5 m/s as the maximum value for the rate of change of centrifugal acceleration, the minimum value of A is:

Equation 5-5: Minimum value of A

$$A_{min} = 0.21(V_D)^{1.5}$$

Where,

V_D = Design Speed (km/h)

The clothoid must have sufficient length to accommodate the super-elevation run-off (Section 5.10). That is:

$$A_{min} = (R.L_{min})^{0.5}$$

and for aesthetic reasons the shape of the curve should be clearly visible therefore

$$A_{min} = {}^{R}/_{3}$$

Finally, the two branches of the clothoid (at either end of the tangent section) should have approximately the same parameters that would therefore produce a similar rate of change of curvature.

5.11 Reverse Curves, Broken-back Curves, and Compound Curves

Curves are more frequent in hilly terrain and tangent sections are shortened. A stage may be reached where successive curves can no longer be dealt with in isolation. Three cases of successive curves are shown in Figure 5-6 and in Chapter 7.

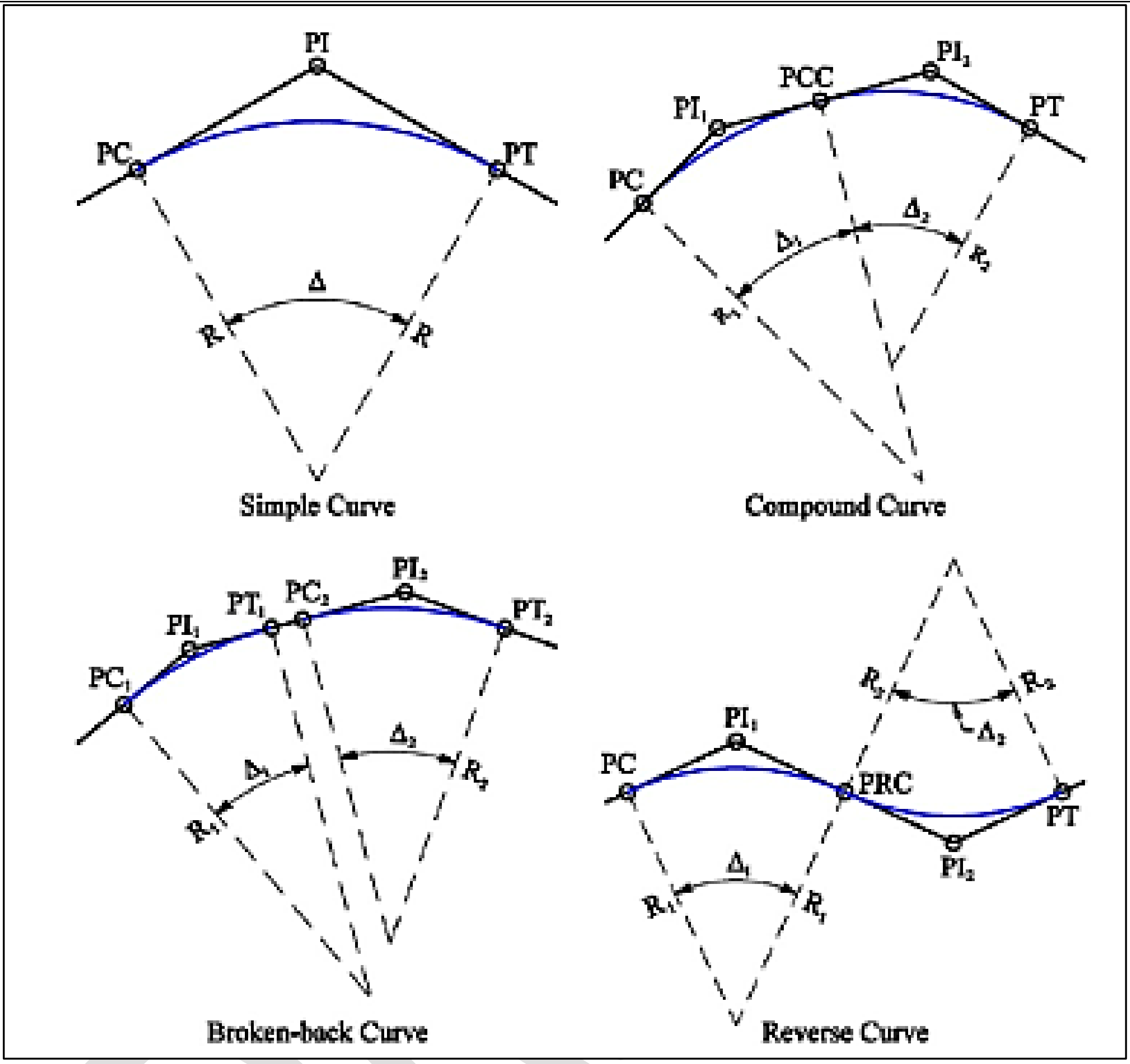


Figure 5-6: Reverse curves, broken-back curves, and compound curves.

5.11.1 Reverse curve

This is a curve followed by another curve in the opposite direction (Figure 5-6 and Figure 5-7). The occurrence of abrupt reverse curves (i.e., a short tangent between two curves in opposite directions) should be avoided. Such geometrics make it difficult for the driver to remain within the lane. It is also difficult to super-elevate both curves adequately, and this may result in erratic operation.

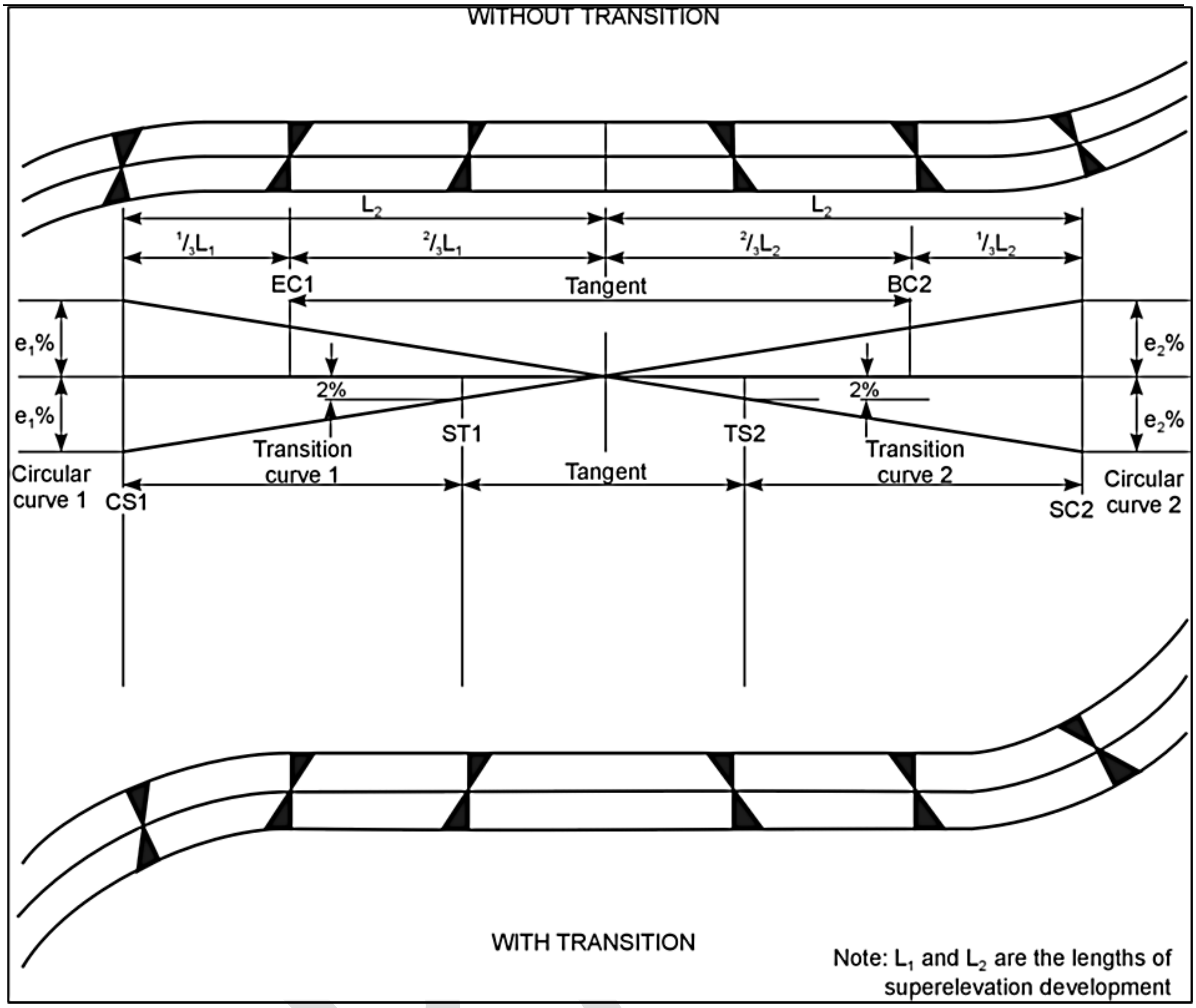


Figure 5-7: Super-elevation of Reverse Curves

5.11.2 Broken-back curve.

This is a curve followed by another curve in the same direction but with only a short tangent in between. Broken-back curves 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 hence safety is compromised. Problems with super-elevation and drainage can also arise. A single curve is preferred if it is physically and economically feasible. The super-elevation is illustrated in Figure 5-8.

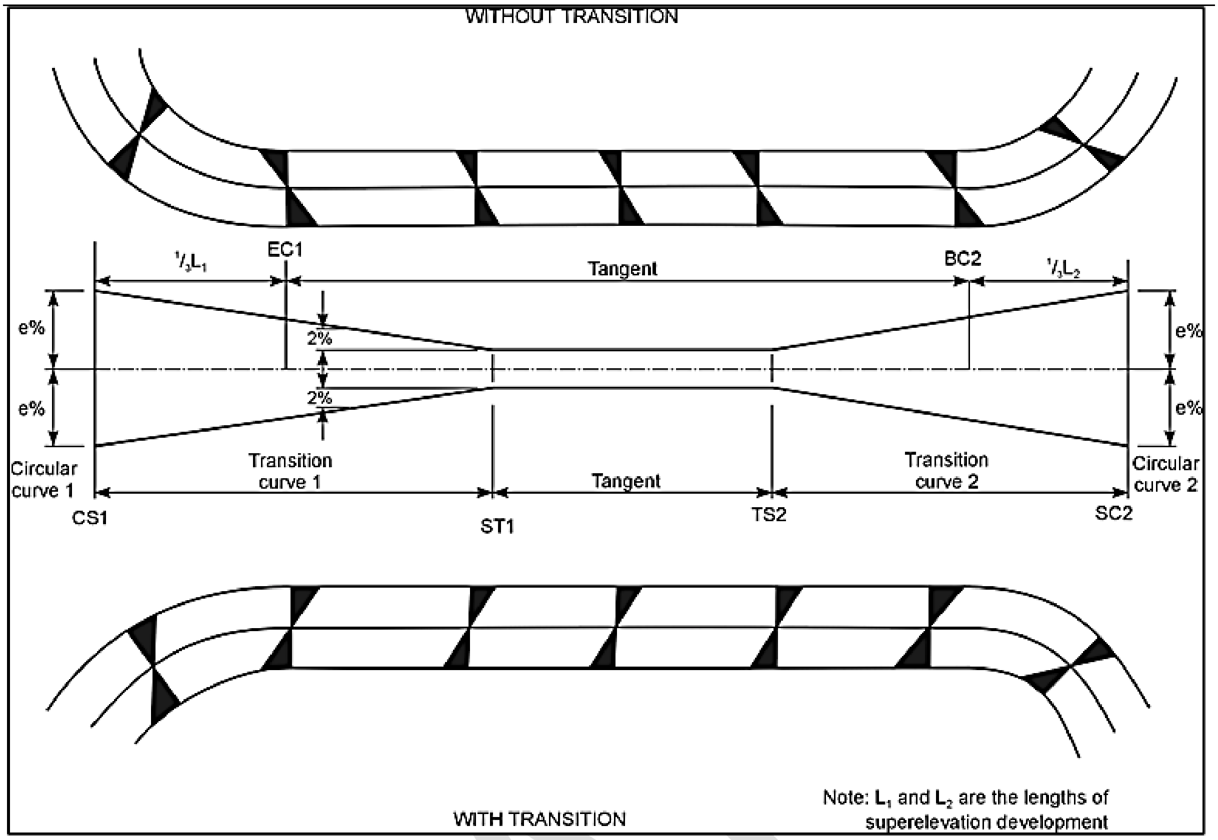


Figure 5-8: Super-elevation of broken-back curves

5.11.3 Compound curve

Compound curves (Figure 5-9) are curves in the same direction but of different radii, and without any intervening tangent section. The use of compound curves provides flexibility in fitting the road to the terrain and other controls. Caution should, however, be exercised in the use of compound curves because the driver does not expect to be confronted by a change in radius once a curve has been entered, hence safety is compromised. Their use should be avoided especially where the curves need to be of short radius.

If two successive circular curves in the same direction cannot be avoided, the connecting tangent should be at least 150 m long. The tangent should have a single crossfall rather than reverting to a normal camber for a short distance.

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 should not be more than 50 % greater than the radius of the sharper arc, i.e., R_1 should not exceed $1.5.R_2$. 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.

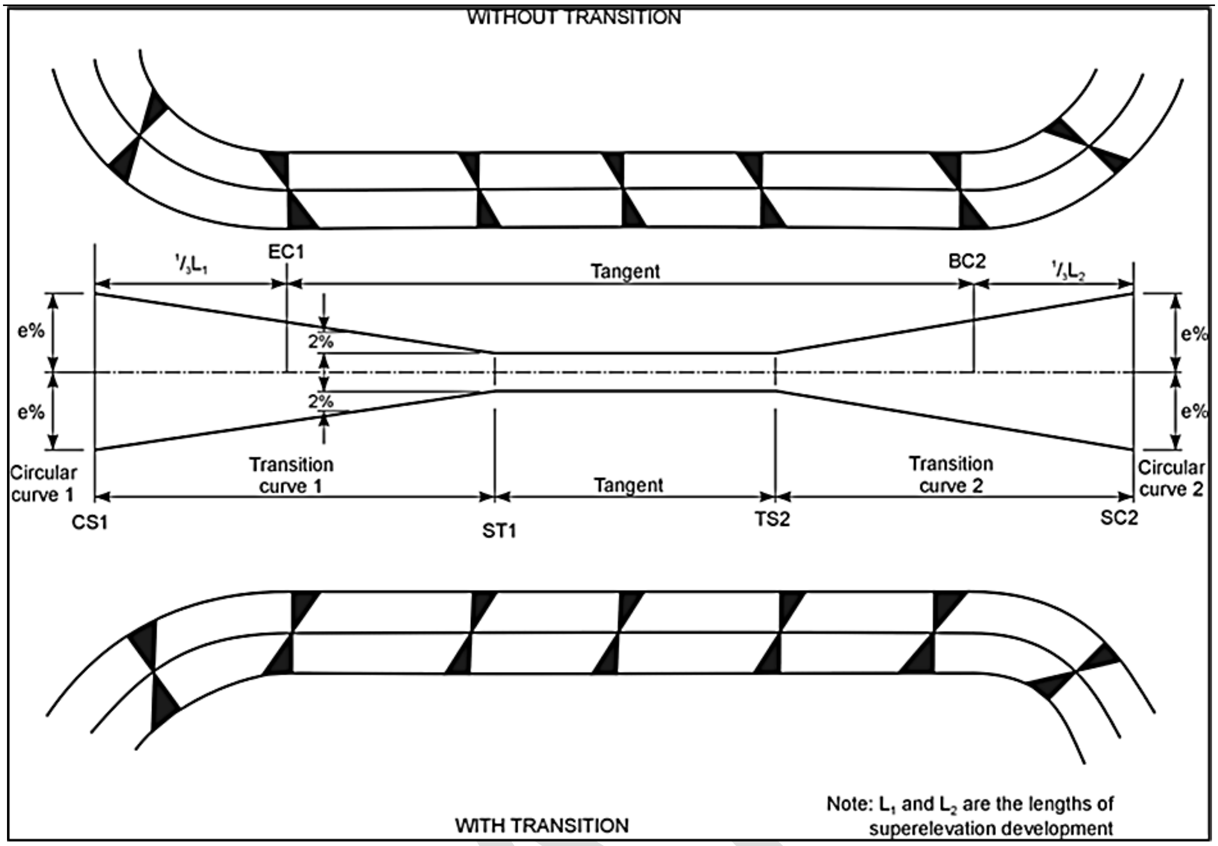


Figure 5-9: Super-elevation of compound curves

5.12 Hairpin Curves

Hairpin curves are used where necessary in traversing mountainous and escarpment terrain. Employing a radius of 20 m or less, with a minimum of 15 m, they are generally outside the standards for all road designs and are specified using the departure from standards guidelines listed in Chapter 16,

Hairpin curves require 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 *Figure* 5-10 and *Figure* 5-11. These figures show that the minimum outer radii for design vehicles DV3 and DV4 are 14.1 m, and 12.5 m, respectively. Hairpin requirements can be determined which allow for:

1) Passage of two opposing DV4 vehicles. This is recommended for Design Standards A, UA, B, UC, and C.
2) Passage of a single DV4 and a DV1. This is recommended for Design Standards C.

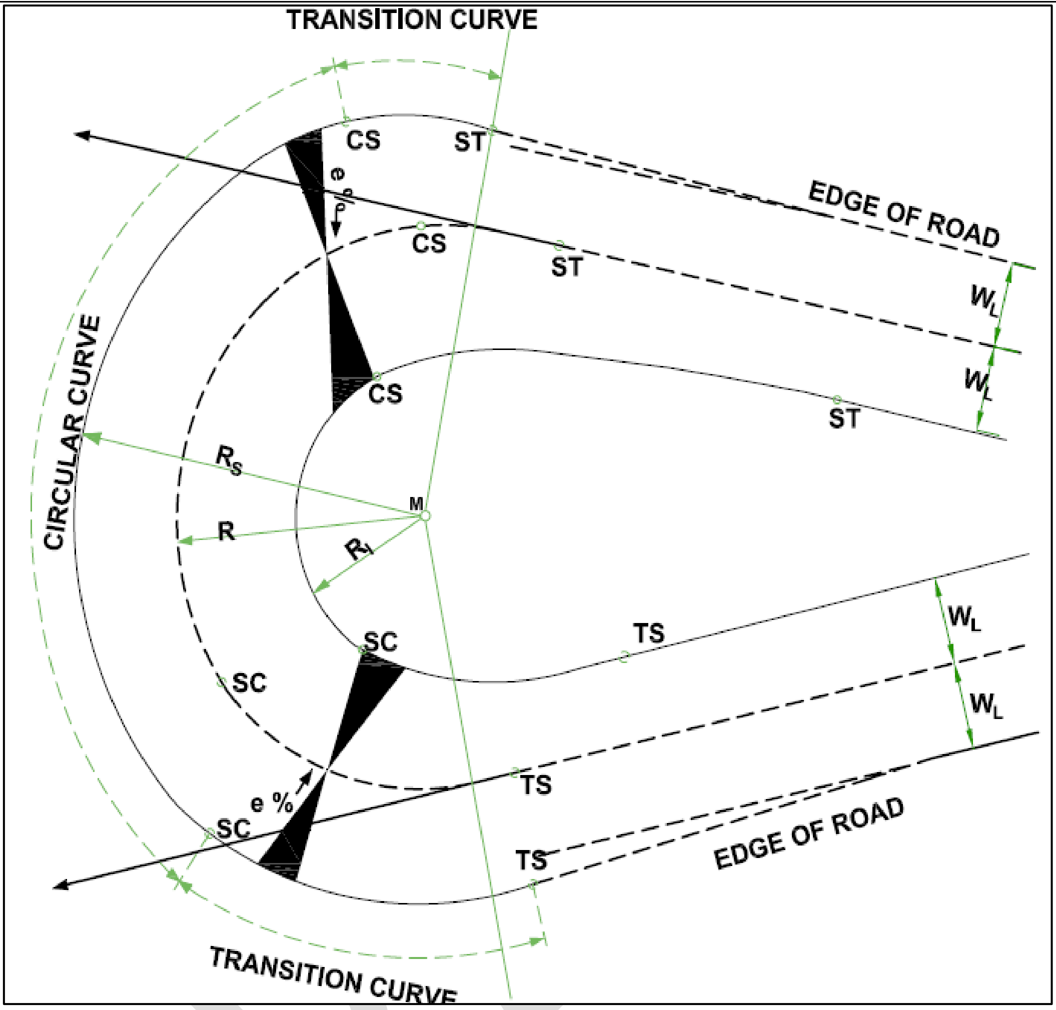


Figure 5-10: Hairpin curve

As an example, consider a road standard which allows for only the passage of a single DV4 vehicle. By superimposing Figure 3-5 or Figure 3-6 for design vehicles DV3 and DV4 over Figure 5-10 or Figure 5-11 to the same scale, the additional requirements can be identified. The normal carriageway width will usually need to be increased at the hairpin curve. Requirements vary depending on passage requirements, radius, deflection angle, and design standard, and a template should be used based on the turning radii of the design vehicle to ensure that the vehicles can negotiate each hairpin.

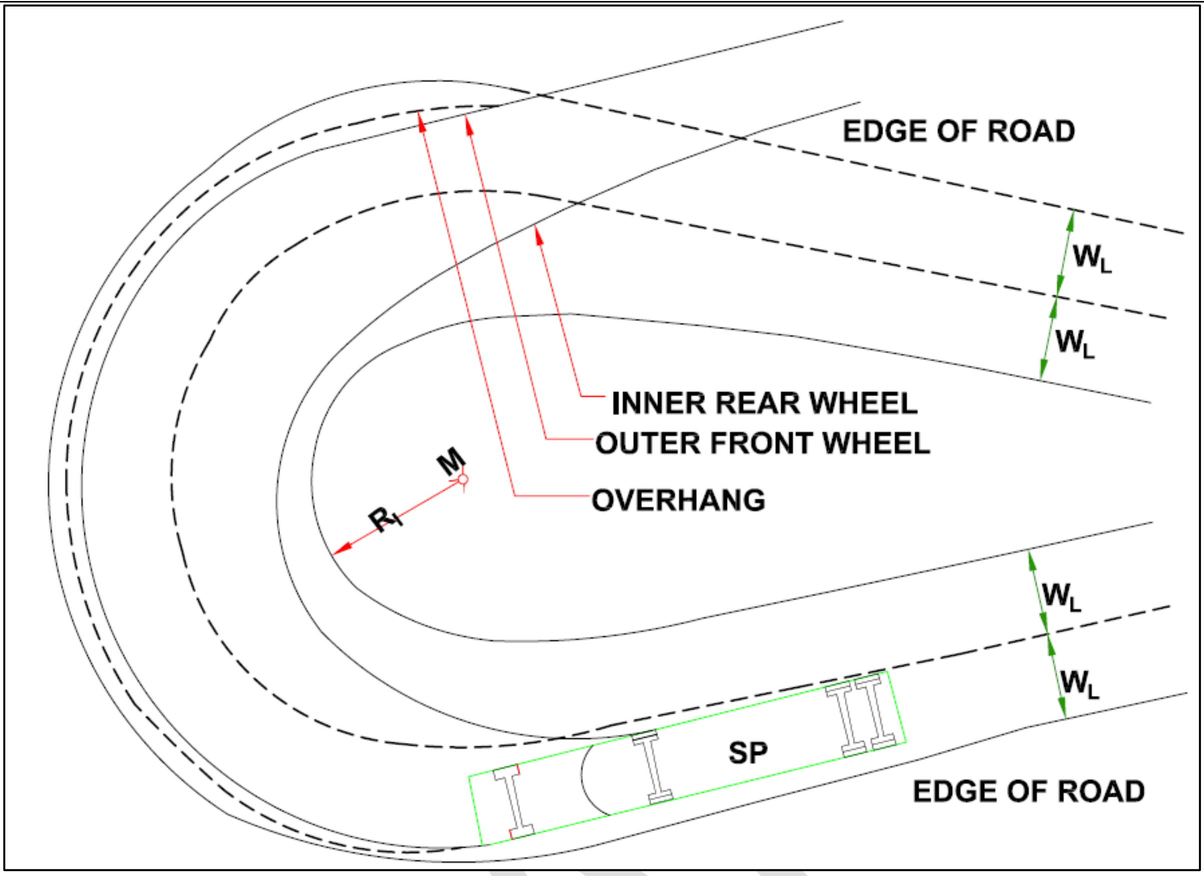


Figure 5-11: Hairpin curve for the passage of single DV4 vehicle

It is important to provide relief from a severe gradient through the switchback. Where switchback curves are unavoidable in mountainous terrain, there is a need to reduce the maximum allowable gradient at any point through the curve. The maximum allowable gradient through a switchback curve is 4 percent.

5.13 Passing Lanes

Passing lanes are normally provided in areas where construction costs are relatively low and where there is an absence of passing opportunities. A passing lane length of about one km is adequate for this purpose. Numerous short passing lanes are preferable to a few long passing lanes (*Figure* 5-12) and it is recommended that they are located at two, four and eight kilometres spacing. A long tangent can cause serious problems of dazzle from approaching headlights which can be extremely dangerous and therefore, if there are large volumes of night traffic, lengths shorter than this should be used where traffic volumes are low, the longest spacing can be used and, as traffic volumes increase, the intervening lanes can be added in a logical manner with one-km passing lanes provided at two-km intervals. They potentially provide safer passing opportunities for drivers who are uncomfortable in using the opposing traffic lane and for those who become frustrated when few passing opportunities exist owing to terrain or traffic density.

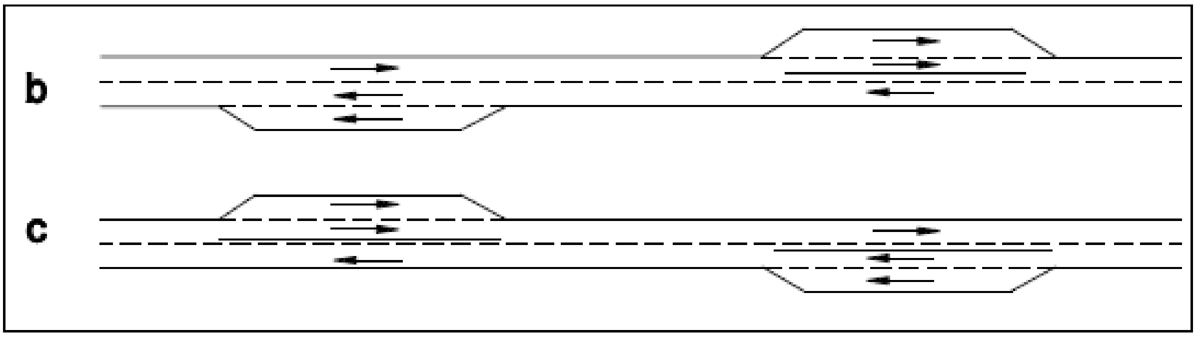


Figure 5-12: Passing lane arrangements.

5.13.1 Three-lane designs

The next level of upgrading is a continuous three-lane cross section, two lanes in one direction (one for overtaking) and a single lane in the opposing direction (often called the 2 + 1 design). See, Figure 5-13 and Figure 5-14. The centre lane is alternately allocated to each of the opposing directions of flow and appropriate road markings and signage are essential. The switch in the direction of flow in the centre lane should be at about two-kilometre intervals. A minimum shoulder width is required as discussed in Chapter 4

Figure 5-13: Example of a passing lane

5.13.2 Entry and exit tapers.

The entrance taper to a passing lane should be a minimum of 100 m in length. The length of the exit taper should be longer to allow adequate time for merging vehicles to find a gap in the through flow. A minimum of 200m is recommended. Since both the entry and the exit tapers indicate a change in operating conditions it is recommended that decision sight distance (Section 3.7.6) should be available at these points.

Without adequate signposting and road marking, erratic and last-second driving manoeuvres occur. Such manoeuvres can be extremely hazardous, especially when merging. It is very important that road markings and signs are always adequate and abundantly clear. Passing lanes provide more numerous passing opportunities and are potentially relatively safe when constructed, marked out, signed, and maintained properly.

5.13.3 Passing lanes and overtaking on gradients

Passing lanes can be used instead of climbing lanes on hilly sections of road. Unlike climbing lanes, passing lanes tend to operate at the speeds prevailing on the rest of the road. Reductions in lane width are thus not recommended and such passing lanes should have the same width as the basic lanes. Figure 5-14 is an example of the layout of a passing lane.

The advantage is that the slow traffic does not change lane at either the beginning or, more importantly, at the end of the passing lane. Such merging and diverging movements are obviously required by the much more manoeuvrable fast traffic, but this is often considered safer than the alternative of slow traffic having to merge with fast traffic. The disadvantage is that, unless the fast traffic does so at the proper location, it will face traffic from the opposite direction travelling towards it in the central lane. The key is good road layout and road signing and marking. Reference should be made to the **Road Design Manual, Vol 6: Part 2 – Traffic Signs** for the recommended signage and road markings. Driver experience and expectations also play an important role.

Neither a passing lane nor a climbing lane are ideal solutions, and each case needs to be examined on its merits.

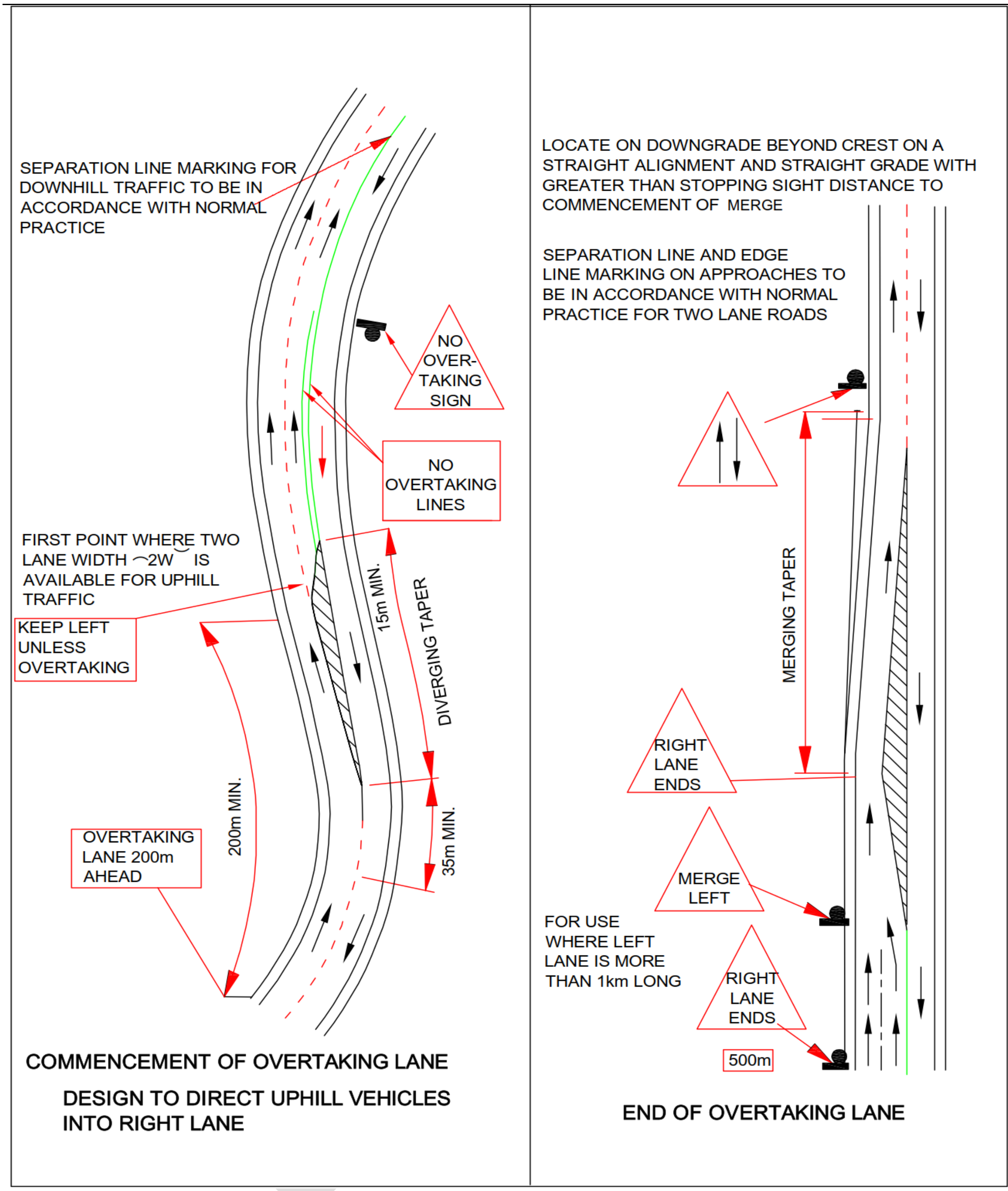


Figure 5-14: Layout for passing lane.

5.14 Safety Considerations in Horizontal Alignment

Sharp curves should not be located at the end of long tangents (AASHTO, 2011a). Speeds tend to increase gradually on long tangents and drivers would not necessarily be aware of their speed in approaching a sharp curve. This could result in a run-off-the-road crash.

If the topography forces the selection of short radius curvature, these curves should be preceded by a series of successively sharper curves. A design speed of 120 km/h could be followed by a series of design speed reduction steps of 10 km/h each until the desired design speed is achieved. If space permits, each section of reduced speed should include at least two curves at the minimum radius for that speed. This is to reinforce the realisation that the previous minimum curve was not just an isolated situation.

It is difficult for drivers to assess the sharpness of a curve in the absence of features such as cut faces, trees and shrubbery or buildings projecting above the road surface. It is thus not desirable to locate sharp curves on high fills.

While compound curves can make it easy for the designer to fit the road within the prevailing topographic constraints, safety must not be sacrificed for ease of design. In terms of human factors design, drivers expect that the curve they are negotiating will maintain a constant radius. A reduction in radius would probably require the driver to brake. The only place where compound curves are acceptable to drivers is on loop ramps at interchanges.

From a safety point of view, the goal should be to use the highest possible value of radius. There is, however, a caveat to this insofar as two-way two-lane roads require tangents long enough to allow for safe overtaking manoeuvres. Generally, the longer the curves, the shorter are the tangent between them. A compromise should therefore be sought between passing sight distance and curve radius.

On a two-lane road, curves to the left enhance sight distance because the driver of a vehicle following a truck can see past the truck without having to move closer to the opposing lane to see approaching vehicles. Unfortunately, a curve to the left for the one direction of flow is a curve to the right for the opposing vehicles. The right-hand curve requires that a driver wishing to overtake would have to venture fairly far to the left, possibly even into the opposing lane to check for a passing opportunity. On minimum radius curves, the probability of a head-on crash is too high to be disregarded.

As an alternative, moving to the right-hand side of the lane and possibly even onto the shoulder may make it possible to see past the right-hand side of the truck. In either case, the overtaking manoeuvre would have to commence at a greater distance behind the truck, generating a need for a longer passing sight distance than would otherwise be the case. In the case of a leading passenger car, drivers can often see through the leading vehicle so that the restriction of sight distance is less severe.

The curve to the left vis-à-vis the curve to the right is an issue in the consideration of the percentage passing sight distance. Obviously, the percentage passing sight distance has to be assessed for both directions. The problem of lack of passing sight distance on curves to the right can be minimised either by shortening the radius of curvature, hence increasing the tangent lengths on either side of the curve, or by increasing the radius of curvature.

Impediment to sight distance caused by trucks is an issue only where there is a high percentage of truck traffic on the road. If the trucks traffic constitute about 10 per cent or less of normal traffic, it is not necessary to make adjustments to the alignment to improve passing sight distance.

Table 5-9: Super-elevation rates for emax = 4.0%

Radius (m)	Design speed (km/h) 40	50	60	70	80	90	100	110	120
7000	n	n	n	n	n	n	n		
5000	n	n	n	n	n	n	n		
4000	n	n	n	n	n	n	n		
3000	n	n	n	n	n	n	rc		
2000	n	n	n	n	rc	rc	rc		
1500	n	n	n	rc	rc	rc	2.2		
1400	n	n	n	rc	rc	rc	2.3		
1300	n	n	n	rc	rc	rc	2.45		
1200	n	n	n	rc	rc	2.1	2.6	Design speeds above 100 km/h are not suitable	
1000	n	n	rc	rc	rc	2.5	3		
900	n	n	rc	rc	2.1	2.7	3.2		
800	n	n	rc	rc	2.3	2.95	3.4		
700	n	rc	rc	rc	2.6	3.2	3.6		
600	n	rc	rc	2.3	2.9	3.45	3.8		
500	n	rc	2.1	2.7	3.25	3.7			
400	rc	rc	2.6	3.1	3.6				
300	rc	2.3	3.1	3.6					
250	rc	2.6	3.4	3.8					
200	2.1	3	3.7						
180	2.3	3.2	3.8						
160	2.5	3.4							
140	2.8	3.6							
120	3.1	3.8							
100	3.4	4.0							
80	3.7								
60	4.0								

Notes

n = *normal crown*
rc = *remove adverse camber*

Table 5-10: Super-elevation rates for emax = 6.0%

Radius	Design speed (km/h)								
(m)	40	50	60	70	80	90	100	110	120
7000	n	n	n	n	n	n	n	n	rc
5000	n	n	n	n	n	n	n	rc	rc
4000	n	n	n	n	n	n	rc	rc	rc
3000	n	n	n	n	n	rc	rc	2.0	24
2000	n	n	n	rc	rc	2.2	2.6	3.0	3.4
1500	n	n	rc	rc	2.3	2.8	3.2	3.7	4.2
1400	n	n	rc	rc	2.5	3.0	3.4	3.9	4.4
1300	n	n	rc	2.1	2.6	3.1	3.6	4.1	4.6
1200	n	n	rc	2.3	2.8	3.3	3.8	4.3	4.8
1000	n	rc	2.1	2.7	3.2	3.7	4.2	4.8	5.3
900	n	rc	2.3	2.9	3.4	3.9	4.4	5.1	5.6
800	n	rc	2.5	3.1	3.6	4.2	4.7	5.4	5.9
700	n	2.1	2.7	3.4	3.9	4.5	5.0	5.7	
600	n	2.4	3.0	3.7	4.2	4.8	5.4		
500	rc	2.7	3.4	4.1	4.6	5.2	5.9		
400	2.3	3.1	3.8	4.5	5.1	5.7			
300	2.8	3.7	4.4	5.1	5.7				
250	3.1	4.0	4.8	5.5					
200	3.6	4.5	5.2	5.9					
180	3.8	4.7	5.4						
160	4.0	4.9	5.6						
140	4.3	5.2	5.9						
120	4.6	5.5							
100	4.9	5.8							
80	5.4								
60	5.9								

Notes

n = normal crown
rc = remove adverse camber

Table 5-11: Super-elevation rates for emax = 8.0%

Radius	**Design speed (km/h)**								
(m)	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**
7000	n	n	n	n	n	n	n	rc	rc
5000	n	n	n	n	n	n	rc	rc	rc
4000	n	n	n	n	n	n	rc	rc	2.2
3000	n	n	n	n	n	rc	rc	2.3	2.9
2000	n	n	n	rc	rc	2.4	2.9	3.5	4.1
1500	n	n	rc	rc	2.6	3.2	3.7	4.3	5.1
1400	n	n	rc	2.1	2.8	3.4	3.9	4.6	5.3
1300	n	n	rc	2.3	3.0	3.6	4.2	4.8	5.6
1200	n	n	rc	2.5	3.2	3.8	4.4	5.1	5.9
1000	n	rc	2.1	2.9	3.6	4.3	4.9	5.8	6.6
900	n	rc	2.4	3.2	3.9	4.6	5.2	6.2	6.9
800	n	2.0	2.7	3.5	4.2	4.9	5.6	6.6	7.3
700	n	2.3	3.0	3.8	4.6	5.3	6.1	7.1	7.8
600	rc	2.6	3.4	4.2	5.0	5.8	6.7	7.6	
500	2.1	3.0	3.9	4.8	5.6	6.4	7.3		
400	2.5	3.5	4.5	5.4	6.3	7.1			
300	3.1	4.2	5.3	6.3	7.2				
250	3.5	4.7	5.9	6.9	7.8				
200	4.0	5.4	6.5	7.5					
180	4.4	5.7	6.8	7.8					
160	4.7	6.0	7.2						
140	5.1	6.4	7.6						
120	5.6	6.9	8.0						
100	6.1	7.4							
80	6.7	8.0							
60	8.0								

Notes

n = *normal crown*
rc = *remove adverse camber*

6 Design of Vertical Alignment

6.1 General

The longitudinal profile of a road consists of a combination of straight grades (tangents) and vertical curves (crest and sag curves) that provide a smooth transition between consecutive gradients. The parabola is specified for the curves because the parabola provides a constant rate of change of curvature and, hence, acceleration and visibility, along its length.

The selection of rates of grade and lengths of vertical curves is based on assumptions about the characteristics of the driver, the vehicle, and the roadway. The resultant design should result in a safe road that is comfortable in operation, pleasing in appearance and adequate for drainage.

Thus, the two major aspects of vertical alignment are vertical curvature, which is governed by sight distance criteria, and gradient, which is related to vehicle performance and level of service. This chapter: -

1) describes the mathematical concepts for defining the vertical curvature of the road;
2) defines the limiting characteristics for each road class;
3) recommends maximum and minimum gradients;
4) recommends gradient requirements through villages.
5) develops the criteria for incorporation of a climbing lane;
6) proposes effective combinations of vertical and horizontal curves and gradients.

A smooth grade line with gradual changes appropriate to the class of road and the character of the topography is preferable to an alignment with numerous short lengths of grade and vertical curves.

The vertical alignment should also be designed to be aesthetically pleasing. As a general guide, a vertical curve that coincides with a horizontal curve should, if possible, be contained within the horizontal curve and should, ideally, have approximately the same length.

6.2 Vertical Curve Formula

A crest curve is a convex vertical curve, and a sag curve is a concave vertical curve. These are illustrated in Figure 6-1 and Figure 6-2 6.2 respectively.

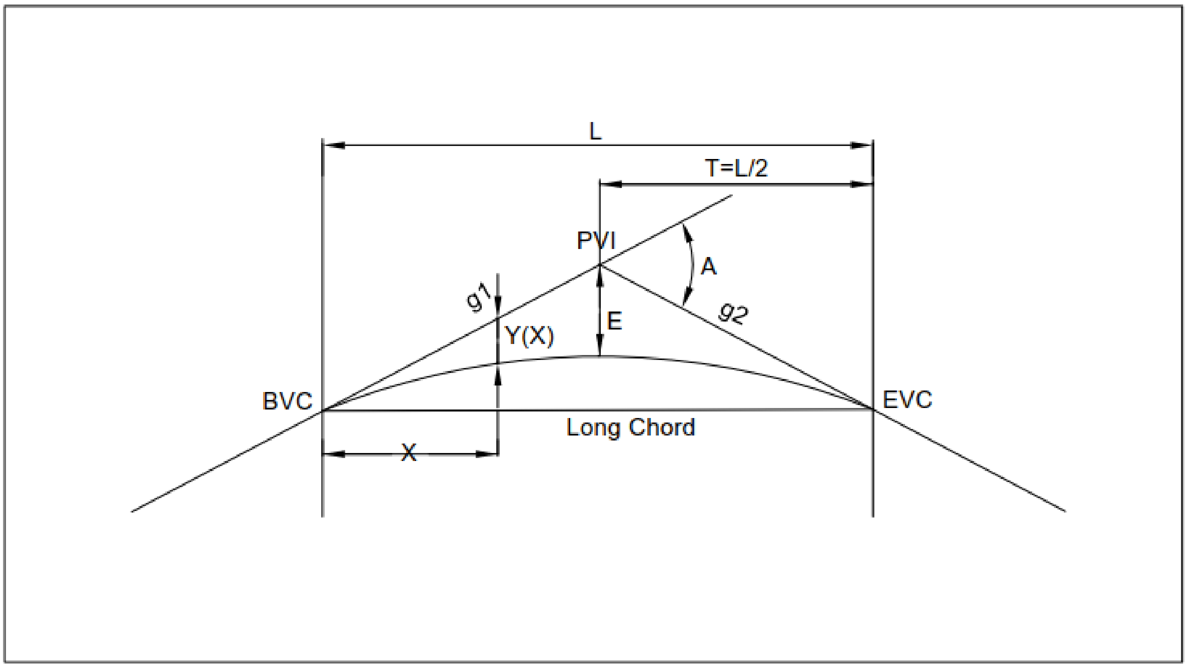


Figure 6-1: Crest Curve

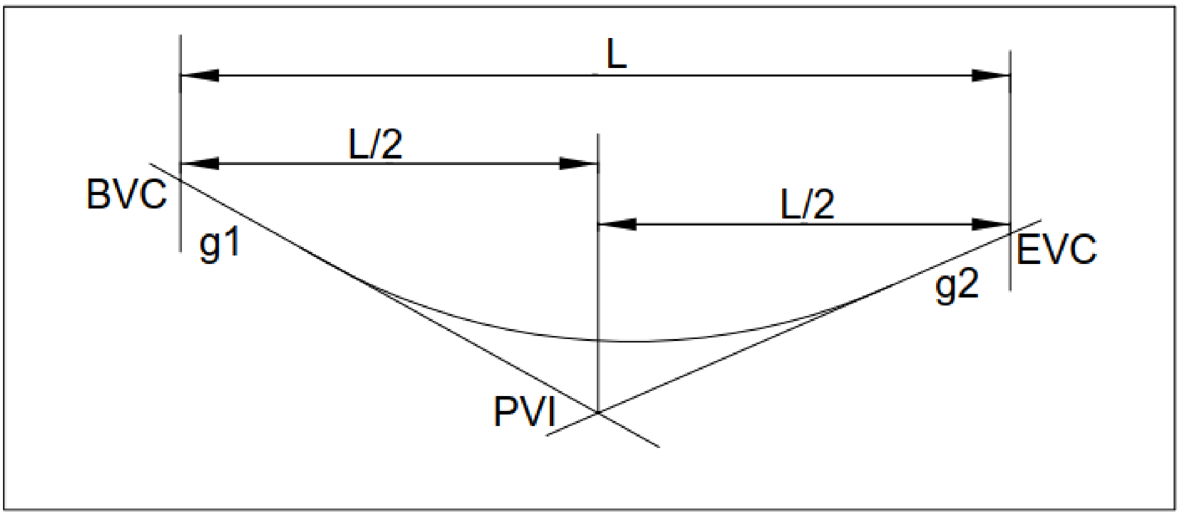


Figure 6-2: Sag Curve

Where:

BVC	=	*Beginning of the vertical curve.*
EVC	=	*End of the vertical curve.*
Y(X)	=	*Elevation of a point on the curve (m)*
X	=	*Horizontal distance from the (BVC) (m)*
r	=	*Rate of change of gradient*

Vertical curves are required to provide smooth transitions between consecutive gradients. The equations relating the various aspects of the vertical curve (both crest and sag) are as follows:

$$Y(X) = \frac{r.X^2}{200} + \frac{X.g_1}{100} + Y_{BVC}$$

Equation 6-1: Equation of a vertical curve

Where:

L = *Length of curve (horizontal distance) in m,*

g_1 = *Starting gradient (%),*

g_2 = *Ending gradient (%),*

r = *Rate of change of grade per section (%/ m)*

= *$(g_2 – g_1)/L$* = *G/L = 1/K*

Useful relationships are;

Equation 6-2: Equation of tangent g1

$$Y(X) = Y(0) + \frac{g_1.X}{100}$$

Equation 6-3: Equation of tangent g2

$$Y(X) = Y(L) + \frac{g_2.(X - L)}{100}$$

Equation 6-4: Equation for y coordinate of the EVC

$$: Y(L) = \frac{(g_1 + g_2)L}{200} + Y(0)$$

Equation 6-5: Point of vertical intersection (PVI)

The Point of Vertical Intersection (PVI) always occurs at an x coordinate of 0.5L hence, from Equation 6-1, the elevation is always;

$$Y(PVI) = Y\left(\frac{L}{2}\right) = Y(0) + \frac{g_1 X}{100} = Y(0) + \frac{g_1 L}{200}$$

<u>Example:</u>

For the crest curve shown in **Error! Reference source not found.**, the two tangent grade lines a re +6% and -3%. The Beginning of the Vertical Curve is at chainage 0.000 and its elevation 100.0m. The length of the vertical curve is 400m. Compute the End of the Vertical Curve and the coordinates of the Intersection Point.

The y coordinate of the EVC is Y(L) = (g_1+g_2) L/200 + Y (0)

= (6 - 3) *400/200 + 100.0 = 106.0

The x coordinate of the EVC is X(L) = 400.0

The coordinates of the VPI are X(IP) = L/2 = 200.0 and

Y(PVI) = Y (0) + 6.400/200

= Y (0) + 12 = 112m

6.3 Crest Curves

Two conditions exist when considering the minimum sight distance criteria on vertical curves. The first is where the sight distance (S) is less than the length of the vertical curve (L), and the second is where sight distance extends beyond the vertical curve. Consideration of the properties of the parabola results in the following relationships for minimum curve length to achieve the required sight distances:

For S < L (the most common situation in practice):

$$L_m = \frac{G.S^2}{200(h_1^{0.5} + h_2^{0.5})^2}$$

Equation 6-6: Vertical curve length (S<L)

and therefore:

$$L_m = K.G$$

Where:

Lm = minimum length of vertical crest curve (m) and large K corresponds with long curves

S = required sight distance (m)

h1 = driver eye height (m)

h2 = object height (m)

K = is a constant for given values of h1 and h2 and stopping sight distance (S) and therefore speed and surface friction.

For S > L

$$L_m = 2S - \frac{[200 * (h_1^{0.5} + h_2^{0.5})^2]}{G}$$

Equation 6-7: Vertical curve length (S>L)

Eye height (h_1) has been taken as 1.05 m, and object heights h_2 of 0.2 m and 0.6 m above the road surface. Minimum values of K for crest curves are shown in Table 6-1 and Table 6-2.

Table 6-1: Minimum values of K for crest vertical curves (paved roads)

Design speed (km/h)	K for stopping sight distance (g = 0%)			K for minimum passing sight distance
	h_2 = 0m	h_2 = 0.2m	h_2 = 0.6m	
25	3	1	1	30
30	5	2	1	50
40	10	5	3	90

Design speed (km/h)	K for stopping sight distance (g = 0%)			K for minimum passing sight distance
	h_2 = 0m	h_2 = 0.2m	h_2 = 0.6m	
50	20	10	7	130
60	35	17	12	180
70	60	30	20	245
80	95	45	30	315
85	115	55	37	350
90	140	67	45	390
100	205	100	67	480
110	285	140	95	580
120	385	185	125	680

Table 6-2: Minimum values for crest vertical curves (unpaved roads)

Design speed (km/h)	K for stopping sight distance (g = 0%)			K for minimum passing sight distance
	h_2 = 0m	h_2 = 0.2m	h_2 = 0.6m	
25	3	1	1	30
30	5	2	2	50
40	11	6	4	90
50	25	11	8	135
60	45	20	15	185
70	75	35	25	245
80	120	58	40	315
85	150	72	50	350
90	185	90	60	390
100	270	130	88	480

Similar calculations can be carried out based on passing sight distance (Section 3.7) rather than stopping sight distance. High values of K (L is large) are obtained (Table 6-1 and *Table* 6-2) and therefore, to achieve the passing sight distance, the volume of earthworks required may also be large. Although as much passing sight distance as possible should be provided along the length of the road, it may be impossible to achieve passing sight distance over the crest curve itself. Encouraging drivers to overtake when sight distances have not been fully achieved is dangerous hence shortening the crest curve to increase the lengths of the grades on either side is a better option.

6.3.1 Minimum Length and Radii of Vertical Curve

Usually, the largest radii vertical curves should be used if they are reasonably economical. However, in difficult situations vertical curves approaching the minimum may be considered where the cost of providing larger curves makes their use prohibitive.

For a particular design speed, the minimum radius of a crest curve is usually governed by sight distance requirements. If the difference between successive grades is small, the intervening minimum vertical curve becomes very short. This can create the impression of a kink in the grade line. The minimum lengths in Table 6-3 should be applied. Riding comfort is generally not considered on crests because sight distance requirements usually dictate the use of a larger radius curve that then satisfies the comfort requirement anyway.

At a small change of grade, the effect on sight distance is small hence, if possibly, the design should be such that a small change in grade is avoided.

Table 6-3: Minimum lengths of vertical curves

Design Speed (km/h)	Length of Curve (m)
40	80
60	100
80	140
100	180
120	220
130	240

Where a crest curve and a succeeding sag curve have a common end (crest) and beginning of curve (sag), the visual effect created is that the road has suddenly dropped away. In the reverse case, the illusion of a hump is created. Either effect is removed by inserting a short length of straight grade between the two curves. Typically, 60 m to 100 m is adequate for this purpose.

6.4 Sag Curves

During daylight hours, or on well-lit streets, sag curves do not present any problems concerning sight distances. For such situations it is recommended that sag curves are designed using a driver comfort criterion of vertical acceleration. A maximum acceleration of 0.3 m/s^2 is often used. This translates into

$$K > \frac{V^2}{395}$$

Equation 6-8: Relationship of K and design speed for sag curves

Where:

V is the speed in km/h.

Where the only source of illumination is the headlamps of the vehicle, the illuminated area depends on the height of the headlights above the road and the divergence angle of the headlight beam relative to the grade line of the road at the position of the vehicle on the curve. Using a headlight height of 0.6m and a beam divergence of 1^{o}, the values of K are approximately twice the values obtained from the driver comfort criterion which should be used for design. The resulting K values for both situations are shown in Table 6-4.

Table 6-4: Minimum Values of K for Sag Curves

Design Speed (km/h)	K for driver comfort	K for headlight distance
20	1.0	2
25	1.5	3
30	2.5	5
40	4	9
50	6.5	14
60	9	19
70	12	25
80	16	32
85	18	36
90	20	40
100	25	50
110	30	60
120	36	70

6.5 Gradient

6.5.1 General

Gradient is the rate of rise or fall on any length or road, with respect to the horizontal. The slope of the grade between two adjacent Vertical Points of Intersection (VPI), is usually expressed in percentage form as the vertical rise or fall in m/100m. In the direction of increasing chainage, up-grades are taken as positive and down-grades as negative.

6.5.2 Maximum Gradients

For very low levels of traffic of only a few four-wheel drive vehicles, various references recommend a maximum gradient in the range 15-18 %. Small commercial vehicles can usually negotiate an 18 % gradient, whilst two-wheel drive trucks can successfully manage gradients of 15-16 % except when heavily laden.

The frequency of crashes increases when the speed differential between trucks and cars increases. If truck speeds decrease by more than about 15 km/h, crashes increase rapidly hence the ‘critical length of grade’ is the length over which a speed reduction of 15 km/h occurs. It is important to note that truck speeds in flat terrain are already lower, on average, than car speeds by, typically, 17 km/h, so another 15 km/h reduction means that the speed differential on uphill grades is of order 32 km/h.

The initial speed of the truck on the grade, which depends on characteristics of the truck and the gradient of the approach (a downhill approach will allow vehicles to gain momentum) both affect the critical length over which a 15 km/h decrease in speed occurs. Also, some authorities use a different speed reduction (e.g., 20 km/h) to define critical lengths of grade.

The ‘critical length of grade’ indicates the maximum length of an upgrade on which a loaded truck can operate without an unreasonable reduction in speed. The ‘maximum absolute gradient and the maximum desirable gradient shown in Table 6 6 are therefore important criteria that greatly affect both the serviceability and cost of the road. A ‘whole life costing’ exercise can be an effective method of calculating the trade-off between construction and maintenance costs and road user costs.

If gradients on which the truck speed reduction is less than 15 km/h cannot be achieved economically it may be necessary to provide auxiliary (climbing) lanes for the slower-moving vehicles (Section 6.6) or passing lanes for the fast-moving vehicles (Section 5.11). A solution often suggested whereby relief gradients of low gradient are provided between steeper sections has proved ineffective because truck drivers prefer to maintain a crawl speed rather than to change gear up and down frequently.

Table 6-5 which indicates the critical lengths for a vehicle with a mass/power ratio of 185 kg/kW and one of mass/power ratio of 275 kg/kW, should be considered as a guide only.

The ‘critical length of grade’ indicates the maximum length of an upgrade on which a loaded truck can operate without an unreasonable reduction in speed. The ‘maximum absolute gradient and the maximum desirable gradient shown in Table 6-6 are therefore important criteria that greatly affect both the serviceability and cost of the road. A ‘whole life costing’ exercise can be an effective method of calculating the trade-off between construction and maintenance costs and road user costs.

If gradients on which the truck speed reduction is less than 15 km/h cannot be achieved economically it may be necessary to provide auxiliary (climbing) lanes for the slower-moving vehicles (Section 6.6) or passing lanes for the fast-moving vehicles (Section 5.13). A solution often suggested whereby relief gradients of low gradient are provided between steeper sections has proved ineffective because truck drivers prefer to maintain a crawl speed rather than to change gear up and down frequently.

Table 6-5: Lengths of grade for a 15 km/h speed reduction by trucks

Gradient (%)	Critical length of grade (m)		Maximum length of gradient (m)
	185 kg/kW	275 kg/kW	
2	550	850	
3	380	620	
4	300	450	900
5	240	345	800
6	180	270	700
7	140	210	600
8	100	160	500

Source: SANRAL and SATCC *Geometric Design Guidelines*

The effect of gradient on traffic flow is not limited to upgrades. Truck drivers frequently adapt their speeds on downgrades to be of similar values to their speeds on upgrades for better control and safety.

Table 6-6: Suggested Maximum Gradients for Paved Sections

Terrain	Maximum Gradient (%), for Paved Sections			
	Design standard S and A		Design standard B	
	Desirable	Absolute	Desirable	Absolute
Flat	3	5	4	6
Rolling	5(1)	7	6	8
Mountainous	7(1)	9	8	10
Escarpment	7(1)	9	8	10
Urban	6(1)	9	8	10

Notes: 1 On freeways a maximum gradient should not exceed 4%

Steeper grades produce variation in speeds between lighter vehicles and the heavier vehicles both in the uphill and downhill directions. This speed variation leads to higher relative speeds of vehicles producing potential for higher crash rates, lower traffic capacity and thus to increased operating cost.

Standards for desirable maximum gradients were set to maintain user comfort and to avoid severe reductions in vehicle speeds. If the occasional terrain anomaly is encountered that requires excessive earthworks to reduce the vertical alignment to the desirable standard, or when these earthworks prove to be incompatible with the surrounding environment in urban areas, an absolute maximum gradient can be used. Employment of a gradient more than the desirable maximum can only be authorized through a formal Departure from Standard as described in Chapter 16.

6.5.3 Gradient and Super-elevation

The line of greatest slope on a pavement is the result of the combination of gradient with the super-elevation. This should not exceed 10%. If this value is calculated to be higher, the value of the gradient should be decreased, not the value of the super-elevation. Alternatively, the horizontal alignment should be modified.

6.5.4 Maximum Gradients at Hairpin Curves

Where hairpin curves are unavoidable in mountainous or escarpment terrain there is a need to reduce the maximum allowable gradient at any point through the curve. The maximum allowable gradient through a hairpin curve itself is 4 % for road standards S, A and B

Corresponding crest and sag curves approaching the hairpin curve must meet the requirements shown in the Tables of Standards in Chapter 8 and the transitions must be completed outside of the hairpin curve.

6.5.5 Minimum Gradients

To maintain water flow and drainage the minimum gradient for the normal situation should be 0.5%. However, flat and level gradients on un-kerbed paved highways and in tangents and non-super-elevated curves are acceptable when the cross slope and carriageway elevation above the surrounding ground is adequate to drain the surface laterally. With kerbed highways or streets, longitudinal gradients should be provided to facilitate surface drainage and adequate drains installed for lateral drainage.

6.5.6 Gradients through Villages

In many cases the natural grade level is flat through villages. The adjacent footpaths in such circumstances can easily become clogged and ineffective. Sometimes they are deliberately blocked to provide easier access to adjacent property or to channel flow for agricultural use. These practices lead to saturation of the subgrade and hence potential pavement failure and should be avoided. Covered drains to provide a footpath may be required in some areas.

6.6 Climbing Lanes and Passing Lanes

6.6.1 General

A climbing lane, also called a truck lane or crawler lane (but not a passing lane), is an auxiliary lane added outside the continuous lanes on a gradient. It reduces congestion by removing slower-moving vehicles from the traffic stream. If the traffic reduction is sufficient, the Level of Service (LoS) (Section 3.6) on the grade will match that on the preceding and succeeding grades. Road safety is also improved by the reduction of speed differentials in the through lane. The requirements for climbing lanes are therefore based on road standard, traffic volume and safety.

A passing lane is also an auxiliary lane that can be provided for the fast traffic on a gradient, but it is also used on level sections of the route to increase passing opportunities. Thus, it is used to raise the overall LoS and capacity of the route. Passing lanes are described in more detail in Section 5.13.

6.6.2 Criteria for Climbing Lanes

The use of climbing lanes is essentially limited to roads in classes S, A and B. Table 6-5 indicates typical conditions for which a climbing lane might be justified. Any grade which exceeds the critical lengths given in Table 6-5 will normally cause truck speeds to be reduced by more than 15 km/h. For an existing road a truck speed profile could be prepared for each direction of flow. This will help to identify those sections of the road where speed reductions of 15 km/h or more may warrant the provision of a climbing lane.

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

1. Climbing lanes will not be required on roads with AADT < 2000 p.c.u. in Design Year 10;
2. Where passing opportunities are limited on the gradients, then climbing lanes must be considered on S, A, B and C Class roads with traffic flows AADT in Design Year 10 in the range from 2000 p.c.u. to 6000 p.c.u.
3. Climbing lanes will normally be required on roads with AADT > 6000 p.c.u. in Design Year 10.

Consideration must always be given to the balance between the benefits to traffic and the initial construction cost. For example, in sections requiring heavy side cut, the provision of climbing lanes may be unreasonably high in relation to the benefits and hence climbing lanes may be omitted leading to reduced "levels of service" over such sections.

Where climbing lanes are to be provided, they shall be introduced on S and A class roads when the speed of a typical heavy vehicle falls by 15 km/h from that speed which this vehicle would maintain on a level or downhill section of the same road. The corresponding fall in speed applicable to B and C class roads shall be 20 km/h.

For design purposes it may be assumed that the highest obtainable speed on a level or downhill section of road for a typical heavy vehicle will be 80% of the design speed or 80 km/h whichever is the lower.

In mountainous terrain, where trucks are reduced to crawl speeds over extended distances and relatively few opportunities for overtaking exist, the cost of construction of climbing lanes may be prohibitive. An alternative solution is to construct short lengths of climbing lane (termed passing bays or partial climbing lanes) instead of a continuous lane over the length of the grade. They are typically 100 to 200 m long. Because vehicles entering the turnout do so at crawl speeds, the tapers can be short (20 to 30m long).

The climbing lane shall be terminated when the speed of a typical heavy vehicle reaches the value at which the climbing lane was introduced. However, it must be verified that a typical heavy vehicle will regain this speed without creating a traffic hazard, i.e., passing sight distances must be adequate. This latter requirement may lead to an extension of the climbing lane beyond that point determined from speed considerations alone.

An alternative to these general criteria for justifying a climbing lane is to consider using an economic analysis. Several software programs have been developed that relate the cost of construction of the climbing lane to the value of time saved by its provision. The analysis is based on calculation of delay that would ensue over the design life of the road if the climbing lane was not provided.

6.6.3 Geometric Properties of Climbing Lanes

The entrance taper to the climbing lane should be 100 m long. The full width of the climbing lane should be maintained until the point is reached where truck speeds have once again increased to be 15 km/h less than the normal speed on a level grade. The exit terminal should also be a simple taper dropping the climbing lane once it has served its purpose but to allow sufficient length for the slow vehicles to find a gap to merge. A minimum length of 200 m is recommended for this. A vehicle that cannot complete the merging manoeuvre at the end of the climbing lane has the shoulder as an emergency escape route to stop.

Climbing lanes usually have the same width as the adjacent basic lane. In very severe terrain, a reduction in width to 3.1 m can be considered because of the low speeds and lane occupancy of vehicles in the climbing lane. In addition, the shoulder width may also be reduced, but to not less than 1.5 m. If the shoulders elsewhere on the road are 3 m wide, the additional construction width required to accommodate the climbing lane and reduced shoulder is thus only 1.6 m.

The introduction and termination of a climbing lane should be accomplished by tapers of 100 m length. The tapers should not be considered as part of the climbing lanes and should be in the flatter section of gradient before the beginning and end of climbing lane.

6.6.4 Safety Aspects of Climbing Lanes

For safety reasons trucks on long downgrades should not travel at much higher speeds than they could maintain if travelling in the opposite direction. If an upgrade warrants a climbing lane and is greater than 1000 m long, the opposite side of the road may be a candidate for a descending lane, especially if traffic levels are high and the LoS is reduced by the slow speed of the trucks.

Safety considerations are important on all long downhill grades. A heavy truck that is not braking will accelerate from 0 km/h to 90 km/h over about 500 m at a descending grade of 5 %. This emphasises the need to provide warning signs for such vehicles at all long continuous grades. The use of brake check areas, “escape lanes” and other safety issues are is discussed in Chapter 13.

The position of the lane-drop must allow the slower vehicles to gain enough speed to merge with the faster vehicles. Lane-drops should not be situated on curves. Figure 6-3 illustrates the recommended layout for climbing lanes.

From a safety point of view, it is important that drivers are made aware of the start and, more particularly, the end of an auxiliary lane. The basic information required in the latter case are:

1) Indication of the presence of a lane drop;
2) Indication of the location of the lane drop; and
3) Indication of the appropriate action to be undertaken.

Climbing lanes must be clearly marked and, where possible, should end on level or downhill sections. This is where speed differences between different classes of vehicles are lowest thereby allowing safer and more efficient merging manoeuvres.

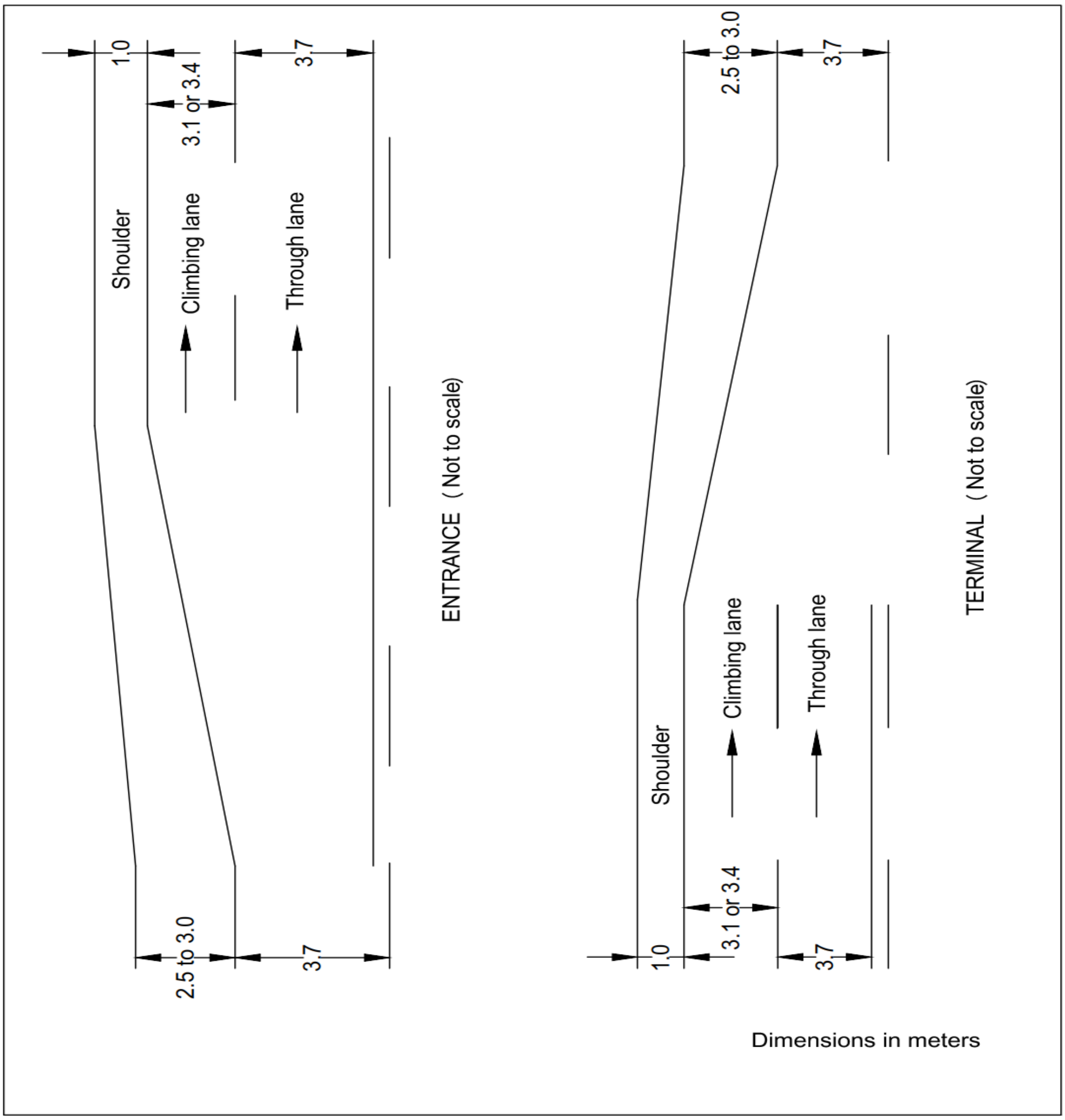


Figure 6-3: Layout of Climbing Lanes

7 Phasing of Horizontal and Vertical Alignment

7.1 Introduction

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 even more important with small radius curves than with large.

Defects may arise if an alignment is mis-phased. Defects may be purely visual and do no more than present the driver with an aesthetically displeasing impression of the road. Such defects often occur on sag curves. When these defects are severe, they may create a psychological obstacle and cause some drivers to reduce speed suddenly. In other cases, the defects may endanger the safety of the user by concealing hazards ahead. A horizontal curve hidden by a crest curve is an example of this kind of defect.

7.2 Types of Mis-phasing and 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.

Several distinct types of mis-phasing occur and are described and illustrated in the subsequent sections.

7.3 Minimum Lengths of Vertical Curves

Especially for trunk and link roads where the algebraic difference between successive gradients is often small, the intervening minimum vertical curve, obtained by applying the formula in Chapter 6, becomes very short. This can create the impression of a kink in the grade line. If the vertical alignment is allowed to contain many curves of short length, the result can be a ‘hidden dip’ profile, and/or a ‘roller coaster’ type profile, as indicated in Figure 7-1. Where the algebraic difference in gradient is less than 0.5 %, a minimum curve length is recommended for purely aesthetic reasons. The minimum length should not be less than twice the design speed in km/h and, for preference, should be 400 m or longer, except in mountainous or escarpment terrain.

A. Hidden Dip (Roller Coaster)

B. Hidden Dips Eliminated

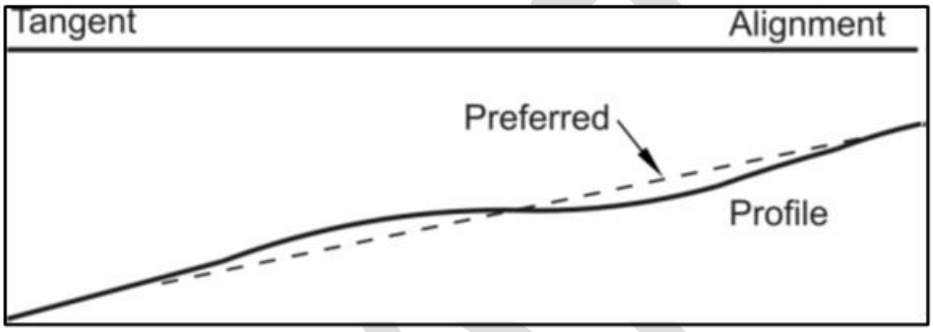
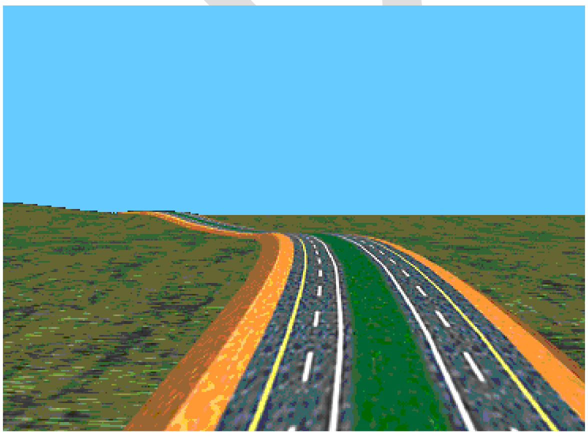


Figure 7-1: Hidden Dip (Roller Coaster) Profile

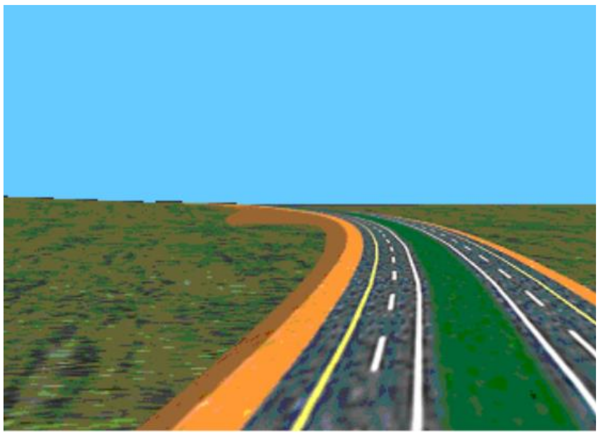
7.4 Crest and Sag Curve Have a Common Beginning or End

Where a crest curve and a succeeding sag curve have a common beginning or end, the visual effect created is that the road has suddenly dropped away. In the reverse case, the illusion of a hump is created. Both effects are removed by inserting a short length of straight grade between the two curves. Typically, 60 m to 100 m is adequate for this purpose.

Figure 7-2 illustrates the appearance of 'humps' when short crests and sags are included on a long horizontal curve. Maintaining a constant grade is the preferred option.



A. Short Humps on Long Horizontal Curve



B. Maintaining a Constant Grade

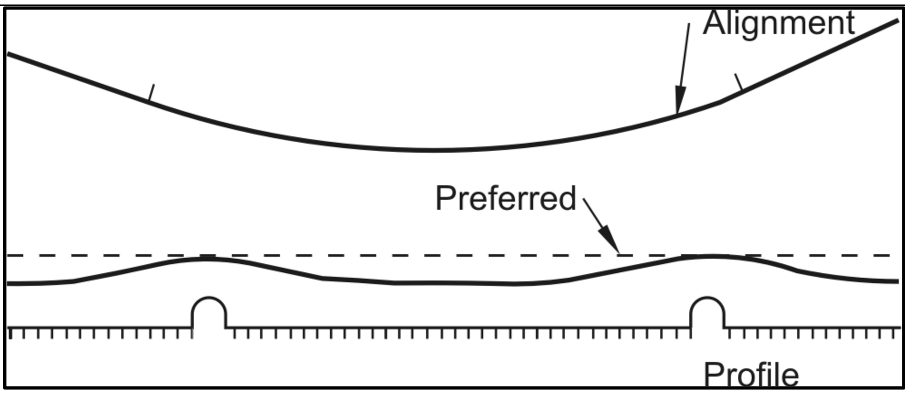


Figure 7-2: Short Humps on Long Horizontal Curve

7.5 Sag Curve at the Start of a Horizontal Curve

A sag curve at the start of a horizontal curve has the effect of enhancing the sharp angle appearance as shown in Figure 7-3 and should be avoided. Raising the preceding grade will move the sag curve downstream. A longer radius on the horizontal curve would cause it to start earlier. Applying both remedial measures should result in a better phasing of the horizontal and vertical alignments.

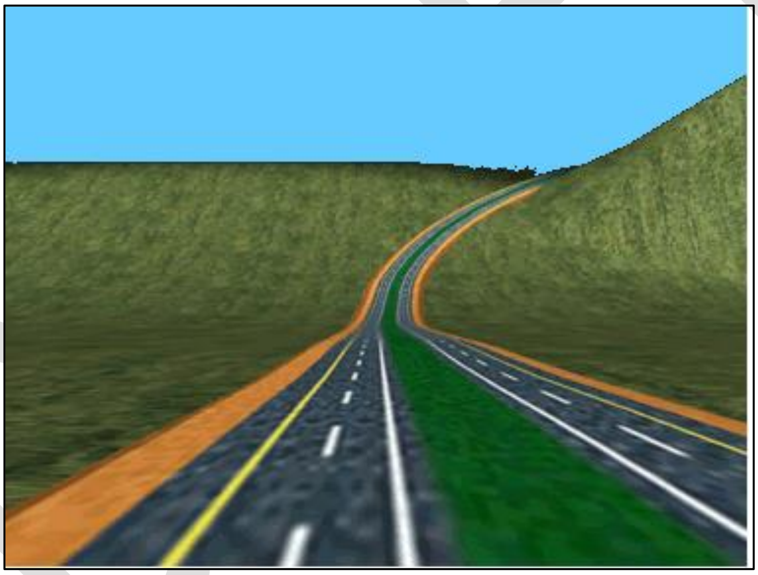
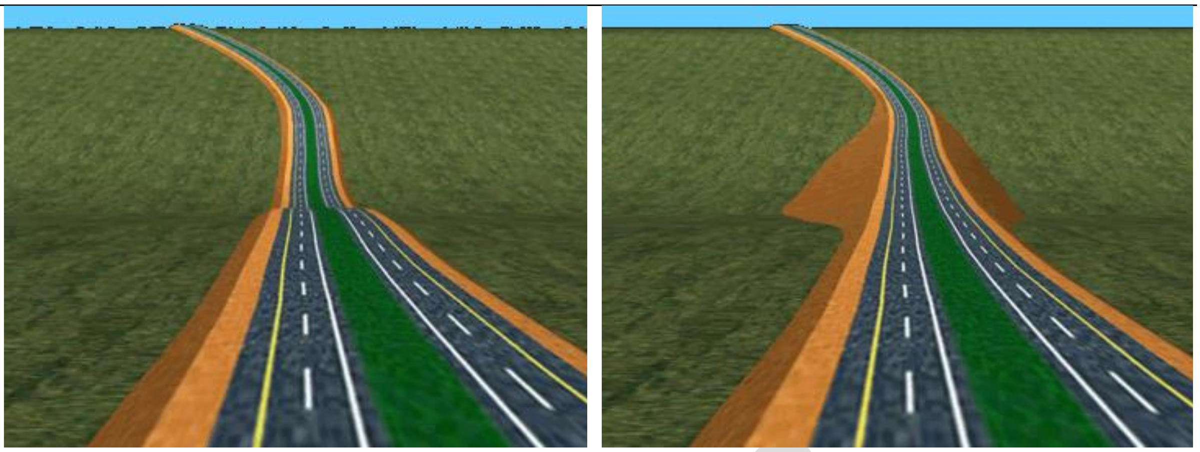


Figure 7-3: Out-of-phase Vertical and Horizontal Alignments

7.6 A Short Dip in the Alignment Preceding a Horizontal Curve

Similar to Section7.4, a short discontinuity or dip in the alignment preceding a horizontal curve creates a particularly discordant view (Figure 7-4). Again, this is similar to the 'roller coaster' profile shown in Figure 7-1 above but with a horizontal component. Eliminating the crest curve in advance and the following sag curve improves the appearance.



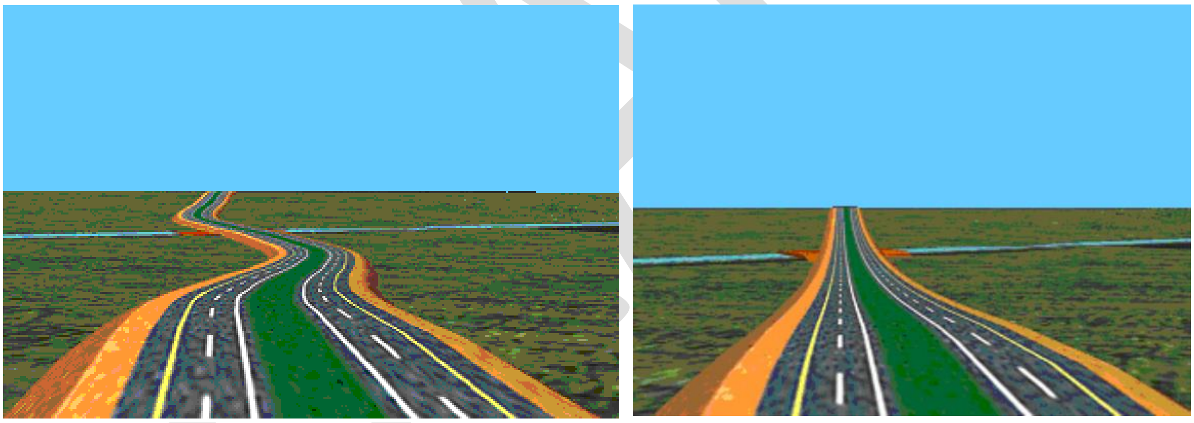
A. Short Hump and Dip Preceding Horizontal Curve

B. Eliminating the Crest Curve in Advance and the Following Sag Curve

Figure 7-4: Short Dip in the Alignment Preceding a Horizontal Curve

7.7 Distorted Alignment

A common fault is illustrated in Figure 7-5. The roadway is often unnaturally curved to cross a small stream at right angles. The advantages in the aesthetics of an alignment with a skew crossing often far outweigh the savings deriving from a square crossing.



A. Distorted Alignment to Create a Square River Crossing

B. Skew Crossing Improves Horizontal Alignment

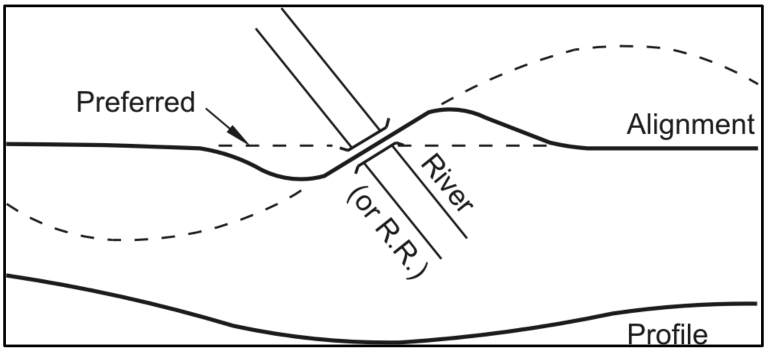
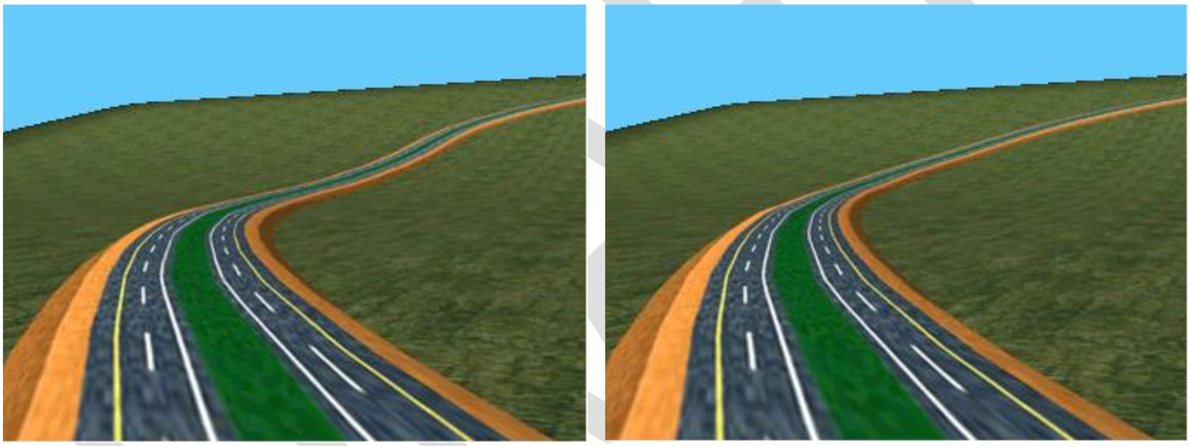


Figure 7-5: Distorted Alignment to Create a Square River Crossing

7.8 Broken Back Curves

Figure 7-6 illustrates a broken-back curve. This is two curves in the same direction separated by a short tangent. Such a combination can be improved. Also, a 'broken plank' grade line, where two long grades are connected by a short sag curve, is equally unacceptable. Using a single radius curve throughout as illustrated is preferred.



A. Broken-back Curve

B. Replacement of Broken-Back Curve by Single Radius Long Curve

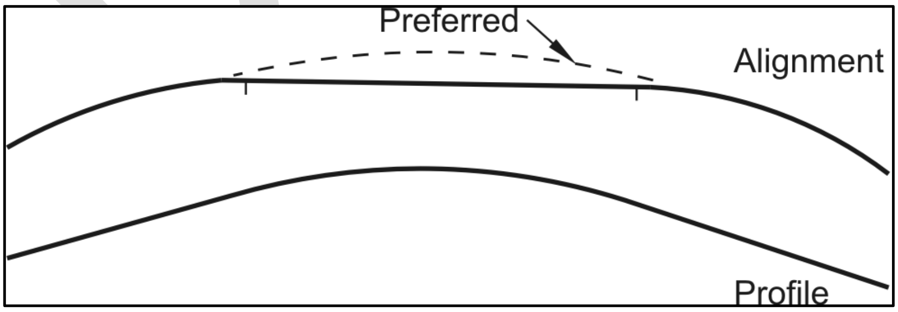


Figure 7-6: Broken-back Curve.

7.9 Variations in Vertical Alignment on Long Horizontal Curves

Significant changes in grade of the vertical alignment, as shown in Figure 7-7 should be avoided on long horizontal curves.

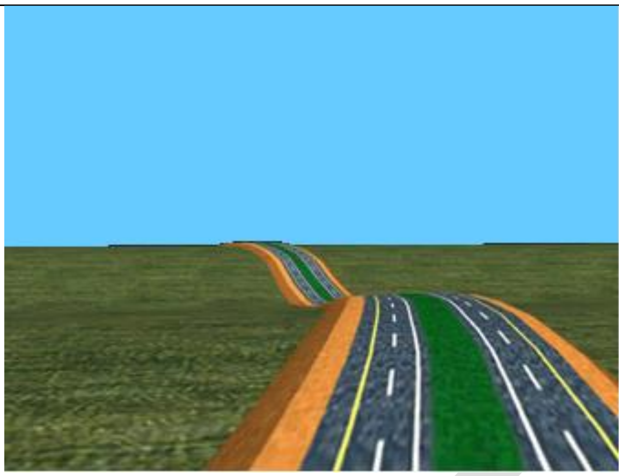


Figure 7-7: Variable Gradients (Rolling Grade-line)

7.10 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. The corrective action is to make both ends of the curves coincident, or to separate them.

7.11 Start of Horizontal Curves not Visible

Figure 7-8 shows the effect when the start of a horizontal curve is hidden by an intervening crest and the continuation of the curve is visible in the distance. The road appears disjointed.

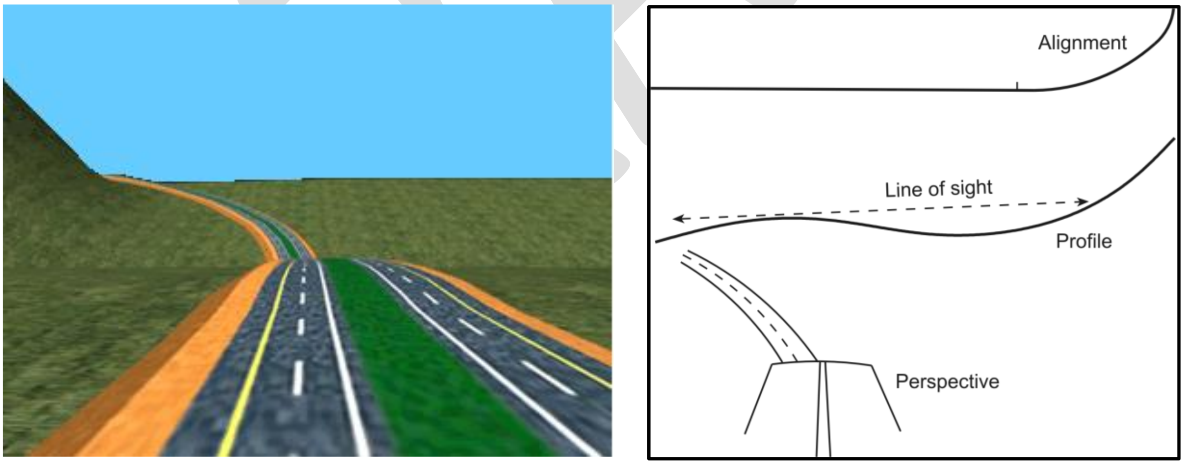


Figure 7-8: Break in Horizontal Alignment

7.12 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, and the resulting crashes are usually head-on collisions. The position of the crest is important because 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.

The defect may be corrected in both cases by completely separating the curves. If this is uneconomic, the curves must be adjusted so that if the horizontal curve is of short radius, they are coincident at both ends, or if the horizontal curve is of longer radius, they need be coincident at only one end.

7.13 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. Corrective action consists of increasing the separation between the curves or making the curves concurrent.

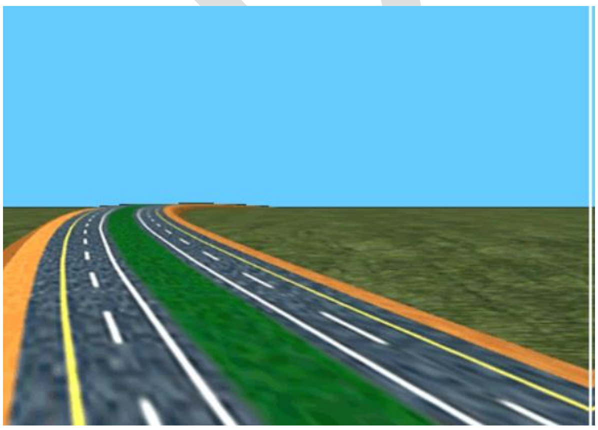
7.14 Vertical Curve Overlaps Both Ends of the Horizontal Curve

If a vertical crest curve overlaps both ends of a sharp horizontal curve, a hazard may be created because a vehicle must 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 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, will appear in the road alignment. The corrective action is to make both ends of the curves coincident or to separate them.

It is important to note that local dips to minimise earthworks that result in a disjointed alignment will be there for the life of the road Figure 7-9 illustrate the advantages of co-ordinating the horizontal and vertical alignment. In each case the vertical curve is contained within the horizontal curve.



A. Well-coordinated Crest and Horizontal Curves

B. Well-coordinated Sag and Horizontal Curves

Figure 7-9: Well-coordinated horizontal and vertical alignment

7.15 The Economic Cost of Good Phasing

The correct phasing of vertical curves restricts the designer in fitting the road to the topography at the lowest cost. Therefore, phasing is usually bought at the cost of extra earthworks and the designer must decide at what point it becomes uneconomic. The designer 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 trial alignments against their elegance.

7.16 Vertical Clearances

Bridges over water normally have a minimum clearance height according to Table 9-1 unless a refined hydraulic analysis has been made. The standard minimum headroom or clearance under bridges or tunnels is 5.1 m for all classes of roads. This clearance should be maintained over the roadway(s) and shoulders. Where future maintenance of the roadway is likely to lead to raising of the road level, then an additional clearance of up to 0.1 m may be provided. Light superstructures (e.g., timber, steel trusses, steel girders, etc) over roadways should have a clearance height of at least 5.3 m. (**RDM Volume 4; Part 1 – Bridge and Culvert Design**) for further reference.

Table 7-1: Vertical Clearance from Superstructure to Design Flood Level

Design Flow at Bridge (m3/s)	Vertical Clearance (m)
5 to 30	0.6
30 to 300	0.9
>300	1.2

Underpasses for pedestrians and bicycles should not be less than 2.4 m. For cattle and wildlife, underpasses must be designed as the normal height of the actual animal plus 0.5 m. Bridges above railways must have a clearance height of at least 6.1 m - if not otherwise stated - to facilitate possible future electrification.

Over existing pipe culverts and box culverts, the roadway elevation cannot be less than as indicated in the RDM Volume 2 Part 2 – Drainage Design.

8 Geometric Design Standards

8.1 General

8.1.1 Standards Development and Application

The development of the standards in these Tables are described in Chapters 2, 3, 4, 5, 6 and 7. As this document is a design manual and not a textbook, the descriptions of the standards in this Chapter are necessarily brief aide memoires. References, of course, are also provided.

The various factors that influence the design standards have been introduced and explained in the previous chapters. To summarise, the main factors are:

1) Functional Class (Chapter 2). This is strongly related to the 'importance' of the road hence the higher the functional class the higher the road design standard within the acceptable bounds.
2) Traffic level (Chapter 3.2). Higher traffic warrants a higher road standard;.
3) Terrain (Chapter 2). Roads in difficult terrain are relatively expensive and do not justify the highest standards.

A balance, as always, must be struck between the competing desire for high standards and the inevitable higher costs that this will require. Economic analysis can assist with this at the planning level but for geometric design the key is for the designer to focus on achieving a high level of safety.

All design standards have been developed based on sound engineering principles but there is a very wide range of situations that must be catered for. The most straightforward and perhaps the most appreciated is the different requirements for roads in urban areas and roads in rural areas. Most design methods deal with this, but many do not deal with the wide range of situations that have arisen in modern urban development. Urban areas must cater for a variety of different primary functions ranging from those associated with the central business district (CBD), smaller 'satellite business areas, or urban 'villages' areas (for want of a better description) areas for light industrial activity, residential areas ranging from high to low density accommodation, recreational areas plus areas close to and influenced by main arterial roads that pass through the urban area. There are various names for some of these functions such as 'peri-urban' roads, 'metropolitan' roads and more but these terms are not universally used but the principle of them all is that they comprise what is now being referred to as components of the all-embracing term and philosophy of 'Movement Networks' which provides a comprehensive approach to dealing with the complexity of modern urban areas and the activities that must be accommodated safely within it.

8.1.2 Interurban Roads Geometric Design Standards

The interurban roads system consists of a network of routes with the following service characteristics:

1) Corridor movement with trip length and density suitable for substantial national or international travel.
2) Movements between all, or virtually all, urban areas including cities and large townships.

3) Integrated movement without stub connections except where unusual geographic or traffic flow conditions dictate otherwise (e.g., international boundary connections or ferry connections)
4) High travel speeds and minimum interference to through movement consistent with the roadway context and considering the range of users to be served.

In the more densely populated areas of Kenya, this class of highway includes most (but not all) heavily travelled routes that might warrant multilane improvements. Interurban roads are stratified into three classifications: Class A, B, and C. The design standards applicable for such corridors must therefore prioritize mobility with separation of low and high-speed vehicles in highly trafficked areas. With low volumes of NMT, the shoulders may be used as footpaths to accommodate pedestrians. If need be, such as when such corridors pas through small urban centres, NMT facilities should be provided.

The applicable design standards for design of interurban roads are as shown in Table 8-1 below. Details of each of the design standards are detailed in Table 8-4 to Table 8-10.

Table 8-1: Applicable design standards for interurban roads.

Functional classification	Applicable design standards
A	DR1, DR2
B	DR2, DR3, DR4
C	DR3, DR4, DR5

8.1.3 Rural Roads Geometric Design Standards

Rural roads generally serve travel of primarily inter-county and inter-sub county importance and constitute those routes on which (regardless of traffic volume) predominant travel distances are shorter than on interurban routes. Consequently, more moderate speeds may be typical and frequent access to roadside development may be provided, consistent with the roadway context and the range of users to be served.

Similar to interurban roads, shoulders may be used as footpaths where pedestrian traffic volumes are low, and footpath provided within sections of high pedestrian activity.

The applicable design standards for design of rural roads are as shown in Table 8-2. Details of each of the design standards are detailed in Table 8-4 to Table 8-10.

Table 8-2: Applicable design standards for rural roads.

Functional classification	Applicable design standards
D	DR4, DR5, DR6
E	DR6, DR7
Minor roads	DR7

8.1.4 Urban Roads Geometric Design Standards

The applicable design standards for design of urban roads are as shown in Table 8-3 with details of application given in Table 8-11 to Table 8-15.

Table 8-3: Applicable design standards for urban roads.

Functional classification	Applicable design standards
UA	DU1
UC	DU2, DU3, DU4
UL	DU4, DU5

8.2 Design Standards for Interurban and Rural Roads

Table 8-4: Design Standard DR1 – Interurban Multilane

	DESIGN ELEMENT			REF. SECTION	CLASS A MULTILANE ROAD					
Design Controls	Design Life				20 years					
	Design Traffic (AADT)									
	Design Speed (Km/Hr)				Flat		Rolling		Mountainous	
					120 – 110		100 - 90		80 - 70	
	Control of Access				Full					
	Level of Service Threshold				C					
Cross-Section Elements	Lane Width				3.65 m					
	Outside Shoulder Width	Paved			2.7 m					
		Un-paved			N/A					
	Median Shoulder Width	Paved			1.5 m					
		Un-paved			N/A					
	Crossfall Slope	Travel Lane			2.5 %					
		Shoulder			4.0%					
	Auxiliary Lanes	Lane Width			3.0 m					
		Shoulder Width			Paved: 1.0 m Unpaved – N/A					
	Median Width	Depressed			2m Minimum					
		Concrete Median Barrier			N/A					
	Road Reserve				Desirable: 60 m; Reduced Desirable: 40 m					
	Roadside Clear Zone	Low Traffic (Fill/Cut)			5 m; 3 m					
		Medium Traffic (Fill/Cut)			8.5 m; 5.5 m					
	Slopes	Cut	Cut Slope		1:4					
			Depth of Ditch		0.75 m					
			Backslope		1:3					
		Fill	Safety Slope (within clear zone)		1:4					
			Fill Slope (outside clear zone)		1:3					
Alignment Elements	**Design Speed (Km/hr)**				**120**	**110**	**100**	**90**	**80**	**70**
	Stopping Sight Distance (m)	g: 0%			285	245	205	170	140	110
		g: 5%			330	285	235	195	155	120
		g: 10%			400	330	280	230	180	140
	Passing Sight Distance (m)	Desirable			780	703	670	615	550	485
		Minimum			395	350	310	275	240	210
	% Passing Opportunity				50		33		25	
	Maximum Super-elevation Rate %				8	8	8	8	8	8

	Minimum Horizontal Curve Radius (m)	SE: 4%		N/A	N/A	525	400	300	205
		SE: 6%		755	595	465	355	265	185
		SE: 8%		665	530	415	320	240	170
	Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %					
		Rolling		Desirable: 4 %; Absolute: 6 %					
		Mountainous		Desirable: 6 %; Absolute: 8 %					
	Minimum Grades			0.5 %					
	Vertical Curve (K-values)	Crest		185	140	100	67	45	30
		Sag (Comfort)		36	30	25	20	16	12
		Sag (Headlights)		70	60	50	40	32	25

Table 8-5: Design Standard DR2 – Rural Single Carriageway Class A, B

	DESIGN ELEMENT			REF. SECTION	CLASS A; CLASS B ROADS			
Design Controls	Design Life				20 years			
	Design Traffic (AADT)							
	Design Speed (Km/Hr)				Flat	Rolling	Mountainous	Escarpment
		Class A			120 – 110	100 - 90	80 - 70	50
		Class B			110 - 90	90 - 85	70 - 60	50
	Control of Access				Partial			
	Level of Service Threshold				C			
Cross-Section Elements	Lane Width				3.65 m			
	Outside Shoulder Width	Paved			2.7 m			
		Un-paved			N/A			
	Median Shoulder Width	Paved			1.5 m			
		Un-paved			N/A			
	Crossfall Slope	Travel Lane			2.5 %			
		Shoulder			4.0 %			
	Auxiliary Lanes	Lane Width			3.0 m			
		Shoulder Width			Paved: 1.0 m Unpaved – N/A			
	Median Width	Depressed			2m Minimum			
		Concrete Median Barrier			N/A			
	Road Reserve				Desirable: 60 m; Reduced: 40 m			
	Roadside Clear Zone	Low Traffic (Fill/Cut)			5 m; 3 m			
		Medium Traffic (Fill/Cut)			8.5 m; 5.5 m			
	Slopes	Cut	Cut Slope		1:4			
			Depth of Ditch		0.75 m			
			Backslope		1:3			
		Fill	Safety Slope (within clear zone)		1:4			
			Fill Slope (outside clear zone)		1:3			

Alignment Elements	**Design Speed (Km/hr)**			**120**	**110**	**100**	**90**	**80**	**70**	**60**	**50**
	Stopping Sight Distance (m)	g: 0%		285	245	205	170	140	110	85	65
		g: 5%		330	285	235	195	155	120	95	70
		g: 10%		400	330	280	230	180	140	105	75
	Passing Sight Distance (m)	Desirable		780	703	670	615	550	485	420	345
		Minimum		395	350	310	275	240	210	180	155
	% Passing Opportunity			50		33		25			
	Maximum Super-elevation Rate %			8	8	8	8	8	8	8	8

	Minimum Horizontal Curve Radius (m)	SE: 4%		N/A	N/A	525	400	300	205	140	95
		SE: 6%		755	595	465	355	265	185	130	85
		SE: 8%		665	530	415	320	240	170	120	80
	Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %							
		Rolling		Desirable: 4 %; Absolute: 6 %							
		Mountainous		Desirable: 6 %; Absolute: 8 %							
	Minimum Grades			0.5 %							
	Vertical Curve (K-values)	Crest		185	140	100	67	45	30	17	10
		Sag (Comfort)		36	30	25	20	16	12	9	6.5
		Sag (Headlights)		70	60	50	40	32	25	19	14

Table 8-6: Design Standard DR3 – Rural Single Carriageway Class B, C

	DESIGN ELEMENT			REF. SECTION	CLASS B; CLASS C ROADS							
Design Controls	Design Life				20 years							
	Design Traffic (AADT)											
	Design Speed (Km/Hr)				Flat	Rolling	Mountainous	Escarpment				
		Class B			110 - 90	90 - 85	70 - 60	50				
		Class C			100 - 80	90 - 65	70 - 50	50				
	Control of Access				Full							
	Level of Service Threshold				C							
Cross-Section Elements	Lane Width				3.65 m							
	Outside Shoulder Width	Paved			2.7 m							
		Un-paved			N/A							
	Median Shoulder Width	Paved			1.5 m							
		Un-paved			N/A							
	Crossfall Slope	Travel Lane			2.5 %							
		Shoulder			4.0 %							
	Auxiliary Lanes	Lane Width			3.0 m							
		Shoulder Width			Paved: 1.0 m Unpaved – N/A							
	Median Width	Depressed			2m Minimum							
		Concrete Median Barrier			N/A							
	Road Reserve				Class B: Desirable: 60 m; Reduced: 40 m							
					Class C: Desirable: 40 m; Reduced: 40 m							
	Roadside Clear Zone	Low Traffic (Fill/Cut)			5 m; 3 m							
		Medium Traffic (Fill/Cut)			8.5 m; 5.5 m							
	Slopes	Cut	Cut Slope		1:4							
			Depth of Ditch		0.75 m							
			Backslope		1:3							
		Fill	Safety Slope (within clear zone)		1:4							
			Fill Slope (outside clear zone)		1:3							
Alignment Elements	**Design Speed (Km/hr)**					**110**	**100**	**90**	**80**	**70**	**60**	**50**
	Stopping Sight Distance (m)	g: 0%				245	205	170	140	110	85	65
		g: 5%				285	235	195	155	120	95	70
		g: 10%				330	280	230	180	140	105	75
	Passing Sight Distance (m)	Desirable				703	670	615	550	485	420	345
		Minimum				350	310	275	240	210	180	155
	% Passing Opportunity						33		25			

	Maximum Super-elevation Rate %				8	8	8	8	8	8	8
	Minimum Horizontal Curve Radius (m)	SE: 4%			N/A	525	400	300	205	140	95
		SE: 6%			595	465	355	265	185	130	85
		SE: 8%			530	415	320	240	170	120	80
	Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %							
		Rolling		Desirable: 4 %; Absolute: 6 %							
		Mountainous		Desirable: 6 %; Absolute: 8 %							
	Minimum Grades			0.5 %							
	Vertical Curve (K-values)	Crest			140	100	67	45	30	17	10
		Sag (Comfort)			30	25	20	16	12	9	6.5
		Sag (Headlights)			60	50	40	32	25	19	14

Table 8-7: Design Standard DR4 – Rural Single Carriageway Class B, C, D

	DESIGN ELEMENT			REF. SECTION	CLASS B; CLASS C; CLASS D ROADS			
Design Controls	Design Life				20 years			
	Design Traffic (AADT)							
	Design Speed (Km/Hr)				Flat	Rolling	Mountainous	Escarpment
		Class B			110 - 90	90 - 85	70 - 60	50
		Class C			100 - 80	90 - 65	70 - 50	50
		Class D			90 – 70	85 – 65	60 - 50	50 - 40
	Control of Access				Partial			
	Level of Service Threshold				A			
Cross-Section Elements	Lane Width				Class B - 3.65 m; Classes C, D – 3.0m			
	Outside Shoulder Width	Paved			N/A			
		Un-paved			N/A			
	Median Shoulder Width	Paved			N/A			
		Un-paved			N/A			
	Crossfall Slope	Travel Lane			2.5 %			
		Shoulder			N/A			
	Auxiliary Lanes	Lane Width			N/A			
		Shoulder Width			N/A			
	Median Width	Depressed			N/A			
		Concrete Median Barrier			N/A			
	Desirable Right of Way				Class B: Desirable: 60 m; Reduced: 40 m			
					Class C: Desirable: 40 m; Reduced: 40 m			
					Class D: Desirable: 25 m; Reduced: 25 m			
	Roadside Clear Zone	Low Traffic (Fill/Cut)			5 m; 3 m			
		Medium Traffic (Fill/Cut)			8.5 m; 5.5 m			
	Slopes	Cut	Cut Slope		1:4			
			Depth of Ditch		0.75 m			
			Backslope		1:3			
		Fill	Safety Slope (within clear zone)		1:4			
			Fill Slope (outside clear zone)		1:3			

					110	100	90	80	70	60	50	40
Alignment Elements	**Design Speed (Km/hr)**				**110**	**100**	**90**	**80**	**70**	**60**	**50**	**40**
	Stopping Sight Distance (m)	g: 0%			245	205	170	140	110	85	65	45
		g: 5%			285	235	195	155	120	95	70	47
		g: 10%			330	280	230	180	140	105	75	50
	Passing Sight	Desirable			703	670	615	550	485	420	345	275

	Distance (m)	Minimum			350	310	275	240	210	180	155	135
	% Passing Opportunity					33		25				
	Maximum Super-elevation Rate %				8	8	8	8	8	8	8	8
	Minimum Horizontal Curve Radius (m)	SE: 4%			N/A	525	400	300	205	140	95	55
		SE: 6%			595	465	355	265	185	130	85	50
		SE: 8%			530	415	320	240	170	120	80	45
	Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %								
		Rolling		Desirable: 4 %; Absolute: 6 %								
		Mountainous		Desirable: 6 %; Absolute: 8 %								
	Minimum Grades			0.5 %								
	Vertical Curve (K-values)	Crest			140	100	67	45	30	17	10	5
		Sag (Comfort)			30	25	20	16	12	9	6.5	4
		Sag (Headlights)			60	50	40	32	25	19	14	9

Table 8-8: Design Standard DR5 – Rural Single Carriageway Class C, D, E

	DESIGN ELEMENT			REF. SECTION	CLASS C; CLASS D; CLASS E ROADS			
Design Controls	Design Life				20 years			
	Design Traffic (AADT)							
	Design Speed (Km/Hr)				Flat	Rolling	Mountainous	Escarpment
		Class C			100 - 80	90 - 65	70 - 50	50
		Class D			90 – 70	85 – 65	60 - 50	50 - 40
		Class E			70 - 50	65 - 40	50 - 30	30
	Control of Access				Partial			
	Level of Service Threshold				A			
Cross-Section Elements	Lane Each Direction				1			
	Lane Width				Class C: 3.5 m; Classes D, E: 3.25 m			
	Outside Shoulder Width	Paved			N/A			
		Un-paved			N/A			
	Median Shoulder Width	Paved			N/A			
		Un-paved			N/A			
	Crossfall Slope	Travel Lane			2.5 %			
		Shoulder			N/A			
	Auxiliary Lanes	Lane Width			N/A			
		Shoulder Width			N/A			
	Median Width	Depressed			N/A			
		Concrete Median Barrier			N/A			
	Road Reserve				Class C: Desirable: 40 m; Reduced: 40 m			
					Class D: Desirable: 25 m; Reduced: 25 m			
					Class E: Desirable: 20 m; Reduced: 20 m			
	Roadside Clear Zone	Low Traffic (Fill/Cut)			5 m; 3 m			
		Medium Traffic (Fill/Cut)			8.5 m; 5.5 m			
	Slopes	Cut	Cut Slope		1:4			
			Depth of Ditch		0.75 m			
			Backslope		1:3			
		Fill	Safety Slope (within clear zone)		1:4			
			Fill Slope (outside clear zone)		1:3			

						100	90	80	70	60	50	40	30
Alignment Elements	**Design Speed (Km/hr)**					**100**	**90**	**80**	**70**	**60**	**50**	**40**	**30**
	Stopping Sight Distance (m)	g: 0%				205	170	140	110	85	65	45	30
		g: 5%				235	195	155	120	95	70	47	31
		g: 10%				280	230	180	140	105	75	50	33

	Passing Sight Distance (m)	Desirable				670	615	550	485	420	345	275	195
		Minimum				310	275	240	210	180	155	135	115
	% Passing Opportunity					33		25					
	Maximum Super-elevation Rate %					8	8	8	8	8	8	8	8
	Minimum Horizontal Curve Radius (m)	SE: 4%				525	400	300	205	140	95	55	30
		SE: 6%				465	355	265	185	130	85	50	26
		SE: 8%				415	320	240	170	120	80	45	24
	Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %									
		Rolling		Desirable: 4 %; Absolute: 6 %									
		Mountainous		Desirable: 6 %; Absolute: 8 %									
	Minimum Grades			0.5 %									
	Vertical Curve (K-values)	Crest				100	67	45	30	17	10	5	2
		Sag (Comfort)				25	20	16	12	9	6.5	4	2.5
		Sag (Headlights)				50	40	32	25	19	14	9	5

Table 8-9: Design Standard DR6 – Rural Single Carriageway Class D, E

	DESIGN ELEMENT			REF. SECTION	CLASS D; CLASS E ROADS			
Design Controls	Design Life				20 years			
	Design Traffic (AADT)							
	Design Speed (Km/Hr)				Flat	Rolling	Mountainous	Escarpment
		Class D			90 – 70	85 – 65	60 - 50	50 - 40
		Class E			70 - 50	65 - 40	50 - 30	30 - 30
	Control of Access				None			
	Level of Service Threshold				A			
Cross-Section Elements	Lanes Each Direction				1			
	Lane Width				Class D: 3.5 m; Class E: 3.25 m – 3.5 m			
	Outside Shoulder Width	Paved			N/A			
		Un-paved			N/A			
	Median Shoulder Width	Paved			N/A			
		Un-paved			N/A			
	Crossfall Slope	Travel Lane			2.5 %			
		Shoulder			4.0 %			
	Auxiliary Lanes	Lane Width			N/A			
		Shoulder Width			N/A			
	Median Width	Depressed			N/A			
		Concrete Median Barrier			N/A			
	Road Reserve				Class D: Desirable: 25 m; Reduced: 25 m			
					Class E: Desirable: 20 m; Reduced: 20 m			
	Roadside Clear Zone	Low Traffic (Fill/Cut)			5 m; 3 m			
		Medium Traffic (Fill/Cut)			8.5 m; 5.5 m			
	Slopes	Cut	Cut Slope		1:4			
			Depth of Ditch		0.75 m			
			Backslope		1:3			
		Fill	Safety Slope (within clear zone)		1:4			
			Fill Slope (outside clear zone)		1:3			

			REF. SECTION							
Alignment Elements	**Design Speed (Km/hr)**			**90**	**80**	**70**	**60**	**50**	**40**	**30**
	Stopping Sight Distance (m)	g: 0%		170	140	110	85	65	45	30
		g: 5%		195	155	120	95	70	47	31
		g: 10%		230	180	140	105	75	50	33
	Passing Sight Distance (m)	Desirable		615	550	485	420	345	275	195
		Minimum		275	240	210	180	155	135	115

	% Passing Opportunity				25					
	Maximum Super-elevation Rate %			8	8	8	8	8	8	8
	Minimum Horizontal Curve Radius (m)	SE: 4%		400	300	205	140	95	55	30
		SE: 6%		355	265	185	130	85	50	26
		SE: 8%		320	240	170	120	80	45	24
	Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %						
		Rolling		Desirable: 4 %; Absolute: 6 %						
		Mountainous		Desirable: 6 %; Absolute: 8 %						
	Minimum Grades			0.5 %						
	Vertical Curve (K-values)	Crest		67	45	30	17	10	5	2
		Sag (Comfort)		20	16	12	9	6.5	4	2.5
		Sag (Headlights)		40	32	25	19	14	9	5

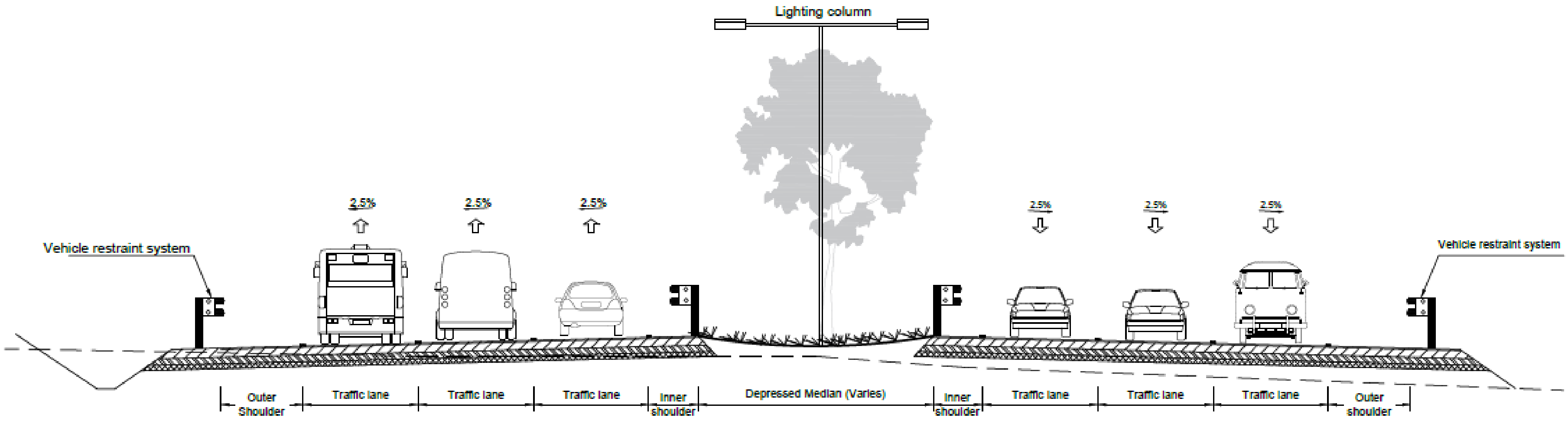
Table 8-10: Design Standard DR7 – Rural Minor Roads Class E, F, G, S, W, T

	DESIGN ELEMENT			REF. SECTION	CLASSES E, F, G, S, W, T			
Design Controls	Design Life				20 years			
	Design Traffic (AADT)							
	Design Speed (Km/Hr)				Flat	Rolling	Mountainous	Escarpment
		Class D			90 – 70	85 – 65	60 - 50	50 - 40
		Class E			70 - 50	65 - 40	50 - 30	30 - 30
	Control of Access				None			
	Level of Service Threshold				A			
Cross-Section Elements	Lane Each Direction				1			
	Lane Width				Class E: 3.25 m; Classes F, G, S, W, T: 2.25 m – 3.5 m			
	Outside Shoulder Width	Paved			N/A			
		Un-paved			N/A			
	Median Shoulder Width	Paved			N/A			
		Un-paved			N/A			
	Crossfall Slope	Travel Lane			2.5 %			
		Shoulder			4.0 %			
	Auxiliary Lanes	Lane Width			N/A			
		Shoulder Width			N/A			
	Median Width	Depressed			N/A			
		Concrete Median Barrier			N/A			
	Road Reserve				Desirable: 20 m; Reduced: 20 m			
	Roadside Clear Zone	Low Traffic (Fill/Cut)			5 m; 3 m			
		Medium Traffic (Fill/Cut)			8.5 m; 5.5 m			
	Slopes	Cut	Cut Slope		1:4			
			Depth of Ditch		0.75 m			
			Backslope		1:3			
		Fill	Safety Slope (within clear zone)		1:4			
			Fill Slope (outside clear zone)		1:3			

	DESIGN ELEMENT		REF. SECTION							
Alignment Elements	**Design Speed (Km/hr)**			**90**	**80**	**70**	**60**	**50**	**40**	**30**
	Stopping Sight Distance (m)	g: 0%		170	140	110	85	65	45	30
		g: 5%		195	155	120	95	70	47	31
		g: 10%		230	180	140	105	75	50	33
	Passing Sight Distance (m)	Desirable		615	550	485	420	345	275	195
		Minimum		275	240	210	180	155	135	115
	% Passing Opportunity				25					

	Maximum Super-elevation Rate %			8	8	8	8	8	8	8
	Minimum Horizontal Curve Radius (m)	SE: 4%		400	300	205	140	95	55	30
		SE: 6%		355	265	185	130	85	50	26
		SE: 8%		320	240	170	120	80	45	24
	Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %						
		Rolling		Desirable: 4 %; Absolute: 6 %						
		Mountainous		Desirable: 6 %; Absolute: 8 %						
	Minimum Grades			0.5 %						
	Vertical Curve (K-values)	Crest		67	45	30	17	10	5	2
		Sag (Comfort)		20	16	12	9	6.5	4	2.5
		Sag (Headlights)		40	32	25	19	14	9	5

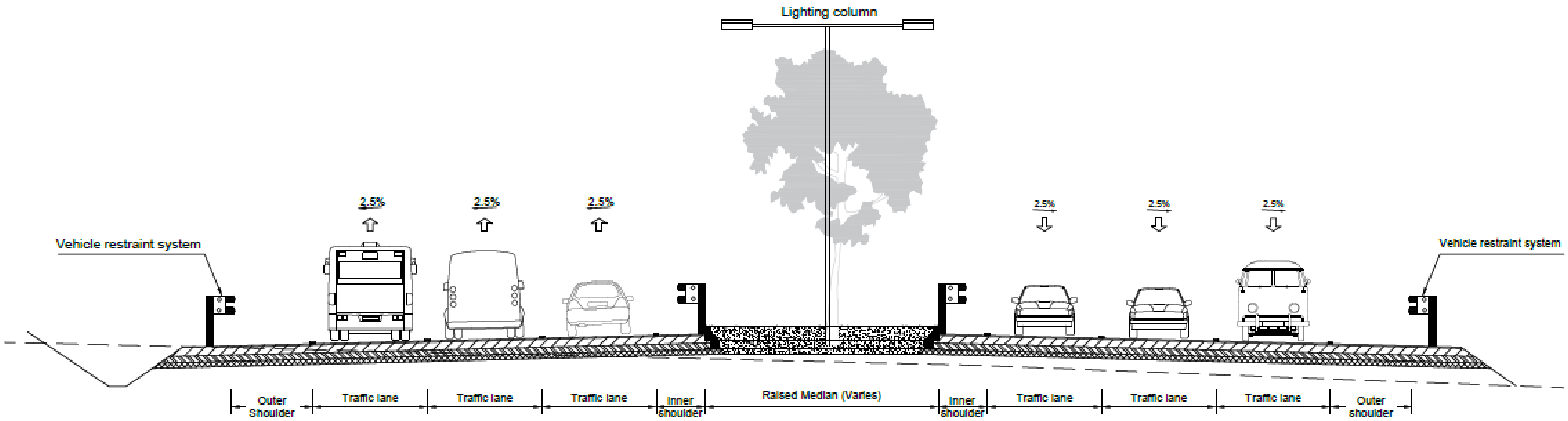
8.3 Typical Cross Sections – Interurban and Rural Roads



Typical cross section - Type 1
(Tripple carriageway with depressed median)

Road Class	International Highways			Remark
Type of Terrain	Width of road element (m)			
	Carriageway	Shoulders	Sidewalk	
Flat	3x7.3	1.5 - 2.5		Tripple Carriageway
Rolling	3x7.3	1.5 - 2.5		
Mountain	3x7.0	1.5		
Escarpment	3x6.5	(1.0) 2.0		

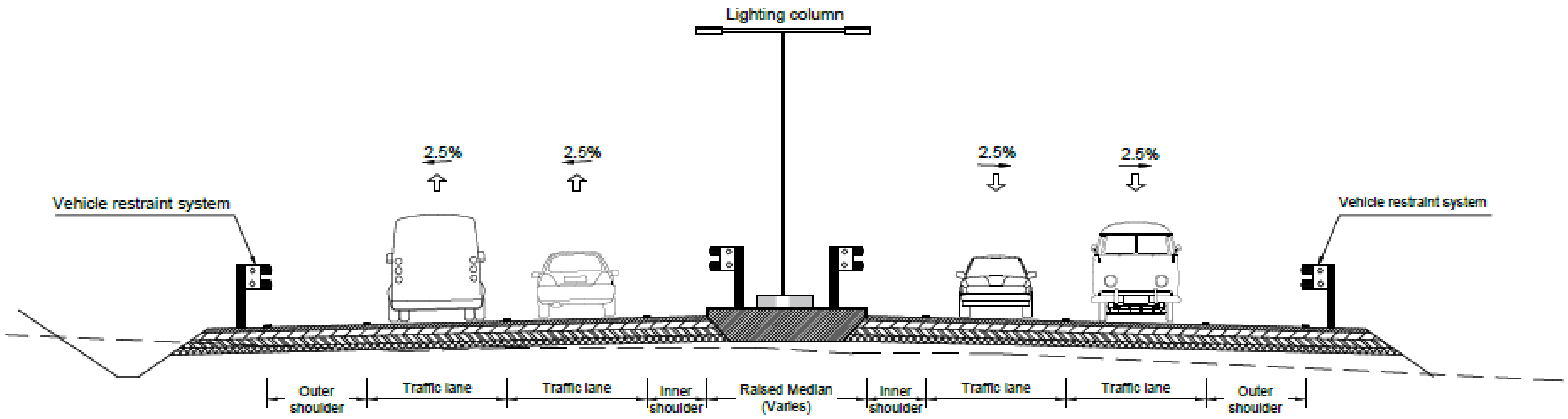
Figure 8-1: Interurban Multilane Road (Depressed Median)



Typical cross section - Type 2
(Tripple carriageway with raised median)

Road Class	International Highways			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Sidewalk	
Flat	3x7.3	1.5 - 2.5		Tripple Carriageway
Rolling	3x7.3	1.5 - 2.5		
Mountain	3x7.0	1.5		
Escarpment	3x6.5	(1.0) 2.0		

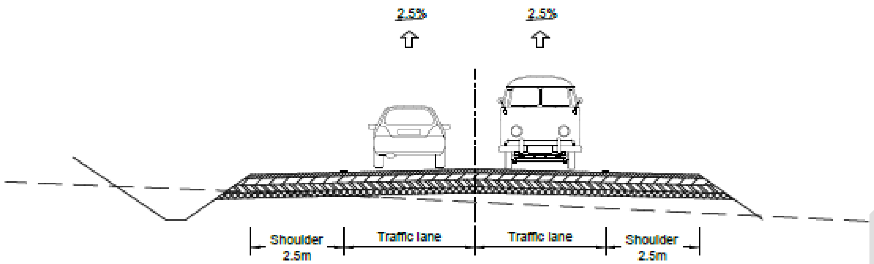
Figure 8-2: Interurban Multilane Road (Raised Median)



Typical cross section - Type 3
(Dual carriageway with raised median)

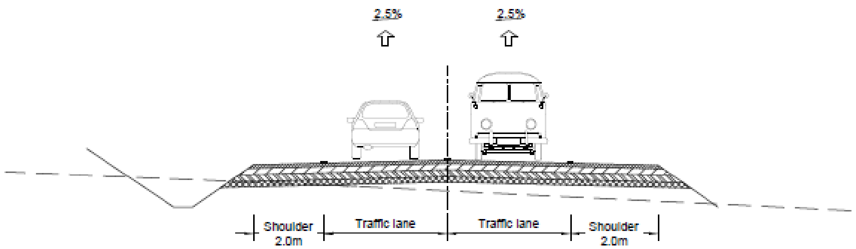
Road Class	International Highways			Remark
Type of Terrain	Width of road element (m)			
	Carriageway	Shoulders	Sidewalk	
Flat	2x7.3	1.5 - 2.5		Dual Carriageway
Rolling	2x7.3	1.5 - 2.5		
Mountain	2x7.0	1.5		
Escarpment	2x6.5	(1.0) 2.0		

Figure 8-3: Dual Carriageway (Raised Median)



Typical cross section Type 4
(Single carriageway with 2.5m shoulders)

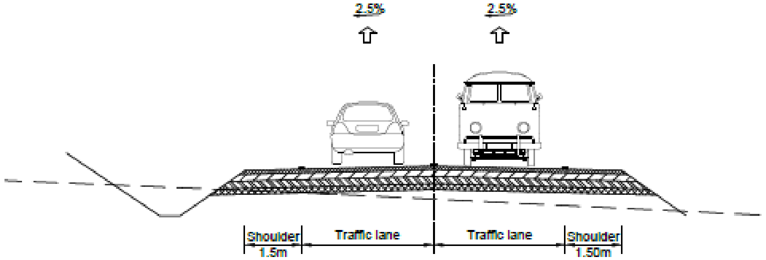
Road Class	National Highways			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Sidewalk	
Flat	7.3	2.5	-	
Rolling	7.3	2.5	-	
Mountain	7	2.5	-	
Escarpment	7	2.5	-	



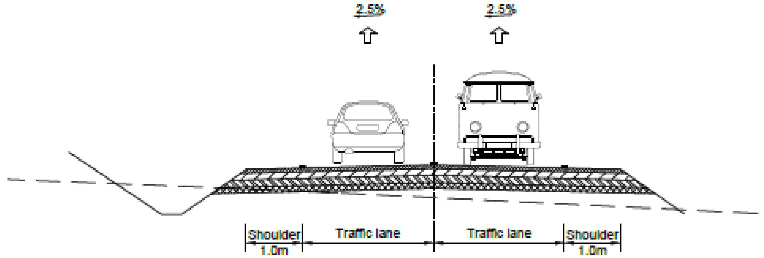
Typical cross section - Type 5
(Single carriageway with 2.0m shoulders)

Road Class	National Highways			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Sidewalk	
Flat	7.3	2	-	
Rolling	7.3	2	-	
Mountain	7	2	-	
Escarpment	7	2	-	

Figure 8-4: Single Carriageway (2.5m and 2m Shoulders)



Typical cross section Type 6
(Single carriageway with 1.5m shoulders)



Typical cross section Type 7
(Single carriageway with 1.0m shoulders)

Road Class	Inter County roads			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Sidewalk	
Flat	7.3	1.5	-	
Rolling	7.3	1.5	-	
Mountain	7	1.5	-	
Escarpment	7	1.5	-	

Road Class	Minor roads			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Sidewalk	
Flat	7.3	1.0	-	
Rolling	7.3	1.0	-	
Mountain	7	1.0	-	
Escarpment	7	1.0	-	

Figure 8-5: Single Carriageway (1.5m and 1m Shoulders)

8.4 Design Standards for Urban Roads

Table 8-11: Design Standard DU1 – Urban Arterial Multi-lane

	DESIGN ELEMENT			REF. SECTION	URBAN ARTERIAL MULTI-LANE	
Design Controls	Design Life				20 years	
Design Controls	Design Traffic (AADT)					
Design Controls	Design Speed (Km/hr)				80 - 100	
Design Controls	Control of Access				Full	
Design Controls	Level of Service Threshold				C	
Cross-Section Elements	No. of Lanes each Direction				2 or 3	
Cross-Section Elements	Lane Width (m)				3.65 m	
Cross-Section Elements	Outside Shoulder Width	Paved			2 m	
Cross-Section Elements	Outside Shoulder Width	Un-paved			N/A	
Cross-Section Elements	Median Shoulder Width	Paved			1 m	
Cross-Section Elements	Median Shoulder Width	Un-paved			N/A	
Cross-Section Elements	Crossfall Slope	Travel Lane			2.5 %	
Cross-Section Elements	Crossfall Slope	Shoulder			2.5 %	
Cross-Section Elements	Auxiliary Lanes	Lane Width			3.5 m	
Cross-Section Elements	Auxiliary Lanes	Shoulder Width			Paved: 1.0 m Unpaved – N/A	
Cross-Section Elements	Median Width	Painted			0.75 m Minimum	
Cross-Section Elements	Median Width	Depressed			2 m Minimum	
Cross-Section Elements	Median Width	Concrete Raised			2 m Minimum	
Cross-Section Elements	Median Width	Concrete Median Barrier			0.75 m	
Cross-Section Elements	On Street Parking – Bay Width	Bus			3.5 m	
Cross-Section Elements	On Street Parking – Bay Width	Car			2.4 m	
Cross-Section Elements	Bus Stop – Bay Width				5 m	
Cross-Section Elements	Cycle Track Width				2 m	
Cross-Section Elements	Footpath Width				1.5 m	
Cross-Section Elements	Service Road Width				3 m	
Cross-Section Elements	Road Reserve				Desirable: 60 m; Reduced: 40 m	
Cross-Section Elements	Roadside Clear Zone	Low Traffic (Fill/Cut)			5 m; 3 m	
Cross-Section Elements	Roadside Clear Zone	Medium Traffic (Fill/Cut)			8.5 m; 5.5 m	
Cross-Section Elements	Slopes	Cut	Cut Slope		1:4	
Cross-Section Elements	Slopes	Cut	Depth of Ditch		0.75 m	
Cross-Section Elements	Slopes	Cut	Back Slope		1:3	
Cross-Section Elements	Slopes	Fill	Safety Slope (within clear zone)		1:4	
Cross-Section Elements	Slopes	Fill	Fill Slope (outside clear zone)		1:3	
Alignment Elements	**Design Speed (Km/hr)**				**80**	**100**
Alignment Elements	Stopping Sight Distance (m)	g: 0 %			140	205
Alignment Elements	Stopping Sight Distance (m)	g: 5 %			155	235
Alignment Elements	Stopping Sight Distance (m)	g: 10 %			180	280

	Decision Sight Distance (m)			310	310
	Passing Sight Distance			Minimum: 240 m; Desirable: 550 m	Minimum: 240 m; Desirable: 550 m
	% Passing Opportunity			50	50
	Maximum Super-elevation Rate %			8	8
	Minimum Horizontal Curve Radius (m)	SE: 4 %		300	525
		SE: 6 %		265	465
		SE: 8 %		240	415
	Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %	
		Rolling		Desirable: 4 %; Absolute: 6 %	
		Mountaneous		Desirable: 6 %; Absolute: 8 %	
	Minimum Grades			0.5 %	
	Vertical Curve (K-values)	Crest		45	100
		Sag (Comfort)		16	25
		Sag (Headlights)		32	50

Table 8-12: Design Standards DU2 – Urban Arterial and Collector

	DESIGN ELEMENT		REF. SECTION	URBAN ARTERIAL	URBAN COLLECTOR
Design Controls	Design Life			20 years	
	Design Traffic (AADT)				
	Design Speed (Km/Hr)			80 – 100	50 - 70
	Control of Access			Full	
	Level of Service Threshold			C	D
Cross-Section Elements	No. of Lanes each Direction			2	
	Lane Width (m)			3.65 m	
	Outside Shoulder Width	Paved		2 m	1.5 m
		Un-paved		N/A	
	Median Shoulder Width	Paved		N/A	
		Un-paved		N/A	
	Crossfall Slope	Travel Lane		2.5 %	
		Shoulder		2.5 %	
	Auxiliary Lanes	Lane Width		3.5 m	
		Shoulder Width		Paved: 1.0 m Unpaved – N/A	
	Median Width	Painted		0.75 m Minimum	
		Depressed		2 m Minimum	
		Concrete Median Barrier		N/A	
	On Street Parking – Bay Width	Bus		3.5 m	3.5 m
		Car		2.4 m	2.4 m
	Bus Stop – Bay Width			5 m	5 m
	Cycle Track Width			2 m	2 m
	Footpath Width			1.5 m	1.5 m
	Service Road Width			3 m	3 m
	Road Reserve			Desirable: 60 m; Reduced: 40 m	Desirable: 60 m; Reduced: 40 m
	Roadside Clear Zone	Low Traffic (Fill/Cut)		5 m; 3 m	4 m; 2.5 m
		Medium Traffic (Fill/Cut)		8.5 m; 5.5 m	5.5 m; 4 m
	Slopes – Cut	Cut Slope		1:4	
		Depth of Ditch		0.75 m	
		Back Slope		1:3	
	Slopes – Fill	Safety Slope (within clear zone)		1:4	
		Fill Slope (outside clear zone)		1:3	

			REF. SECTION	URBAN ARTERIAL		URBAN COLLECTOR	
Alignment Elements	**Design Speed (Km/hr)**			**80**	**100**	**50**	**70**
	Stopping Sight Distance (m)	g: 0 %		140	205	65	110
		g: 5 %		155	235	70	120
		g: 10 %		180	280	75	140

	Decision Sight Distance (m)			240	240	240	240
	Passing Sight Distance			Minimum: 240 m; Desirable: 550 m	Minimum: 240 m; Desirable: 550 m	Minimum: 155 m; Desirable: 345 m	Minimum: 210 m; Desirable: 485 m
	% Passing Opportunity			25			
	Maximum Super-elevation Rate %			8			
	Minimum Horizontal Curve Radius (m)	SE: 4 %		300	525	95	205
		SE: 6 %		265	465	85	185
		SE: 8 %		240	415	80	170
	Maximum Grades			Desirable: 7 %; Absolute: 9 %			
	Minimum Grades			0.5 %			
	Vertical Curve (K-values)	Crest		45	100	10	30
		Sag (Comfort)		16	25	6.5	12
		Sag (Headlights)		32	50	14	25

Table 8-13: Design Standard DU3 – Urban Collector

	DESIGN ELEMENT			REF. SECTION	URBAN COLLECTOR	
Design Controls	Design Life				20 years	
	Design Traffic (AADT)					
	Design Speed (Km/hr)				50 - 70	
	Control of Access				Full	
	Level of Service Threshold				D	
Cross-Section Elements	No. of Lanes each Direction				2	
	Lane Width (m)				3.65 m	
	Outside Shoulder Width	Paved			1.5 m	
		Un-paved			N/A	
	Median Shoulder Width	Paved			N/A	
		Un-paved			N/A	
	Crossfall Slope	Travel Lane			2.5 %	
		Shoulder			2.5 %	
	Auxiliary Lanes	Lane Width			3.5 m	
		Shoulder Width			Paved: 1.0 m Unpaved – N/A	
	Median Width	Depressed			2m Minimum	
		Raised Concrete			2m Minimum	
		Concrete Median Barrier			N/A	
	On Street Parking – Bay Width	Bus			3.5 m	
		Car			2.4 m	
	Bus Stop – Bay Width				5 m	
	Cycle Track Width				2 m	
	Footpath Width				1.5 m	
	Service Road Width				3 m	
	Road Reserve				Desirable: 60 m; Reduced: 40 m	
	Roadside Clear Zone	Low Traffic (Fill/Cut)			4 m; 2.5 m	
		Medium Traffic (Fill/Cut)			5.5 m; 4 m	
	Slopes	Cut	Cut Slope		1:4	
			Depth of Ditch		0.75 m	
			Back Slope		1:3	
		Fill	Safety Slope (within clear zone)		1:4	
			Fill Slope (outside clear zone)		1:3	
Alignment Elements	**Design Speed (Km/hr)**				**50**	**70**
	Stopping Sight Distance (m)	g: 0 %			65	110
		g: 5 %			70	120
		g: 10 %			75	140
	Decision Sight Distance (m)				210	210
	Passing Sight Distance				Minimum: 155 m; Desirable: 345 m	Minimum: 210 m; Desirable: 485 m

	% Passing Opportunity			33	33
	Maximum Super-elevation Rate %			8	8
	Minimum Horizontal Curve Radius (m)	SE: 4 %		95	205
		SE: 6 %		85	185
		SE: 8 %		80	170
	Maximum Grades			Desirable: 5 %; Absolute: 7 %	
	Minimum Grades			0.5 %	
	Vertical Curve (K-values)	Crest		10	30
		Sag (Comfort)		6.5	12
		Sag (Headlights)		14	25

Table 8-14: Design Standards DU4 – Urban Collector and Local

	DESIGN ELEMENT			REF. SECTION	URBAN COLLECTOR		URBAN LOCAL	
Design Controls	Design Life				20 years			
	Design Traffic (AADT)							
	Design Speed (Km/Hr)				50 – 70		30 - 50	
	Control of Access				Full		Partial	
	Level of Service Threshold				D		B	
Cross-Section Elements	No. of Lanes each Direction				1 or 2		1	
	Lane Width (m)				3.65 m		3.5 m	
	Outside Shoulder Width	Paved			1.5 m		0 - 1 m	
		Un-paved			N/A			
	Median Shoulder Width	Paved			N/A			
		Un-paved			N/A			
	Crossfall Slope	Travel Lane			2.5 %			
		Shoulder			2.5 %			
	Auxiliary Lanes	Lane Width			3.5 m		3 m	
		Shoulder Width			Paved: 1.0 m Unpaved – N/A			
	Median Width	Depressed			N/A			
		Concrete Median Barrier			N/A			
	On Street Parking – Bay Width	Bus			3.5 m		3.5 m	
		Car			2.4 m		2.4 m	
	Bus Stop – Bay Width				5 m		5 m	
	Cycle Track Width				2 m		2 m	
	Footpath Width				1.5 m		2.4 m	
	Service Road Width				3 m		N/A	
	Road Reserve				Desirable: 60 m; Reduced: 40 m		Desirable: 60 m; Reduced: 40 m	
	Roadside Clear Zone	Low Traffic (Fill/Cut)			4 m; 2.5 m		2.5 m; 2.5 m	
		Medium Traffic (Fill/Cut)			5.5 m; 4 m		3 m; 3 m	
	Slopes	Cut	Cut Slope		1:4			
			Depth of Ditch		0.75 m			
			Back Slope		1:3			
		Fill	Safety Slope (within clear zone)		1:4			
			Fill Slope (outside clear zone)		1:3			
Alignment Elements	**Design Speed (Km/hr)**				**50**	**70**	**30**	**50**
	Stopping Sight Distance (m)	g: 0 %			65	110	30	65
		g: 5 %			70	120	31	70
		g: 10 %			75	140	33	75
	Decision Sight Distance (m)				155			
	Passing Sight Distance				Minimum: 155 m; Desirable: 345 m	Minimum: 210 m; Desirable: 485 m	Minimum: 115 m; Desirable: 195 m	Minimum: 155 m; Desirable: 345 m

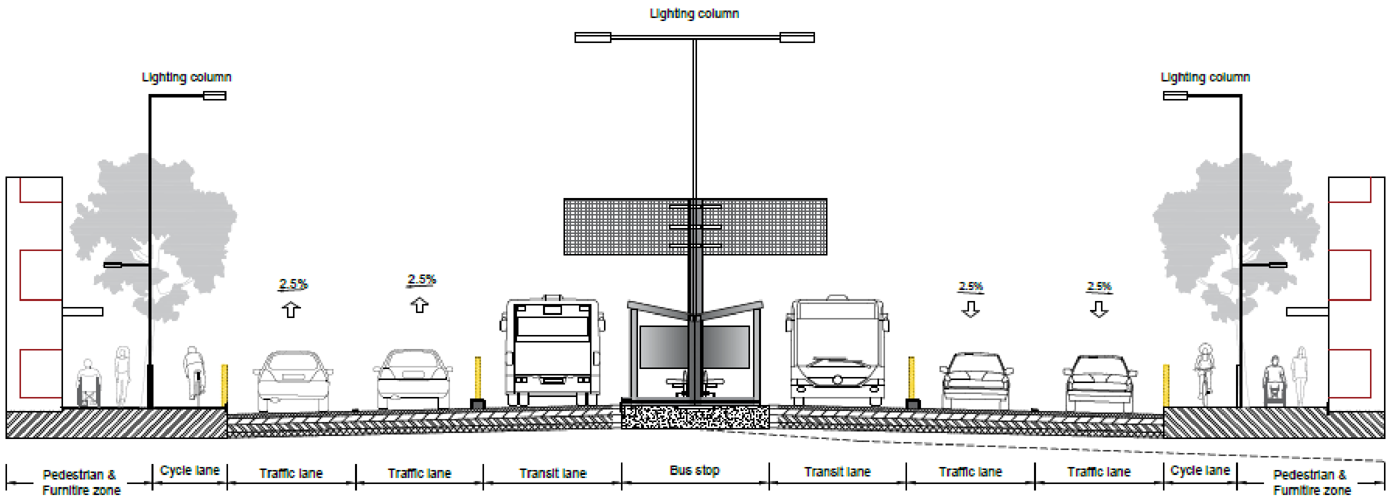
	% Passing Opportunity			50			
	Maximum Super-elevation Rate %			8	8	8	8
	Minimum Horizontal Curve Radius (m)	SE: 4 %		95	205	30	95
		SE: 6 %		85	185	26	85
		SE: 8 %		80	170	24	80
	Maximum Grades			Desirable: 3 %; Absolute: 5 %			
	Minimum Grades			0.5 %			
	Vertical Curve (K-values)	Crest		10	30	2	10
		Sag (Comfort)		6.5	12	2.5	6.5
		Sag (Headlights)		14	25	5	14

Table 8-15: Design Standard DU5 – Urban Local (CBD and Residential)

	DESIGN ELEMENT			REF. SECTION	CBD/INDUSTRIAL	RESIDENTIAL
Design Controls	Design Life				20 years	
	Design Traffic (AADT)					
	Design Speed (Km/Hr)				30 - 50	
	Control of Access				Partial	
	Level of Service Threshold				B	
Cross-Section Elements	No. of Lanes each Direction				1	
	Lane Width (m)				3 m – 3.5 m	
	Outside Shoulder Width	Paved			0 - 1 m	
		Un-paved			N/A	
	Median Shoulder Width	Paved			N/A	
		Un-paved			N/A	
	Crossfall Slope	Travel Lane			2.5 %	
		Shoulder			2.5 %	
	Auxiliary Lanes	Lane Width			3.0 m	
		Shoulder Width			Paved: 1.0 m Unpaved – N/A	
	Median Width	Depressed			N/A	
		Concrete Median Barrier			N/A	
	On Street Parking – Bay Width	Bus			N/A	3.5 m
		Car			2.4 m	
	Bus Stop – Bay Width				5 m	
	Cycle Track Width				2 m	
	Footpath Width				2.4 m	
	Service Road Width				N/A	
	Road Reserve				Desirable: 25 m; Reduced: 25 m	
	Roadside Clear Zone	Low Traffic (Fill/Cut)			2.5 m; 2.5 m	
		Medium Traffic (Fill/Cut)			3 m; 3 m	
	Slopes	Cut	Cut Slope		1:4	
			Depth of Ditch		0.75 m	
			Back Slope		1:3	
		Fill	Safety Slope (within clear zone)		1:4	
			Fill Slope (outside clear zone)		1:3	
Alignment Elements	**Design Speed (Km/hr)**				**30**	**50**
	Stopping Sight Distance (m)	g: 0 %			30	65
		g: 5 %			31	70
		g: 10 %			33	75
	Decision Sight Distance (m)				155	155
	Passing Sight Distance				Minimum: 115 m; Desirable: 195 m	Minimum: 155 m; Desirable: 345 m

	% Passing Opportunity			50	50
	Maximum Super-elevation Rate %			8	8
	Minimum Horizontal Curve Radius (m)	SE: 4 %		30	95
		SE: 6 %		26	85
		SE: 8 %		24	80
	Maximum Grades			Desirable: 3 %; Absolute: 5 %	
	Minimum Grades			0.5 %	
	Vertical Curve (K-values)	Crest		2	10
		Sag (Comfort)		2.5	6.5
		Sag (Headlights)		5	14

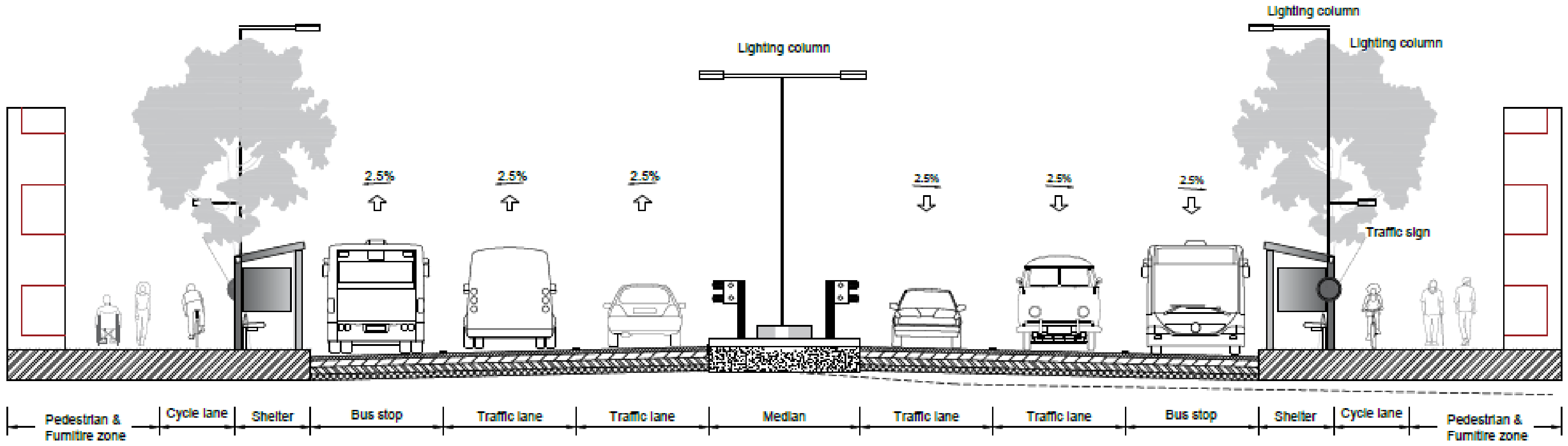
8.5 Typical Cross Sections – Urban Roads



Typical cross section - Type 1
(Urban Arterial road with bus stop)

Road Class	Urban Arterial roads			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Sidewalk	
Flat	3x7.3	1.5 - 2.5		Tripple Carriageway
Rolling	3x7.3	1.5 - 2.5		
Mountain	3x7.0	1.5		
Escarpment	3x6.5	(1.0) 2.0		

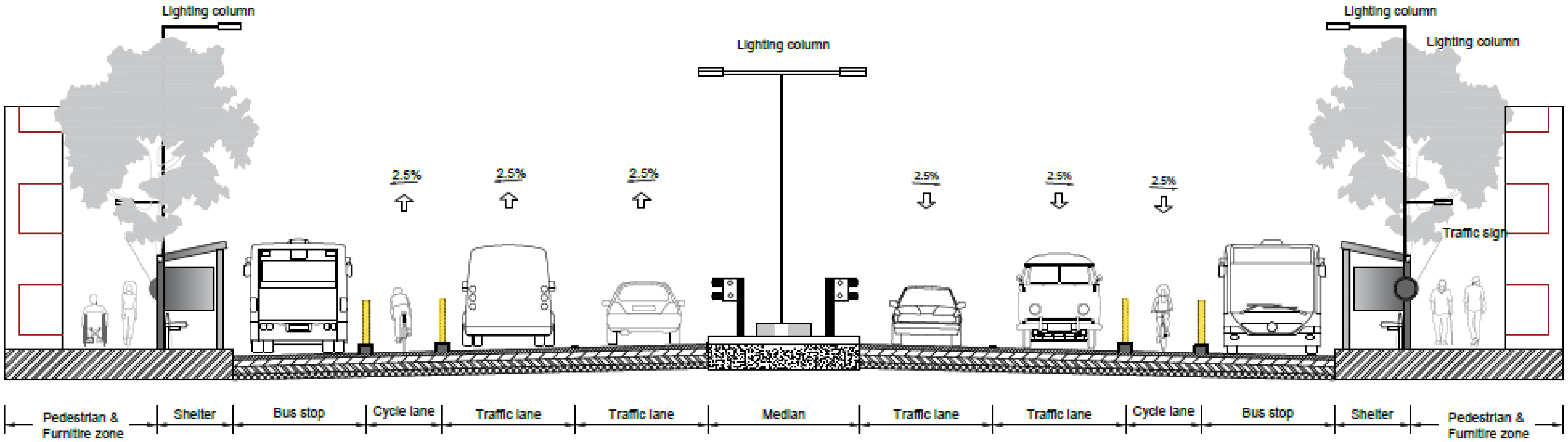
Figure 8-6: Urban Arterial Road (BRT Bus-bay on Median)



Typical cross section - Type 2
(Urban arterial with raised median and bus stops)

Road Class	Urban arterial roads			Remark
Type of Terrain	Width of road element (m)			
	Carriageway	Shoulders	Sidewalk	
Flat	3x7.3	1.5 - 2.5		Tripple Carriageway
Rolling	3x7.3	1.5 - 2.5		
Mountain	3x7.0	1.5		
Escarpment	3x6.5	(1.0) 2.0		

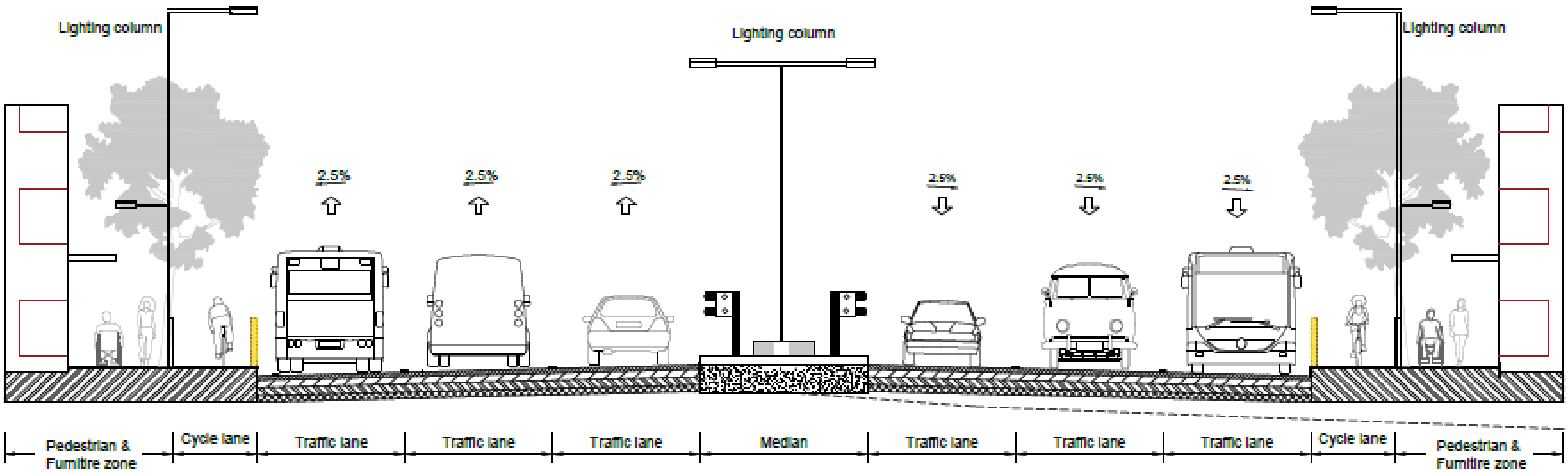
Figure 8-7: Urban Arterial Road (Raised Median and Roadside Bus Stops)



Typical cross section - Type 3
(Urban arterial with raised median, bus stops and cycle lanes infront of bus stops)

Road Class	Urban arterial roads			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Sidewalk	
Flat	3x7.3	1.5 - 2.5		Tripple Carriageway
Rolling	3x7.3	1.5 - 2.5		
Mountain	3x7.0	1.5		
Escarpment	3x6.5	(1.0) 2.0		

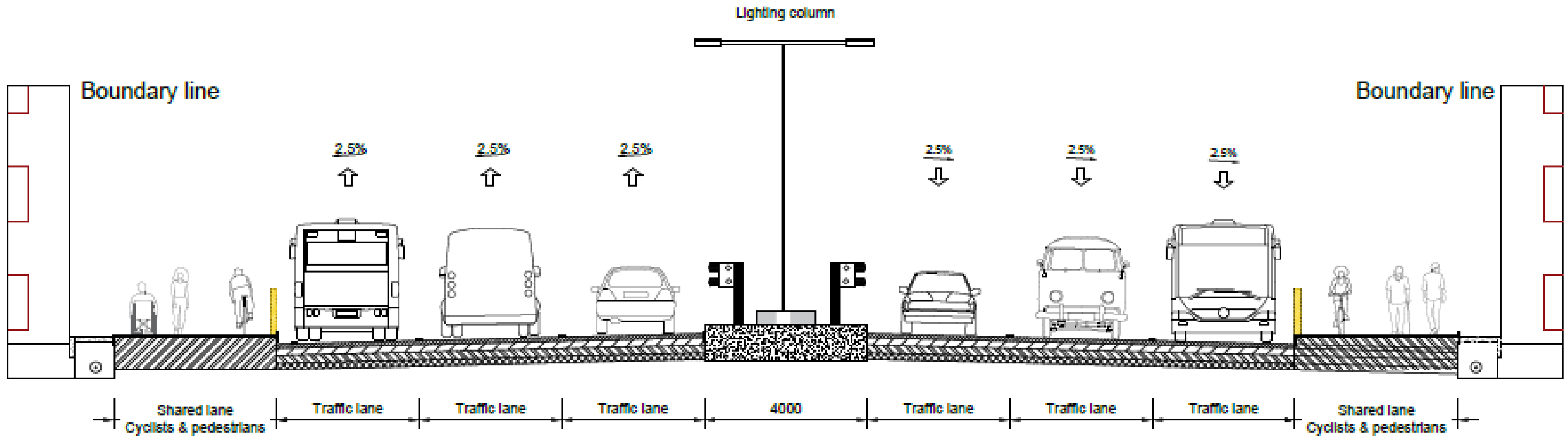
Figure 8-8: Urban Arterial Road (Raised Median, Roadside Bus Stops, Cycle Lanes)



Typical cross section - Type 4
(Urban arterial with raised median)

Road Class	Urban arterial road			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Sidewalk	
Flat	3x7.3	1.5 - 2.5		Tripple Carriageway
Rolling	3x7.3	1.5 - 2.5		
Mountain	3x7.0	1.5		
Escarpment	3x6.5	(1.0) 2.0		

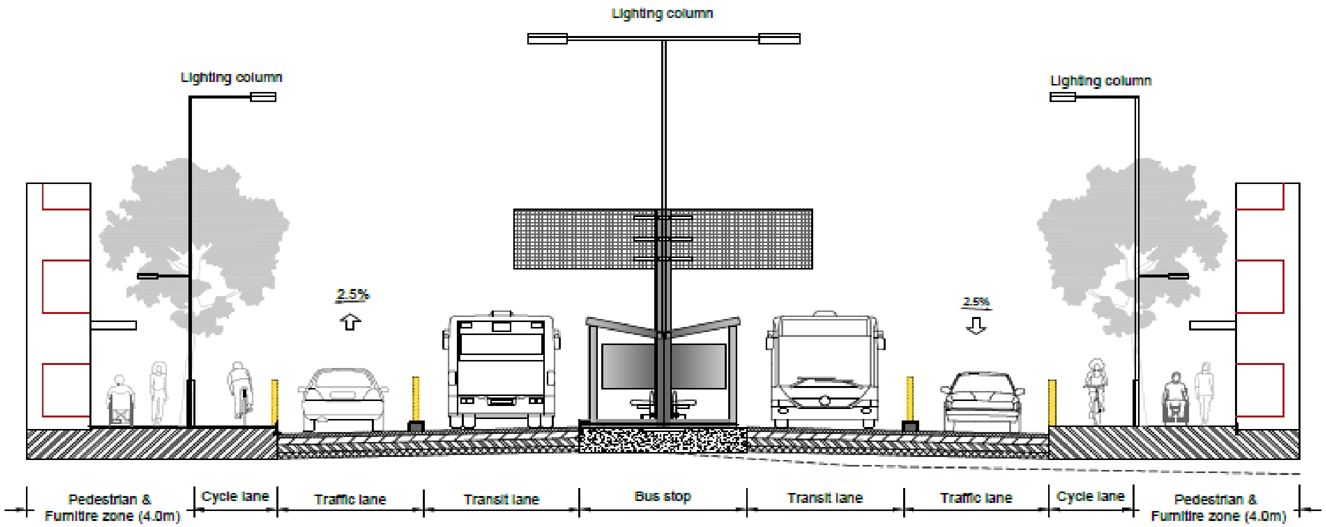
Figure 8-9: Urban Arterial Road (Raised Median)



Typical cross section - Type 5
(Urban arterial with shared lane for cyclists and pedestrians)

Road Class	Urban arterial roads			Remark
Type of Terrain	Width of road element (m)			
	Carriageway	Shoulders	Sidewalk	
Flat	3x7.3	1.5 - 2.5		Tripple Carriageway
Rolling	3x7.3	1.5 - 2.5		
Mountain	3x7.0	1.5		
Escarpment	3x6.5	(1.0) 2.0		

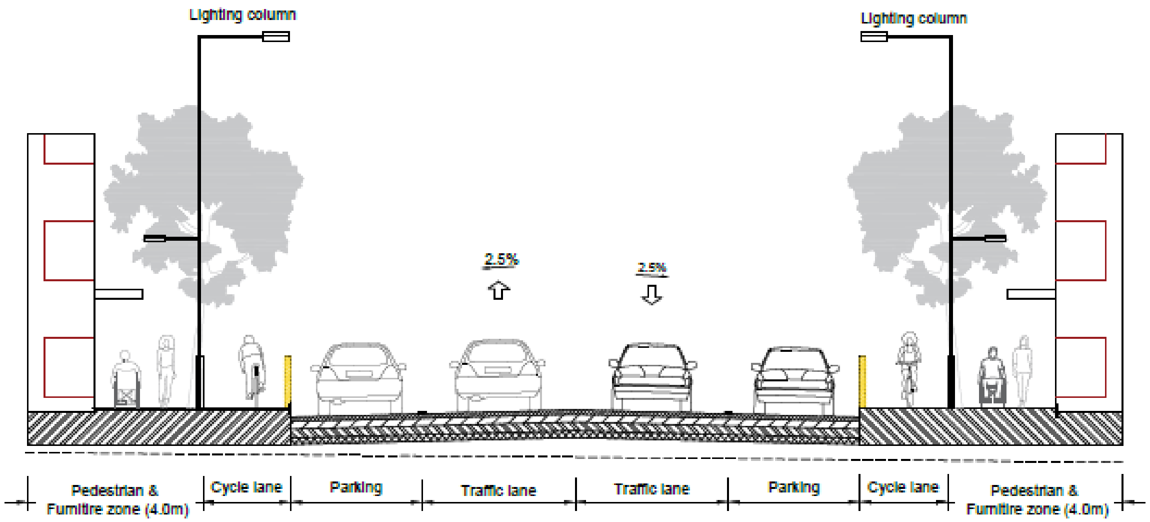
Figure 8-10: Urban Arterial Road (Raised Median and Shared Lane for Pedestrians and Cyclists)



Typical cross section - Type 6
(Urban Collector road with bus stop)

Road Class	Urban collector road			Remark
Type of Terrain	Width of road element (m)			
	Carriageway	Shoulders	Sidewalk	
Flat	2x7.3	1.5 - 2.5		Double Carriageway
Rolling	2x7.3	1.5 - 2.5		
Mountain	2x7.0	1.5		
Escarpment	2x6.5	(1.0) 2.0		

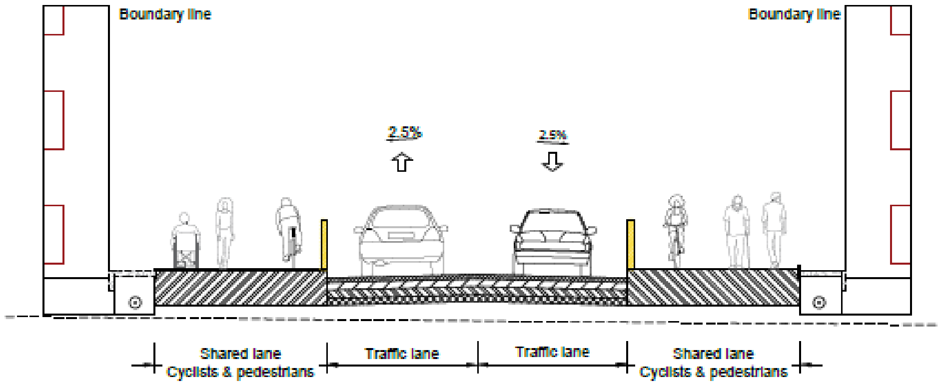
Figure 8-11: Urban Collector Road (Median BRT Bus Stops)



Typical cross section - Type 7
(Urban Arterial road with parking)

Road Class	Urban Collector road			Remark
Type of Terrain	Width of road element (m)			
	Carriageway	Shoulders	Parking	
Flat	7.3	1.5 - 2.5	3.0	Single Carriageway
Rolling	7.3	1.5 - 2.5	3.0	
Mountain	7.0	1.5	3.0	
Escarpment	6.5	(1.0) 2.0		

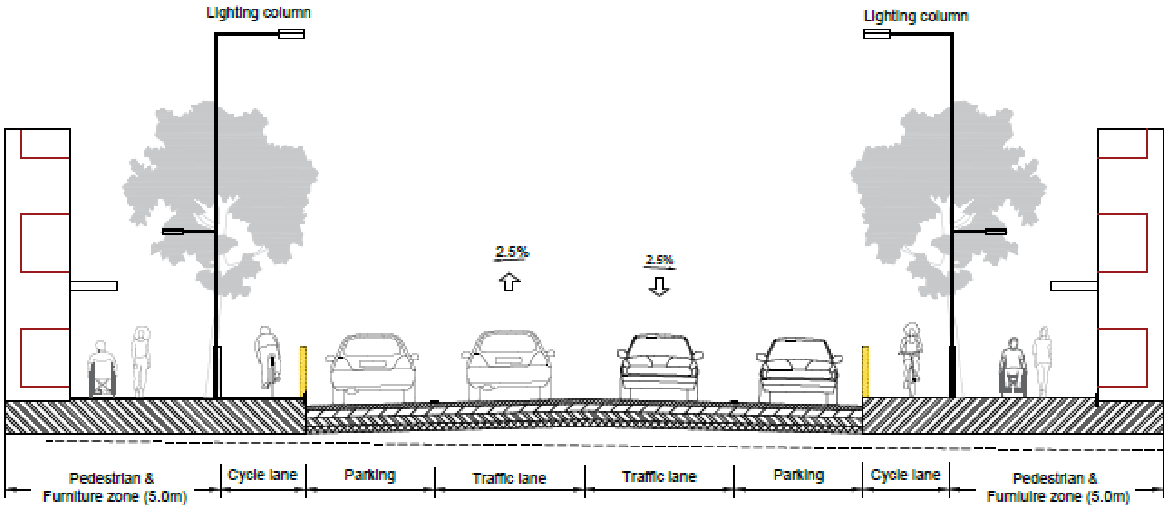
Figure 8-12: Urban Arterial Road (On-street Parking and Raised Cycle Lane)



Typical cross section - Type 8
(Urban collector road - Single Carriageway)

Road Class	Urban Collector road			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Sidewalk	
Flat	7.3	1.5 - 2.5		Single Carriageway
Rolling	7.3	1.5 - 2.5		
Mountain	7.0	1.5		
Escarpment	6.5	(1.0) 2.0		

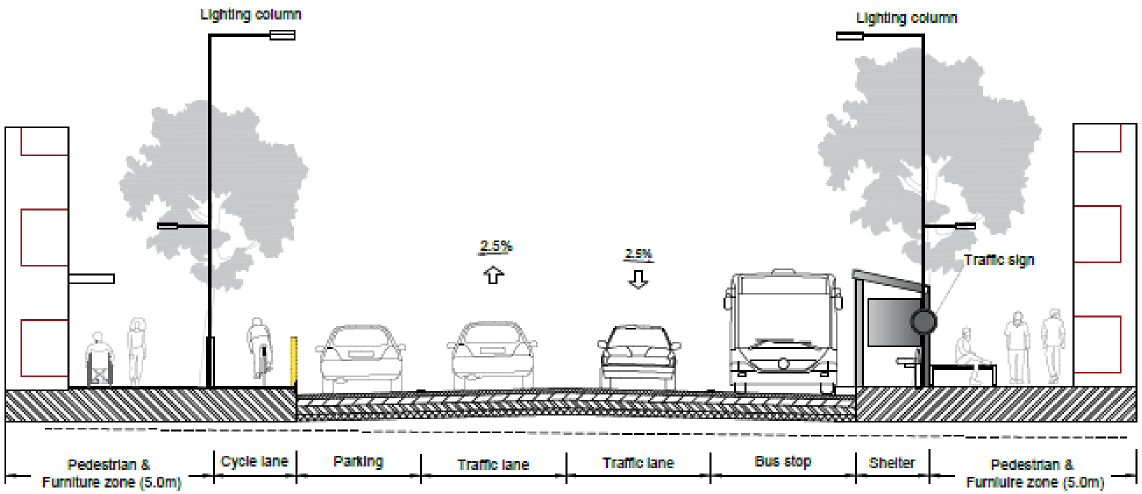
Figure 8-13: Urban Collector Road (with Shared Lane for Pedestrians and Cyclists)



Typical cross section - Type 9
(Urban local street with parking)

Road Class	Urban local street			Remark
Type of Terrain	Width of road element (m)			
	Carriageway	Shoulders	Parking	
Flat	7.3	1.5 - 2.5	3.0	Single Carriageway
Rolling	7.3	1.5 - 2.5	3.0	
Mountain	7.0	1.5	3.0	
Escarpment	6.5	(1.0) 2.0	3.0	

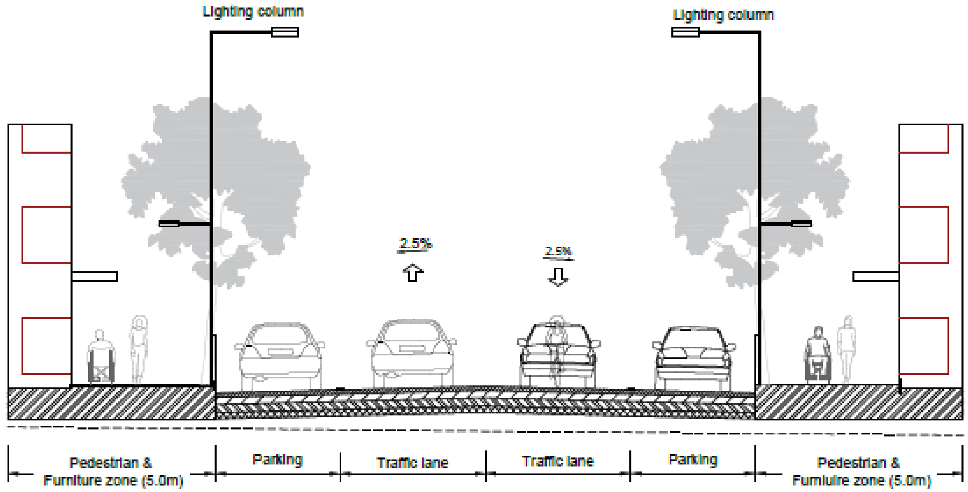
Figure 8-14: Urban Local Street Road (On-street Parking)



Typical cross section - Type 11
(Urban local street with parking bus stop)

Road Class	Urban local street with parking and bus stop			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Parking	
Flat	7.3	1.5 - 2.5		Single Carriageway
Rolling	7.3	1.5 - 2.5		
Mountain	7.0	1.5		
Escarpment	6.5	(1.0) 2.0		

Figure 8-15: Urban Local Street (On-street Parking, Pedestrian and Cyclist Lanes and Roadside Bus Stop)



Typical cross section - Type 10
(Urban local street - shared lane for live traffic and bicycles)

Road Class	Urban local street			
Type of Terrain	Width of road element (m)			Remark
	Carriageway	Shoulders	Parking	
Flat	7.3	1.5 - 2.5	3.0	Single Carriageway
Rolling	7.3	1.5 - 2.5	3.0	
Mountain	7.0	1.5	3.0	
Escarpment	6.5	(1.0) 2.0		

Figure 8-16: Urban Local Street (On-street Parking and Pedestrian Lane)

9 Design of At-Grade Intersections

9.1 Introduction

9.1.1 General

The unique characteristic of intersections is that vehicles, pedestrians, and bicycles travelling in many directions, must share a common area, often at the same time. The mitigation of the resulting conflicts is a major objective of intersection design. One of the consequences of this is that the task of drivers is inherently more demanding and the primary objective of the design of the intersection is to make the drivers task as easy as possible. This is made more difficult because many junctions usually must cater for pedestrians and the behaviour of pedestrians is considerably less predictable than the behaviour of drivers. Thus, guides for drivers and pedestrians to minimise points of contact and clear paths for all users are important issues in junction design.

9.1.2 Vehicle Characteristics

The size and manoeuvrability of vehicles is a governing factor in intersection design, particularly when channelisation features are being selected. Because of the importance of vehicle characteristics in the operation of an intersection, the selection of an appropriate vehicle, as described in Section 3.11 will theoretically influence the design. However, vehicles that can negotiate other elements of the highway system satisfactorily should have no difficulty with a properly designed junction, but this should be checked using the design vehicle described in Section 3.11.

9.1.3 Traffic Manoeuvres and Conflicts

The manoeuvres that result in vehicle conflict are merging, crossing diverging and weaving (Figure 9-1) and Table 9-13 and Table 9-14 for more details.

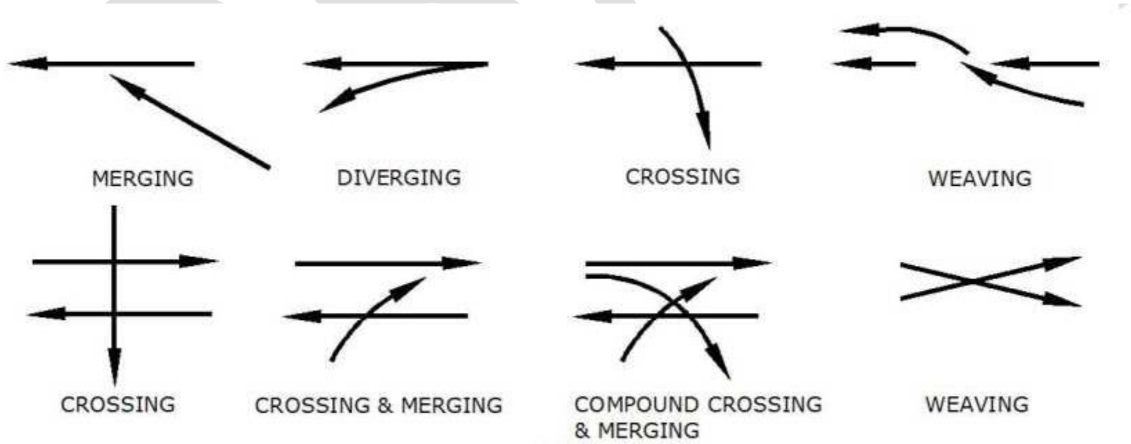


Figure 9-1: Junction Manoeuvres

Crossings should be direct (i.e., the angle between intersecting directions should be between 75° and 120°) Other angles of skew should be avoided. A staggered intersection might be considered as an alternative.

The conflict at intersections created by the various manoeuvres leads to a unique set of operational characteristics. Understanding these is central to intersection design. The most important characteristics are safety and capacity. Traffic volume is, not surprisingly, the most important factor affecting crashes. The crash potential caused by conflict increases as traffic on the approach legs increases.

The type of traffic control also influences crashes. More rear-end and sideswipe crashes tend to occur at signalised intersections than at other types of control. Stop and Give-Way controls tend to increase the frequency of crossing crashes. Table 9-1 lists many of the condition that can lead to crashes and the geometric and control measures that are used to mitigate the number and severity of crashes.

Most of the characteristics listed are difficult to quantify and difficult to rank. A review of local crash data should be made for similar intersections and the expertise of an experienced road safety expert or auditor should also be sought.

Table 9-1 Features contributing to crashes and remedial measures.

Features or conditions contributing to crashes	Traffic engineering actions that reduce crashes and severity
Poor approach sight distance	Addition or installation of exclusive turning movement lanes. Speed management.
Poor corner sight distance	Upgrade traffic control scheme
Steep grades at intersections	Improve sight distances
Lack of conspicuity especially at night	Install improved lighting
Multiple approaches	Remove fixed objects
Presence of curves within intersection	Increasing corner radii
Number of adjacent driveways or access points. Inappropriate kerb radii.	Application of channelisation, and improved signage
Narrow lanes	Removal of clutter, signs
Poor drainage and skid resistance	Improve drainage path and roughen surface e.g. surface treatment

9.2 Intersection Types

9.2.1 General

There are five main types of intersection distinguished by the amount of traffic that they can carry satisfactorily and by the method and degree of control of the traffic that they employ as described in Table 9-2.

Table 9-2: Basic Types of Intersections

Intersection Type		Characteristics
Simple uncontrolled for low levels of traffic (Sections 9.2.1 and 9.2.2)	Crossroads	A simple at-grade junction of two roads that cross approximately at right angles
	T- junctions	A simple at-grade junction of two roads, at which, usually, the minor road joins the major road approximately at right angles
	Staggered T-junctions	An at-grade junction of three roads, at which the major road is continuous through the junction and the minor roads connect with the major road to form two opposed T-junctions (Figure 9.5).
Basic Priority Junctions (Section 11.2.1)	Crossroads	Similar to the simple crossroads but with road markings, give-way or stop signs on the minor road, channelizing islands and/or ghost islands. For unsignalized junctions, traffic from all directions must come to a complete stop and give priority to those joining from the right (Figure 9-2).
	T-Junction	Similar to the simple T junction but with road markings, give-way or stop signs, channelizing islands and/or ghost islands shaped and located to direct traffic movement (Figure 9.5). For unsignalized T junctions, adjoining traffic should give way to traffic on the 'major' road in both directions.
	Staggered Junction	Staggered junction with channelizing and ghost islands shaped and located to direct traffic movement (Figures 9.5 and 9.6)
	Skew or Y junction	An at-grade junction of two roads, at which the minor road approaches the major road at an oblique angle and terminates at the junction (Figure 9.6).
Roundabouts. (Section 10.2.3 and Chapter 10)	For low to medium traffic flows, primarily for urban and metropolitan conditions	They provide minimum delays at lower flows and are safer than priority junctions. They require attention to pedestrian movements and the accommodation of slow-moving traffic. Roundabouts are discussed in Chapter 10.
Intersections where traffic is controlled by traffic signals. (Section 9.2.4)	As for priority intersections	As for priority junctions but for higher traffic flows and more complex conditions such as additional routes.

Intersection Type		Characteristics
Grade-Separation (Chapter 11)		Grade separated interchanges are expensive and used only for high flows, but they result in minimum delays. Pedestrian movements need special consideration. These interchanges are dealt with in Chapter 11.

Table 9-3: Types of Traffic Control for At-grade Intersections

Basic category	Traffic Control		Intersection Type
	Major Road	Minor Road	
Simple	None	None	Simple
Priority	Priority	Stop or give way signs	A Unchanneled T intersection B Partly Channelized T intersection C Channelized T intersection
Roundabouts	Priority to traffic already on the roundabout but can also be traffic signal controlled		D Priority E Signalized
Traffic controlled	Traffic signals or give way signs		E Signalized Intersection

Once designed and constructed, almost all will be unique. This chapter deals with basic principles and examples of some of the popular options.

The principal characteristics of intersections are that vehicles, pedestrians and non-motorised traffic travelling in many directions must share a common area, often at the same time. The mitigation of the resulting conflicts is therefore a major objective of intersection design.

Good intersection design allows through movement and transitions from one route to another with minimum delay and maximum safety. Thus, the layout and operation of the intersection should be obvious to vehicle drivers as they approach, with good visibility between conflicting movements.

Intersections for the higher traffic levels are expensive and, like bridges and other major structures, should be designed for a period of at least 30 years. Therefore, a careful assessment of likely future traffic flows is required to ensure that a structure will perform satisfactorily into the future.

9.2.2 Simple and Priority Crossroads and T-Intersections

The basic intersection layouts for rural roads are crossroads and T-junctions with the major road traffic having priority over the minor road traffic (e.g. Figure 9.2). However, except where traffic is low on all arms, the crossroads form of priority intersection is not recommended because it has a much higher accident risk than any other kind of intersection.

An existing crossroads should, where possible, be converted to a staggered intersection, (Figure 9-5) or roundabout or be controlled by traffic signals.

Simple uncontrolled junctions are appropriate for most minor junctions on single carriageway roads but must not be used for wide single carriageways or dual carriageways. For new rural junctions they should only be used when the design flow in the minor road is not expected to exceed about 300 vehicles 2-way AADT and that on the major road is not expected to exceed 13,000 vehicles 2-way AADT.

At existing rural and urban junctions, upgrading a simple junction to provide a right turning facility from the major road should always be considered because vehicles waiting on the major road to turn right inhibit the through flow and create a hazard.

A right turn from the main road is a dangerous manoeuvre hence different junction designs are used to cater for increasing traffic levels. The use of partial channelisation, full channelisation, ghost islands, single lane dualling, and traffic signal control are all techniques used to provide safe right turn facilities for increasing traffic flows. Examples are illustrated in, Figure 9-3, and Figure 9-6

Where the flow levels are not large enough to justify the provision of a right turning facility, and a right turning problem remains, a nearside passing bay should allow through vehicles to pass the vehicles waiting to turn right, albeit at a reduced speed.

Intersections with more than four arms are not recommended. Where more complex layouts involving the intersection of four or more roads are encountered, these should be simplified by redesign to two junctions, or a roundabout should be used. Experience in some countries has shown that converting crossroads into roundabouts can reduce crashes by more than 80 per cent.

T-intersections include the staggered T-intersection (Figure 9-5 and Figure 9-6), which cater for cross-traffic. Staggered T-intersections are often the result of a realignment of the minor route to improve the angle of the skew of the crossing.

When traffic on the main road is quite high and a staggered T-intersection is required, there are two options namely the ‘turn left onto the main road then turn right onto the minor road’ and the ‘turn right then turn left’ stagger. Both options have two conflict locations where a vehicle must merge with one stream and cross the other stream at the same location (i.e., the driver must identify two gaps at the same time) but the ‘turn right then turn left stagger’ is preferred because any required storage occurs in the minor roads. The ‘turn left then turn right’ stagger might require an auxiliary lane in the main road to store vehicles before they can turn across the opposite stream of the main road into the minor road.

The basic designs are modified for higher traffic flows and for higher traffic speeds. Additional lanes are provided and traffic control by means of additional channelisation (e.g., Figure 9-3 and Figure 9-4) and/or traffic signals.

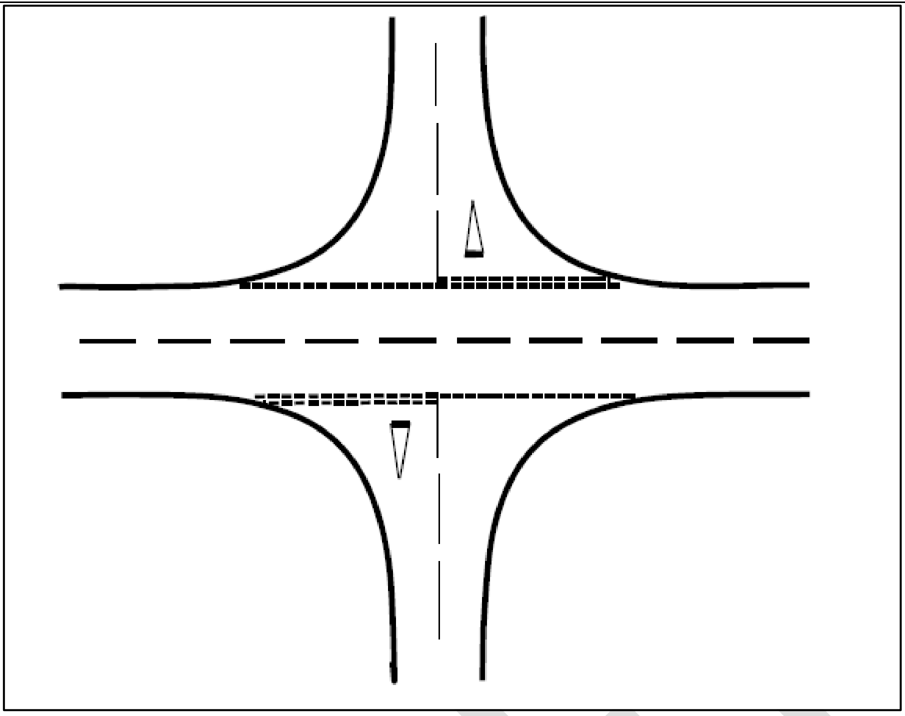


Figure 9-2: Crossroads with Stop Lines

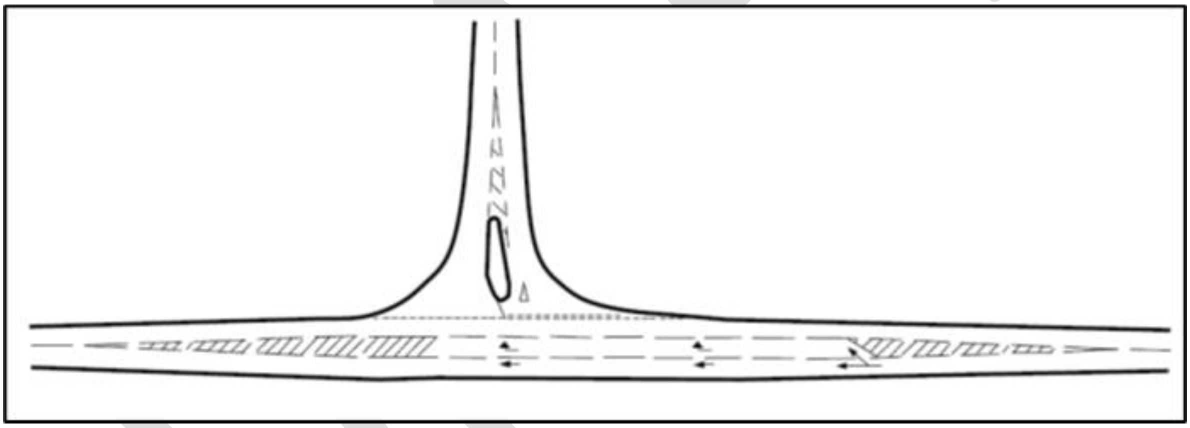


Figure 9-3: Partly Channelised T-Intersection with Ghost Islands

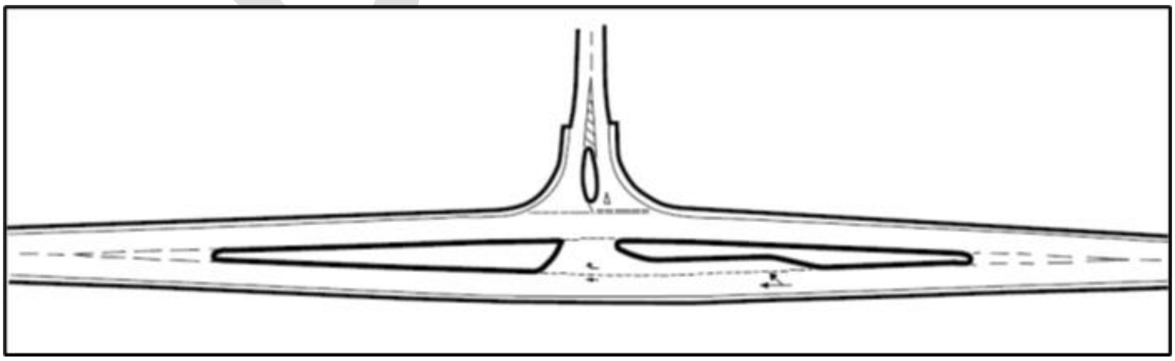


Figure 9-4: T-Intersections with Channelisation

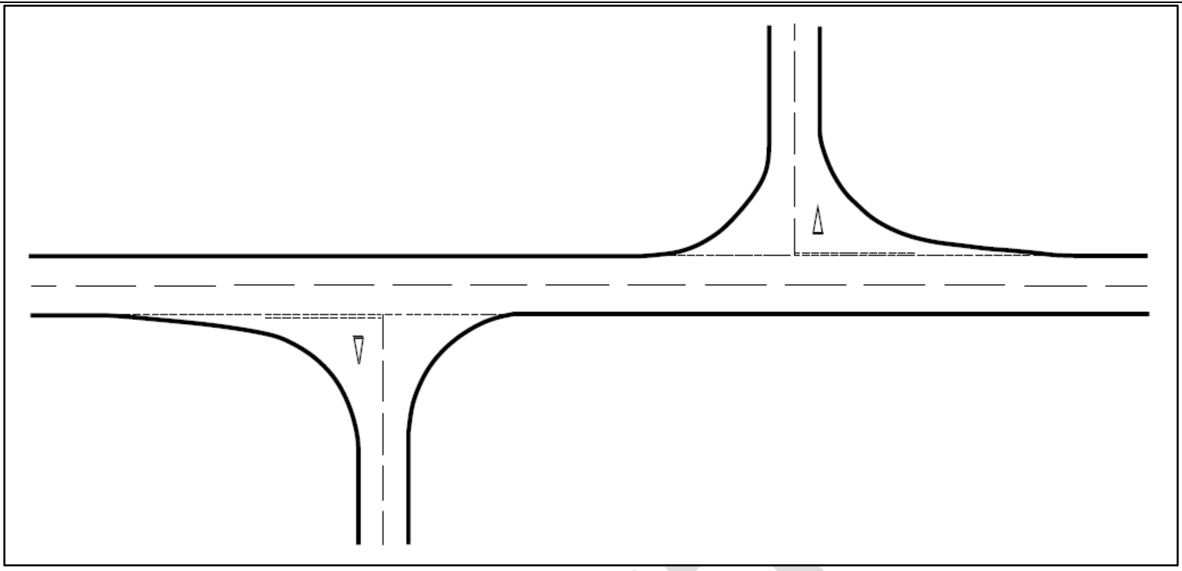


Figure 9-5: Staggered Intersection

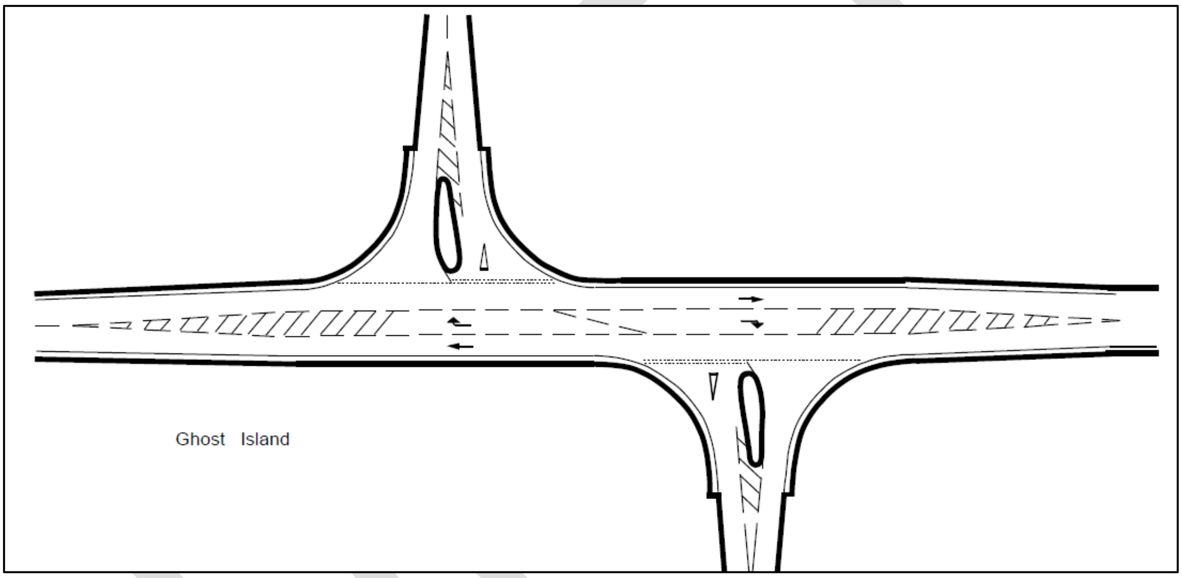


Figure 9-6: Partly Channelised Staggered Intersection with Ghost Islands

9.2.3 Skew Intersections

The angle of intersection of two roadways influences both the operation and safety of an intersection (Figure 9-7). Large skews are undesirable because:

1) The area of the pavement is increased and therefore also the area of possible conflict.
2) Crossing vehicles and pedestrians are exposed for longer periods.
3) The driver's sight angle is more constrained and gap perception becomes more difficult.
4) Vehicular movements are more difficult and large trucks require more pavement area.
5) Defining vehicle paths by channelization is more difficult.

For new intersections the crossing angle should preferably be in the range 75° to 120°. The absolute minimum angle of skew is 60° because drivers, particularly truck drivers, have difficulty at this angle of skew in seeing vehicles approaching from one side or the other. The designer should justify using an angle of skew of less than 75°. In the remodelling of existing intersections, the accident rates and patterns will usually indicate whether a problem exists and provide evidence of any problems related to the angle of skew.

The location of an intersection may require modification to improve the angle of skew between the intersecting roads. If the angle of skew is less than 60^{O}, the intersection can usually be replaced by two relatively closely spaced T-intersections.

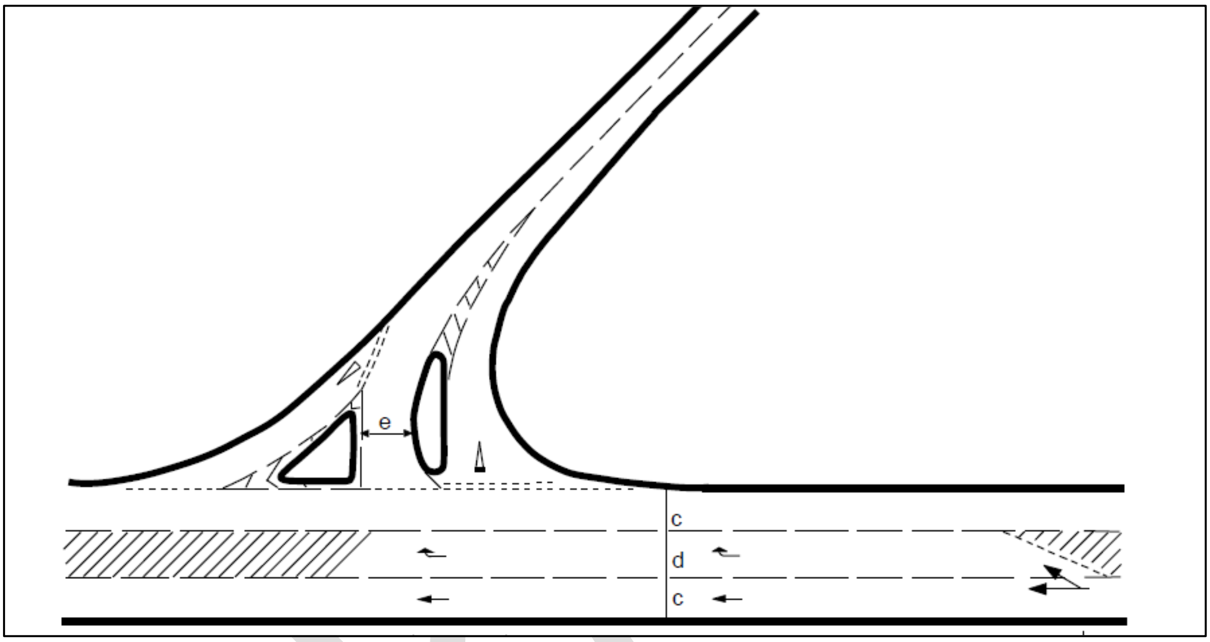


Figure 9-7: Skew or Y Intersection with Ghost and Channelising Islands.

9.2.4 Roundabouts

The key feature of roundabouts is that traffic entering the roundabout must give way to circulating traffic already on the roundabout. Ideally the minor road incoming traffic should be at least 10-15% of the total incoming traffic. Roundabouts are discussed in Chapter 9.

9.2.5 Controlled Intersections

Controlled or signalised intersections are interchanges where the traffic is controlled by traffic lights, Figure 9-8. No traffic conflicts are allowed between straight through traffic movements. The choice of controlled intersections is discussed in Section 9.5 and their design in Section 9.9.

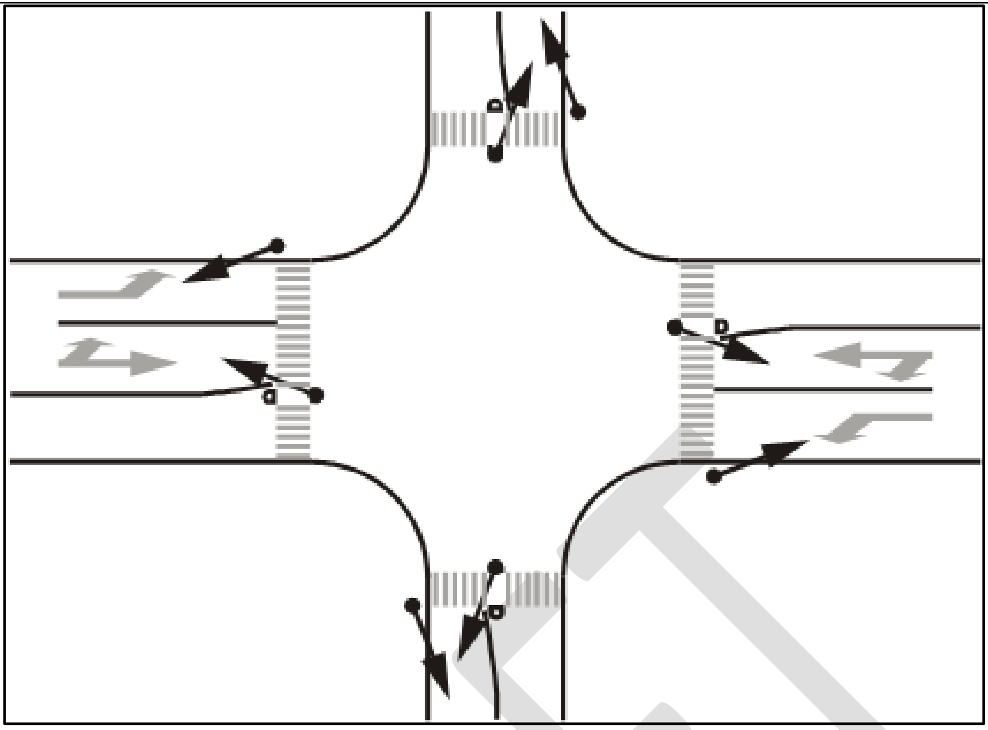


Figure 9-8: Traffic Signal Controlled Crossroads

9.3 Other Considerations

9.3.1 Non-motorised Traffic and Non- Road Users

Intersection design requires suitable facilities to be provided for non-motorised road users and for non-road users who need to negotiate the engineered structures.

Correctly located crossings are critical to walking and cycling activities and can help overcome severance created by busy roads. A balance needs to be struck between the legitimate needs of all road users. This balance will be influenced by the location of the intersection and the volume of pedestrian and cycle traffic.

9.3.2 Pedestrian Requirements

From a pedestrian perspective an ideal crossing facility would:

1) Be very safe.
2) Provide the desired routing requirements.
3) Allow for crossing the intersection in all directions;
4) Have adequate capacity; and
5) Possess a quick response to demand.

In practice this ideal is difficult to achieve and therefore some compromises are necessary, but these should not be at the risk of reduced safety.

In an urban situation the location of at-grade bicycle or pedestrian crossings, whether controlled or uncontrolled, at locations where approach speeds are likely to be in excess of 50 km/h should be avoided. The crossings should also be positioned away from locations where drivers might be applying maximum acceleration. In such circumstances segregated facilities may be more appropriate.

The requirements of pedestrians with impaired mobility must also be considered.

9.3.3 Traffic Control

Pedestrian facilities are sometimes provided by stopping all traffic movements and introducing a ‘pedestrian-phase’ during which pedestrians can cross all arms of the intersection. The disadvantage is that the pedestrian-phase results in considerable lost time which impacts the capacity of the intersection and forces the use of long signal cycle times.

Pedestrian facilities can often be designed in such a way that pedestrians are able to cross when non-conflicting streams of traffic are running. In this case a specific signal indicates when it is appropriate for pedestrians to cross. These are referred to as ‘walk-with-traffic’ pedestrian facilities. The provision of walk-with-traffic pedestrian facilities separates pedestrian routes into a series of relatively short sections between safe refuges. As a result, shorter ‘cross now’ periods are required at the points of conflict and the pedestrian-to-traffic inter-green periods are shorter.

9.3.4 Shared Facilities

Pedestrian and cycle flows should initially be considered as two different movements. If their individual requirements turn out to be similar, then consideration should be given to providing joint facilities. The shared use of space by pedestrians and cyclists should only be considered as a last resort when all other solutions have been dismissed. Unsegregated shared use should be avoided, particularly in heavily used urban contexts.

9.4 Factors Affecting Selection of Intersection Type and Design

9.4.1 Functionality

The functionality of the road is the key to identifying the most appropriate designs. In Chapter 2 over 60 design standards covering many aspects of functionality are introduced. Intersections are required on all roads hence their design must be flexible, and compromises must be made in many circumstances.

Table 9-4 and Figure 9-9 provide general guidance on the type of intersection required based on daily traffic flows, but high peak flows must also be catered for in many cases (Section 9.7.2).

Table 9-4: Intersection Selection Based on Traffic Flow (1000 vpd)

	Type of Intersection			
Traffic Flow on Major Road	**Traffic Flow on Minor Road**			
(1000)	**(<) Less than**	**(<) Less than**	**Range**	**(>) Greater than**
	Simple	**Priority**	**Roundabout/ or Signalised**	**Grade Separation**

<10	1.2			
<10		1.2 - 7.0		
10 - 12		7.0	7.0 - 12	>12
12 - 14		6.5	6.5 - 11	11
14 - 16		5.0	5.0 - 10	10
16 - 18		4.0	4.0 – 9.5	9.5
18 - 20		3.5	3.5 – 9.0	9.0
20 - 22		3.5	3.5 – 8.0	8.0
22 - 24		3.0	3.0 – 7.5	7.5
24 - 26		2.5	2.5 – 6.5	6.5
26 - 28		2.5	2.5 – 6.0	6.0
28 - 30		2.0	2.0 – 5.0	5.0
30 - 32		2.0	2.0 – 4.5	4.5
32 - 34		2.0	2.0 – 4.0	4.0
34 - 36		2.0	2.0 – 3.0	3.0
36 - 38		2.0		2.0
38 - 40		2.0		2.0

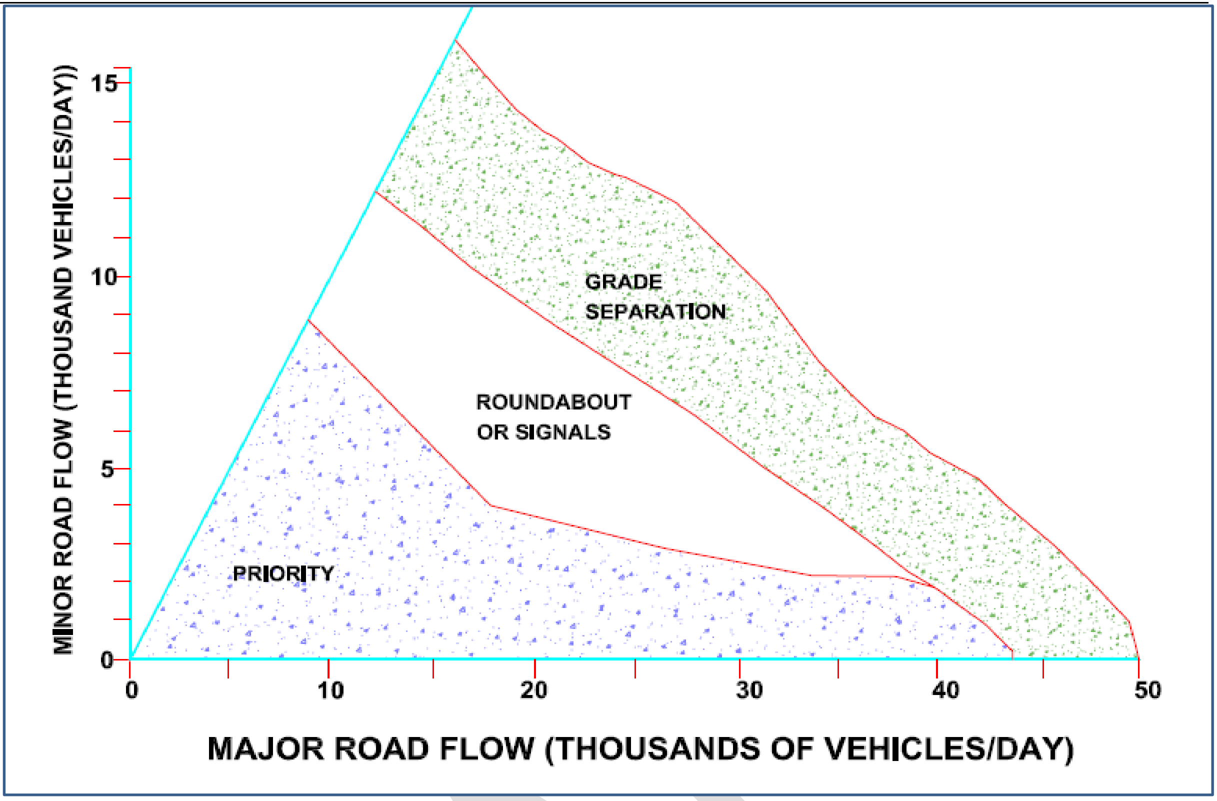


Figure 9-9: Intersection Selection Based on Traffic Flows

Source: The Highways Agency, UK.

9.5 Principles of Intersection Design

9.5.1 General

The objectives of good intersection design are to:

1) Ensure that the traffic capacity satisfies the forecast requirements.
2) Keep the number of points of potential conflict to the minimum compatible with efficient operation.
3) Reduce the complexity of conflict areas wherever possible.
4) Limit the frequency of actual conflicts; and to
5) Limit the severity of those conflicts that might occur.

An intersection is considered safe when it is visible, comprehensible and of dimensions that allow easy vehicle movements. The design is therefore concerned with the following aspects:

1) Location;
2) Design Speed;
3) Sight distances;
4) Safety and operational comfort;
5) Vehicle characteristics;
6) Capacity and proportion of traffic on each approach;

7) Local environment;

8) Economy.

The physical aspects of the intersection that enable these objectives to be realised are as follows:

9.5.2 Location

Intersection should be located according to the following principles:

1) Intersections should not be located on horizontal curves with radii less than those indicated in Table 9-5;
2) Intersections should not be located on gradients steeper than 3%. The gradient is more critical on the minor road than on the major road because all vehicles on the minor road have to stop or yield; and
3) In a collision between vehicles, either or both may leave the road. Therefore intersections should not be located on high fills.

Table 9-5: Minimum Radii for Location of Intersections on Curves

Design Speed (km/h)	Minimum Radius (m)
40	250
50	375
60	550
70	750
80	1000
90	1220
100	1500

Source: SANRAL. Geometric Design Guidelines.

9.5.3 Design Speed

It is the effective traffic speed on the major road in the vicinity of the intersection, but it is not the design speed of the main road because drivers tend to slow down when approaching intersections, even when they are travelling on the major road.

The design speed is the main design parameter upon which the capacity and the geometrical layout of an intersection is based and greatly affects the safety and efficiency of the intersection and the construction cost. For safety reasons, it should never be less than 20 km/h lower than the average design speed for the major road.

The time available to carry out a driving manoeuvre depends on the speed of traffic in the lanes to be crossed. Mathematical models have been developed for carrying out these calculations but require many assumptions and are not reliable. The best information is obtained from local empirical data.

The driver of a vehicle approaching an intersection should have an unobstructed view of the entire intersection, including any traffic control devices, and sufficient distance along the intersecting highway to permit the driver to anticipate and avoid potential collisions. Decision sight distance must be available on all the approach legs (Section 3.7.6).

The recommended perception and reaction times for calculating sight distances are shown in Table 9-6.

Table 9-6: Human Factors for Intersection Design

Human Factor	Design Values	Design Elements Affected
Perception/reaction time	2.0 to 4.0 seconds	Intersection sight distance
Gap acceptance	5.5 to 7.5 seconds	Intersection sight distance
Driver eye height	1.05 m	Sight distance
Pedestrian walking speeds	1.0 to 1.5 m/s	Pedestrian facilities

It may be necessary to modify the alignment of either the major or the minor road, or both, to ensure that adequate sight distances are available. If this is not possible, the options available to the designer are to:

1) Relocate the intersection;
2) Provide appropriate Stop control; or
3) Provide a Jug-handle (also called a Quarter link) interchange, as shown in Figure 11-4.

9.5.4 Sight distances

The important factors are the time required to carry out the manoeuvre and the time available to do so based on the sight distance and the speed of traffic. The sight distance needs to be at least as great as the product of the traffic speed and the time required to carry out the manoeuvre.

On a basic crossroad intersection, the traffic manoeuvres are left turns and right turns from both the minor road and the major road and crossing manoeuvres across the intersection. The time required to carry out a manoeuvre depends upon:

1) Whether the vehicle is in motion and at what speed when it reaches the intersection (yield control or approach control) or whether the vehicle begins from a stopped position (stop or departure control).
2) The type and power of the vehicle.
3) The length of the vehicle.
4) The distance the vehicle needs to travel (number of lanes plus median if present);
5) The gradient of the road which the vehicle must negotiate.

The required sight distances also depend on driver behaviour. Driver behaviour generally depends on driving history and drivers’ experiences and is not static, hence taking it into account is not a simple process. To improve road safety, road safety research, which includes analysis and review of road crashes, should be a continuous process.

9.5.4.1 Empirical Method of Estimating Sight Distances

It is not a simple task to calculate the optimum or minimum sight distances applicable to different intersection designs, different road classes and different mixes of traffic. A pragmatic approach is to utilise the available empirical data but to select conservative options for safety. **Xxxx** summarises the empirical approach and shows how sight distances can be calculated for most manoeuvres and for different design vehicles.

The greatest sight distances are needed for the manoeuvres that take the longest to execute (required time) and involves joining fast traffic (short available time). This means that heavy truck and trailer combinations require the greatest sight distances when joining a main road. Catering for this situation is not always possible. The methods described in **Xxxx** can be used to calculate sight distances for different design vehicles, but it is prudent to use vehicle DV4 (Table 3-24) for most designs.

9.5.4.2 Sight Triangles

Each quadrant of an intersection should contain a triangular area free of obstructions that might block an approaching driver's view of potentially conflicting vehicles. These specified areas are known as clear sight triangles. The dimensions of the legs of the sight triangles depend on the design speeds of the intersecting roadways and the type of traffic control used at the intersection. Two different forms of sight triangle are required. The approach triangle must have sides with sufficient lengths on both intersecting roadways such that drivers can see any potentially conflicting vehicle in sufficient time to slow, or to stop, if need be, before entering the intersection. For the departure sight triangle, the line of sight described by the hypotenuse of the sight triangle should be such that a vehicle just coming into view on the major road will, at the design speed of this road, have a travel time to the intersection corresponding to the gap acceptable to the driver of the vehicle on the minor road.

Practical sight distances are summarised in Table 9-7 and Table 9-8.

Table 9-7: Minimum Sight Distances for 'Yield' or 'Approach' Conditions

Main road design speed (km/h)	40	50	60	70	85	100	120
Sight distance, LA (m)	80	95	115	140	190	215	270

Table 9-8: Minimum Sight Distances for 'Stop' or 'Departure' Conditions

Main road design speed (km/h)	40	50	60	70	85	100	120
Sight distance L_S (m)	130	160	190	225	275	320	385

9.5.5 Visibility

In addition to adequate sight distances, good visibility of the intersection is obviously an equally vital component of safety and can often be improved by good design. The following should be provided:

1) The intersection should be sited so that the major road approaches are all readily visible;

2) Early widening of the intersection approaches;

3) Provision of visibility splays which ensure unobstructed sight lines to the left and right along the major road;
4) Use of traffic islands in the minor road to emphasize a ‘yield’ or ‘stop’ requirement;
5) Use of medians;
6) Use of early and eye-catching traffic signs;
7) Optical guidance by landscaping and the use of road furniture, especially where an intersection must be located on a crest curve;
8) Suitable pavement tapers and transitions;
9) The angle of intersection of the major and minor roads should be between 70 and 110 degrees;

The use of single lane approaches is preferred on the minor road to avoid mutual sight obstruction from two vehicles waiting next to each other to turn or cross the major road.

9.5.6 Safety and Operational Comfort

Intersections are inherently complex and therefore human factors, as discussed in Section 0 and elsewhere, are of vital importance. Intersection design should reflect and make provision for the characteristics of the drivers and their expectations on the various classes of the roads. The designs must not only be visible but also comprehensible:

1) The right of way should follow naturally and logically from the intersection layout;
2) The types of intersections used throughout the whole road network should be as similar as possible;
3) Kerbs, traffic islands, road markings, road signs and other road furniture must be clearly visible.

Crashes are minimised by appropriate design of the intersection based primarily on the volume of traffic on both the major and the minor roads and on their speed. As traffic levels and speed increases, the degree of control of the traffic must increase to maintain safety standards.

Poor design leads to significantly higher injury and higher crash rates. Table 9-9 summarises the features of crashes at intersections and possible measures to minimise them. Most of these have been mentioned but designs are rarely perfect and can probably be improved, even if only slightly, by double checking hence it is appropriate to re-emphasise the issues.

Table 9-9: Features Contributing to Crashes at Intersections.

Geometric features contributing to crashes at intersections	Traffic engineering actions that reduce accident incidents and severity
Poor approach and sight distances Poor corner sight distances	Improve of sight distances

Construction on high fill	Construction should not be on high fills
Curves within intersection	Avoid building on a horizontal curve
Inappropriate kerb radii	Increasing corner radii
Inappropriate traffic control	Upgrading of traffic control scheme
Multiple approaches	Addition of exclusive turn lanes
Poor lighting	Installation of improved lighting
Narrow lanes	Use of channelization
Poor drainage	Improved drainage paths
Low surface friction	Improved surface skid resistance
Steep grades at intersections Number of adjacent driveways or access points.	Avoid building on a gradient greater than 3 % (stopping sight distances increase quickly as down gradient increases) Speed management

9.5.7 Vehicle characteristics

The size and manoeuvrability of vehicles is theoretically a governing factor in intersection design, especially where channelised design is required. However, it is adequate to consider only the class of road, corresponding channel, lane width and geometric dimensions unless particularly large and unusual vehicles are to be catered for.

Table 9-10 summarises the factors that affect the manoeuvrability but the standards of the various classes of both urban and rural roads should be fully appreciated by the owners of large vehicles (that are not suitable for use on all classes).

Table 9-10: Vehicle Factors for Intersection Design

Vehicle Characteristics	Design Element Affected
Length	Length of storage lanes
Width	Width of lanes
	Width of turning roadways
Wheelbase	Nose placement
	Corner radius
	Width of turning roadways
Acceleration	Acceleration tapers and lane lengths
	Gap acceptance
Deceleration and braking capability	Length of deceleration lanes and tapers
	Stopping sight distance

Source: SANRAL. Geometric Design Guidelines

9.5.8 Local Environment

The type of area and adjacent land use governs the selection of an appropriate intersection. In urban areas, pedestrian flows, on-street parking and bus and taxi activity are commonplace. In residential areas, bicycles and school crossings need to be considered. They are usually absent in rural areas, where utility and delivery vehicles are more common.

9.5.9 Economy and long-term transport plan

The cost of interchanges is always an important factor. Provided that safety is not compromised, the lowest cost option may be acceptable, but it should be borne in mind that interchanges, especially the large ones and those constructed in urban areas where space is restricted, need to be designed for long design lives (at least 30 years) and for future traffic that may not be easy to predict accurately. A long-term transport plan is required if the network is to develop logically and economically.

9.6 Capacity

9.6.1 General

The traffic flow on intersections can be based on ‘priority’ or be ‘controlled’ depending on the relative traffic flows on each branch of the intersection. 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 or, perhaps, only at peak times when the traffic is high.

9.6.2 Providing easy manoeuverability

To provide for ease of manoeuverability:

1) All traffic lanes should be of adequate width for the appropriate vehicle turning characteristics;
2) To accommodate truck traffic, turning radii must be at least 15 m. In restricted urban areas this could be reduced to an absolute minimum of 12m;
3) The edges of traffic lanes should be clearly indicated by road markings;
4) Traffic islands and kerbs should not conflict with the natural vehicle paths;
5) Active traffic control by means of traffic signals is an essential component of intersections whenever traffic flow exceeds certain thresholds;
6) Depending on location, traffic conditions and speed limits, different types of priority or control intersection should be selected;
7) For the highest traffic levels, grade separated interchanges (Chapter 11) are required.

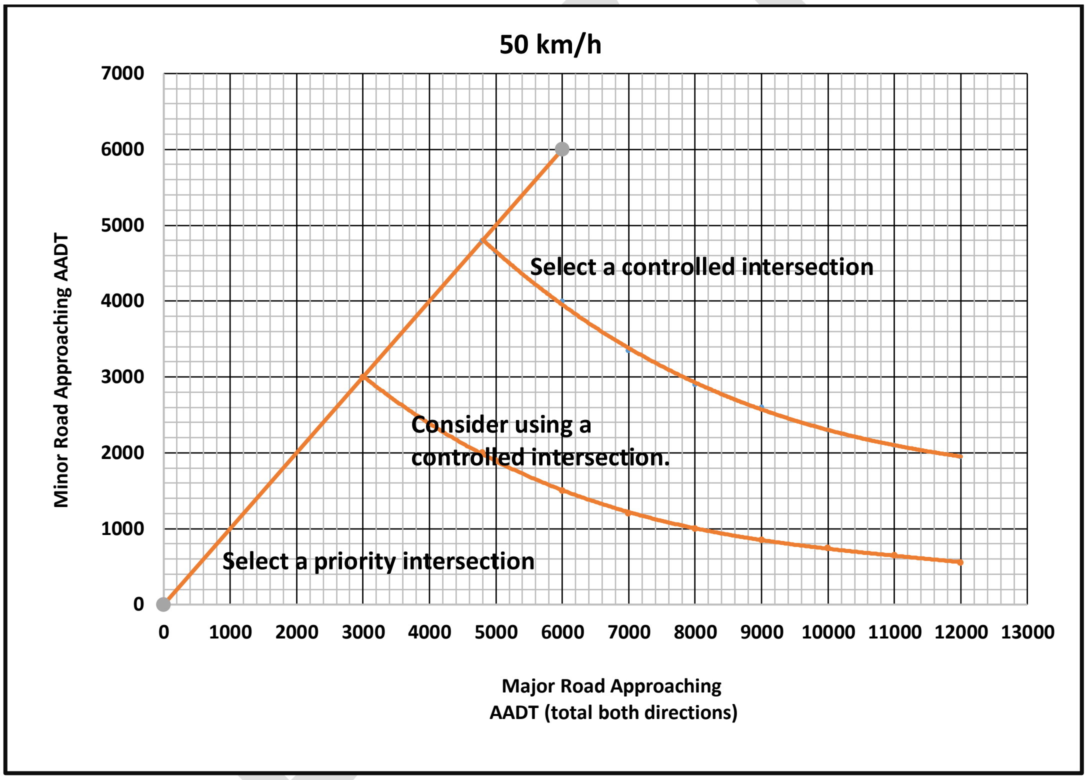
9.7 Intersections

9.7.1 Priority and Controlled Intersections

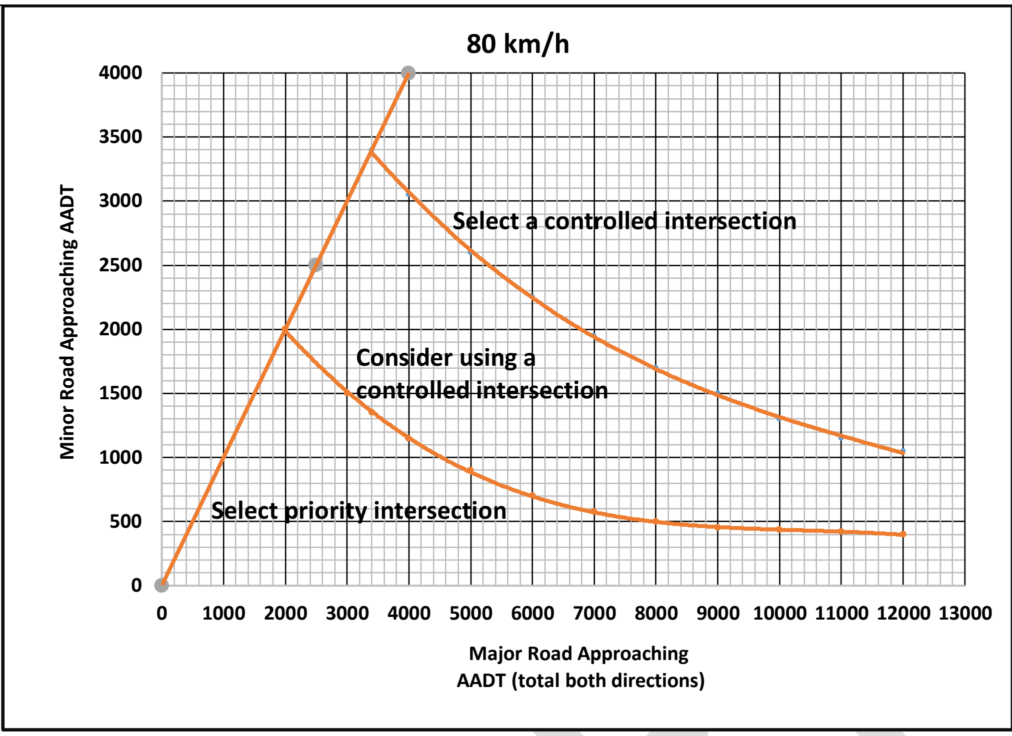
Priority intersections are introduced in Section 9.2.2 and examples are shown in Figure ***9-2*** through to Figure 9-8. The selection of a priority intersection should be based mainly on safety. The selection can be made by using known relationships between safety levels and the average daily approaching traffic volumes (AADT in vehicles/day) based on accident statistics. Figure 9-11 indicates the traffic flows where priority intersections are recommended for T-intersections on 2-lane roads with 50, 80 and 100 km/h design speed. Crossroads should be avoided. The number of right turning vehicles should also affect the decision.

A partly channelised T-intersection would normally be used if needed to facilitate pedestrian crossings and if a minor road island is needed to improve the visibility of the intersection.

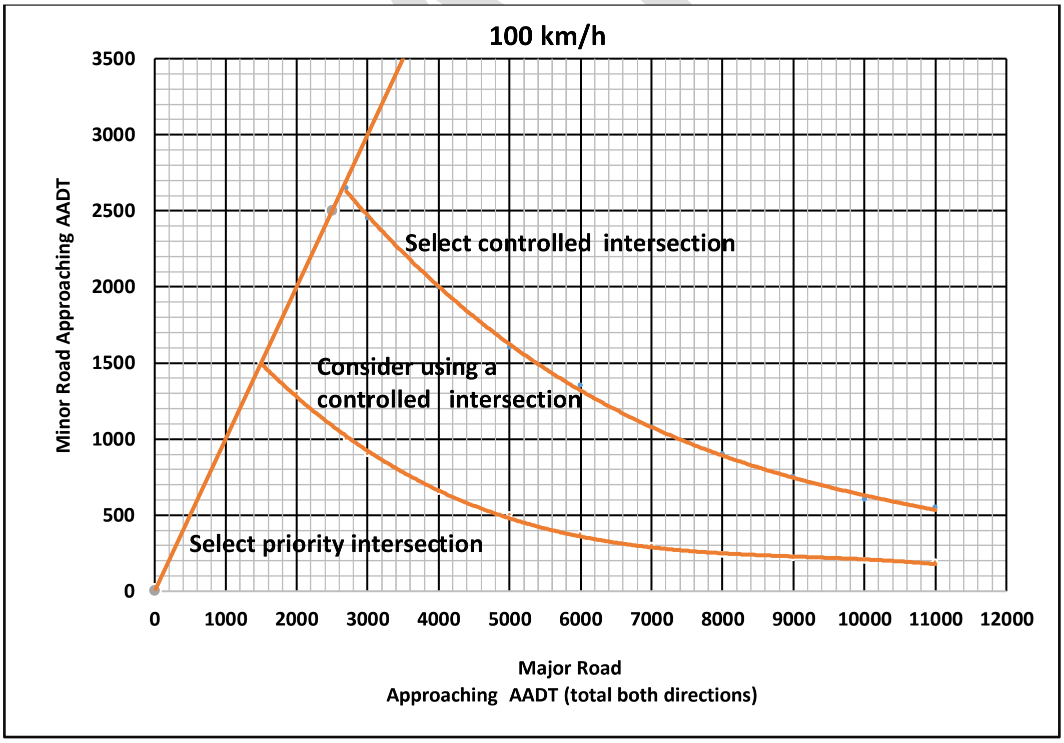
Many intersections must cope with peak hour flows for a short period of the day and much lower traffic flows at other times. Thus, sometimes just a priority interchange is required but at others a controlled interchange is necessary. In these circumstances a signalised interchange is required. Figure 9-11 also illustrates the relationship between recommended intersection type and traffic on the major and minor roads and traffic speed on the major road based on safety considerations. Traffic is expressed as Figure 9-10a (design speed 50km/h) is primarily for urban interchanges, Figure 9-10b (80km/h) is for metropolitan or rural interchanges and Figure 9-10(c) (100km/h) is for rural interchanges only.



(a) (50 Km/hr)



(b) (80 Km/hr)



(c) (100 Km/hr)

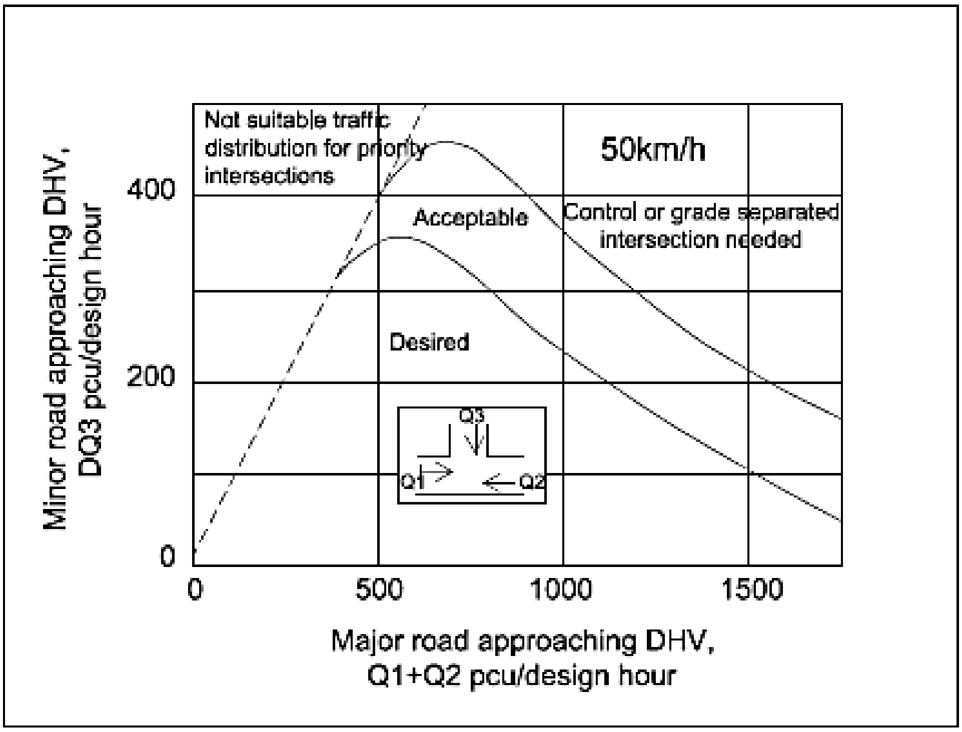
Figure 9-10: Selection of Intersection Types

Note: The lines in these figures cannot be precise. They are for guidance.

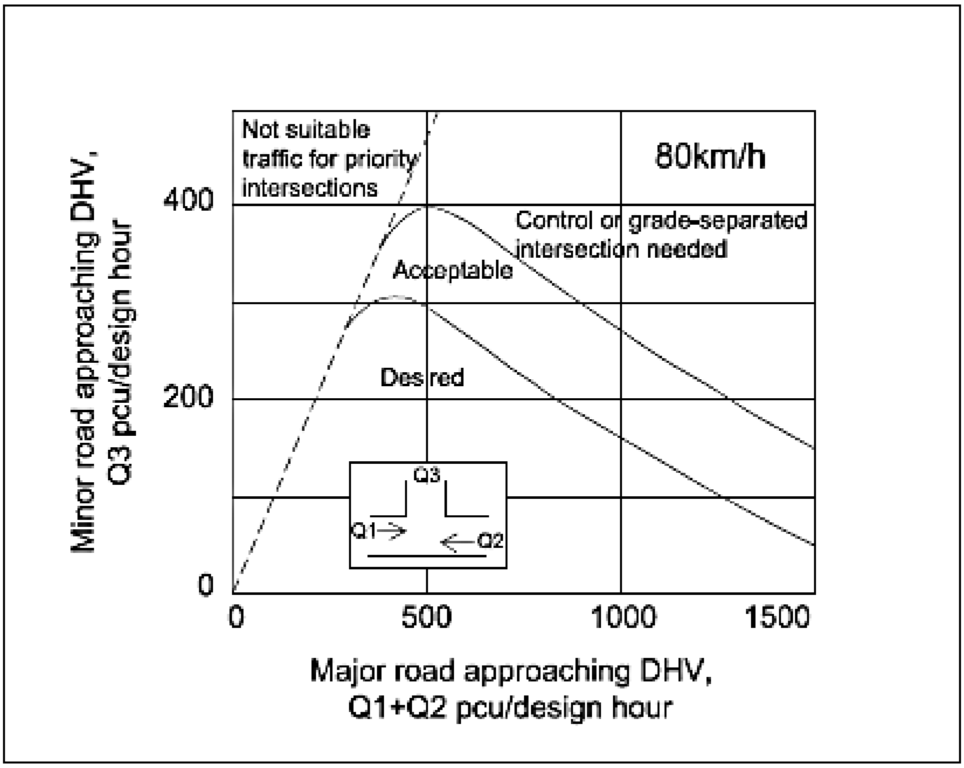
The traffic flows are not constant throughout the day. Significant peaks occur at certain times and interchange design must cater for peak flows to prevent congestion and minimise traffic delays.

9.7.2 Priority Intersections based on Traffic in the peak hour.

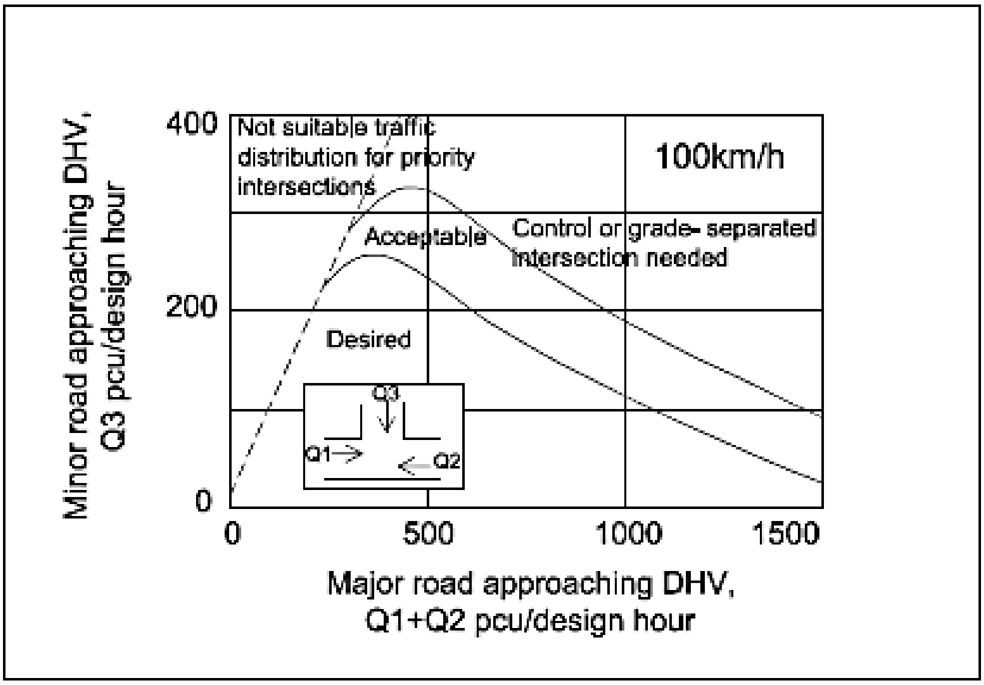
Figure 9-11 shows the relationships between capacity and traffic volumes in PCU/h approaching the interchange during the 'design hour'. The diagrams are for T-intersections on 2-lane roads with speed limits of 50, 80 and 100 km/h. Table 8- summarises the important traffic levels. The 'desired' level 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.



(a) (50 Km/hr)



(b) (80Km/hr)



c) (100 km/hr)

Figure 9-11: Capacity and traffic volumes in PCU/h during the 'design hour'

Note: The lines in these figures cannot be precise. They are for guidance.

Table 8-11: Typical Maximum Traffic Volumes

Road Type	Design Traffic Volume (two-way vehicles/h)		
Major Road	500	1000	1500
Minor Road	500	250	100

When traffic exceeds these values, additional features need to be included as described in the following Sections.

9.8 Signal Controlled Intersections

For higher traffic levels Table 9-4 and Figure 9-12 provides guidance. Roundabouts are suitable for almost all situations provided there is enough space. Roundabouts have been found to be safer than signalised intersections. At very high traffic volumes they tend to become blocked because drivers fail 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 to obtain the maximum capacity from each intersection.

The ideal flow rate through an intersection is the saturation flow rate per hour of 'green' time. The vehicular 'green' time is the time dedicated to presenting vehicular traffic with a green (or proceed) indication. The value selected is affected by the initial driver reaction, vehicle acceleration and the behaviour of following vehicles. The capacity of an approach or leg of an intersection is proportional to the green time for that approach within the signal cycle in accordance with:

$$C_a = s.\frac{g}{c}$$

Equation 9-1: Capacity of an approach or leg of an intersection

Where,

Ca = *capacity (PCU/h)*

s = *saturated flow rate (PCU/h)*

g/c = *the ratio of useful green time to signal cycle time.*

The important factors affecting saturation flow are:

1) Number of lanes including turning lanes;
2) Widths of lanes;
3) Proportion of heavy vehicles;
4) Gradients in excess of 3%;
5) On-street parking;

6) Pedestrian activity;
7) Type and phasing of signals.

The critical factors are the total number of lanes and the need for exclusive turning lanes at each approach.

Examples of signal-controlled intersections are shown in Figure 9-12 and Figure 9-13. Note the locations of the pedestrian crossings.

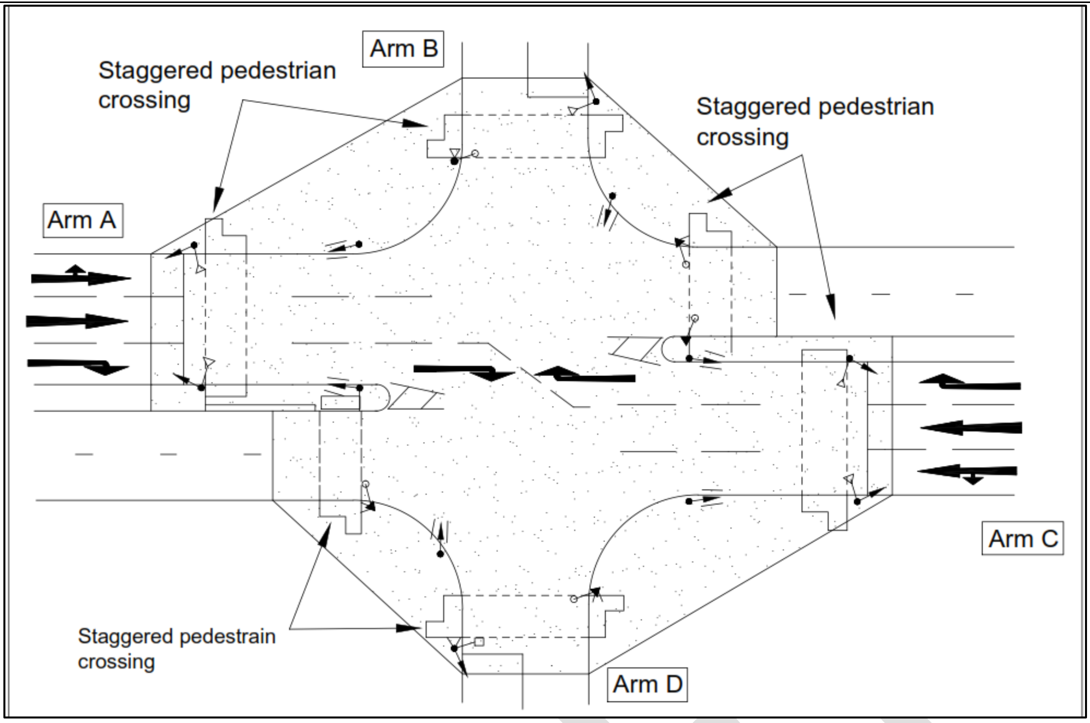


Figure 9-12: Example of a Signal-Controlled Crossroads

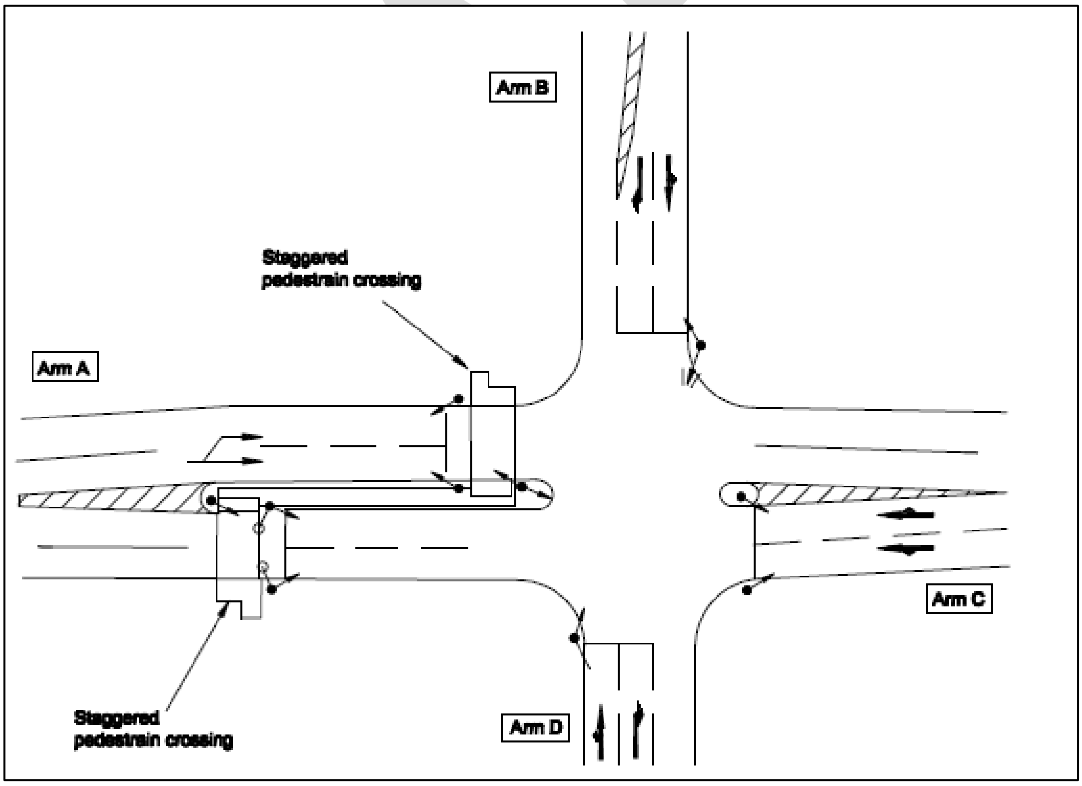


Figure 9-13: Signal-Controlled Crossroads with a Staggered Pedestrian Crossing.

9.9 Distance between Adjacent Intersections

Designers seldom have influence on the spacing of roadways in a network because it is largely predicted by the original or the developed land use. Nevertheless, the spacing of any type of intersections has an impact on the operation, level of service and capacity of a roadway.

Therefore, new intersection spacing should be based on road function and traffic volume. Table 9-11 shows the recommended minimum spacing between successive uncontrolled intersections.

Table 9-11: Minimum Spacings (m) between Uncontrolled Intersections

Design Speed (km/h)	Access Class	
	Marginal	Partial and Full
40	20	80
50	35	110
60	50	130
70	70	175
80	100	200
100	170	300
120	250	350

(Source: SANRAL*, Geometric Design Guidelines*)

9.10 Selection of Intersection Type

9.10.1 Steps in the Selection and Design Procedure

Table 9-12 lists the steps required for intersection design. In practice this process will also be dependent on current methods in use.

Table 9-12: Steps in the Selection and Design Procedure

Step	Activity
1	Data collection
2	Defining the major road and determining the intersection design speed).
3	Selecting the intersection category and type.
4	Consider the requirements of all road users
5	Preliminary landscape recommendations
6	Develop traffic flows
7	Assemble preliminary details of example design elements
8	Assess key geometric standards
9	Checking that it offers adequate safety and capacity for the predicted traffic manoeuvres. If not review a different intersection type
10	Determine requirements for connecting roads
11	Check that an effective signing system can be provided
12	Carry forward to appraisal stage

For priority intersections the minimum distance between consecutive intersections should, preferably, be equal to (10 x V_D) m; where V_D is the major road design speed in km/h. Where it is not possible to provide this minimum spacing, then the design shall incorporate either, or both, of the following:

1) A distance between minor road centre lines equal to the passing sight distance appropriate for the design speed of the junction plus half the length of the widened major road sections at each junction, or
2) A grouping of minor road junctions into pairs to form staggered T-junctions and a distance between pairs as in (1) above.

For signalized roads, the traffic signal cycle lengths and traffic speed should be consistent with the intersection spacings as indicated in Figure 9-13. The minimum spacing should be at least 400 m. Where spacings closer than the minimum already exists and the LoS is deemed deficient, several improvement are possible, for example:

1) Two-way flows can be converted to one-way operation;
2) Minor connecting roads can be closed or diverted;
3) Channelisation can be used to restrict turning movements.

LoS and driver perception are both affected by the spacing of intersections. In certain cases, it may be necessary to limit the number of intersections for reasons of safety and serviceability.

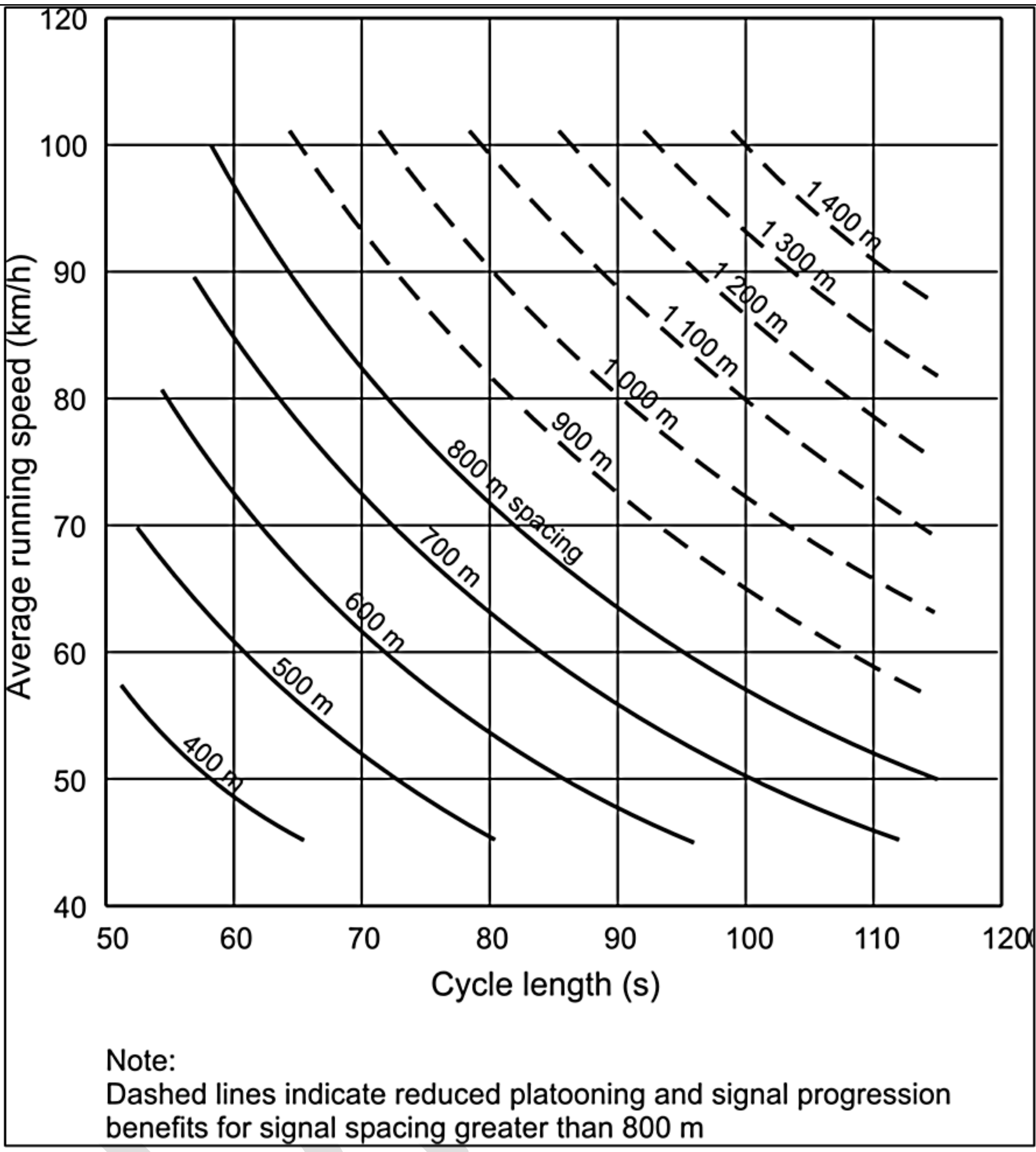


Figure 9-14: Desirable Spacings of Controlled Intersections

Source: SANRAL, Geometric Design Guidelines)

9.11 Design of the Elements of Priority Junctions

The detailed principles of intersection design are described under the following headings:

1) Horizontal and vertical alignment;
2) Lane widths and shoulders, central reserves and traffic islands;
3) Ghost islands;
4) Channelisation;
5) Medians;
6) Splitter islands;

7) Speed change lanes;
8) Merging and diverging;
9) Turning roadways;
10) Private access.

9.11.1 Horizontal and Vertical Alignment

Simple alignment design should enable drivers to recognise the intersection as early as possible and provide a timely focus on the intersecting traffic and manoeuvres that must be prepared. The following are specific operational requirements at intersections:

1) The alignments should not restrict the required sight distances.
2) The alignments should allow for the frequent braking and turning associated with intersections.
3) The environment around the intersection should not cause any distractions to drivers.
4) The alignments itself should not require a driver's attention to be detracted from the intersection manoeuvres and avoidance of conflicts.
5) The intersection should not be over a crest, in a sag or on a curve. If there is no choice, the horizontal curve radii at intersections should not be less than the radii shown in Table 9-5
6) For high-speed roads with design speeds in excess of 80 km/h, approach gradients should not be greater than -3 %. For low-speed roads in an urban environment this can be increased to -6 %.

9.11.2 Lane Widths and Shoulders

Through lanes width should normally be unchanged through the intersection. However, if they are wider than 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.

A right turning lane width should normally be 3.0 m.

The width of a traffic island depends on the type. Thus, an island created with road markings is normally 0.35 m wide for a double centre line. For a kerbed island, space is required for:

1) A 'pass left side only' traffic sign, 0.4 to 0.9 m.
2) Lateral clearances, minimum 0.3 m
3) An inner hard shoulder, if needed, in the opposite direction, 0.25 to 0.5 m for an edge line.

The widths of paved shoulders are as per the design class of road but should be narrowed in two-lane roads to 0.5 m 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, the central reserve should be widened to provide some protection if the driver decides to make the turn in two stages (i.e., crosses one major road traffic direction at a time).

9.11.3 Ghost Islands

A ghost island is a traffic island comprising oblique parallel line markings on the road to indicate that vehicles should not enter the painted area. Ghost islands have several uses but one of the most important is at intersections to encourage drivers to maintain lane discipline, especially when required to merge or diverge with another traffic stream.

They effectively discourage overtaking where it is likely to be hazardous and provide space for turning traffic to wait. Ghost islands should be used on new single carriageway roads, or in the upgrading of existing junctions to provide right turning vehicles with a degree of shelter from the through flow. They are effective in improving safety, and are relatively cheap, especially on wide 2-lane single carriageway roads where very little extra construction cost is involved. Examples are illustrated in the Figure 9-3, Figure 9-6 and Figure 9-7.

9.11.4 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 m 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 m 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 should 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 (mountable) flush kerbs unless lighting is provided, in which case raised kerbs may be used.

9.11.5 Channelisation

A traffic island is a defined area between traffic lanes for the control of vehicle movements. Figure 9-4 and Figure 9-7 show some examples. Traffic islands are used for 'channelling' traffic to manage the conflicts that are inherent in any intersection by guiding vehicles safely through the intersection area from an approach leg to the selected departure leg.

There are various aspects of good channelisation:

1) Traffic streams should cross at close to right angles and merge at flat angles.
2) Vehicle paths should be clearly defined.
3) It provides protection and storage for turning and crossing vehicles.
4) Points of conflict should be separated whenever possible.
5) Undesirable or 'wrong way' movements should be discouraged or prohibited.
6) Islands provide locations for traffic signs.
7) The islands provide refuges for pedestrians and the handicapped where appropriate.

8) The design should encourage safe vehicle speeds.

9) High priority flows should have the greater degree of freedom to manoeuvre.

10) Decelerating, slow-moving or stopped vehicles should be separated from higher speed through lanes.

Guidance is provided by lane markings that clearly define the required vehicle path and indicate auxiliary lanes for turning movements. For example, road markings are used to indicate that turns from selected lanes, either to the left or to the right, are mandatory.

The fully channelised T-junction design, (Figure 9-4) is for intersections with a moderate volume of turning traffic. An important feature is that there is only one through lane in each direction on the major road. This form of junction is designed to prevent overtaking and excessive speeds through the conflict zones. It is formed by widening the major road to provide a central reservation, a right turning lane and space for vehicles waiting to turn right from the major road into the minor road. A limiting factor is the left-hand sideways visibility from the driver's seat, which can be very restricted in some cabs and leaves the driver with no option but to make the manoeuvre in one stage. It usually has a traffic island in the minor road. In urban areas this would normally be kerbed to provide a refuge for pedestrians.

Typical island shapes are illustrated in Figure 9-15 (Note traffic control signals are not shown in the Figures).

Islands are generally either long or triangular, with the circular shape being limited to application in roundabouts. They are situated in areas not intended for use in vehicle paths. Directional islands are typically triangular with their dimensions and exact shape being dictated by:

1) The corner radii and associated tapers.

2) The angle of skew of the intersection; and

3) The turning path of the design vehicle.

Drivers tend to find an archipelago of small islands confusing and are liable to select an incorrect path through the intersection area. As a general design principle, a few large islands are preferred to several small islands.

The designer should bear in mind that islands are hazards and should be less hazardous than whatever they are replacing. Islands should not be less than 5 m^2 in area to ensure that they are easily visible to approaching drivers and, where necessary, additional guidance should be given by carriageway markings in advance of the nose supplemented, if necessary, by speed humps.

Islands may be kerbed, painted or simply non-paved. Kerbed islands provide the most positive traffic delineation and are normally used in urban areas to provide some degree of protection to pedestrians and traffic control devices. The island kerbs should be offset a minimum of 0.3 m from the edge of through-traffic lanes even if they are mountable.

Painted islands are usually used in suburban areas where speeds are low (in the range of 50 km/h to 70 km/h) and space is limited.

Traffic islands bordered by raised kerbs should not be used on the major road. In rural areas, kerbs are not common and, at the prevailing speeds in these areas, typically 100 km/h or more, they are a potential hazard. If it is necessary to employ kerbing at a rural intersection, the use of mountable kerbing should be considered.

As an additional safety measure, a kerbed island should always be preceded by a painted island with oblique parallel line (chevrons) markings limited by continuous longitudinal lines.

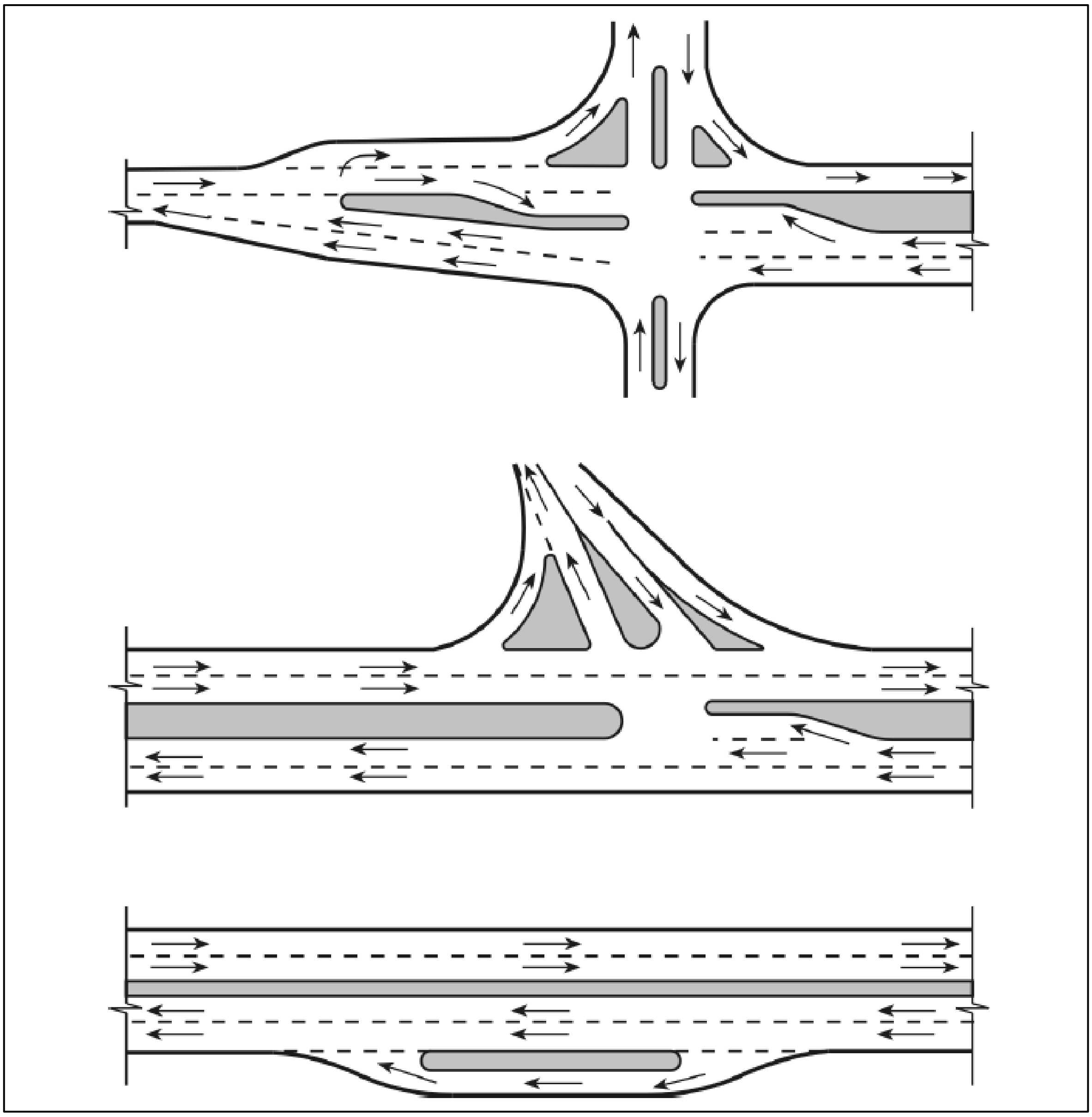


Figure 9-15: Typical Traffic Islands

Non-paved islands are defined by the pavement edges and are usually used for large islands at rural intersections. These islands may have delineators on posts and may be landscaped.

A typical triangular island is illustrated in Figure 9-16. The approach ends of the island usually have a radius of about 0.6 m as shown and the offset between the island and the edge of the travelled way is typically 0.6 m to 1.0 m to allow for the effect of kerbing on the lateral placement of moving vehicles. Where the major road has shoulders, the nose of the island is offset about 1.0 m from the edge of the usable shoulder. The side adjacent to the through lane is tapered back to terminate at the edge of the usable shoulder, thus offering some guidance and redirection. A kerbed cross-section on the major road suggests that the nose of the island should be offset by about 1.6 m from the edge of the travelled way, with the side adjacent to the through lane being tapered back to terminate 0.6 m from the edge of the through lane.

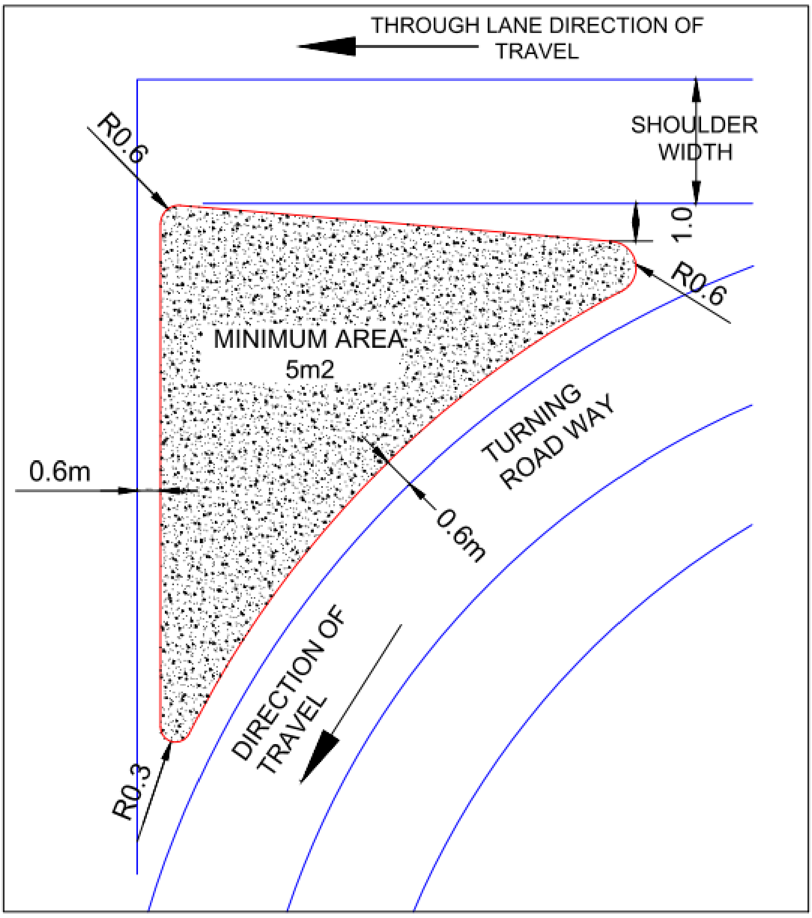


Figure 9-16: Typical Triangular Island

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.

9.11.6 Medians

Median islands are very useful and discussed in Chapter 4. The general layout of median openings at intersections is normally dictated by wheel-track templates. However, median openings should not be shorter than:

1) The surfaced width of the crossing road plus its shoulders.
2) The surfaced width of the crossing road plus 2.5 m (if kerbing is provided).
3) 12.4 m.

A further control on the layout of the median opening is the volume and distribution of traffic passing through the intersection area. If the median is wide enough to accommodate them, it may be advisable to make provision for speed-change and storage lanes. The additional lanes reduce the width of the median at the point where the opening is to be provided and thus influence the median end treatment.

The median end treatment is determined by the width of the median. Where the median is 3 m wide or less, a simple semicircle is adequate. For wider medians, a bullet nose end treatment is recommended. The bullet nose is formed by arcs dictated by the wheel paths of turning vehicles and an assumed nose radius of 0.6 to 1.0 m. This results in less intersection pavement area and a shorter length of opening than the semi-circular end.

Median width of 5 m and above, the width of the minor road controls the length of the opening. A flattened bullet nose, using the arcs as for the conventional bullet nose but with a flat end as dictated by the width of the crossing road, is recommended. These end treatments are illustrated in Figure 9-17.

The bullet nose and the flattened bullet nose have the advantage over the semi-circular end treatment that the driver of a turning vehicle has a better guide for the manoeuvre for most of the turning path. Furthermore, these end treatments result in an elongated median which provides a better refuge area for pedestrians crossing the dual carriageway road.

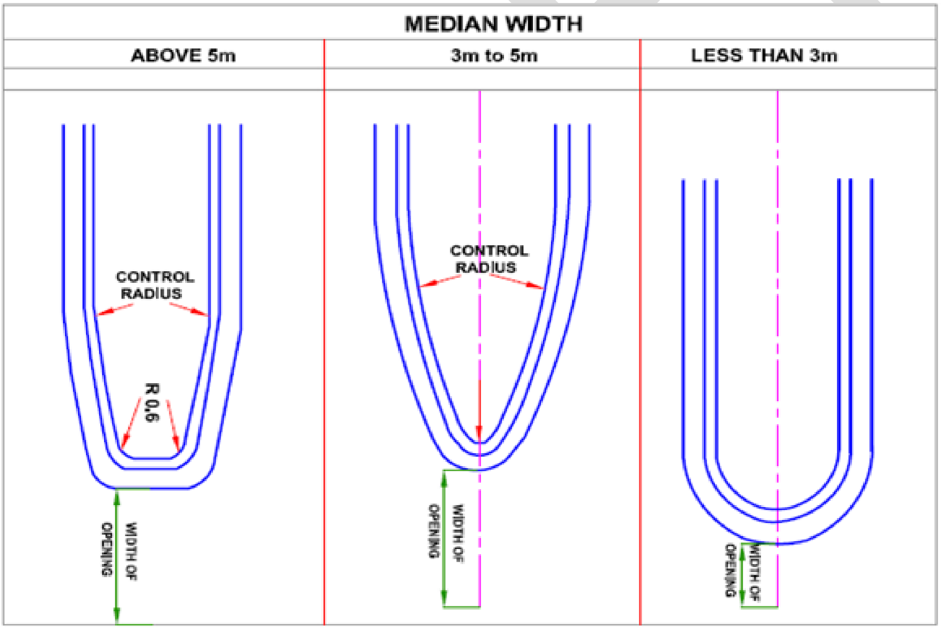


Figure 9-17: Median End Treatment

9.11.7 Splitter Islands

Dividing, or splitter, islands usually have a teardrop shape as shown by the splitter island in Figure 9-18 and Figure 9-19. They are often employed on the minor legs of an intersection where these legs have a two-lane, two-way or four-lane undivided cross-section. With 4-lanes there should also be traffic lights.

The principal function of a dividing island is to warn the driver of the presence of the intersection. This can be achieved if, at the widest point of the island, its edge is in line with the edge of the approach leg. To the approaching driver, it appears as though the entire lane had been blocked off by the island. If space does not permit this width of island, a lesser blocking width must be applied, but anything less than half of the approach lane width is not effective.

Splitter islands are also used in the approach to roundabouts where there is a need to redirect vehicles entering a roundabout through an angle of not more than 30°.

Dividing islands are usually kerbed to ensure that the island is visible within normal stopping sight distance. However, it may be advisable to draw the driver's attention to the island by highlighting the kerbs with paint or reflective markings. As in the case of the triangular island, the nose of the dividing island should be offset by 0.6 m from the centreline of the minor road. For the sake of consistency, the radius of the nose should be of the order of 0.6 m.

The balance of the shape of the island is defined by the turning paths of vehicles turning both from the minor road to the major road and from the major road to the minor.

9.11.8 Channelised Intersection Layout

A channelised intersection, as shown in Figure 9-3 is to be used whenever a separate right turn lane is required. The layout in Figure 9-6 also includes a left turn lane.

The partly channelised intersection shown in Figure 9-4 is to be used whenever a separate right turn lane is not required. The layout shown in Figure 10.3 does not include a left turn lane, but such a lane may be included if required, as in Figure 9-6.

A variety of intersections and recommended dimensions are provided in the UK's Highways Agency publication, TD42/95 *Geometric Design of Major/Minor Priority Junctions.*

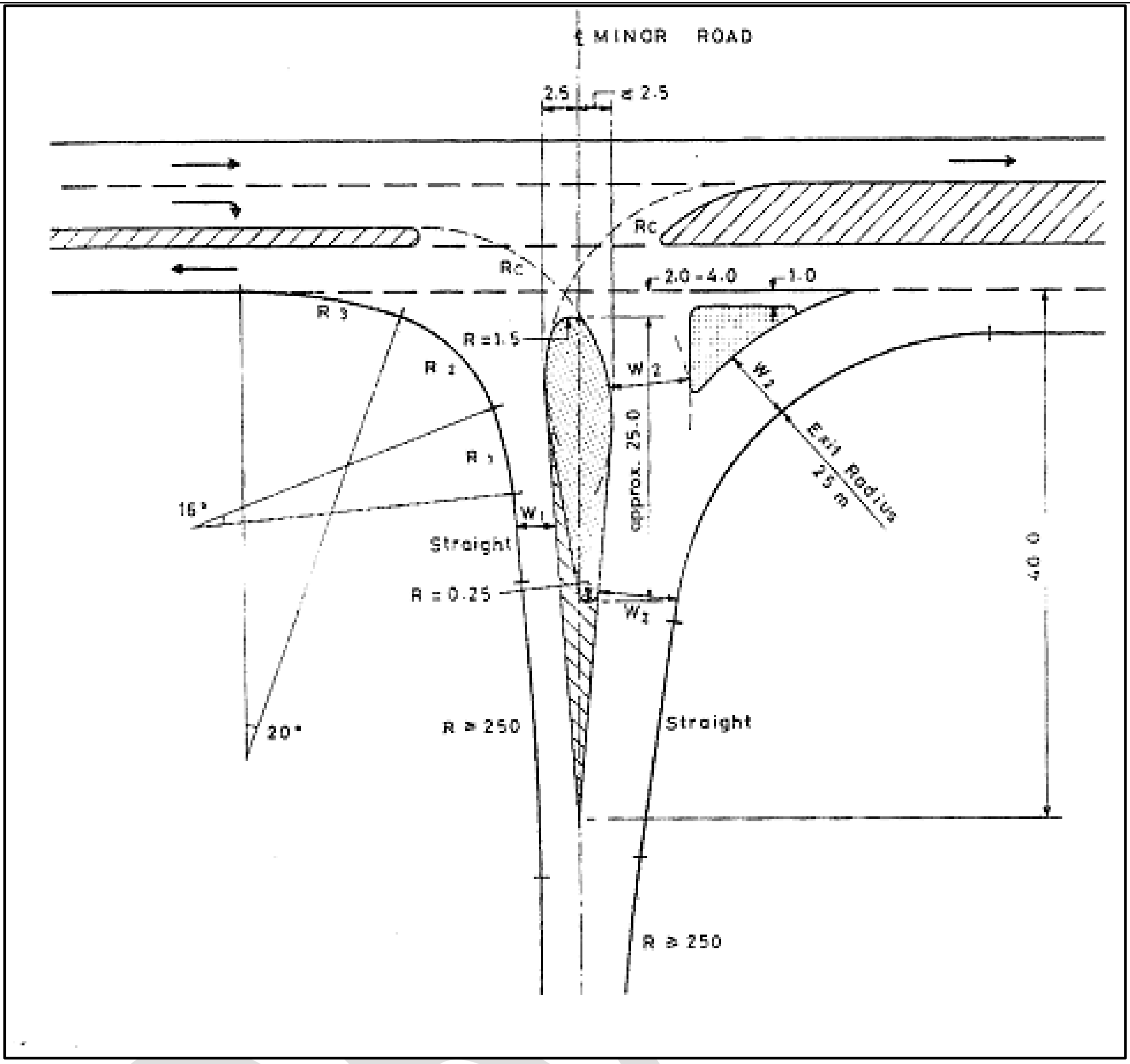


Figure 9-18: Layout of a Channelised Intersection

Notes:

R_C = Central radius dependent upon vehicle turning characteristics (minimum turning radius) recommended value: 15m.

The ratio R_1: R_2: R_3 to be 2:1:3 and the recommended value for R_2 is 12.0m.

W_1 is equal to the minor road lane width but shall not be less than 3.0m.

W_2 is 5.5m (excluding offsets to raise kerbs)

For detail of major road widening, see Section 9.11.9

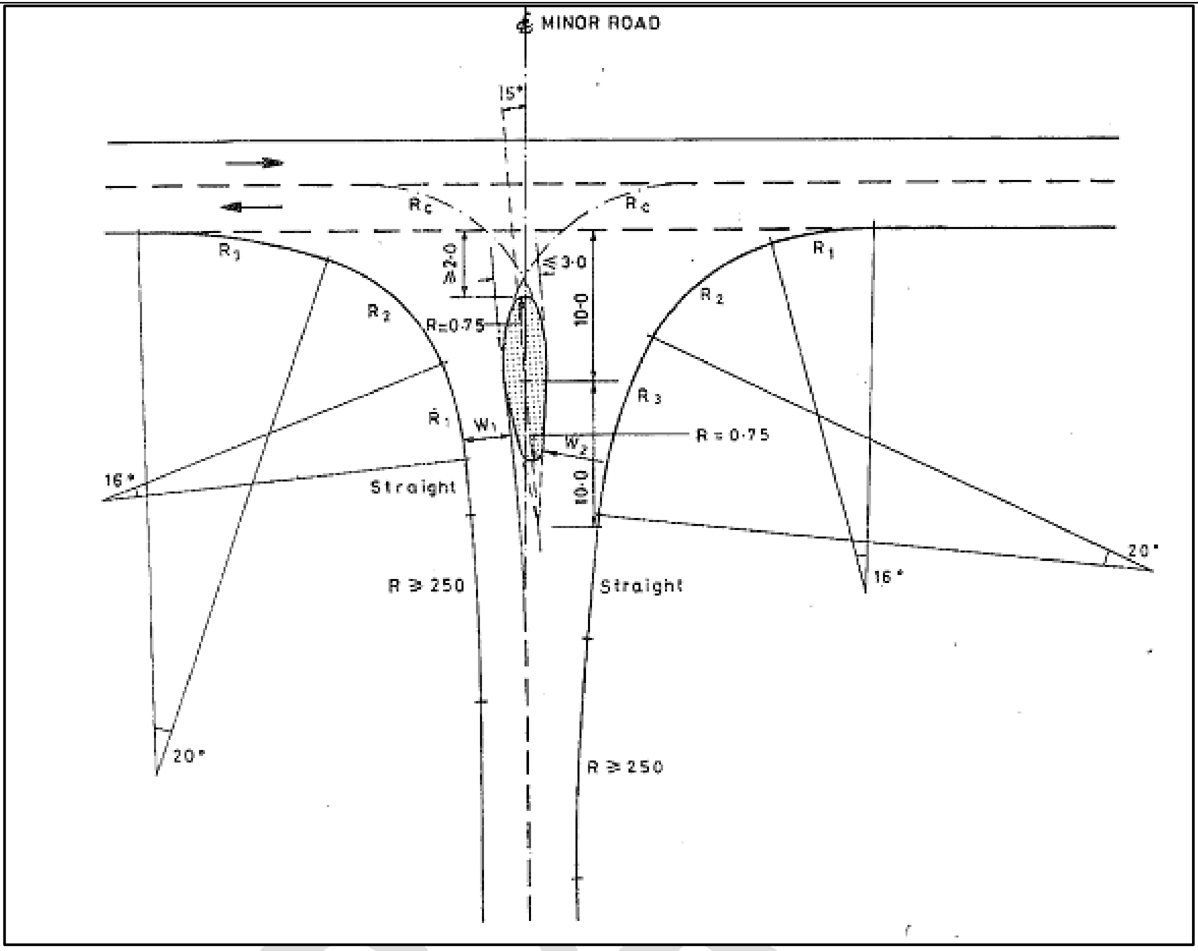


Figure 9-19: Layout of a Partially Channelised Intersection

Notes:

R_C = Central radius dependent upon vehicle turning characteristics.

The ratio R_1: R_2: R_3 to be 2:1:3 and R_2 will be dependent on vehicles turning characteristics and proportion of large vehicles. Recommended range for R_2 is 8.0-12.0m.

W_1 is equal to the minor road lane width.

W_2 is dependent upon vehicle turning characteristics.

9.11.9 Widening of the Major Road at the Intersection

To accommodate the 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 should be the same as the approach lanes and the widening should be designed so that the through lanes are given a smooth and optically pleasing alignments.

On straight alignments, the widening should be provided by the deviation of the through lane opposite the minor road. This deviation should be effected gradually by introducing a radius of between 5,000 and 10,000m at the beginning and end of the widening and 1 in 45 tapers.

On curved alignments, a smooth alignment for the through lanes can be achieved by widening on the inside of the curve. This is done by introducing transition curves which approximate to 1 in 45 tapers.

If the intersection is located on a crest or in a horizontal curve it is advisable to lengthen the island because this will make the intersection more visible to approaching traffic.

Excessive intersection widths should be avoided to discourage high speeds and overtaking.

Where intersecting roadways have shoulders or sidewalks, the shoulder of the main road should be continued through the intersection. Lane widths should be 3.7 m for through lanes and 3.6 m for turning lanes. Where conditions are severely constrained, lane widths as low as 3.3 m can be considered if approach speeds are below 80 km/h. In constricted urban conditions on low speed-roadways, lane widths of 3.0 m should be the minimum adopted.

All traffic lanes should be of adequate width and radius for the appropriate vehicle characteristics. To accommodate truck traffic, turn radii must be a minimum of 12 m.

Offsets from the edge of a turning roadway to kerb lines should be 0.6 to 1.0 m.

The edges of traffic lanes should be clearly indicated by road markings.

9.11.10 Speed Change Lanes

Deceleration lanes for vehicles turning left or right from a major road are of particular importance on higher speed and higher volume roads when a vehicle slowing down to leave the major road may impede the following vehicles and cause a hazardous situation. Similarly, a vehicle joining a high-speed road will also cause a hazardous situation unless it can increase its speed to that of the traffic on the road before merging; hence an acceleration lane is also desirable. Thus, speed change lanes comprising a taper section and deceleration lane should be provided for:

1) T1, T2 class roads;
2) T3 roads (and others) if the design speed exceeds 85km/h;
3) The present year traffic on the major route exceeds 1500 AADT or the peak hour flows exceed the values shown in Figure 9-20.
4) The present turning traffic onto the minor route exceeds 750 AADT or the peak hour flows exceed the values shown in Section 9.11.11

The length of such speed-change lanes is based on acceptable levels of discomfort for decelerating (and for accelerating) which are approximately half of those used in the calculation of stopping sight distance because the latter is concerned with emergency braking. These lengths are therefore greater than stopping sight distances.

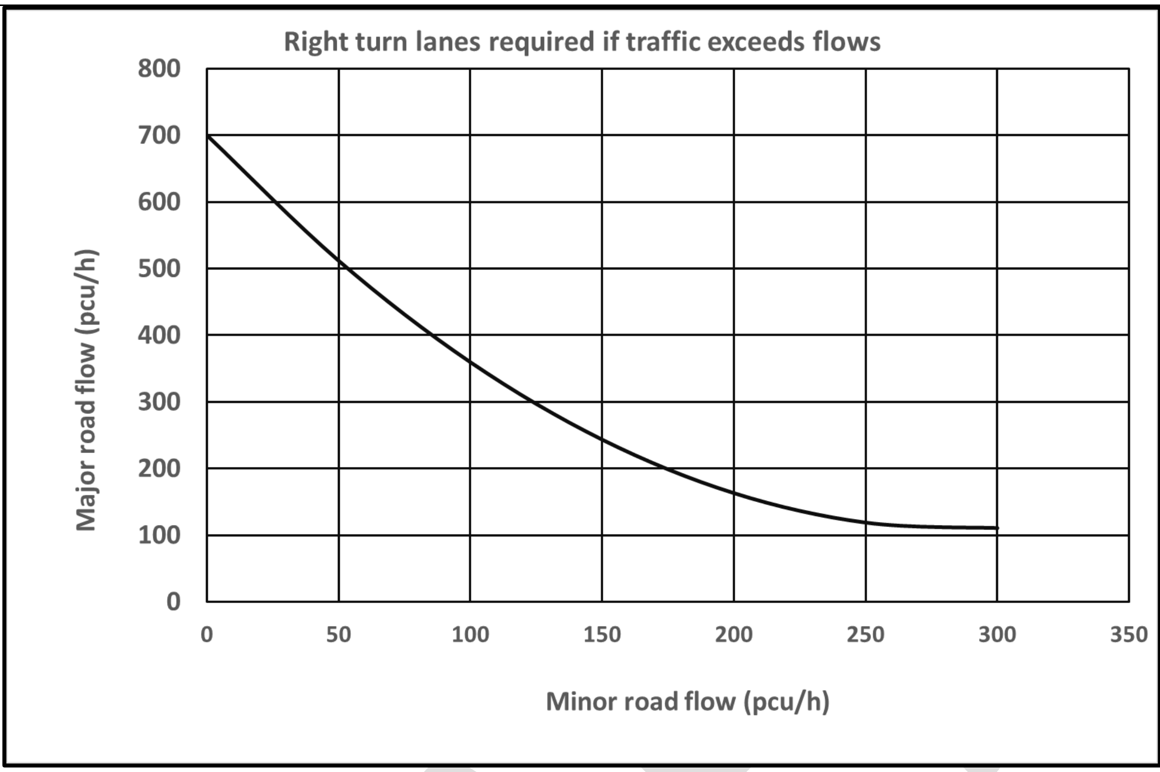


Figure 9-20: Conditions Requiring a Right Turn Lane

9.11.11 Decelerating Lane: Left Turn

It is assumed that a vehicle will leave the through lane at operating speed and negotiate the taper at unaltered speed, i.e., zero speed differential, and then decelerate on the portion of the lane that is parallel with the through lane.

Deceleration rates are a function of the design speed of the major road and the ramp or exit control speed. As both speeds increase, so does the deceleration rate, which varies between 1.0 and 2.0 m/s^2. The rate used to develop Table 8.14 is 2.0 m/s^2.

A detail of the layout for the Left Turn Lane is shown in Figure 9-21

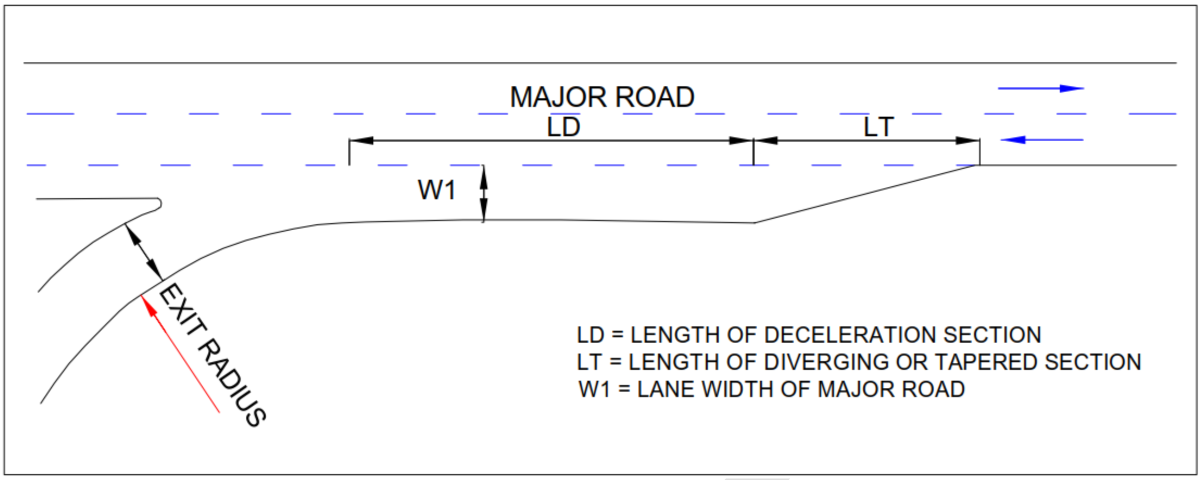


Figure 9-21: Layout for Left Turn Lane

The length of the left turn lane including the taper, measured as shown in the Figure 9-21, is related to design speed as indicated in Table 9-13. On up-hill gradients these distances are shorter and on down-hill grades they are longer. The increase or decrease in length is linear and is 5% for every 1% change in grade. Thus, for example, for a down-hill grade of 4 % the length should be increased by 20%.

Table 9-13: Length of Left Turn Lane

Main road design speed (km/hr)	Length of diverging (taper) (LT) (m)	Length of deceleration section (LD)							
		Exit control speed (km/hr)							
		0	40	50	60	70	80	90	100
60	65	70	40	25	-	-	-	-	-
70	75	95	60	50	25	-	-	-	-
80	80	125	90	75	55	30	-	-	-
90	85	155	125	110	85	60	30	-	-
100	90	190	160	145	125	100	70	35	-
110	100	235	200	190	165	140	110	75	40
120	110	280	245	230	210	180	155	120	85
130	115	325	290	275	255	230	200	170	135

(Source: SANRAL, Geometric Design Guidelines)

The actual entrance or exit lane from the major road to the minor road can take the form of a taper or a parallel lane. A taper is preferred. Table 9-14 indicates the taper rates for exit lanes.

The width of the major approach lane must be the same as the width of the traffic lanes.

Table 9-14: Taper Rates for Exit Lanes (or Ramps)

Design Speed (km/h)	Radius (m) for 2% super-elevation	Taper rate	Taper Length (m)
60	1000	1:14	67
70	1500	1:17	76
80	1500	1:17	76
90	2000	1:20	86
100	2500	1:22	92
110	3000	1:25	102
120	3500	1:27	108
130	4000	1:28	112

9.11.12 Acceleration Lanes

Acceleration lanes are less useful than deceleration lanes because entering drivers can always wait for an opportunity to merge without disrupting the flow of through traffic. Their principal application is on high volume roads where, at peak periods, gaps between vehicles are infrequent and short.

The ideal length of an acceleration lane depends on the acceleration of the slower vehicles, namely large, heavy trucks. The wide range of truck sizes and designs means that these acceleration characteristics also cover a wide range of values hence choosing an acceptable value that provides a satisfactory speed for merging, and therefore a satisfactory level of safety, at acceptable cost is essentially a matter of judgement. An acceleration rate of 0.7 m/s^2 has been selected and the lengths of the acceleration lanes for different speed differentials are as shown in Table 9-15. Acceleration also takes place on the taper, which is thus included in the overall length of the acceleration lane.

Table 9-15: Length of Acceleration Lanes including Taper (m)

Main road design speed (km/h)	Entry/ramp control speed (km/h							
	0	40	50	60	70	80	90	100
60	200	150	150	-	-	-	-	-
70	270	180	150	150	-	-	-	-
80	350	265	215	155	150	-	-	-
90	450	360	310	250	175	150	-	-
100	550	460	415	350	280	200	150	-
110	670	580	530	470	395	315	220	150

120	790	705	655	595	525	440	345	240
130	930	840	795	735	660	580	485	380

The designs are incorporated into the Standard Detailed Drawings for all intersections on trunk and link roads.

9.11.13 Right Turn Lanes.

Right turn lanes are required when the traffic flows exceed the values shown in Figure 10-20 It consists of a taper section, a deceleration section, and a storage section. The minimum lengths are as for left turn lanes and shown in Table 9-16

Details of the layout for a right turn lane are shown in Figure 9-22 for a single carriageway and in Figure 9-23 for a dual carriageway. Both figures also show additional details of a right turn lane.

Table 9-16: Lengths of Storage Sections for Right Turn Lanes

Right-Turning Traffic (AADT)	Length of Storage Section (L_S) (m)
0-1500	20
1500-3000	40
>3000	60

Provision of right turn lanes can be made for the major road. On single carriageway roads, a painted central reserve must always be used, and *traffic control is also necessary* (not shown in the Figure). To accommodate a right turn lane, the carriageway must be widened to provide the required width (Section 9.11.9). The widening must be designed so that the through lanes are given smooth and optically pleasing alignments. The width of the through lanes at the intersection must be the same as the approach lanes.

The widening must be provided by the deviation of both through lanes from the centreline. This should be achieved by introducing a taper of 100 m length at the beginning and end of the widening or by introducing a horizontal curve of large radius as described in Section 9.11.1

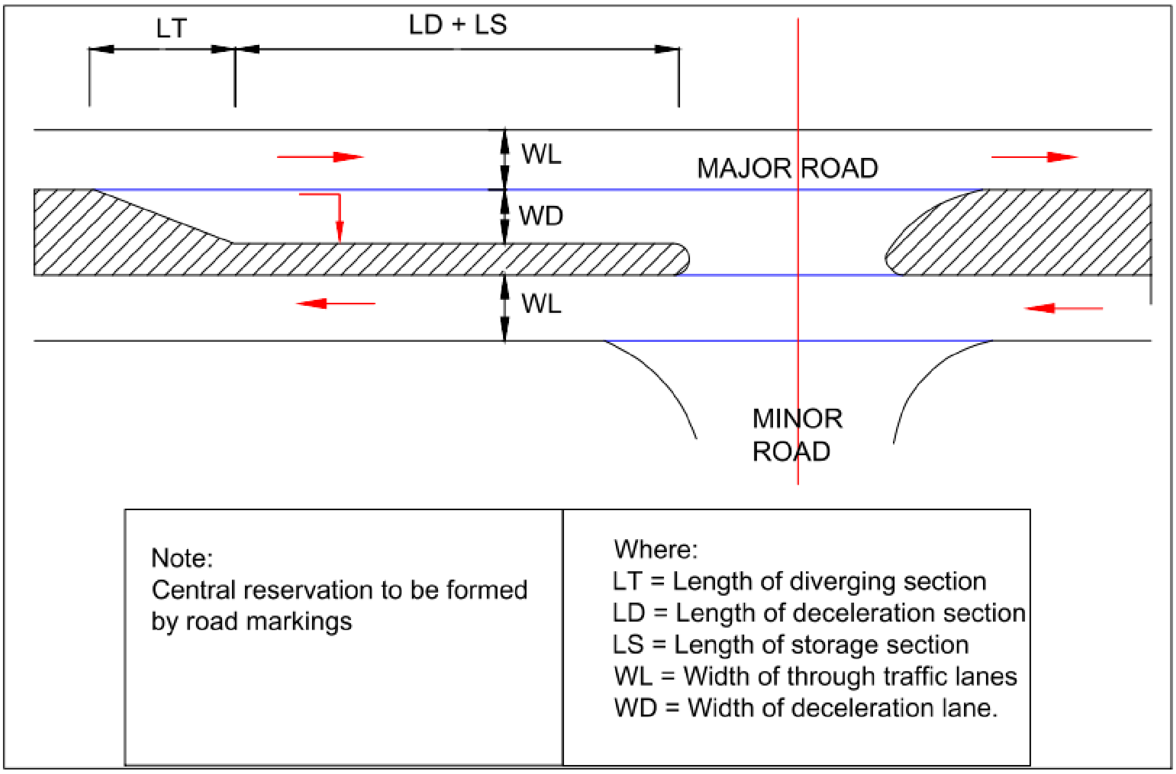


Figure 9-22: Layout for Right Turn Lane, Single Carriageway

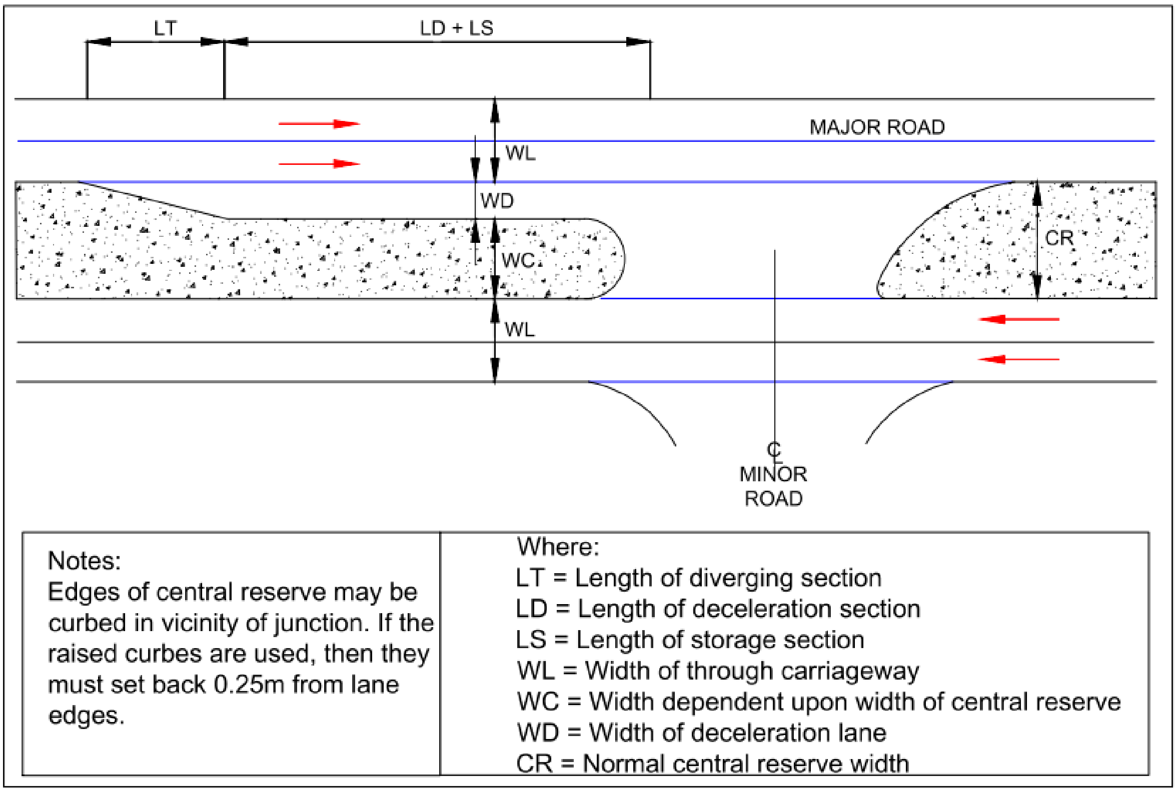
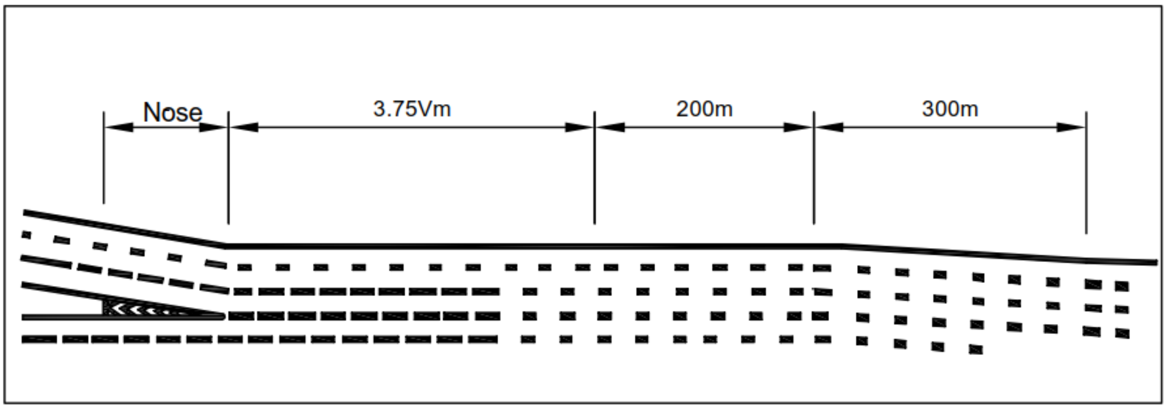


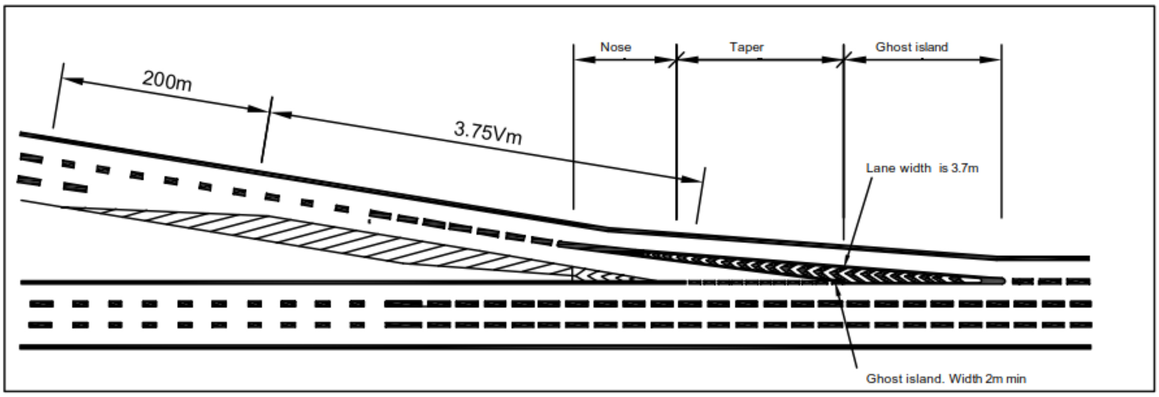
Figure 9-23: Layout for Right Turn Lane, Dual Carriageway

9.11.14 Merging and Diverging

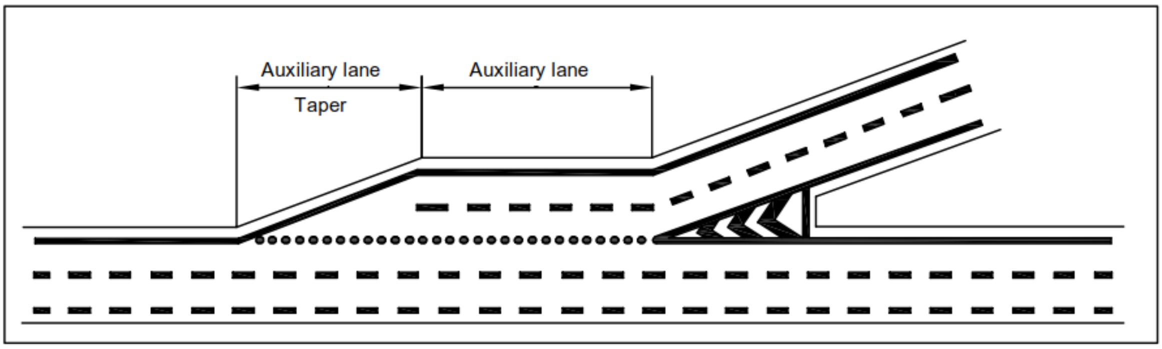
Merging and diverging lanes are required for most intersections. Where more than one lane merging with one other is being designed, ghost islands or other means of keeping traffic safely separated are required. Figure 9-24 illustrates some typical designs.



(a) Simple parallel merge with offside lane drop showing typical lengths



(b) Lane drop on merging carriageway using ghost islands



(c) Simple parallel diverge

Figure 9-24 Examples of Merge and Diverge Lanes

9.11.15 Turning Roadway

Turning roadways are channelised roadway sections at an at-grade intersection. Traffic movements are accommodated either within the limitation of the crossing roadway widths or through the application of turning roadways. Turning roadways can be designed for three possible types of operation:

1) Case 1 One-lane one-way with no provision for passing stalled vehicles.
2) Case 2 One-lane one-way with provision for passing stalled vehicles.
3) Case 3 Two-lane one-way operation.

Three traffic conditions must also be considered:

1) Condition A Insufficient trucks in the traffic stream to influence design.
2) Condition B Sufficient trucks to influence design.
3) Condition C Sufficient semi-trailers in the traffic stream to influence design.

The lengths of turning roadways at intersections are normally short, so that design for Case 1 operation is sufficient. Even in the absence of traffic counts, there will usually be enough trucks in the traffic stream to warrant consideration hence Condition B is normally adopted for design purposes. Widths of turning roadway for the various cases and conditions are shown in Table 9-17. The radii in the Table refer to the inner edge of the pavement. The values supersede any value quoted elsewhere that do not include provision for large semi-trailers.

Table 9-17 Turning Roadway Widths

Inner Radius (m)	Case 1			Case 2			Case 3		
	Condition			Condition			Condition		
	A	B	C	A	B	C	A	B	C
15	4.0	5.5	7.9	6.1	8.8	13.4	7.9	10.7	15.2
20	4.0	5.2	6.7	5.8	8.2	11.0	7.6	10.1	12.8
30	4.0	4.9	6.4	5.8	7.6	10.4	7.6	9.4	12.2
40	3.7	4.9	6.4	5.5	7.3	8.8	7.3	9.1	10.7
60	3.7	4.9	5.2	5.5	7	8.2	7.3	8.8	10.1
80	3.7	4.6	5.2	5.5	6.7	7.6	7.3	8.6	9.4
100	3.7	4.6	4.9	5.2	6.7	7.3	7.0	8.4	9.1
150	3.7	4.3	4.6	5.2	6.7	7.3	7.0	8.2	9.1

Source: SATCC: *Code of Practice for the Geometric Design of Trunk Roads.*

Figure 9-25 shows the design of a typical crossroad intersection illustrating turning sections, channelisation islands, deceleration lanes, tapers, medians, and mountable kerbs.

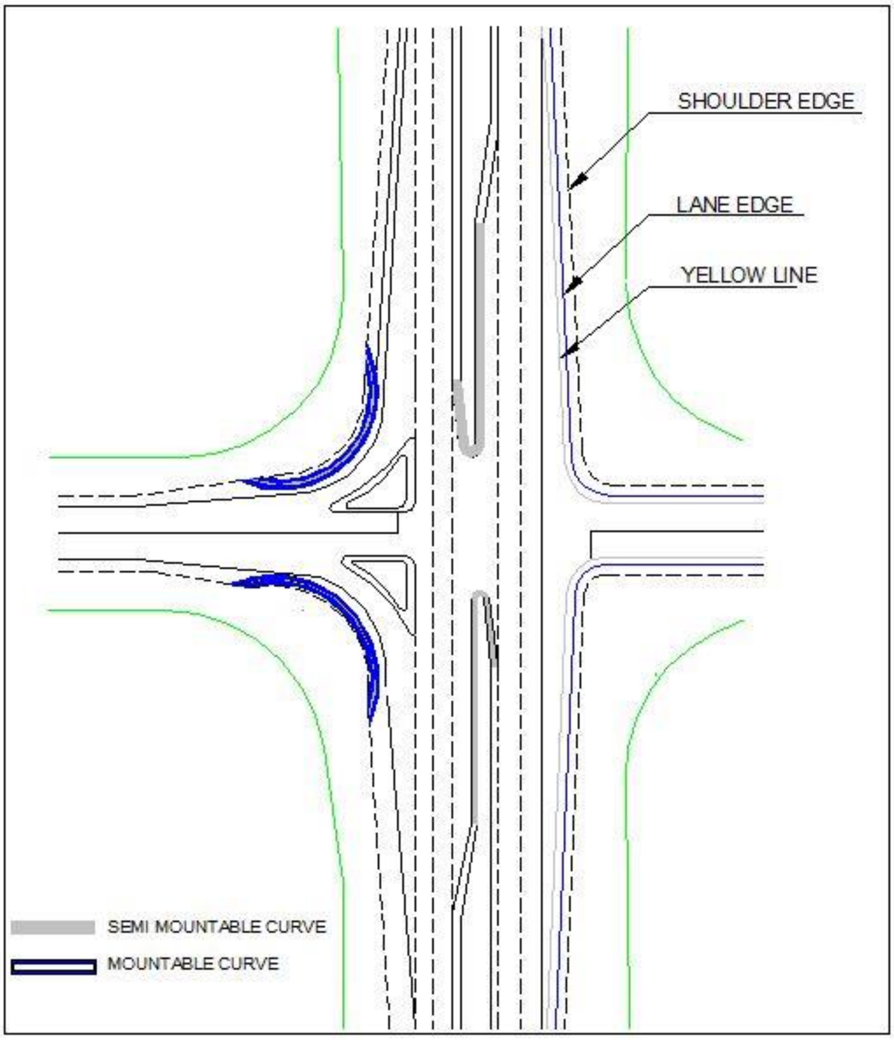


Figure 9-25 Typical Crossroad Intersection Showing all Elements.

9.11.16 Private Access

A private access is defined as the intersection of an unclassified road with a classified road. An access must have entry and exit radii of 6 m or greater, depending upon the turning characteristics of the expected traffic. The minimum width must be 3 m. A typical access is shown Figure 9-26. The location of the access must satisfy the visibility requirement for 'stop conditions' described in Section 4.5. A traversable drainage pipe must be placed as required. The access must be constructed back to the road reserve line, with a taper to match the existing access.

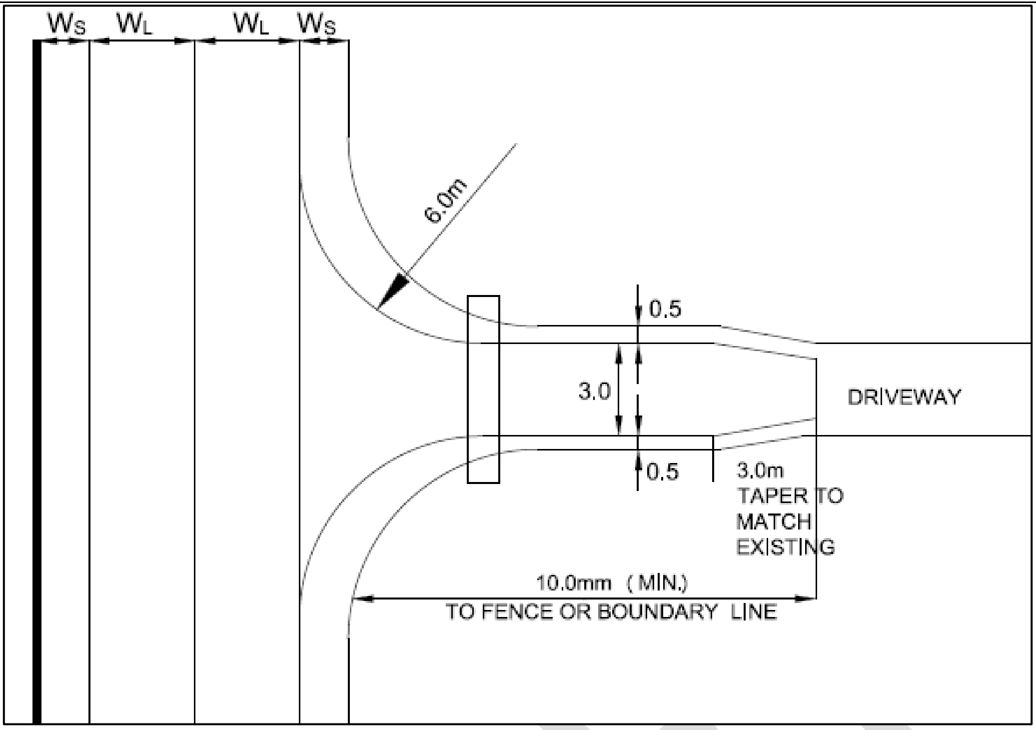


Figure 9-26 Typical Access

9.12 Design of Signalised Intersections

Well-designed signal control at an intersection enhances traffic safety and efficiency by reducing congestion and conflicts between vehicle movements. The principle is that vehicles passing a steady green signal or green arrow signal (a protected turn) must not encounter any primary conflicts (i.e., crossing vehicles). However lower order conflicts (i.e. with turning vehicles) may be acceptable in some circumstances.

When total peak hour traffic is similar on both the major and minor road, traffic signal control is usually justified when the sum of vehicle flows on the major road plus that on the minor road exceeds 900 vph. When the traffic on the minor road is low (<100 vph) this threshold guideline increases to 1500 vph.

The major advantages compared to priority-controlled intersections are:

1) maximum waiting time is fixed and known (if the intersection capacity is not reached);
2) available capacity is distributed fairly between approaches; and,
3) drivers on the minor road do not have to make a judgment about when it is safe to proceed.

Close co-operation is necessary with the signal control and electrical engineers throughout the design process, especially in the early stages.

Most of the safety problems that arise with signalised intersections are related to drivers passing the signal at red. Rear-end collisions also occur when the signal changes from green to red because some drivers attempt to cross late. This has implications for signal visibility and timings.

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 9-27.

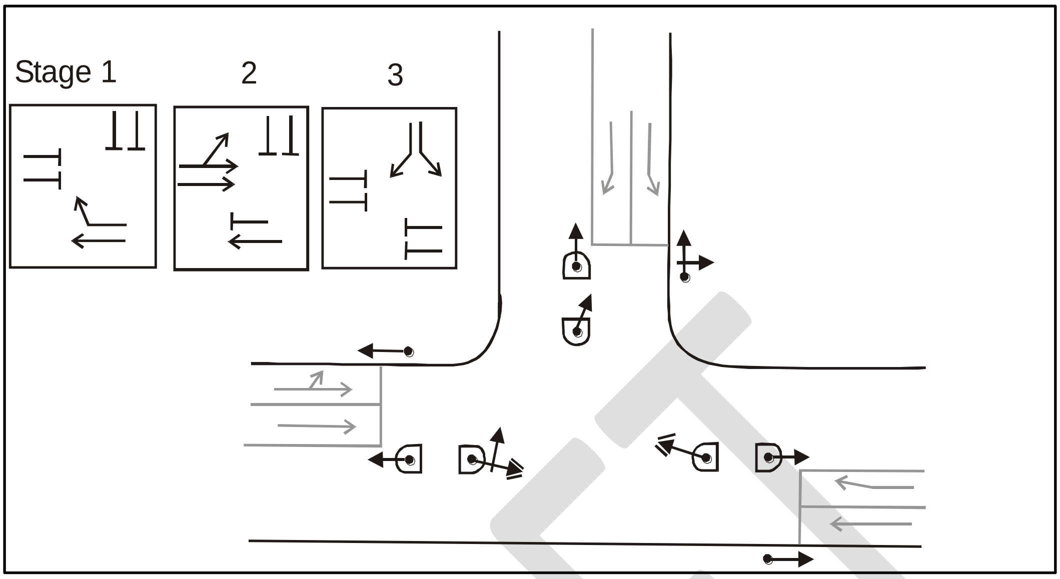


Figure 9-27 Protected Right Turn Sequence

The signal control can work on fixed or vehicle-actuated timings which adapt to traffic conditions by means of vehicle detectors. Vehicle actuated (i.e., demand-responsive) signals are much more efficient and drivers are more likely to comply with them.

On a vehicle-actuated system each stage has a minimum and maximum green time. There should always be an inter-green period (i.e., a short period when no green signals are showing) between conflicting stages to allow for safe stage changes. The length of the inter-green period depends on the size of the intersection, the speed limit and whether pedestrians and cyclists are being accommodated. For details of traffic signal management, the reader should consult FHWA (2008), *Traffic Signal Timing Manual,* Report HOP-08-024.

9.12.1 Control Strategy and Layout

Signalised intersections should normally be restricted to roads with a speed limit 50 km/h and never where the speed exceeds 70km/h. Where signals are needed on roads with speeds higher than 50km/h additional equipment is needed to ensure safety, for example, overhead mounted signals on each high-speed approach. For more information see the SADC Road Signs Manual.

Protected right turns are always 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 can be longer.

Pedestrian crossing signals may also be provided at signalised intersections. Ideally, they should have their own stage during which there should be no conflicts with vehicle movements. Table 9-18 shows the criteria to be met for installing pedestrian crossing signals.

Table 9-18 Criteria for Traffic Signalisation of Cross Walks

Speed Limit (km/h)	Traffic Volume (ADT)	Pedestrians/Cyclists (Number in maximum hour)
30	5000 – 8000	>30
	>8000	>20
40	5000 – 8000	>20
	>8000	>10
50	5000 – 8000	>20
	>8000	>10
60	2000	>20
70	1500	>20

The capacity analysis should be based on expected traffic volumes during the design hour, normally both morning and evening peaks.

9.12.2 Visibility

Each traffic lane must have clear vision of at least one primary signal head associated with its movement from the desirable stopping sight distance (70m at 50 km/h and 110m at 70 km/h). It is also important that the desirable stopping sight distance is available to all traffic entering the queue estimated from the capacity and traffic flow calculations. Warning sign for traffic signals must be used where the visibility is impaired.

The intersection inter-visibility 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 8.28.

Designers should aim to achieve the greatest level of inter-visibility within this zone to permit manoeuvres to be completed safely once drivers, cyclists and pedestrians have entered the zone.

Signalisation may be an appropriate accident countermeasure for higher volume intersections with restricted sight distance and a history of sight-distance related crashes. However, traffic signals may fail from time to time. Furthermore, traffic signals at an intersection are sometimes placed on two-way flashing operation under off-peak or night-time conditions. To allow for either of these eventualities, the appropriate departure sight triangles as set out for **Case B in Appendix C**, should be provided for the minor-road approaches to ensure a minimum level of safety when the signals are out of order.

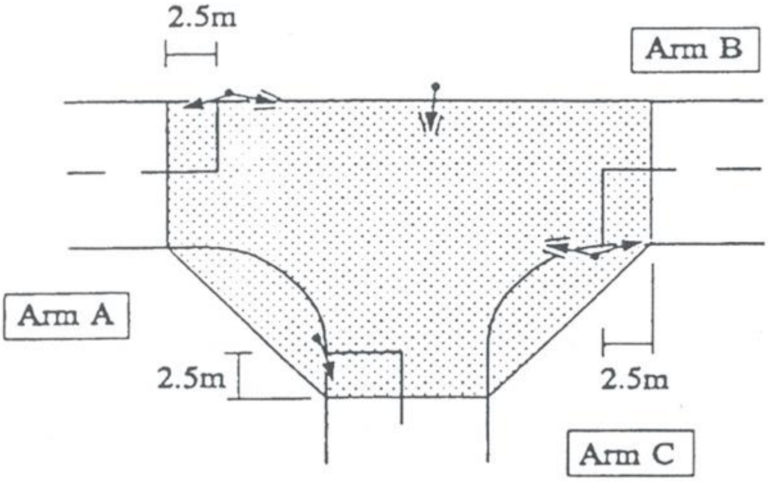


Figure 9-28 Inter-visibility Zone without Pedestrian Crossing

9.12.3 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 may 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.

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 inter-green periods.

The entry taper, Li, (Figure 9-29) of a kerbed entry lane should be a minimum of 30 m (taper 1:10) to allow a semi-trailer to cope with it. Tapers can be narrowed to 1:5 to allow more queuing space within the same total length.

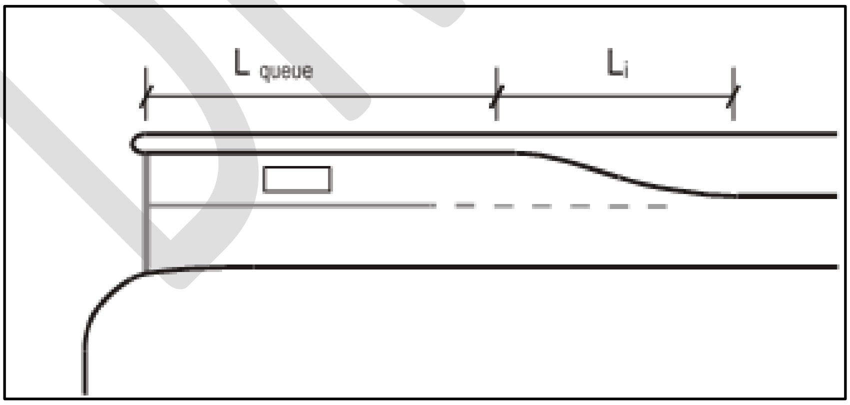


Figure 9-29 Right Turn Lane Design

Minimum design measurements for a right turn with a ghost island are shown in Figure 9-30.

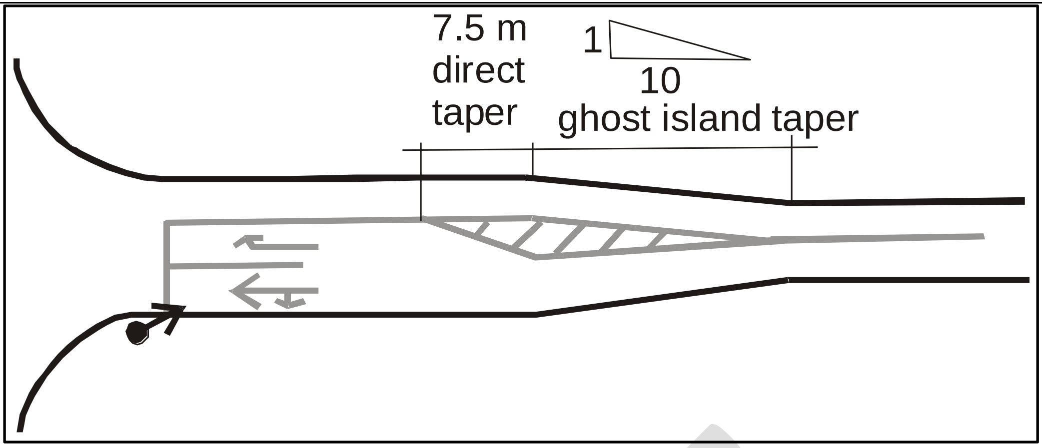


Figure 9-30 Ghost Island Layout

Filter lanes for left turning vehicles can be signalised or uncontrolled (i.e., give way signs and markings). They can be used when left turn manoeuvres for large vehicles are required (Figure 9-31). Uncontrolled left turn lanes improve the efficiency of the traffic signal control, as inter-greens can be decreased, especially at high left turn volumes. Uncontrolled traffic should be separated with a triangular separation island.

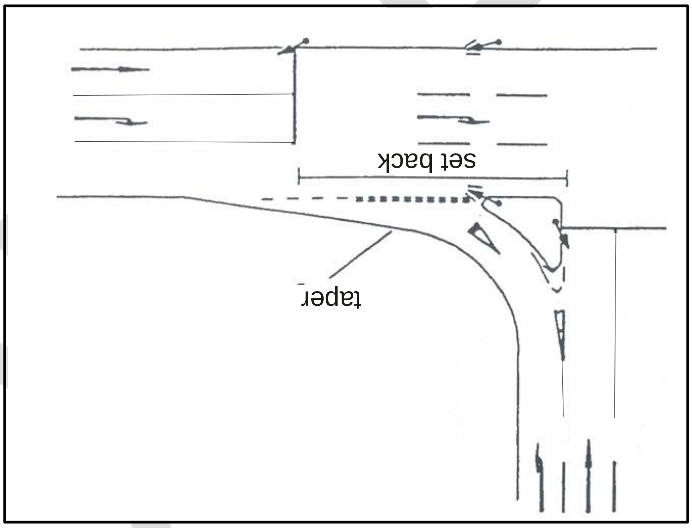


Figure 9-31 Left Turn Filter Lane with Taper to Facilitate Large Vehicles

If left turn filter lanes are used, a consistent design approach should be adopted for ease of understanding. Uncontrolled filter lanes can be confusing for pedestrians. Uncontrolled and controlled pedestrian crossings should not be mixed within the same intersection.

The number of straight-ahead entry and exit lanes should be balanced to reduce conflicts caused by traffic merging or diverging within the intersection (Figure 9-32). 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 condition.

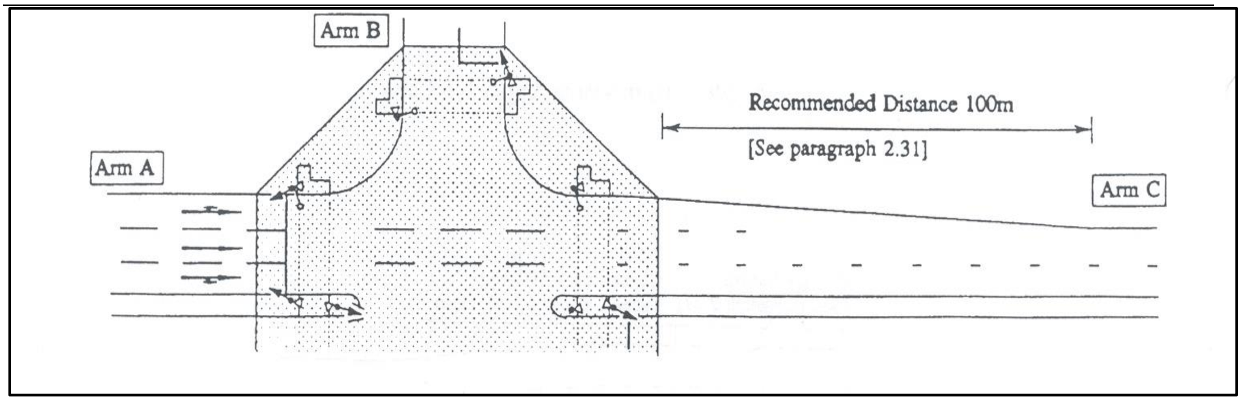
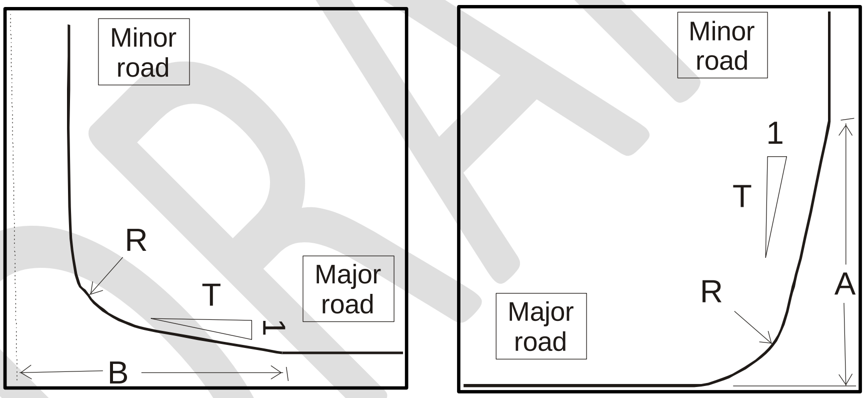


Figure 9-32 Lane Drop Design Principles

9.12.4 Swept Paths and Corner Curves

The design of corner curves and channel width depend on what design vehicle and level-of-service is chosen (Chapters 3 and 4). Signalised intersections with very low volumes of large trucks and buses could have simple 6 m corner radii to minimise the intersection area and optimise the signal control strategy. The radius should be increased to 10 m if rigid trucks or buses are common and 15 m in rural areas where larger trucks might operate. The following combinations of tapers and corner radii can be used in urban areas to accommodate semi-trailers, see Figure 9-33.



	R	T	A	B
Urban	10	5	30	30
Rural*	15	10	25	25

Figure 9-33 Combinations of Tapers and Corner Radii

It is also essential to ensure that adequate turning radii are provided for the swept paths of all types of vehicles using the intersection as shown in Figure 9-34. Swept paths must be checked for all permitted turning movements to control locations of traffic islands, signals etc. The example on the left of the Figure 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, if available at the correct scale, can be used for checking whether semi-trailers can negotiate intersections, but the use of specialist computer software gives a more accurate simulation.

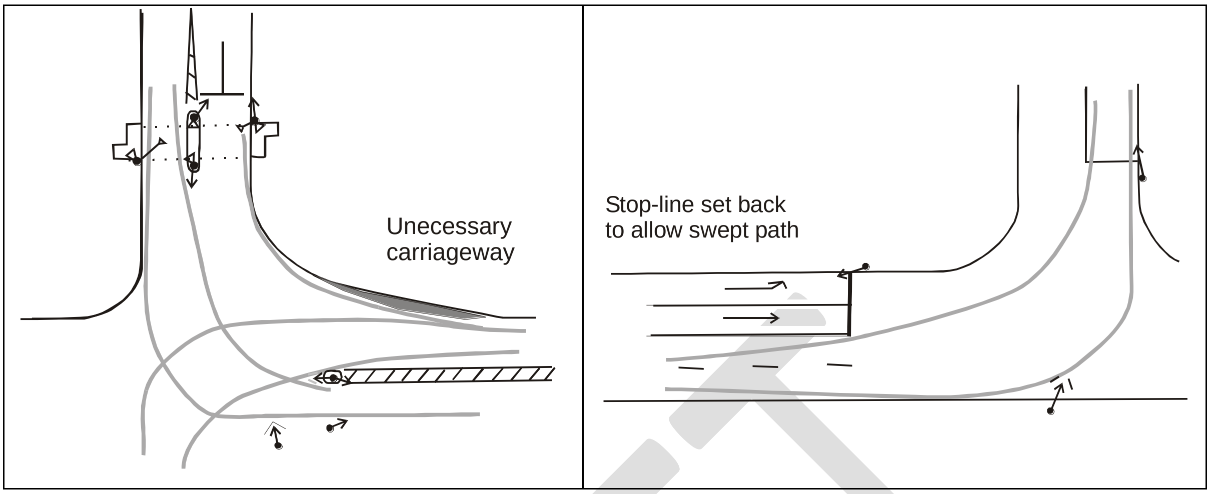


Figure 9-34 Examples of Swept Path Checks

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 and must be controlled if the super-elevation is over 2.5 %.

9.12.5 Signals

There should be at least two signals visible from each approach, usually comprising a primary and a secondary signal, and stop-lines (Figure 9-36) (see SATCC Road Traffic Signs Manual). 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 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 Figure 9-37.

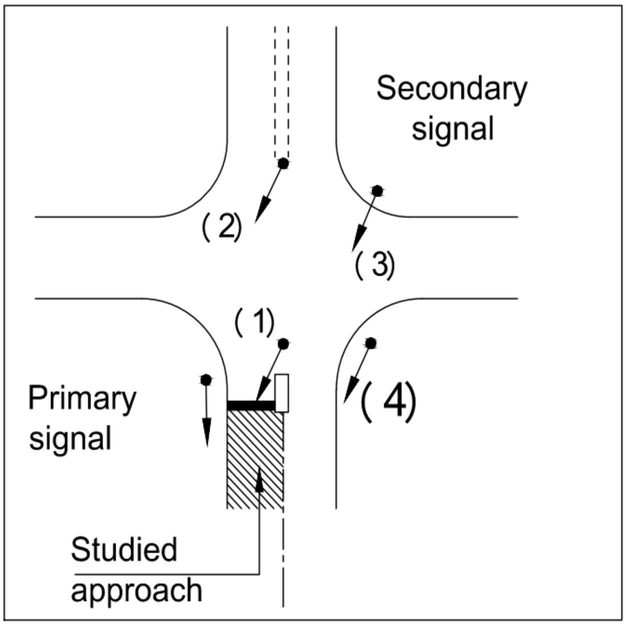
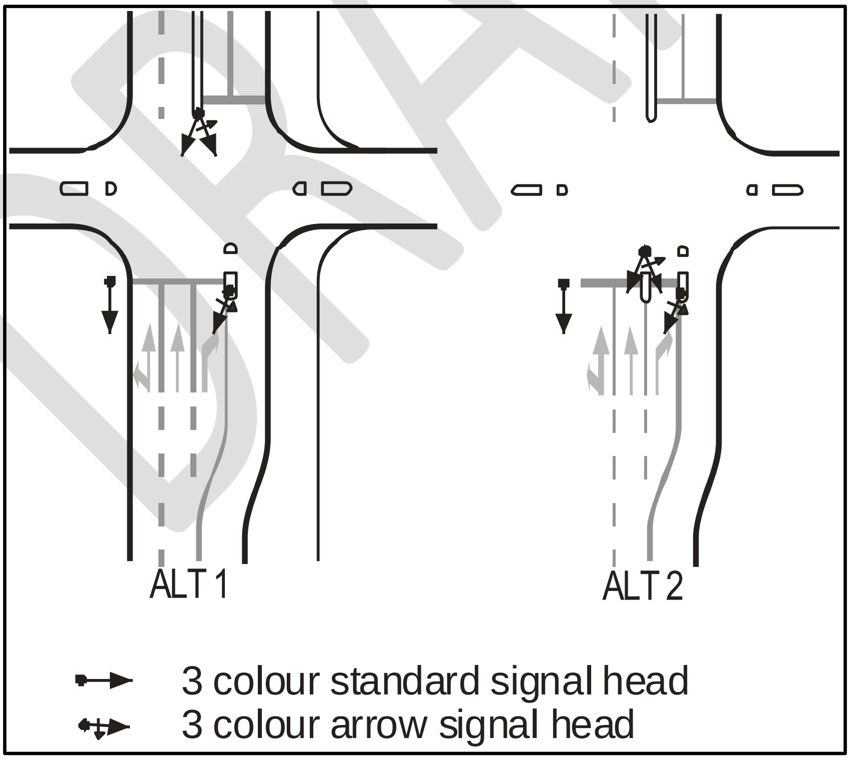


Figure 9-35 Signal Location

The following alternative designs may be used where there are approaches with three or more traffic lanes and protected right turns. The primary right turn arrow is mounted on the exit separation island, Figure 9-36a, or on an extra separation island in the approach, Figure 9-37b.



(a) **(b)**

Figure 9-36 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 or from 0.9 m to 1.65 m. Wider islands can be needed if they are also to serve as pedestrian refuges.

9.12.6 Pedestrian and Cyclist Facilities

Pedestrian crossings should be perpendicular to the edge of the carriageway to assist inter-visibility and to benefit visually impaired people. The footway should have a dropped kerb.

Minimum measures for pedestrian refuges for pedestrian crossings are timed to permit crossing in one movement. The normal width should be 2.5 m, with 1.5 m as the absolute minimum.

Pedestrian phases should preferably have no conflicts with turning traffic. This could be arranged with staggered pedestrian crossings as illustrated in Figure 9-37.

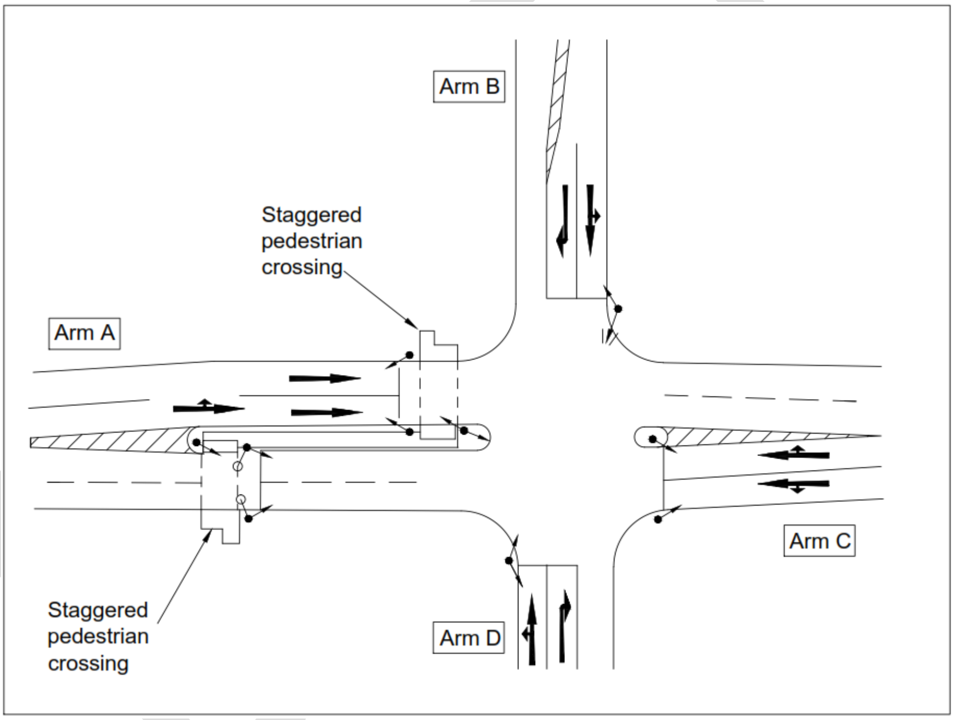


Figure 9-37 Signal-Controlled Intersection with a Staggered Pedestrian Crossing

9.13 Checklist for Intersection Design

The following is a checklist of factors that need to be considered in the design of intersections.

1) Will the intersection be able to carry the expected/future traffic levels without becoming overloaded and congested?
2) Have the traffic and safety performance of alternative intersection designs been considered?
3) Is the route through the intersection as simple and clear to all users as possible?
4) Is the presence of the intersection clearly evident at the decision sight distance to approaching vehicles from all directions?
5) Are warning and information signs placed sufficiently in advance of the intersection for a driver to take appropriate and safe action given the design speeds on the road?
6) Are warning and information signs visible and readable at the operational speed?
7) On the approach to the intersection, is the driver clearly aware of the actions necessary to negotiate the intersection safely?
8) Are turning movements segregated as required for the design standard?
9) Are drainage features sufficient to avoid the presence of standing water?
10) Is the level of lighting adequate for the intersection, location, pedestrians, and the design standard?
11) Are the warning signs and markings sufficient, particularly at night?
12) Have the needs of pedestrian and non-motorised vehicles been met?
13) Are sight lines sufficient and clear of obstructions including parked and stopped vehicles?
14) Are accesses prohibited a safe distance away from the intersection?
15) Have adequate facilities such as footpaths, refuges, and crossings, been provided for pedestrians?
16) Do the design, road marking and signing clearly identify the designated passageways and priorities?
17) Is the design of the intersection consistent with road types and adjacent intersections?
18) Are the turning lanes and tapers where required of sufficient length for speeds and storage?

10 Design of Roundabouts

10.1 Introduction

10.1.1 General

A roundabout is a one-way circulatory system around a central island, entry to which is controlled by markings and signs. Internationally roundabouts operate on the "Give-way-on-entry" rule so that, in Kenya and where vehicles drive on the left, vehicles give way to vehicles from the right. Thus, priority is given to traffic already on the roundabout. To help facilitate this, roundabouts operate by deflecting the vehicle paths to slow the traffic and promote yielding.

Roundabouts provide relatively high capacity and minimum delay. They also have a good safety record because traffic speeds are low and the number of potential traffic conflicts is greatly reduced, typically by 75%, and should crashes occur, they are not as severe as crashes on roads with traffic travelling at normal speeds. The key elements (Figure 10-1) of a roundabout are:

(i) Entries and exits.

(ii) Splitter islands

(iii) The circulatory roadway.

(iv) The central island.

(v) Sight distances.

The main types of roundabouts are normal (in size) roundabouts, compact mini, and signalised and grade separated.

10.1.2 Normal Roundabouts.

A 'normal' roundabout has a kerbed central island at least 4 m in diameter (Figure 10-1). Its approaches may be dual or single carriageway roads. Usually, a normal roundabout has flared entries and exits to allow two or three vehicles to enter or leave the roundabout on a given arm at the same time. If so, its circulatory roadway needs to be wide enough for two or three vehicles to travel alongside each other on the roundabout itself. If four or more arms are required, the roundabout becomes large and a signalised roundabout is usually required.

If there is sufficient space available on site, provision of a left turn slip lane is beneficial on approaches where a significant proportion of the traffic turns left. In some cases, the use of a left turn slip lane can avoid the need to build an additional entry lane.

10.1.3 Multilane Roundabouts

1) The number of circulating lanes from any particular approach must be equal to or greater than the maximum number of entry lanes on that approach.

2) The number of exit lanes must not be greater than the number of circulating lanes. On multi-lane roundabouts, the number of exit lanes is based on the lane usage as determined by the pavement arrows on the approaches.

10.1.4 Compact Roundabout

A compact roundabout has single-lane entries and exits so that only one vehicle can enter or leave it from a given arm at any one time (Figure 10-2). The width of the circulatory carriageway is such that it is not possible for two cars to pass one another. On roads with a speed limit of 65 km/h or less a compact roundabout may have low values of entry and exit radii in conjunction with high values of entry deflection and is particularly suitable where there is a need to accommodate the movement of pedestrians and cyclists. A central overrun is normally provided to allow longer vehicles with a wider swept path to navigate the compact roundabout safely.

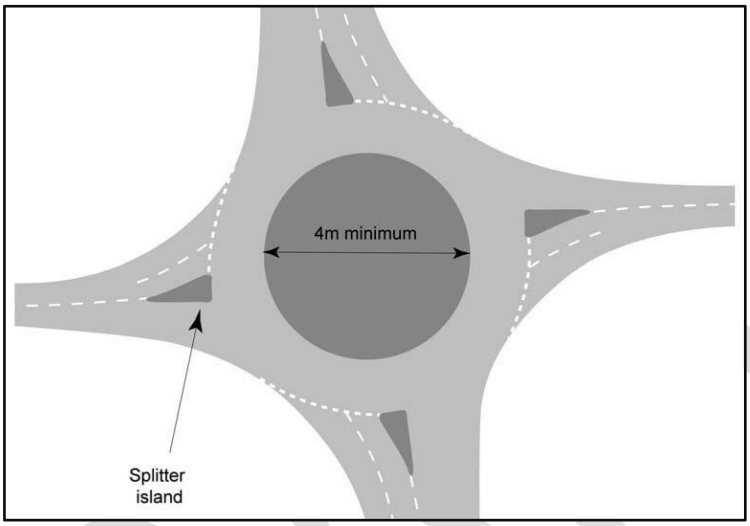


Figure 10-1: Basic Roundabout Showing Key Features

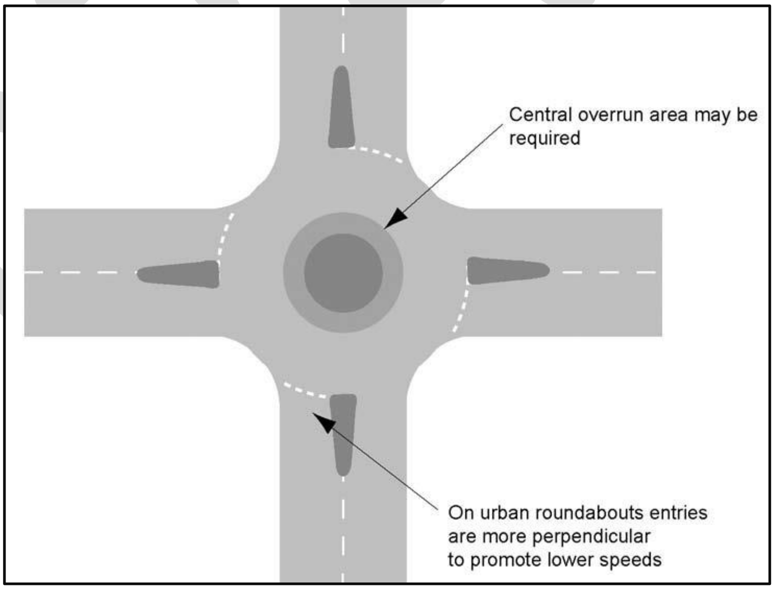


Figure 10-2: A Compact Roundabout

10.1.5 Mini Roundabout

A mini roundabout does not have a kerbed central island. The central island is a flush or domed circular marked area between 1 and 4 m in diameter capable of being driven over where unavoidable.

10.1.6 Signalised Roundabout

A signalised roundabout has traffic signals on one or more of the approaches and at the corresponding point on the circulatory carriageway itself. An example is illustrated in Figure 10-20.

10.2 When to use Roundabouts

Roundabouts should be considered when:

1) Intersection volumes do not exceed 3,000 vph at three-legged intersections or 4,000 vph at four-legged intersections.
2) Roundabouts should generally be used if the major road flow is less than 3 times the minor road flow.
3) The use of roundabouts may also be considered close to built-up areas where the through road may be crossed by local roads carrying high traffic volumes.
4) At intersections where traffic volumes on the intersecting roads are such that traffic signals would result in greater delays than a roundabout (in many situations roundabouts provide a similar capacity to signals, but may operate with lower delays and better safety, particularly in off-peak periods).
5) At intersections where there are high proportions of left-turning traffic. - unlike most other intersection treatments, roundabouts can operate efficiently with high volumes of left-turning vehicles.
6) As a traffic calming measure.

They may not be appropriate in the following circumstances:

1) Where traffic flows are unbalanced, with high flows on one or more approaches leading to serious delays to traffic on the major road;
2) Where there are substantial pedestrian flows;
3) Roundabouts in urban areas are not compatible with Urban Traffic Control (UTC) systems unless they are controlled by traffic signals and are part of the UTC itself. These systems move vehicles through their controlled areas in platoons by adjusting traffic signal times to suit the required progress. Roundabouts without signals can interfere with platoon movement;
4) In the presence of ‘reversible’ lanes;
5) Where semi-trailers and/or abnormal vehicles are a significant proportion of the total traffic passing through the intersection and where there is insufficient space to provide the required layout;
6) Where traffic congestion downstream (e.g., from a signalised intersection) causes a queue to back up through the roundabout; and
7) Two-lane roundabouts with more than four legs may cause operational problems and should be avoided.

Roundabouts generally take more land than fully channelised intersections. The additional land acquisition costs for roundabouts should be balanced with the increased capacity offered and lower maintenance cost.

Roundabouts are usually more difficult for pedestrians to cross than normal intersections hence arrangements should be made to provide adequate facilities.

10.3 Design Requirements

10.3.1 Safety and Speed Control

The design speeds of the roundabout and its approach roads are the critical standards in controlling safety. Entry deflection is the most important factor governing safety because it governs the speed of vehicles through the roundabout. Approaching vehicles must be slowed down to 50km/h or less. To help reduce speed, a deflection is forced on the vehicles of up to 30^{O}. All roundabouts must be designed with some entry deflection.

Entry width and sharpness of flare are the most important determinants of capacity.

Splitter islands should also be used to help achieve adequate speed reductions and to:

1) Allow drivers time to perceive the roundabout as soon as practicable.
2) Provide space for a comfortable deceleration distance.
3) Physically separate entering and exiting traffic.
4) Prevent deliberate and highly dangerous wrong-way driving.
5) Control entry and exit deflections.
6) Provide a refuge for pedestrians and cyclists.
7) Provide a place to mount traffic signs.

The sizes of splitter islands are dictated by the dimensions of the central island and inscribed circle. As a general guideline they should have an area of at least 10 m^2 to ensure their visibility to the oncoming drivers. The length of splitter islands should be equal to the comfortable deceleration distance from the design speed of the approach to that of the roundabout. Ideally, the nose of the splitter island should be offset to the right of the approach road centreline by about 0.6 m to 1 m.

Entry and exit deflection angles and a suitable central island radius should prevent speeds in excess of 50 km/h on the roundabout. This is accomplished by maximising the difference between the shortest path a driver could take through the roundabout versus the straight-line distance from an entry to the opposite exit. No vehicle path should allow a vehicle to traverse the roundabout at a radius greater than 100 m, which corresponds to the recommended design speed of 40 to 50 km/h (Figure 10-13).

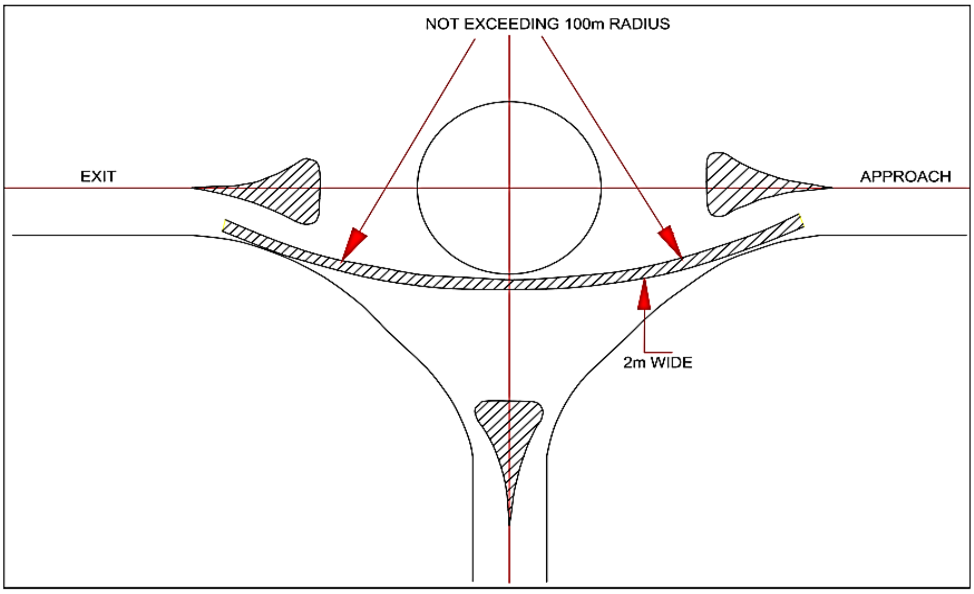


Figure 10-3: Vehicle Path through Roundabout

10.3.2 Key Requirements

The key requirements are:

1) The roundabout must be easily seen and identified when drivers are approaching it.
2) The layout must be simple and easy to understand; and it should be clearly signed and marked.
3) The global exit capacity must exceed the global entry capacity.
4) Speed patterns on entry, exit and through the roundabout should follow a safe and acceptable profile and be guaranteed by application of adequate deflection to the vehicles' movements.
5) Vehicles should be guided through the full length of the trajectory from upstream to downstream of the roundabout by applying adequate channelisation principles and solutions.
6) Adequate sight distances must be provided at the different positions of the vehicles throughout the full trajectories.
7) Where possible, lighting at night should be considered for safety.

10.3.3 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 60 degrees 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 centrelines 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.

10.3.4 Visibility and Sight Distances

Roundabouts should be located where approaching drivers will have a good overview of the roundabout with its entries, exits and circulating carriageway. Therefore, roundabouts should not be located on crest curves. Stopping sight distances must be provided at every point within the roundabout and on all approaches.

The visibility splays shown in Figure 10-4 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.

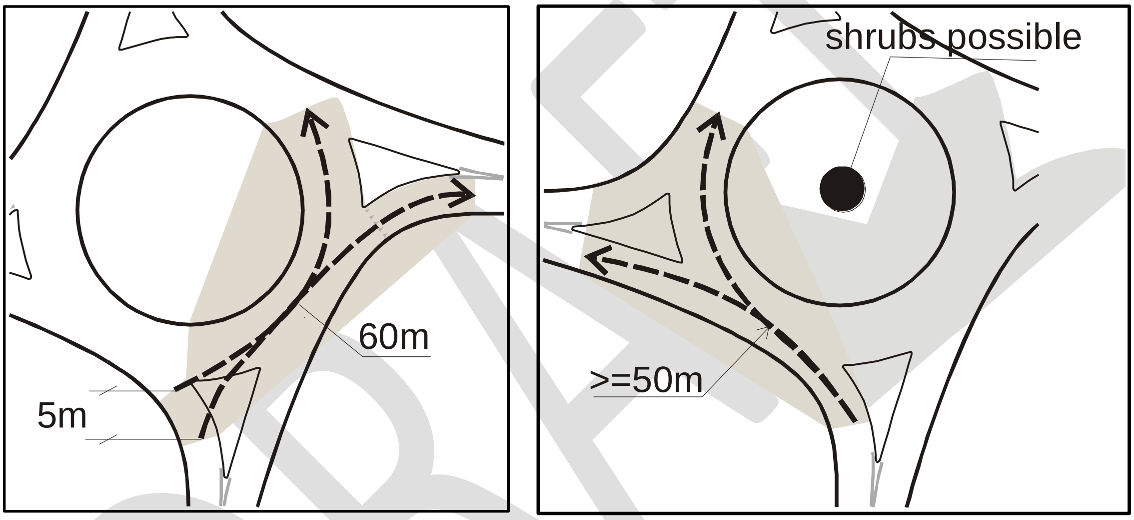


Figure 10-4: Required visibility towards approaching vehicles from the right and required visibility forwards to the left

Once within the roundabout, drivers must be able to see the area shown in Figure 10-5. Signs and landscaping on the centre island should be designed and located so that they do not obstruct the view more than is necessary, as illustrated.

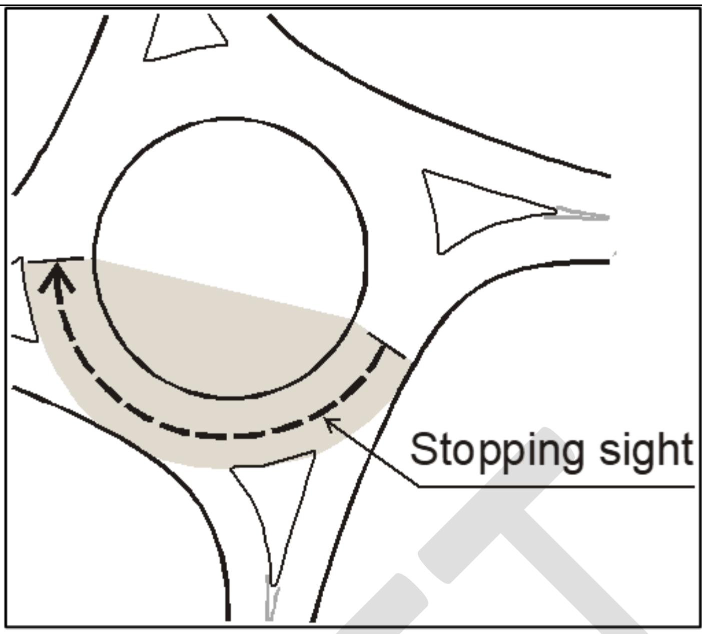


Figure 10-5: Required Visibility for Drivers within a Roundabout.

10.4 Dimensions of Roundabouts

The dimensions of roundabouts are defined by the radii and widths shown in Figure 10-6and Table 10-1

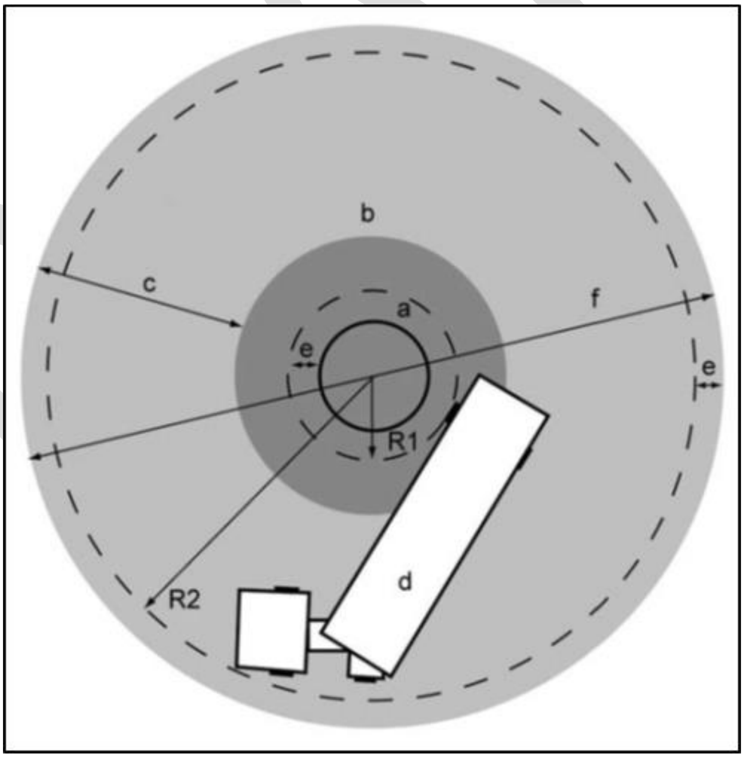


Figure 10-6: Nomenclature for Dimensions of Roundabouts Showing Design Vehicle.

The labels are defined as follows:

a *Main central island radius;*

b *Central overrun area, where provided;*

c *Remaining circulatory carriageway width = 1.0-1.2 x maximum entry;*

d *Vehicle;*

e *1 m clearance minimum;*

f *Inscribed Circle Diameter (ICD);*

R_1 *a + e.*

Table 10-1: Roundabouts Dimensions

Central Island Diameter (m)	R_1	R_2	Minimum ICD (m)
4	3	13.0	28
6	4	13.4	28.8
8	5	13.9	29.8
10	6	14.4	30.8
12	7	15.0	32
14	8	15.6	33.2
16	9	16.3	34.6
18	10	17.0	36.0

The inscribed circle diameter (f) of the roundabout is the diameter of the largest circle that can be fitted into the junction outline Figure 10-6). The inscribed circle diameter of a normal roundabout should not exceed 100m. Large inscribed circle diameters can lead to vehicles exceeding 50 km/h on the circulatory carriageway.

The minimum value of the inscribed circle diameter for a normal or compact roundabout is the smallest roundabout that can accommodate the swept path of the design vehicle.

If the inscribed circle diameter lies between 28m and 36m, a compact roundabout should be considered if the traffic flows can be accommodated.

The circulatory carriageway of normal or compact roundabouts should generally be circular and of constant width. The width of the circulatory carriageway must be between 1.0 and 1.2 times the maximum entry width.

A suitable design vehicle is an articulated vehicle with a single axle at the rear of the trailer, of length 15.5 metres. The turning space requirements of this vehicle on a roundabout with an inscribed circle diameter of between 28 m and 36 m are also shown in Figure 10-6. The turning requirements of such a vehicle are greater than those for all other vehicles within the normal maximum dimensions permitted. The requirements are less onerous for many other vehicles including an 11 m long rigid vehicle, 12m long coach, 15 m bus, 17.9m, 18.35 m drawbar-trailer combination, and a 16.5 m articulated vehicle.

A mountable area or apron may be added to the central island to accommodate occasional large heavy vehicles and to allow the circulatory width to be reduced to 9.5 m. The apron should have crossfall steeper than that of the circulatory road, principally to discourage passenger vehicles from driving on it, and a crossfall of 4 to 5% is recommended.

10.4.1 Normal Roundabouts

These are roundabouts where the radius at the edge of the carriageway is at least 18m and the central island radius is between 10m and 25m. 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.

10.4.2 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 an articulated vehicle and two-lane roundabouts are designed for an articulated vehicle and a passenger car. Figure 10-7 shows the minimum width of circulating carriageway after determining the design vehicle and the inscribed diameter (outer diameter).

At normal and grade-separated roundabouts, the width of the circulatory carriageway should not exceed 15 m. At compact roundabouts, it should not exceed 6 m, although an additional overrun area may be required for small values of inscribed circle diameter, depending on the types of vehicles using the roundabout.

For normal one-lane roundabouts (central island radius 10 m or greater) and two-lane roundabouts, the central island radius, the edge of carriageway radius and the width of the circulating carriageway are determined by the graphs in Figure 10-7 and Figure 10-8

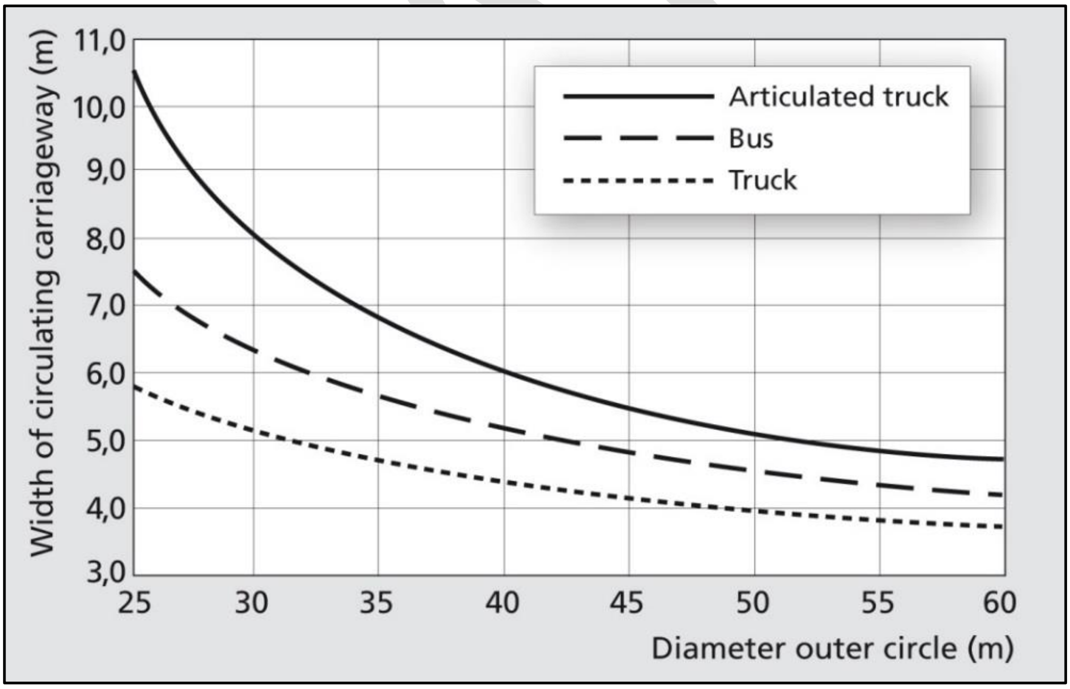


Figure 10-7: Minimum Width of Circulating Carriageway – One Lane

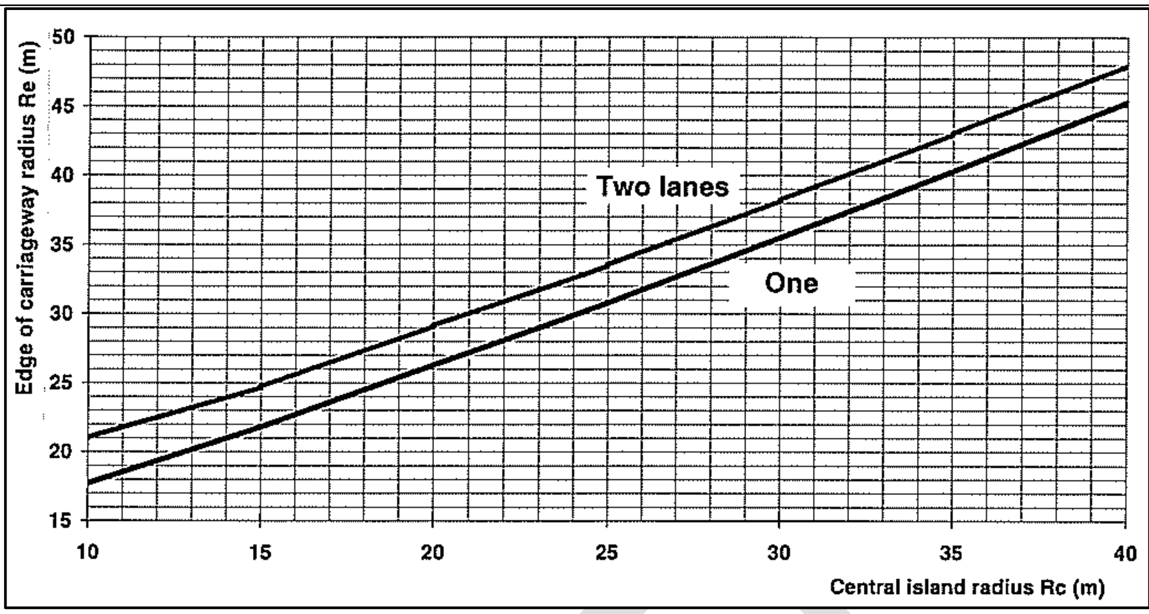


Figure 10-8: Radius of Central Island and Circulating Carriageway-Normal Roundabouts.

The circulating carriageway must be no more than about 1.2 times the maximum entry width. Very wide carriageways encourage unsafe speeds, but the circulatory roadway should be sufficiently wide to allow a stalled vehicle to be passed. The minimum roadway width for single-lane operation is therefore about 6.5 m between kerbs. Two-lane operation requires a roadway width of about 8.5 m. If trucks are present in the traffic stream in sufficient numbers, the circulatory road width should be increased by 3 m both in the single-lane and in the two-lane situation. A significant proportion of semi-trailers would require the width of the circulatory road to be increased even more to 13 m and 16 m in the single-lane and the two-lane situation respectively.

The width should be constant throughout the circle. Drivers tend to position their vehicles close to the outside kerbs on entering and exiting the roundabout but close to the central island between these two points. The vehicle path, being the path of a point at the centre of the vehicle, should thus have an adequate offset to the outside and inside kerbs. For a vehicle with an overall width of 2.6 m, the offset should be not less than 1.6 m, with 2.0 m being preferred.

A circulatory road width of 13 m makes it possible for passenger cars to traverse the roundabout on relatively large radius curves and at correspondingly high speeds. To avoid this possibility, the central island should be modified as discussed in the subsequent section.

The cross-slope on the roadway should be away from the central island and equal to the camber on the approaches to the intersection.

10.4.2.1 The Central Island

The central island consists of a raised non-traversable area (except in the case of mini roundabouts) that is usually circular. The island is often landscaped but the landscaping must not obscure the proper sight lines across the island as shown in Figure 10-4 and Figure 10-5.

It is important that a direct sight line across the island is not provided. Such a sight line is a distraction for drivers and is not required by them. The sight line across the central island must be obscured by the elevation of its core, or the central island when there is no apron. This elevation should be in the form of a cone (slant slope = 15%), planted with bushes, to cut the sight line across the island.

The central island should be circular and at least 4 metres in diameter. Mini roundabouts have central markings rather than kerbed islands with diameters of up to 4 m capable of being driven over where unavoidable.

The inscribed circle diameter, the width of the circulatory carriageway and the central island diameter are interdependent. Once any two of these are established, the remaining measurement is determined automatically.

10.4.3 Small Roundabouts (Compact and Mini)

These are roundabouts where the radius of the edge of carriageway is less than 18 m and should have an inner central island radius of at least 2 m. Where space is limited, such as in built-up areas, a slightly different design of roundabout is needed to accommodate long trucks without sacrificing speed controlling features**.**

The problem with small roundabouts is that it is difficult to control car speeds because the circulating carriageway must be very wide 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. 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. Guidance on the selection of central island radii and traversable area is given in Figure 10-9.

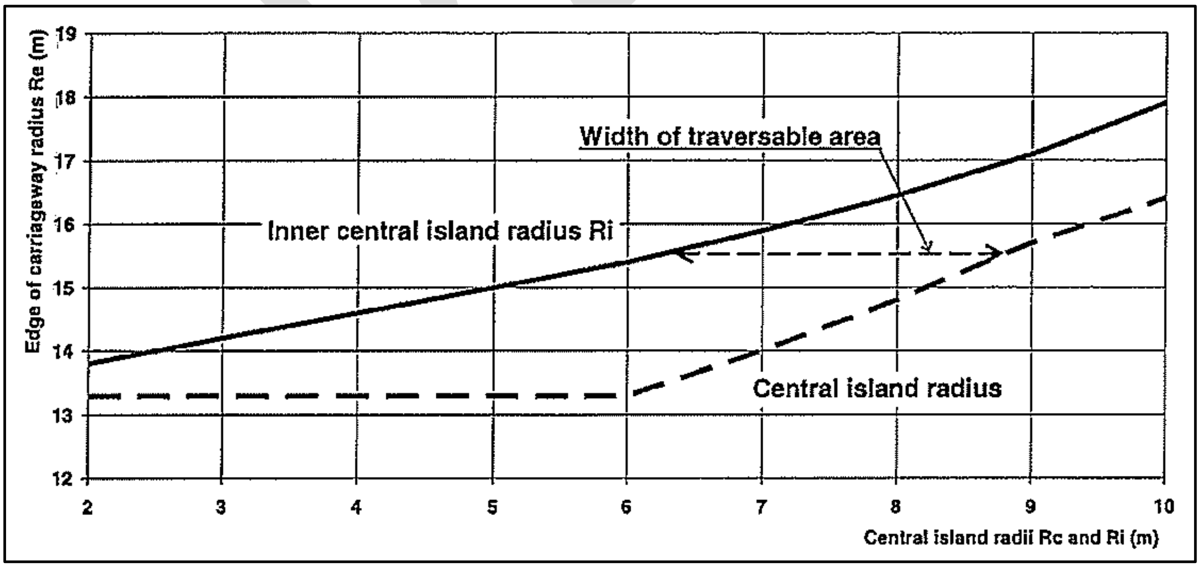


Figure 10-9: Roundabout Radii for Small Roundabouts

10.5 Entries

10.5.1 Number of Entry Lanes

One-lane is preferred from a safety viewpoint. For higher traffic volumes, a two-lane roundabout circulating roadway may be necessary. Guidance is provided in Figure 10-10.

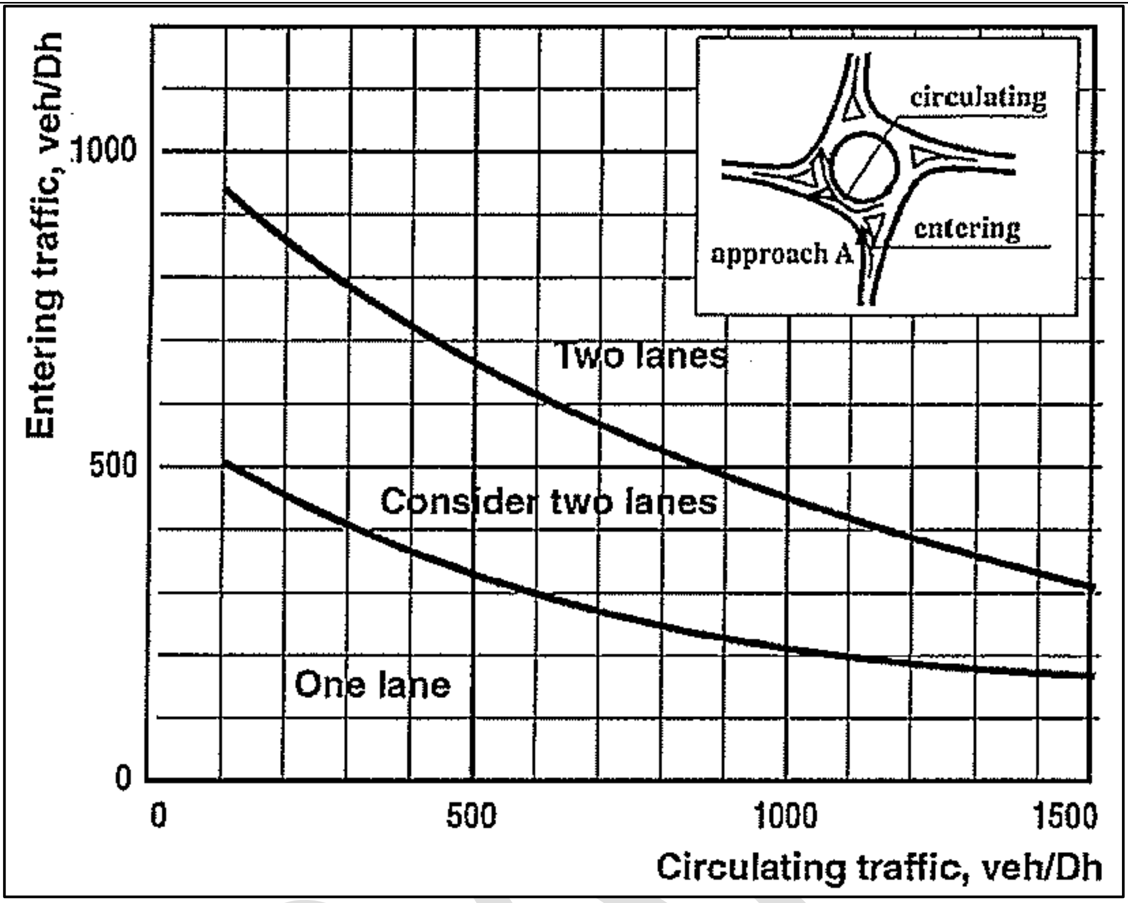


Figure 10-10: Number of Entry Lanes

The need for two lanes must be checked for each entry and circulating flows during the design hour. If two lanes are necessary for one entry, the whole roundabout should be designed with two lanes.

10.5.2 Approach Alignment

Entry deflection is essential 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 10-11 shows one way of achieving entry deflection. The designer should avoid making the deflection too sharp because 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. Typically, the centrelines of each of the approaches should intersect in the middle to form a polygon as shown in Figure 10-11

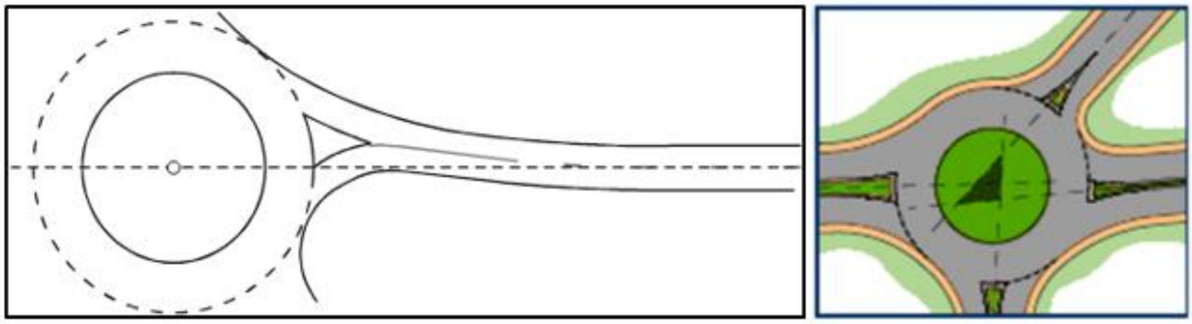


Figure 10-11: Design of Approach Deflection.

10.5.3 Splitter Islands

Splitter islands are used on each arm, located and shaped so as to separate and direct traffic entering and leaving the roundabout. They are usually kerbed, but if there is insufficient space to accommodate a kerbed island, they may consist entirely of markings. Markings may also be used to extend a splitter island on the approach, the exit or the circulatory carriageway.

Kerbed splitter islands can act as pedestrian refuges provided that they are large enough to give adequate safe standing space for accompanied wheelchair users and pedestrians with pushchairs or pedal cycles. Signs and other street furniture can be sited on kerbed islands if there is sufficient room to maintain the required clearances.

10.5.4 Entry Design

Several variables need to be considered in selecting an entry design which is safe and has adequate capacity. These variables are:

1) approach half width;
2) entry width;
3) entry flaring;
4) entry angle;

10.5.4.1 Approach Half Width

The approach half width, ‘v’ in Figure 10-12) is the width of the approach carriageway, excluding any hatching, in advance of any entry flare. It is the shortest distance between the median line, or the edge of the central reserve on dual carriageway roads, and the nearside edge of the road. Where there is white edge lining or hatching, the measurement should be taken between markings rather than kerb to kerb.

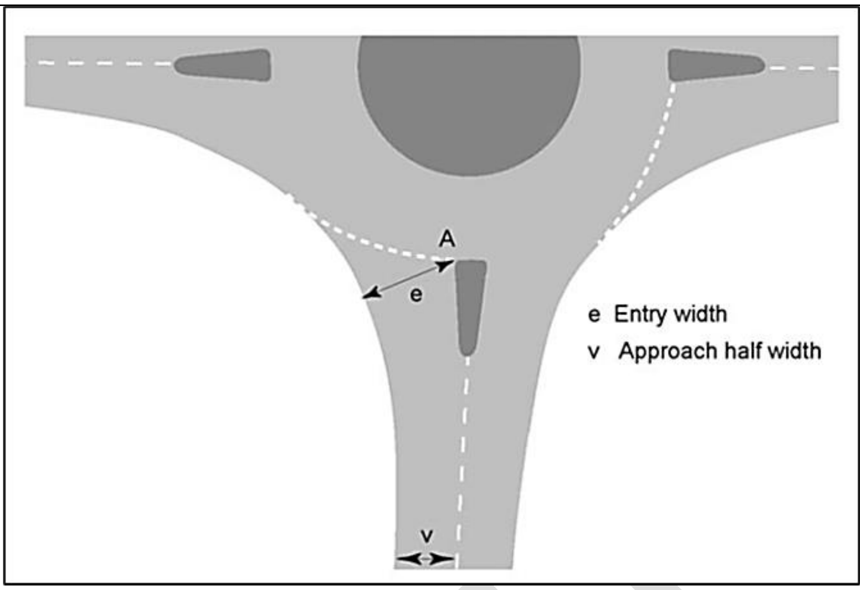


Figure 10-12: Approach Half Width and Entry Width

10.5.4.2 Entry Radius and Width

The entry width, ‘e’, is the width of the carriageway at the point of entry. It is measured from the point A at the right-hand end of the give way line along the normal to the nearside kerb (Figure 10-12). For capacity assessment, the measurement should be taken as the total width of the lanes which drivers are likely to use i.e., the effective width, which is normally between any white edge lining or hatching.

The entry width is one of the most important factors in increasing the capacity of the roundabout and can be increased above the width of the approach by flaring, i.e., by providing a passive taper with a taper rate of 1:12 to 1:15. If the approach volumes are high, the flaring could add a full lane to the left of the entry to increase capacity. As a rule, not more than two lanes should be added.

If space is scarce, especially likely in an urban environment, this could be reduced to a minimum of 6 m.

Lane widths at the give way line (measured along the normal to the nearside kerb, as for entry width) must be not less than 3m or more than 4.5 m, with the 4.5 m value appropriate at single lane entries and values of 3 to 3.5 m appropriate at multilane entries.

On a single carriageway approach to a Normal Roundabout, the entry width must not exceed 10.5 m. On a dual carriageway approach to a Normal Roundabout, the entry width must not exceed 15 m.

If flaring is provided, tapered lanes should have a minimum width of 2.5m.

On a single-carriageway road, where predicted flows are low and increased lane width is not operationally necessary, a Compact Roundabout with single lane entries should be used. The entry may need to be closed to carry out any form of maintenance so the design of traffic management for maintenance should be discussed at an early stage in the design process.

The development of entry lanes must take account of the anticipated turning proportions and possible lane bias, since drivers often tend to use the nearside lane. The use of lane bifurcation where one lane widens into two should maximise use of the entry width.

For highway improvement schemes on trunk roads, it is usual to consider design year flows sometime after opening. This can result in roundabout entries with too many lanes for initial flows, subsequently leading to operational problems. A layout based on projected flows will determine the eventual land requirements for the roundabout, but for the early years it may be necessary for the designer to consider an interim stage. This approach can result in reduced entry widths and entry lanes.

The entry widths in Table 10-2 should normally be used for one and two-lane roundabouts respectively. The transition to normal lane width should be at least 30 metres long.

Table 10-2: 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

10.5.4.3 Alignment of Entry Lanes

The alignment of entry lanes is critical. Except at compact roundabouts in urban areas, the kerb-line of the splitter island (or central reserve in the case of a dual carriageway) should lie on an arc which, when projected forward, meets the central island tangentially (see Figure 10-13 to reduce the likelihood of vehicle paths overlapping.

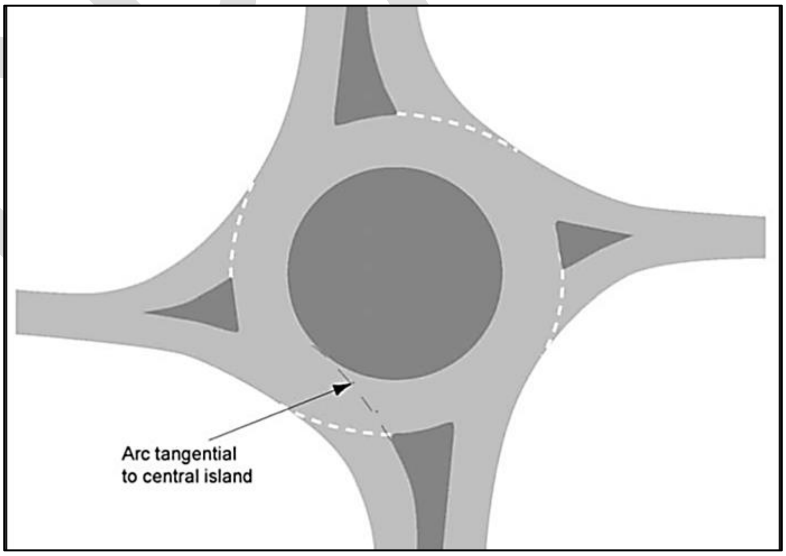


Figure 10-13: Example Showing an Arc Projected Forwards from the Splitter Island and Tangential to the Central Island.

10.5.4.4 Design of Multilane Entries

On multilane entries, it is important to ensure that entries are used equally. On flared entries, the queue from an overused lane may back up and block access to other lanes.

10.5.4.5 Entry Flaring

Entry flaring is localised widening at the point of entry. Normal roundabouts usually have flared entries with the addition of one or two lanes at the give way line to increase capacity. Single lane entries e.g., those at compact roundabouts, should be slightly flared to accommodate large goods vehicles. Even a small increase in entry width may increase capacity.

The effective flare length, ‘l’, is the length over which the entry widens. It is the length of the curve CF', shown in Figure 10-14.

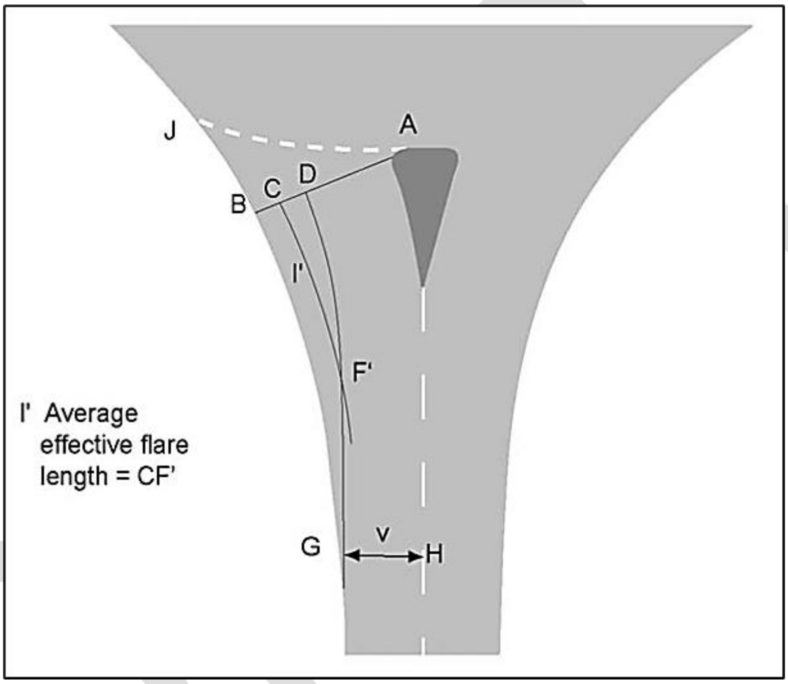


Figure 10-14: Average Effective Flare Length

Notes:

a. AB = entry width.

b. GH = v (approach half width at point G which is the best estimate of the start of the flare).

c. GD is parallel to AH and distance v from AH (v is measured along a line perpendicular to both AH and GD.

d. CF' is parallel to BG and distance ½ BD from the kerb line BG.

The total length of the entry widening (BG) will be about twice the average effective flare length.

The capacity of an entry can be improved by increasing the average effective flare length. However, effective flare lengths greater than 25 m may improve the geometric layout but have little effect in increasing capacity. A minimum length of about 5m in urban areas and 25 m in rural areas is desirable.

The entry width and the flare length are related. The capacity of a wide entry combined with a short flare can be similar to that of a narrow entry combined with a long flare.

10.5.4.6 Entry Path Radius

The entry path radius is a measure of the deflection to the left imposed on vehicles entering a roundabout. The smallest radius of this path occurs on entry as it bends to the left before joining the circulatory carriageway (Figure 10-15). It is the most important determinant of safety at roundabouts because it governs the speed of vehicles through the junction and whether drivers are likely to give way to circulating vehicles.

The entry path radius must not exceed 70 m at Compact Roundabouts in urban areas (where the speed limit and the design speed within 100 m of the give way line on any approach do not exceed 70 kph. At all other roundabout types, the entry path radius must not exceed 100 m.

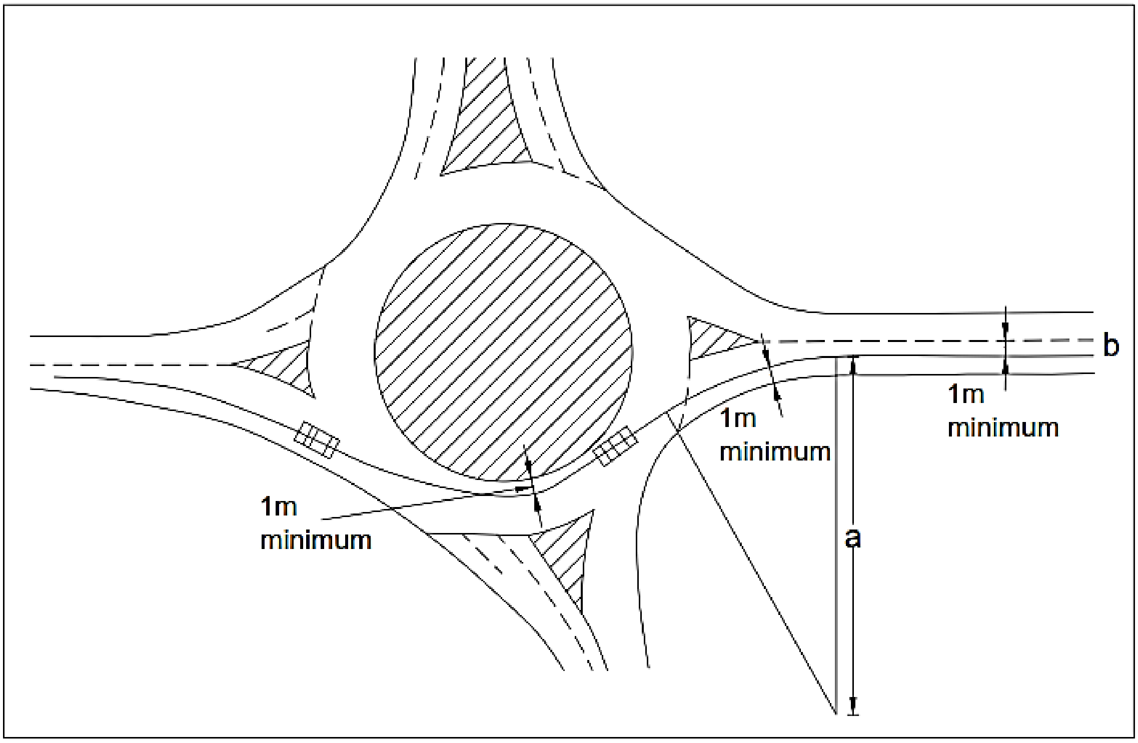


Figure 10-15: Determination of Entry Path Radius for Ahead Movement at a 4-arm Roundabout

Notes

a. Entry path radius should be measured over the smallest best fit circular curve over a distance of 25m occurring along the approach entry path in the vicinity of the give way line, but not more than 50m in advance of it.

b. Commencement point 50m from the give way line and at least 1m from the nearside kerb or centre line (or edge of central reserve).

10.6 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, the designer must check that all possible 2 m wide driving paths for passenger cars fulfil the requirement R1≤ R2≤ R3≤ 100 m to achieve speed control.

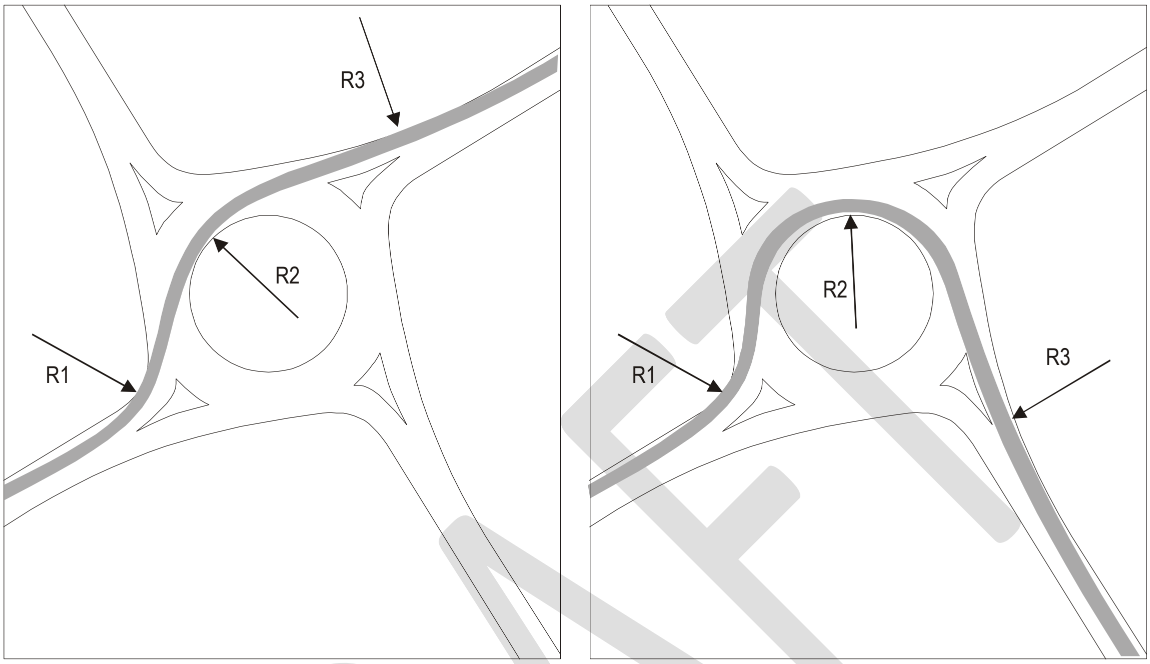


Figure 10-16: Driving Paths for Passenger Cars

It is preferable to avoid reverse curvature between the entry and the following exit (Figure 10-17). 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.

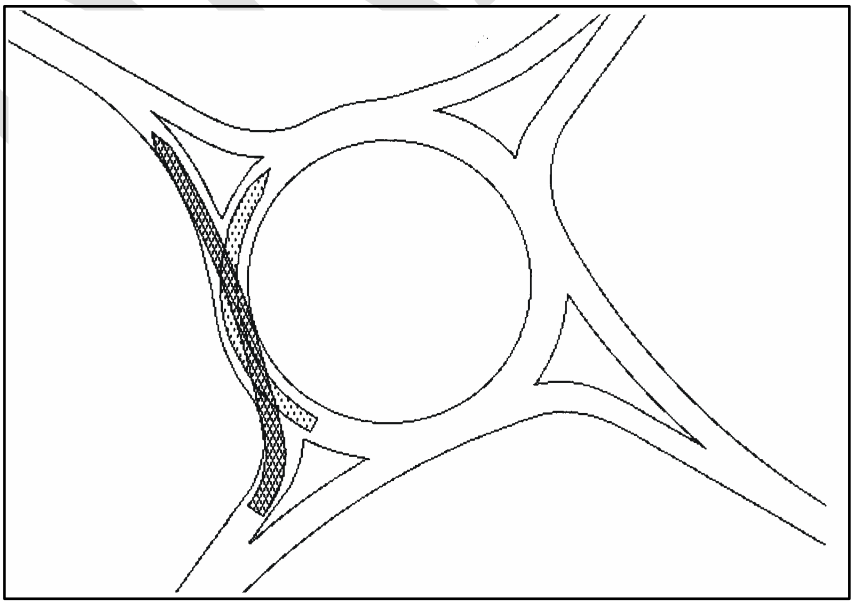


Figure 10-17 Alignment between entry and exit

A method for creating entry deflection at a Normal Roundabout is to stagger the arms as shown in Figure 10-18. This will:

1) reduce the size of the roundabout;
2) minimise land acquisition;
3) help to provide a clear exit route with sufficient width to avoid conflicts.

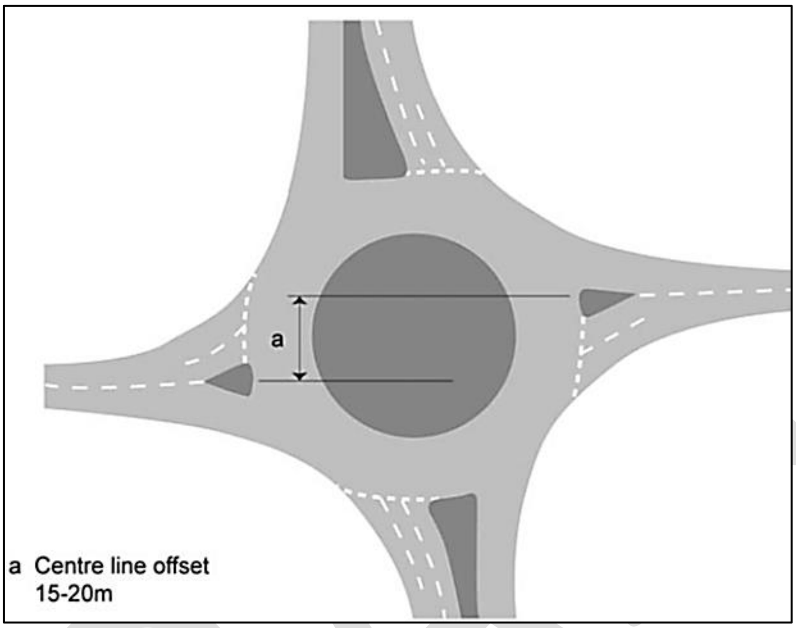


Figure 10-18: Staggering of East-West Arms to Increase Deflection

10.7 Exit Width

The exit width is the width of the carriageway on the exit. It is the distance between the nearside kerb and the exit median (or the edge of any splitter island or central reserve) where it intersects with the outer edge of the circulatory carriageway. It is typically similar to or slightly less than entry widths (exits have less flaring). Except for compact roundabouts, the exit width should, where possible, accommodate one more traffic lane than is present on the link downstream.

For example, at a normal roundabout, if the downstream link is a single carriageway road with a long splitter island, the exit width should be between 7m and 7.5m and the exit should taper down to a minimum of 6m (Figure 10-19), allowing traffic to pass a broken down vehicle. If the link is an all-purpose two-lane dual carriageway, the exit width should be between 10m and 11m and the exit should taper down to two lanes wide.

The width should be reduced in such a way as to avoid exiting vehicles encroaching onto the opposing lane at the end of the splitter island. Normally the width would reduce at a taper of 1:15 to 1:20.

Where the exit is on an up gradient, the exit width may be maintained for a short distance before tapering in. This helps reduce intermittent congestion caused by slowly accelerating large goods vehicles by giving other drivers an opportunity to overtake them. If the exit road is on an up gradient combined with an alignment which bends to the left, it may be necessary to maintain the exit width over a longer distance to help ensure that overtaking manoeuvres can be completed before the merge is encountered.

At a compact roundabout, the exit width should be similar to the entry width.

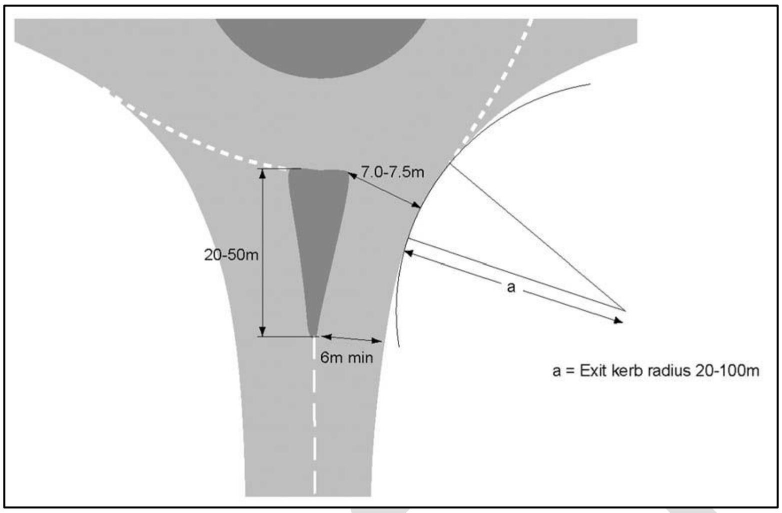


Figure 10-19: Single Carriageway Exit with a Long Splitter Island

10.8 Signalised Roundabouts

For large flows and therefore large roundabouts, traffic control will be required. Figure 10-20 is an example of such a roundabout. For details about the design of such roundabouts the reader is referred to CD116[2] (Design Manual for Roads and Bridges) published by the UKs Highways England or any similar publication.

10.9 Capacity of Roundabouts

The capacity of roundabouts has been the subject of much study and has led to comparatively simple relationships which have proved remarkably robust. Of the significant variables, three are of particular importance namely entry width, approach width and flare length. The remaining geometries have lesser effects.

Typical values are an entry capacity of 2000 pcu/h when the circulating flow is 1000 pcu/h and no pedestrian facilities, decreasing to 1700 pcu/h when pedestrian crossing facilities are required.

More detailed information is available from CD116 (Design Manual for Roads and Bridges) published by the UKs Highways England.

[2] CD116: https://www.standardsforhighways.co.uk/dmrb/search/2b5901c6-3477-4826-b780-cf99003fb5e0

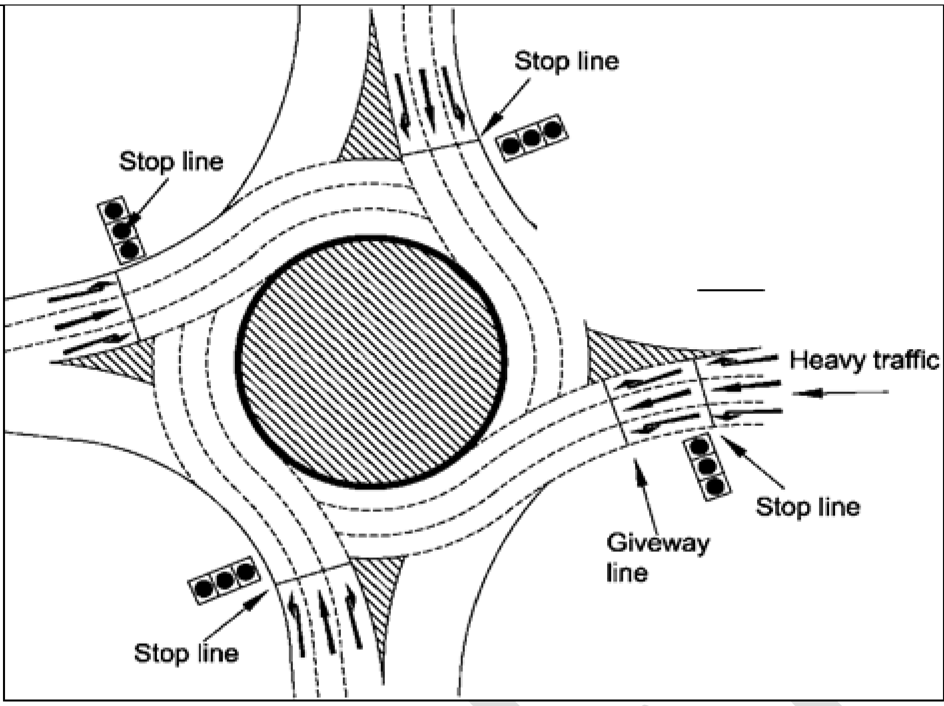


Figure 10-20: Example of a Signalised Roundabout

10.10 Pedestrian and Cycle Crossings

Pedestrian/cycle crossings are normally placed according to one of the two alternatives shown in Figure 10-21. They should never be located within the roundabout.

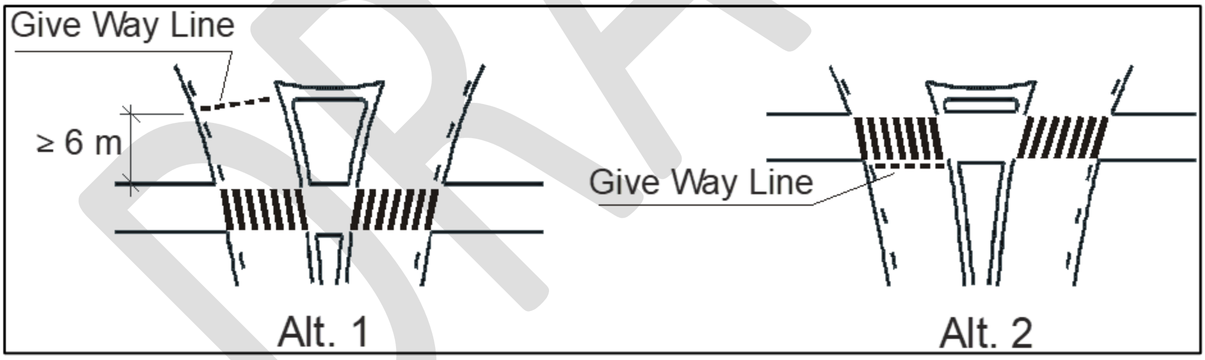


Figure 10-21: Location of pedestrian crossings

In alternative 1 the give way line is placed after the pedestrian crossing. In alternative 2 it is placed before the pedestrian crossing.

With a sufficient distance between the crossing and the 'give way' line (alternative 1), vehicles can yield separately for the pedestrian crossing and the roundabout. This improves capacity but safety might be compromised. An existing vehicle can also 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 an extra detour.

10.11 Detailed Design Procedure

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 outer 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. 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.

11 Design of Grade-separated Interchanges

11.1 General

Grade-separated interchanges are divided into two functional classes, referred to as ‘access’ interchanges, minor interchanges or merely grade separated interchanges), and ‘systems’ (or major) grade separated interchanges.

Systems/major interchanges are the nodes of the main network itself, linking roads of functional classes A and B (i.e. those that are subject to full access control) plus some roads of classes C and D with only partial access control. Minor, or simply ‘access’ interchanges link individual highways with no access control into a cohesive linked whole. These two fundamentally different applications require different types of interchange layout.

The fundamental difference between a freeway and any other road is that it is subject to rigid control of access. Entrances and exits to and from a freeway may take place only at specified points, typically remote from each other, and then only at very flat angles of merging and diverging. A freeway is characterised by the fact that all junctions along its length are systems interchanges.

11.2 Scope

This chapter illustrates and describes the most common and popular designs of major grade-separated interchanges between restricted access freeways (motorways) although, by their very nature, most designs are likely to be unique in some way, however small. More comprehensive design guidelines for this topic are contained in the *Geometric Design Guidelines* published by the South African National Roads Authority Limited, Pretoria, RSA, and the *Code of Practice for the Geometric Design of Trunk Roads* published by the Southern African Transport and Communications Commission (SATCC). More detailed design information is also available in the *Advice Notes* of the UKs Highways Agency listed in the References that are freely available on the internet.

Systems interchanges have ramps with free-flowing terminals at both ends. The volume of turning movements is high so there is a need for high design speeds on the ramps. They provide uninterrupted movement for vehicles moving from one main route to another using connector roads with a succession of diverging and merging manoeuvres. All turning movements are separated, and, ideally, weaving in the interchanges is reduced to a minimum. The layout of these interchanges involves a substantial area and possibly more than one structure constructed on two or more levels.

The most efficient form of grade separation is that which presents drivers with the minimum number of clear unambiguous decision points as they drive through the interchange and in merging and diverging. Additionally, on a motorway or an all-purpose road that is generally grade separated, consistency of design for successive interchanges is an important consideration involving the adoption of the same design speed. This need for consistency also applies to signing and road markings.

The circumstances in which the use of a grade-separated interchange is warranted are usually as follows:

1) Where roads cross or connect to motorways.

2) Insufficient capacity of an at-grade junction. An interchange is then justified economically from the savings in traffic delays and accident costs.
3) Grade separation is cheaper on account of topography or on the grounds that expensive land appropriation can be avoided by its construction.
4) Reduction in accident rates.

11.3 Safety Considerations

Some at-grade interchanges on heavily travelled urban interchanges exhibit high crash rates that cannot be lowered by improvements to the geometry or the use of control devices. Crash rates also tend to be high at interchanges on heavily travelled rural arterials where there is a proliferation of ribbon development. A third area of high crash rates is at interchanges on lightly travelled low volume rural locations where speeds tend to be high. In these cases, low-cost interchanges may be a suitable solution.

Closely spaced successive off-ramps are often a source of confusion to the driver leading to erratic responses and manoeuvres. Thus, an interchange should have only a single exit for each direction of flow and exits should be located in advance of the interchange structure. Directing traffic to alternative destinations on either side of the motorway should then take place clear of the motorway itself. Thus, drivers are required to make two separate decisions. First, to leave the motorway or not and, if not, to decide which route to take for their next destination. This spreads the workload and simplifies the decision process, hence improving the operational efficiency and safety of the entire facility.

Single entrances are also preferred. Merging manoeuvres by vehicles entering the motorway are a perturbation to the free flow of traffic in the left lane. Closely spaced entrances exacerbate the problem and could influence the adjacent lanes as well.

There are several advantages in carrying the minor crossing road over the motorway rather than under it.

1) Exit ramps on up-grades assist deceleration and the corresponding entrance ramps on downgrades assist acceleration.
2) Rising exit ramps are highly visible to drivers and provide advanced warning of the interchange ahead requiring an early decision from the driver whether to stay on the motorway or to depart from it.
3) Placing the motorway into cut reduces noise levels to surrounding communities and reduces visual intrusion.

11.4 Types of Interchange

Each major type is introduced in Table 11-1 with reference, where appropriate, to the basic line diagram layouts shown in Figure 11-1. For additional designs the reader should consult (the *Highway Capacity Manual,* (Transportation Research Board*)*.

Grade-separated interchanges generally fall into four categories depending upon the number of roads involved and their relative importance. These categories are as follows:

1) Three-way junctions.

2) Junctions of major/minor roads.
3) Junctions of two major roads.
4) Junctions of more than two major roads.

Table 11-1: Characteristics of some common Grade-separated Interchanges

Type of interchange	Basic Properties	Considerations
A and B	Grade separation of only one traffic stream	This configuration is appropriate for traffic volumes of up to 30,000 AADT on the four-lane major road (3,000 vehicles per hour). With a single loop lane, it is appropriate for loop traffic of 1,000 vehicles per hour.
C	The simplest for major/ minor road junctions that transfer the major traffic conflicts to the minor road.	Layout C shows the ‘half clover leaf’ type of junction which has the advantage of being easily adapted to meet difficult site conditions.
D		Layout D shows the normal ‘diamond’ junction which requires the least land appropriation. The choice between these options is generally dependent on land requirements.
E	Layouts E and F show the two basic layouts for use where high traffic flows make the simpler layouts unsatisfactory. They are appropriate for traffic volumes on both crossing roads of between 10,000 and 30,000 AADT (3,000 vehicles per hour).	Layout E shows a ‘full clover leaf’ junction involving only one bridge but requiring a large land appropriation.
F	Layout F shows a typical roundabout interchange. It is only suitable if the secondary road containing the roundabout is of a relatively low design speed but carries a comparatively high volume of traffic.	Layout F shows a typical roundabout interchange involving two bridges.

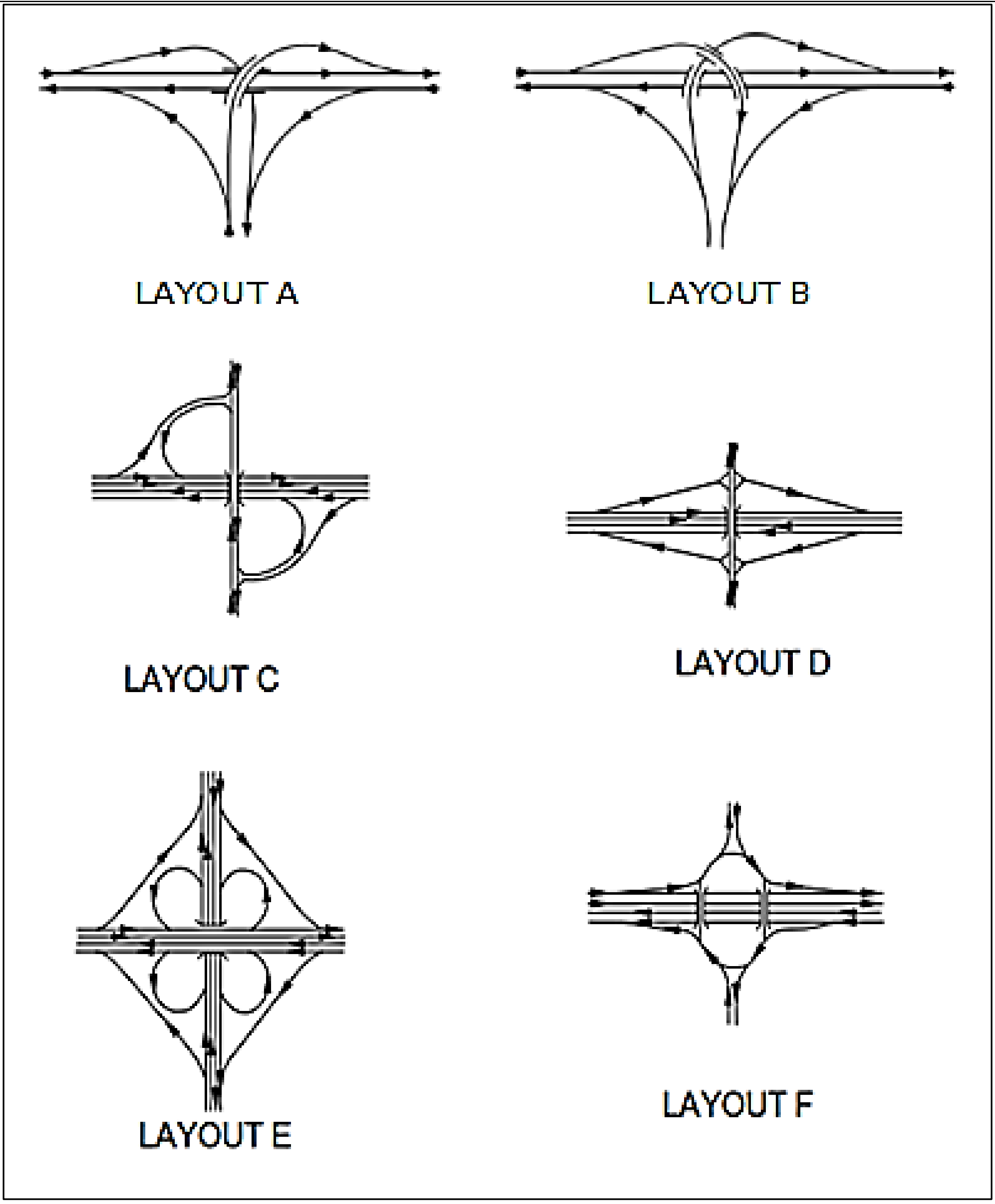


Figure 11-1: Typical Layouts for Grade-Separated Interchanges.

11.4.1 Three-way Interchanges (Layouts A and B).

The Y-Interchange (also called a Trumpet Interchange) is a three-legged interchange where one motorway terminates at its interchange with another motorway. It is therefore the interchange equivalent of a T-junction.

There are two possible layouts for the interchange, both of which make provision for direct ramps for all but one of the movements (Figure 11-1). Type A has the loop ramp before the structure. The alternative (Type B) has the loop ramp after the structure. A traffic study will show which flows are the highest and the local topography and existing or planned developments around the site will all influence the final layout.

The Y-interchange is also suitable when the intersecting roads are on a skew angle (i.e. not 90^o).

The Y-interchange may also be a part of a phased construction, for example, if the motorway which currently ends at the interchange, is planned to continue at some time in the future. In such a case the alignment of the motorway should be in the final position, or as near to it as possible, to minimise future construction work, and the bridges should be built in their final position, so that the future extension can make use of them.

11.4.2 Junctions of Major/Minor Roads (Layouts C and D)

Layouts C and D are the simplest for major/minor road junctions and both transfer the major traffic conflicts to the minor road. Layout C shows the 'half clover leaf' type of interchange. Layout D shows the normal 'diamond' interchange.

Approaching the interchange from either direction, an off-ramp diverges only slightly from the major road and runs directly across the minor road becoming an on-ramp that returns to the major road in a similar way. The two places where the ramps meet the minor road are treated as conventional priority interchanges with stop signs or traffic lights. This form of interchange is very common, particularly in rural areas.

The diamond interchange uses less space than most types of motorway interchange and avoids the interweaving traffic flows that occur in interchanges such as the cloverleaf. Thus, diamond interchanges are most effective in areas where traffic is light, and a more expensive interchange type is not needed. But where traffic volumes are higher additional traffic control measures such as traffic lights and extra lanes dedicated to turning traffic are required.

The ramp intersections with the minor road can be configured as a pair of roundabouts. This is the 'dumbbell' layout shown in Figure 11-2. The advantages are that it can be adapted to fit either a diamond or half cloverleaf; it has increased junction capacity and reduced land take compared with the diamond.

Roundabouts can generally handle traffic with fewer approach lanes than other intersection types therefore this configuration allows other roads to form approach legs to the roundabouts and allows easy U-turns.

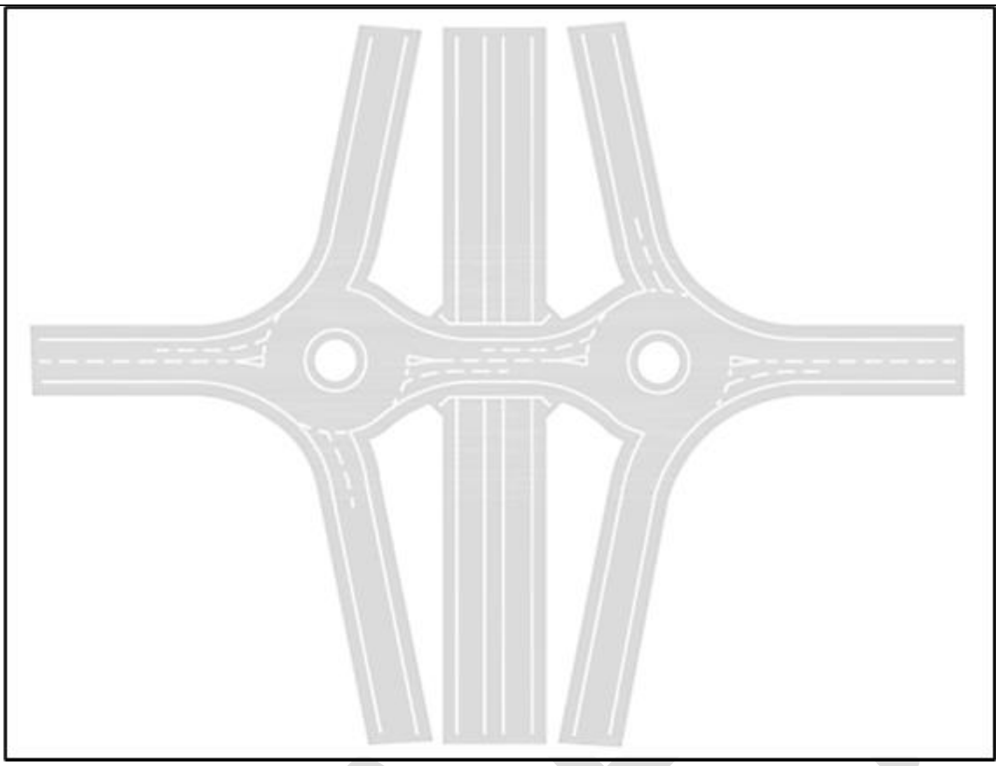


Figure 11-2: Dumb-bell Layout (One Bridge, Two Roundabouts)

11.4.3 Interchange between Two Major Roads

Layouts E and F show the two basic junction layouts for use where high traffic flows make the simpler layouts unsatisfactory. They are appropriate for traffic volumes on both crossing roads of between 10,000 and 30,000 AADT (3,000 vehicles per hour).

Layout E shows a 'full clover leaf' interchange involving only one bridge but requiring a large land appropriation. It is well suited for the intersection of two motorways especially in rural or suburban locations where space is available. The cloverleaf is characterised by having all the right–turning movements accommodated by loop ramps, i.e., 270 degrees change of direction. To maximise capacity all left turns are enabled before reaching the intersection by ramps (referred to as 'collector-distributer' roads) shown on the outside of each quadrant in the Figure 11-1 To facilitate the turning and weaving movements, auxiliary lanes should be provided on each carriageway thereby allowing unhindered traffic flow on the straight-through motorway lanes.

The right turn movements occur on the loop ramps, with a design speed of 30 to 50 km/h, giving radii of 30 - 100 m respectively. The additional travel distance around the loop varies from 200 - 500 m. The larger the radius of the loop ramp, the larger the radius of the left turn ramps must be as well; thus, the overall size of the interchange area also increases.

There are two (relatively minor) disadvantages of the cloverleaf. Firstly, the low radius, and consequent low design speed, of the loops, restricts them to being single lane thereby limiting their capacity. Secondly the requirement to turn left first to turn right is not intuitive to a driver. The advantage of the cloverleaf interchange is that it can handle large volumes of traffic and, unless the traffic on one ramp becomes very high, can serve most situations adequately.

Interchanges with loops in all four quadrants are referred to as full cloverleafs and all others are referred to as partial cloverleafs. A full cloverleaf may not be warranted at major-minor crossings where, with the provision of only two loops, freedom of movement for traffic on the major roadway can be maintained by confining the direct at-grade left turns to the minor roadway.

Layout F (Figure 11-1) shows an interchange involving two bridges. This layout is suitable if the secondary road containing the minor circulating roadway is of a relatively low design speed but carries a comparatively high volume of traffic. If high speeds on the circulating roadway occur, it can lead to problems for joining traffic hence the dimensions of the circulating roadway need to be selected to avoid this or traffic control can be used to alleviate this problem.

11.4.4 Roundabout Layouts

The 'dumb-bell' roundabout layout (Figure 11-2) has the advantage of reduced cost (only one bridge) and less land take than the two-bridge arrangement of Layout F. An alternative to layout F is to reduce the size of the interchange by using a roundabout as shown in Figure 11-3. The same considerations apply as for layout F and traffic control will be required.

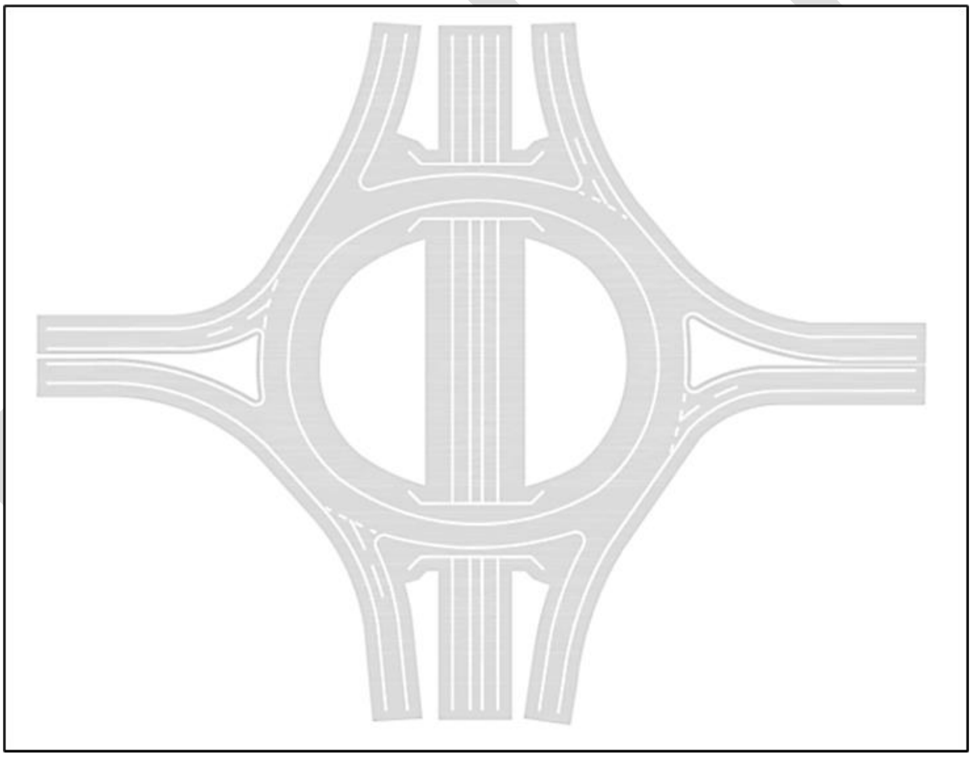


Figure 11-3: Use of a large roundabout

11.4.5 Junctions of more than Two Major Roads

Interchanges of more than two main roads are difficult to design; they occupy large areas of land; require numerous bridges; and are extremely expensive.

The need for this type of interchange can often be reduced by changes in the major road alignments (which will simplify the traffic pattern) to a combination of the simpler and more economic layouts described above.

11.5 Siting of Interchanges

The distance between two successive grade-separated junctions is an element of great importance in ensuring the desired level of service. Rural interchanges are typically spaced at distances of 8 km or more.

In an urban metropolitan environment closer spacing is often needed. In such an environment the spacing should provide sufficient distance for the weaving manoeuvres required between interchanges and, most importantly, for the sequence of signs required to inform drivers who are unfamiliar with the road of the location of exits to specific destinations. The recommended absolute minimum distance is 2.5 km.

11.6 Choice of Scheme

The following factors should be considered:

1) Predicted traffic volumes.
2) Cost.
3) Congestion control.
4) Trip lengths (travel distance).
5) Size of urban areas.

From a study of conflicting traffic movements, it should be apparent which traffic streams must be grade-separated, leaving the other streams to be dealt with by interchanges at grade. The choice of these will depend upon the capacities needed. A study of the characteristics of various types of grade-separated junctions is necessary, and several alternative designs should be prepared. The final choice of scheme must satisfy capacity requirements, geometric standards, and operational needs, and represent an economical design.

The exact layout and configuration of the ramps of the interchange depends on the interchange angles between the motorway and the intersecting road, the position of the interchange in the network and the layout of the total network. All movements may not necessarily be provided in all interchanges, however, the non-availability of certain movements in the interchange may lead to unwanted behaviour by drivers. If all movements are not provided from the outset, it is good practice to plan for the possible future inclusion of additional ramps and to acquire adequate road reserve.

In some cases, the choice of a particular design will be determined by the adoption of two-stage construction, i.e., constructing an at-grade junction first and providing grade separation later.

11.7 Interchanges on Non-motorway Roads

Major routes that warrant interchanges are usually motorways. A major route that is not a motorway is unlikely, but it might arise where traffic flows are so heavy that a signalised intersection cannot provide sufficient capacity, or it might be an intersection with a particularly poor accident history that requires upgrading. As a general rule, a simple and relatively low-cost interchange (grade-separated junction) should suffice (e.g., Figure 11-4) The crash history should provide some indication of the required type of interchange.

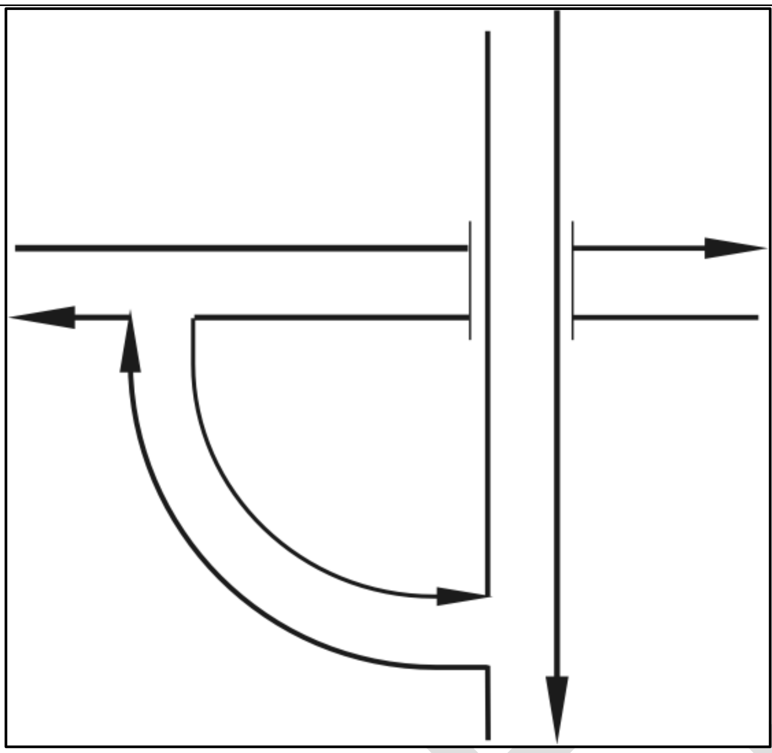


Figure 11-4: Jug Handle Interchange

11.8 Geometric Standards of Grade-separated Interchanges

11.8.1 Design Speed

The design speed for through traffic movements is determined in accordance with Chapter 4. Stopping sight distances appropriate for the design speed should always be provided.

The design speeds for loops and ramps depend on whether their terminations are free flowing or a stop junction. The term 'free flowing' implies that the ramp terminals can be negotiated at more or less the speed prevailing on the through road. Traffic on the terminals thus diverges from or merges with traffic on the through road at very flat angles.

For the ramps or loops of access-type interchanges, where the end of the exit loop terminates at a road junction, the design speed should, ideally, be 40-50 km/h. Higher design speeds require higher radii of curvature and longer loops and therefore have a significant cost implication. The design speed should not be so low that it is requires drivers who are leaving the motorway to reduce speed too quickly hence either compound curves are required suitable for an entry speed of 65% of the design speed of the motorway or a deceleration lane must be provided on the motorway.

If a high volume of turning (exiting) traffic is expected, free flowing terminals at each end of the loop or ramp will accommodate traffic entering and leaving at speeds close to the operating speeds of the through and intersecting roads. A lower design speed in the middle of the loop or ramp will have a restrictive effect on the capacity of the ramp and is therefore unacceptable.

Where a dual carriageway intersects with another dual carriageway (a major interchange), the interchange between the facilities must be designed so that the linking ramps do not entail any significant reduction in the design speeds of the crossing carriageways. That is, a sufficient deceleration to cause discomfort to vehicle occupants.

Deceleration and acceleration lanes must also be provided on the motorway.

11.8.2 Acceleration and Deceleration Lanes

The minimum standards to be applied for left turn deceleration lanes are the same as for at-grade junctions (Section 9.11.11 and 9.11.12). The total length of an acceleration lane (i.e., not including the merging taper) must never be less than 150 m or more than 400 m.

11.8.3 Horizontal Curves and Super-elevation

The geometric principles described in this manual apply equally to the ramps for interchanges. The maximum super-elevation for loops is 8% which, at a design speed of 50 km/h, leads to a minimum radius of 80 m. Where smaller radii are unavoidable, warning signs are necessary.

Where transitions occur from high to low speeds, the curves must be compound or transitional, the radius at any point being appropriate for the vehicle speed at that point.

11.8.4 Vertical Curves and Gradients

To ensure reasonable standards of visibility, comfort and appearance, vertical curves should be introduced at all changes in gradient. Vertical curve lengths should be determined in accordance with Chapter 6 to provide safe stopping sight distances.

11.8.5 Widths and Gradients of Ramps

If a stalled vehicle blocks an off-ramp, the line of stopped vehicles will soon extend back to the motorway creating a hazardous situation and will, therefore, also affect the quality of traffic flow on the motorway. The blocking of an on-ramp will lead to the blocking of the stop-condition terminal, impeding the flow of traffic along the crossing road. An overall ramp width of 8.0 m, comprising a shoulder of 2.0 m on the nearside and 1.5 m on the far side (widened by 0.5 m where a safety barrier is required) is adequate and allows for future conversion of the single lane into two narrower lanes.

For ramps on radii of 150 m or less, the minimum carriageway width must be in accordance with Table 11-2.

Table 11-2: Minimum Widths for Ramps

Radius (m)	25	30	40	50	75	100	150
Carriageway Width (m)	5.3	5.0	4.6	4.5	4.5	4.5	4.0

The maximum up gradient should be 5% and the maximum down gradient should be 7%.

11.8.6 10.8.6 Clearances

The required vertical and horizontal clearances must be in accordance with those described in this manual for principal roads.

11.8.7 Capacity

Grade-separated junctions are generally designed using traffic volumes given in terms of the Daily High Volume (DHV) rather than Annual Average Daily Traffic (AADTs). A detailed traffic study and analysis can be made to determine these values. In the absence of such a study, it can be assumed that the DHV in an urban area is 10% of AADT and 15% in rural areas. The capacity of each traffic lane, in DHV, is normally about 1000 vehicles per hour. For a design traffic flow of 10,000 to 15,000 AADT, for example, the expected DHV is 1000 to 1500. The capacity of this facility would be exceeded at more than 1000 vehicles per hour per lane, which equates to 4,000 vehicles per hour for all four lanes, hence capacity will not be exceeded at 15,000 AADT.

The DHV values are necessary for selecting the number of lanes for the ramps corresponding to the junction.

11.8.8 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. Alternatively stated, the basic number of lanes is a constant number of lanes assigned to a route, exclusive of auxiliary lanes. Deceleration and acceleration lanes are classified as auxiliary lanes in this context; hence the term 'balance' is not strictly correct except in the context defined here and as illustrated in Figure 11-5 where auxiliary lanes for diverging and merging are shown.

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. This problem can be overcome using auxiliary lanes but 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 crashes, 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. The application of lane balance and coordination with basic number of lanes is illustrated in Figure 11-5.

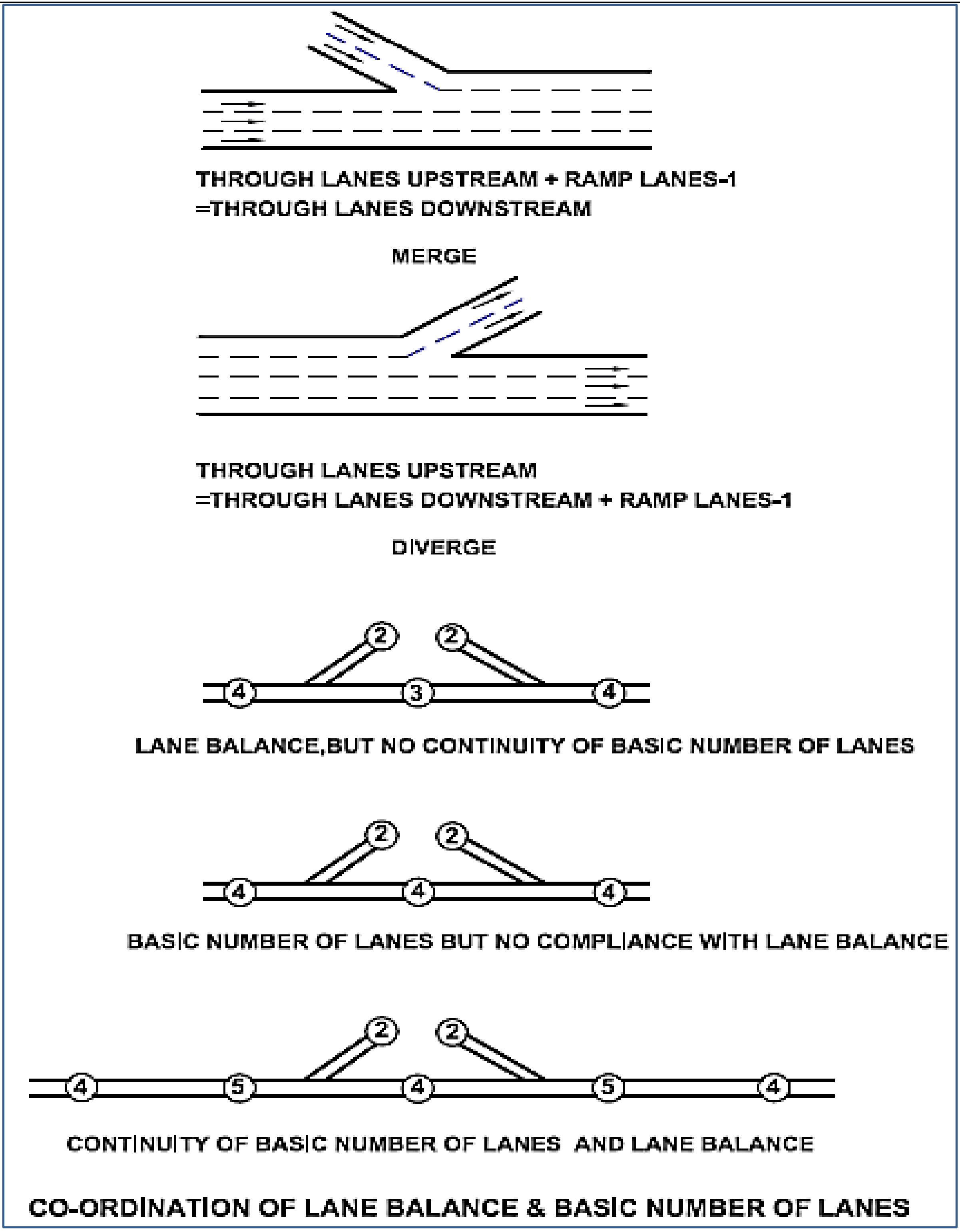


Figure 11-5: Principles of Lane Balance

At merges, the number of lanes downstream of the merge should be one less than the sum of the number of lanes upstream of the merge plus the number of lanes in the merging ramp. 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 less than the total number upstream of the diverge plus the number of lanes in the diverging ramp. 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 motorway, the above rule indicates that the number of motorway 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.

10.9 Design Principles

Specific design principles apply to grade-separated intersections. These are:

1) The high speeds normally found on roads where grade separation is required, and the low design speeds of ancillary roads make it necessary to pay particular attention to the transitions between high and low speed. This not only influences the use of long speed-change lanes and compound curves but also the choice of types of interchange which do not result in abrupt changes in vehicle speeds.
2) Weaving between lanes on the main roadway within the interchange is undesirable and can be avoided by arranging for diverging points to precede merging points.
3) On a road with many grade-separated interchanges, a consistent design speed is desirable for ramps. This speed must be not less than 65% of the speed of the adjoining major road.
4) As a rule, right-turning movements that are grade separated should be made through a left-hand loop.
5) Unexpected, prohibited traffic movements, especially where traffic is light, are difficult to enforce and cause danger. If possible, the geometric layout should be designed to make prohibited movements difficult, for example on one-way ramps, entry contrary to the one-way movement can be restricted by the use of suitably shaped traffic islands to supplement the traffic signs.

10.10 Design Procedures

Step 1 Develop a Basic Plan

It is important that grade-separated intersections and lengths of the principal roadway between interchanges are considered together from the outset. Choices of one will affect the other and vice versa.

Step 2 Determine Design Year

Careful consideration of the design year is required, bearing in mind the design year strategy adopted for the routes connected by the interchange. It will often be easier to add capacity to a motorway route than to reconstruct a major interchange and therefore high design year traffic flows should be considered.

Step 3 Establish Urban or Rural Standards

Major interchanges will normally be located on inter-urban routes designed to rural standards. However restricted space available around existing interchanges may require consideration of speed restrictions and possibly lower urban design standards, especially in peri-urban areas. A clear and definite change between rural and urban standards will be required so that drivers are made aware of the changed driving environment. This can be made by the introduction of a transition zone including posted speed limit either for the whole complex or for those elements linking directly to the local urban network.

Step 4 Determine Constraints

Choice of location for major interchanges on existing routes will be limited, compared with new routes. In many instances development, attracted by easy access to the motorway system, may have extended up to the existing highway boundary. Constraints may include the following:

1) Environmental Constraints:
2) Land take
3) Effect on
4) property
5) landscape
6) ecology
7) rights of way
8) heritage
9) Noise and air quality
10) Visual impact

A. Engineering Constraints:

1) Condition of existing structures;
2) Topography;
3) Geology;
4) Existing traffic flows;
5) Existing interchange layout;
6) Ability to manage traffic during construction;
7) Ability to manage traffic during maintenance.

Step 5 Develop Local Network and Interchange Strategy

This stage follows initial consideration of the broad network strategy and constraints. It includes assessment of the need to maintain provision for all existing traffic movements at the interchange or the redirection of traffic to adjacent junctions or interchanges via motorway link roads or other routes.

Step 6 Select Options for Appraisal

The aim is to identify a satisfactory minimum cost solution. A comparison of at least two solutions should be made, even for relatively straightforward problems. For more complex problems several solutions should be prepared for analysis. Options should include those with minimum effect on existing traffic together with options that may cause greater disturbance during construction but would provide a potentially more efficient and/or compact layout for future use. The incremental cost of each should be compared with the quantified benefits/costs of the alternative solutions.

Step 7 Select Appraisal Criteria

It may be appropriate to apply different weighting to different criteria, depending on local factors but this should be agreed within the Ministry. Where identified, minimum values to be achieved should be included.

Step 8 Develop Traffic Flows

Derive low and high growth design year traffic flows for each section of mainline and connector road in accordance with ministry procedures.

Step 8 Lane Requirements

Determine mainline and connector road lane requirements for each option.

Step 9 Merge, Diverge and Weaving Requirements

Check merge, diverge and weaving layouts including lane balance. If the route is particularly constrained by the proximity of interchanges or by high weaving flows, controlled speed environments may need to be considered, either all day or for part of the day.

Step 10 Road Signs

Ensure that an effective road signing scheme has been designed incorporating both advance direction signing and subsequent route confirmation.

Step 11 Appraisal Process

In many cases the scale and effect of the works required will necessitate preparation of a full environmental appraisal either for the interchange works alone or in conjunction with adjacent motorway widening or construction proposals.

The Public Consultation type framework for the comparison of several options provides a suitable basis for the assessment. This will therefore ensure that consideration is given to:

1) The effects on travellers;
2) The effects on occupiers of property;
3) The effects on users of facilities;
4) Conservation policies;
5) Development and transport policies;
6) Costs.

The effects on travellers will include an appraisal of the complexity and safety of the proposed interchange layouts. Where there are significant differences between the times and/or distances involved in negotiating the interchange, economic assessments of operating costs and time savings or delays should be carried out.

Driver stress and driver comprehension of the layout will depend on the number and timing of decisions and manoeuvres required. These will be affected by the speed of traffic and its density which may mean short gaps for manoeuvres and increased stress when weaving.

Travellers will also be affected by delays during construction and the economic assessment must take account of these costs. Solutions that result in the best final arrangement may cause the greatest disturbance to traffic during construction. It is therefore important that consideration is given to the provision of temporary works. Such measures, while increasing construction costs, can significantly reduce the cost of delays. It is also important that the costs of future maintenance, including traffic delay costs, are considered.

Safety of both motorway users and construction personnel is of prime importance in the design of major interchange improvement schemes. It is essential that designers consider the safety implications of the construction methods and traffic management measures necessary for execution of the work.

It will also be necessary to establish the importance given to the feasibility of providing additional capacity at a future date for each option.

Environmental factors are likely to be significant. There will often be limitations on the land available for new highway works and amelioration measures due to the presence of development along some parts of the motorway boundaries. The use of long lengths of elevated carriageway or the provision of additional levels over existing interchanges is likely to be environmentally intrusive.

12 Road Safety Systems

12.1 The Road Accident Situation

12.1.1 General

Kenya ranks as one of the countries with high traffic fatality rates and the number of serious injuries resulting from road crashes is equally alarming hence large reductions should be possible.

Economic analysis has shown conclusively that this high level of road crashes has economic consequences for the country in terms of property damage, loss of earnings or production and hospital costs resulting from physical injury, that is equivalent to a reduction of between 2% and 3% of GDP. This is a very significant drain on the economy. Furthermore, the consequences of the road crashes impose a great deal of grief and anguish on a considerable proportion of the population. Every effort should therefore be made to reduce the number of serious crashes. Safety and economy are the foundations on which competent design rests. Inadequate consideration of either will automatically result in inadequate design.

It is difficult to correct many safety defects at a later stage without major reconstruction hence designing for safety should occur at the very beginning of a road project. Road safety audits by an independent team should be undertaken during each stage of the design and a system for doing so has been included in the manuals (PAM Volume 6, Road Safety Audits)

Good geometric design of roads thus has an important part to play in reducing the number and severity of road crashes. Road safety aspects have been highlighted throughout this manual:

1) Human factors have been addressed in the design process;
2) Both horizontal and vertical alignments have been designed for maximum safety through suitable curvature, adequate sight distances, and suitable design speeds and the designs of junctions (Chapters 8, 9 and 10) are largely based on safety considerations, especially when they are controlled junctions but also when they are basic priority junctions.
3) Road and shoulder widths have been increased were necessary to accommodate pedestrians, NMT, and intermediate forms of transport (IMT);
4) Parking places and lay-bys for buses have been included in populated areas;
5) Account has been taken of reduced friction on unpaved roads.

Thus, although many aspects of geometric design that have been described in previous chapters are dictated by road safety requirements, safety issues are important in all aspects of road design The scope of this chapter is to introduce the design and specifications of other important safety features that have not been covered in detail in earlier chapters. However, some of the principles, for example, the effects of human factors, are given additional emphasis.

Miscellaneous design items in this chapter include bus lay-bys and parking bays, parking lanes, traffic calming, safety barriers, emergency escape ramps, brake check areas, safety rest areas and scenic overlooks, public utilities, and railway grade crossings.

12.1.2 Pedestrians

About 35-40 % of all road fatalities are pedestrians. Methods and designs that improve the safety of pedestrians are therefore particularly vital.

1) Pedestrian actions are less predictable than those of motorists. They tend to select paths that are the shortest distance between two points and avoid using underpasses or overpasses that are not convenient. As a consequence, they frequently take risks that vehicle drivers have difficulty anticipating;
2) Walking speeds vary from 1.0 m/s to 1.8 m/s, with an average of 1.4 m/s. For design purposes 1.0 m/s is recommended to accommodate pedestrians that may be carrying load, with children or the aged;
3) In urban areas it is necessary to make provision for:
   (i) Passengers boarding and alighting on and off from public transport.
   (ii) Disabled persons; and
   (iii) Other non-vehicular users of the facility in addition to accommodating pedestrians and cyclists.
4) On rural roads, speeds are high so that crashes involving pedestrians are inevitably serious and often fatal. Provision should be made for protecting pedestrians on rural roads, even though their numbers may be low.
5) Pedestrian safety is enhanced by the provision of median refuge islands of sufficient width at wide intersections (Section 4.16), and lighting at complex locations.
6) In urban areas, the presence of large numbers of pedestrians will require adequate sidewalk widths .
7) Age is an important factor that may explain some behaviour that leads to collisions. It is recommended that older pedestrians be accommodated, not only by assuming lower design speeds as stated above, but also by using simple designs that minimize crossing widths. Where complex elements such as channelisation and separate turning lanes are featured, the designer should assess alternatives that will assist older pedestrians.

12.1.3 Cyclists

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

1) Wider paved shoulders.
2) Bicycle-safe drainage gratings (flat metal grids to prevent unwanted debris from entering a drain underneath but with transverse bars and slots which cannot snag bicycle wheels).
3) Maintaining a smooth, clean riding surface.

Cyclists can often be accommodated on the normal travelled lanes but, when the number of cyclists increases, it may be necessary to widen these lanes or to provide cycle paths adjacent to or, for preference, away from the travelled lane. At certain locations it may be appropriate to supplement the existing road system by providing specifically designated cycle paths.

12.1.4 Improving Safety

There are several other steps that can be taken to improve safety. These include:

1) Carrying out a road safety audit (RSA) at all stages of design.
2) Traffic calming measures to reduce the speeds of vehicles in populated area.
3) Improved road markings, signing and lighting.
4) Segregating pedestrians and motorised vehicles in populated areas.
5) Providing safety barriers at dangerous locations.

Many of these have been discussed in the appropriate chapters of this manual but for more detail various publications on safer design by World Bank, WHO, World Road Congress, TRL, iRAP etc are recommended.

12.2 Traffic Calming

12.2.1 General

The seriousness of road crashes increases dramatically with speed and hence significant improvements to road safety are possible if traffic can be slowed down. This process is called traffic calming. All such methods have both advantages and disadvantages, and the effectiveness of the methods also depends on aspects of driver behaviour that can vary considerably from country to country. Therefore, research also needs to be carried out in Kenya to identify the most cost-effective approaches.

The likely effect of any traffic calming measure on all the road users should be reviewed before they are installed. Some are unsuitable if large buses are part of the traffic stream; some are very harsh on bicycles, motorcycles, and motorcycle taxis; and some are totally unsuitable when there is any animal-drawn transport.

Traffic calming measures such as road bumps should not be provided on the higher road classes designed for high traffic volumes. It is advisable to provide pedestrian overpasses and underpasses for safe crossing on high-speed roads.

The most common methods are:

1) Chicanes
2) Rumble strips and other textured surfacing
3) Speed reduction humps and cushions
4) Roundabouts
5) Horizontal deflection of a straight road when approaching a roundabout or junction
6) Narrowing the width of the carriageway by, for example, channelising the traffic
7) Road markings such as painted chevrons, ghost islands, go slow signs, speed limit signs etc.
8) Gateways (so-called) marking the entrance to a village area.
9) Prohibition of certain vehicle types by physical width or height restrictions or signs (but with enforcement)

10) Adding crossing facilities for pedestrians and cyclists.

The effectiveness of these methods are quite variable; those that require a physical intervention and those that directly affect have been found to be more effective. For more detail on what works, reference can be made to the World Bank document on Guide for Road Safety Interventions: Evidence of What Works and What Does Not Work[3].

12.2.2 Chicanes

These are designed to produce minor turning movements along straight streets in established urban areas by reducing the width of the road to one lane for a very short distance (3-5 m) at intervals (typically 300 m) along it. They are usually built on alternate sides of the road. They cause drivers to slow down provided that the traffic level is high enough to make it very probable that they will meet an oncoming vehicle. The method is obviously unacceptable if traffic flow is high because the congestion that it causes will be severe. For safety, they must be illuminated at night.

12.2.3 Rumble Strips

These are essentially a form of artificial road texture that causes considerable tyre noise and vehicle vibrations if the vehicle is travelling too fast. They are used in two ways. The first is to delineate areas where vehicles should not be. Here they are provided as a line running parallel to the normal traffic flow so that if a vehicle inadvertently strays onto or across the line the driver will receive adequate warning.

Secondly, they are used across the road where they are placed in relatively narrow widths of 2 to 4 m but at intervals along the road of typically 50 to 200 m. They are uncomfortable to drive across at speed, although not significantly for heavy vehicles, hence they are effective in providing moderate slowing down of the traffic. They do not need to be illuminated at night.

12.2.4 Speed Reduction Humps and Cushions

A road hump is a device for controlling the speed of vehicles consisting of a raised area across the roadway. There are two main types of road humps i.e., circular topped, which are intended for traffic speed reduction only, and flat-topped humps, which are intended for speed reduction and for use as a pedestrian crossing.

The shape of the hump is important to reduce the severity of the shock when a vehicle drives over it. Ideally, they should cause driver discomfort but not vehicle damage. Road humps may be used on roads where it is proven to be necessary. Standard circular road humps are 100mm high with standard lengths of 3.7 m and 9.5 m for speed limits of 30 and 50km/h respectively as shown in Figure 12-1 .

[3] Guide for Road Safety Interventions: Evidence of What Works and What Does Not Work https://www.roadsafetyfacility.org/publications/guide-road-safety-interventions-evidence-what-works-and-what-does-not-work

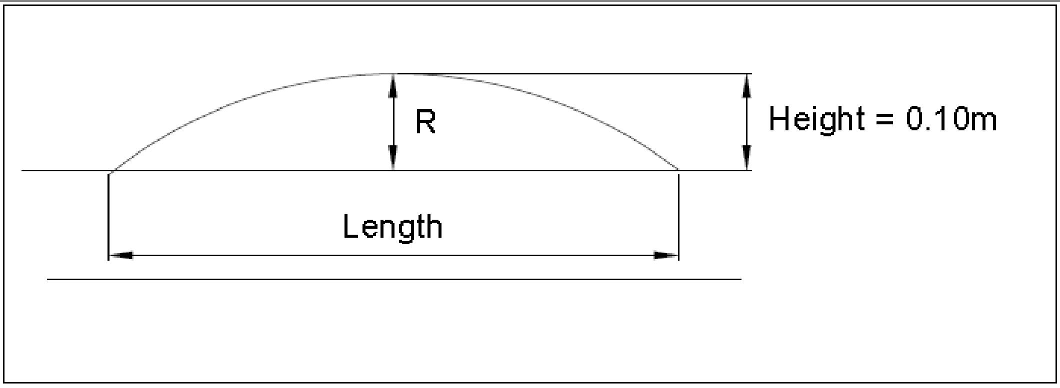


Figure 12-1: Circular humps

However, other sizes may be adopted depending on site conditions as indicated in Table 12-1. The circular road hump must also have a short run-on fillet at both ends to smooth the passage of vehicles onto the hump.

Table 12-1: Design of Circular Road Humps (Height = 0.1 m)

Car speed (km/h)		**Truck speed (km/h)**	**Radius (m)**	**Length (m)**
30		15	20	3.7
35		20	31	5.0
40		25	53	6.5
45		30	80	8.0
50		35	113	9.5
		40	180	12.0

Note: The lead on and lead off fillets are not shown

Flat-topped road humps (Figure 12-2) are an alternative to the circular road humps but are longer, with a flattened top, and used to give pedestrians a level crossing between footways. They can be especially useful where there are a lot of pedestrians. Normally, pedestrian crossings should only be installed at busy crossing points. Where it is necessary to use traffic calming measures to reduce speed, the most suitable arrangement is to install circular road humps a short distance from the pedestrian crossing. If it is necessary to provide a hump at the crossing, a flat-topped hump should be used, which is easier for pedestrians.

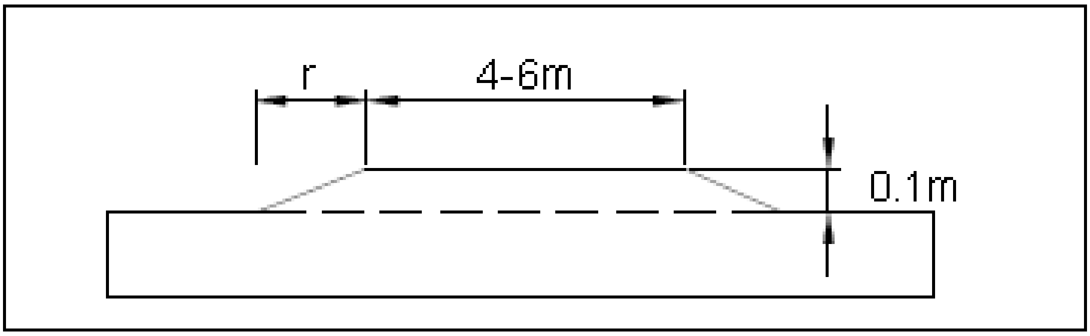


Figure 12-2: Flat-topped Humps

The standard size in usually 8.4 m long and 100 mm high. However, other sizes may be adopted depending on site conditions as indicated in Table 12-2

Table 12-2: Design of Flat-topped Road Humps

Car speed (km/h)	Truck speed (km/h)	Ramp Length (m)	Grade of step i %
30	15	1.0	10
35	20	1.3	7.5
40	25	1.7	6.0
45	30	2.0	5.0
50	35	2.5	4.0
	40	3.3	3.0

Based on a similar principle to the speed hump, speed reducing cushions are more versatile. They are essentially very similar to the speed hump, but the hump is not continuous across the road. The width of a two-lane road is usually covered by two or three cushions with gaps between them of 750 – 1200 mm. The idea is that large heavy vehicles will not be able to pass without at least one wheel running over one of the humps, but bicycles and motorcycles can pass between them without interference. If suitably designed, the wheels of animal drawn carts can also avoid the humps.

12.3 Treatment of Trading Centres

12.3.1 Introduction

The roads serving trading centres and small urban centres are often required to serve two conflicting functions in that they must cater for both inter- and intra-urban traffic and urban related functions and users. As a result, traffic entering the centres often does so at speeds that are much too high for the environment where there is slow moving, turning traffic, parking, roadside vending, shops and stalls and pedestrians who require to move along or across the road. Such a situation requires the need for a comprehensive treatment of the trading centres which will induce, or even force, a driver to reduce speed significantly.

The traffic calming measures described above can be introduced within such environments to contribute to reducing vehicle speed and thus improving the safety of road users. Specific measures include calming traffic with speed humps, rumble strips, road narrowing, pedestrian crossings, and specially demarcated low speed zones. However, the functional characteristics of the through road will dictate to some extent the kind of safety measures that are acceptable.

The objective of the approach to traffic calming is to develop a perception that the trading centre is a low-speed environment and to encourage drivers to reduce speed because of this perception. To this end, the road through the trading centre/small urban centre is divided into three zones, namely:

1) The approach zone.
2) The transition zone.
3) The core zone.

12.3.2 The Approach Zone

The Approach Zone is the section of road prior to entry into the centre where the driver needs to be made aware that the open road speed is no longer appropriate. This is the section of road where speed should be reduced typically from above 60/70 km/h down to 50 km/h before entering the village.

The entry should be marked by an obvious Gateway that marks the beginning of the village. Drivers should be clearly informed that they are entering a section, where they are required to drive more slowly and carefully, and this can best be done by installing a gate or gateway at the point where the built-up area begins. The gateway sign shall be double-sided and combines the speed limit sign with a panel showing the place name.

Gates are likely to achieve greater speed reductions if they incorporate traffic islands that prevent the driver from continuing straight ahead - some typical designs are shown in Figure 12-3. However, to avoid the islands becoming a hazard, especially at night, they must be very clearly marked with reflective markings and road studs, and they should be designed so that, if errant vehicles hit them, the consequences are not severe.

The gate should preferably be designed such that the toughest vehicle path for a passenger car through the gate or portal should have an entry radius below 100 m for 50 km/h speed control and 50 m for 30 km/h speed control. Curves that follow 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 as shown.

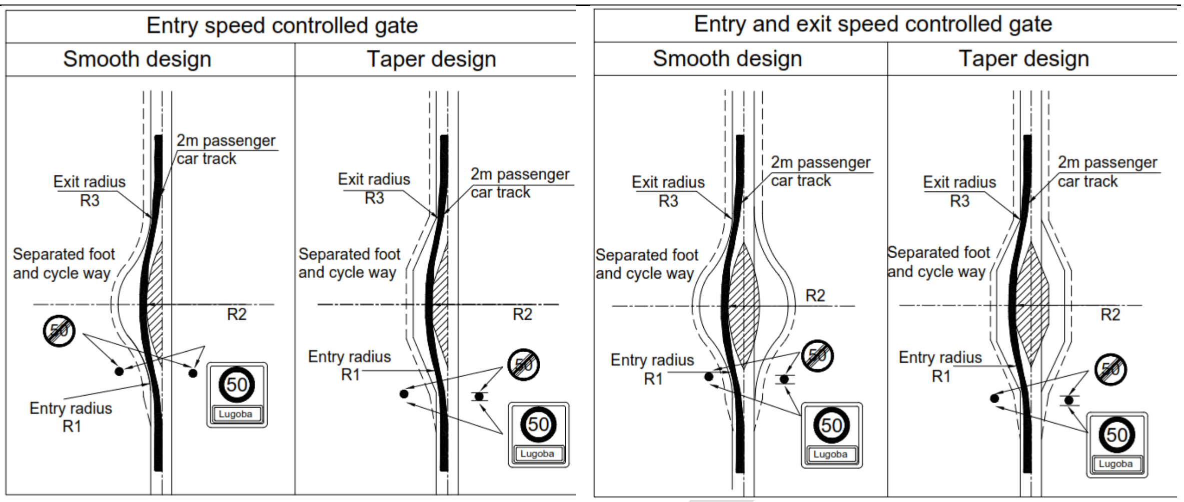


Figure 12-3: Entry and Exit Gateway Designs

12.3.3 The Transition Zone

The Transition Zone is the section of road between the Gateway, and the core zone of the centre. The target speed, and posted speed limit in this zone should be maintained at typically 50 km/h. The first road hump or humps in a series of humps should be sited in this zone. In this context, with adequate advance warning provided by the approach zone and Gateway, properly designed road humps should be quite safe.

12.3.4 The Core Zone

The Core Zone is the section identified as being in the centre of the trading area where most of vehicle/pedestrian conflicts are expected to take place. This would normally be where most shops, bus-bays or other pedestrian generating activities are located. This is the section where pedestrian crossing facilities are most likely to be established and where the target speed, and posted speed limit, should be reduced to 40 km/h or lower. Road humps should be provided within this zone with advisory speed limits to enforce the lower speed environment required.

12.4 Brake Check Areas

Brake check area for trucks are areas set aside before a steep descent. They provide opportunity for cooling the brake system and they ensure that drivers begin the descent at low speed and in a low gear that may make the difference between controlled and out-of-control operation on the downgrade. They also provide an opportunity to display information about the grade ahead, escape ramp locations and maximum safe recommended descent speeds. They should be provided on routes that have long, steep downgrades and commercial vehicle numbers of around 500 per day, especially on National Roads and principal traffic routes. They need to be large enough to store several semi-trailers, the actual numbers depending on volume and predicted arrival rate. Their location requires good visibility and acceleration and deceleration tapers. Adequate signage should be provided to advise drivers in advance of the facilities.

In addition to brake check areas, emergency escape ramps might also be required (Section 12.6).

12.5 Safety Barriers

12.5.1 General

Many crashes on high-speed roads involve vehicles leaving the road and coming into collision with hazardous obstacles such as trees, bridge supports, or simply rolling down a high embankment. Similarly, a vehicle leaving a lane on a dual carriageway runs the risk of collision with an oncoming vehicle.

All elements of risk along the roads such as obstacles, steep side slopes, bridges and underpasses might cause serious personal injuries in the case of a vehicle crash.

The design of a safe roadside requires the identification of the dangerous obstacles that are present. Once identified, it is possible to establish strategies or measures necessary to protect the traffic from them.

When a roadside hazard is identified, the best solution is to remove the hazard. Where the hazard is a drop, it is worth considering whether the slope can be flattened to make it less hazardous. If this cannot be done, it may be possible to shield the hazard with a barrier.

Safety barriers are used to prevent vehicles from hitting or falling into a hazard; for example, falling down a steep slope, hitting an obstruction, 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 because of excessive speed, lack of concentration, failure of a tyre or a collision. A safety barrier is also an element of risk and should only be used when it is more dangerous to drive off the road than to hit a safety barrier.

Two classes of dangerous objects are defined. Point obstacles and linear obstacles. These classes correspond to different procedures for the selection of mitigation measures. However, the general strategy applied to both classes is common and consists of the following steps:

1) Assessment of dangerous obstacles;
2) If possible, removal of the dangerous obstacle out of the free zone;
3) When removal is not possible, assess the possibility of modification of the dangerous obstacle;
4) If these options are not possible, protect the traffic with a vehicle restraint system.

12.5.2 Basic Principles

Safety barriers should be the last resort to protect traffic from existing hazardous obstacles in the road zone. The presence of these devices is an acceptance that the removal of a dangerous obstacle is not practicable or economically possible. However, the high number of fatalities with fixed obstacles in which collisions with safety barriers are considered the most dangerous, demonstrate that this protection is not a completely effective solution.

Safety barriers are designed to either attenuate the impact of a vehicle or to redirect errant vehicles. They also provide guidance for pedestrians or other road users. Ideally, the safety barrier will:

1) Prevent the vehicle from passing through the barrier.

2) Absorb (cushion) the impact of the vehicle without injuring the occupants (i.e., no severe deceleration);
3) Re-direct the vehicle along the road parallel to the other traffic.
4) Enable the driver to retain control of the vehicle (no spinning or overturning of the vehicle)
5) Reduce the severity of crashes.

Table 12-3 summarises the requirements for barriers for a range of roadside obstacles.

Table 12-3: Roadside Obstacles and Barrier Requirement

Obstacle or Terrain	Barrier Requirement/Comment
Bridge piers, abutments, railing ends	Shielding analysis required
Boulders	Judgement: nature of object: likelihood of impact
Culverts, pipes (smooth)	Judgement: based on size, shape, location
Cut slopes (smooth)	Shielding analysis not generally required
Cut slopes (rough)	Judgement: based on likelihood or impact
Ditches (parallel)	Analysis generally required
Embankments	Judgement: based on fill height and slope
Retaining walls	Judgement: based on wall smoothness and angle of impact
Sign and lighting supports	Shielding analysis for isolated signals in the clear zone on speed (80 km/h or greater) facility.
Traffic signal supports	Shielding analysis for isolated signals in the clear zone on speed (80 km/h or greater) facility
Trees	Judgment: site specific
Utility poles	Judgment: case by case basis
Permanent bodies of water	Judgment: depth of water, likelihood of encroachment

There is little or no standardisation of the configuration of safety barriers, but safety barriers should be placed sufficiently far from the carriageway edge so that they themselves do not constitute a hazard to vehicles, nor reduce the effective width of the carriageway. A description of each type of safety barrier and a brief discussion of the positive and negative elements of each type follows below.

There are various types of safety barrier

1. Longitudinal barriers:
   i) Flexible steel beam guardrail with posts made of steel or timber;
   ii) Wire rope barrier

iii) Concrete barrier.

2. Terminals
    i) Redirective
    ii) Non redirective

3 Various Transitions

It should be noted and understood that the specification, installation, and maintenance of safety barriers are highly technical subjects, and this manual can only give a brief introduction to the subject. The designer should always seek advice from experts. A safety barrier can be ineffective and even dangerous if not properly designed and installed. The components of safety barrier should always be purchased from a specialist manufacturer and their advice obtained. If possible, arrangements should be made for the manufacturers to install it, or to supervise its installation.

12.5.3 Containment Requirements for Safety Barriers

Various standards of safety barriers and bridge parapets are defined depending on the level of impact that they need to withstand. Table 12-4 indicates the requirements of the containment levels.

Table 12-4: Containment Levels

Level	Road Type and Condition
1	Roads with speed limits ≤ 60 km/h and AADT ≤12,000 Roads with speed limits ≥70 km/ and AADT ≤1,500
2	Roads with speed limits ≥ 60 km h and AADT > 12,000 Roads with speed limits ≤ 70 km/ h and AADT > 1,500 Motorways carrying high traffic volumes
3	Bridges and large culverts Retaining walls with drop > 4 m Cliff or a rock face with a drop of more than 4 metres and a slope steeper than 1:1.5) (On condition of satisfactory space for deformation and fastening of the post) Narrow medians (<2m) on motorways with design speed >80 km/h and high portion of HGV (>20%) Sensitive locations where errant vehicles may cause substantial damage e.g., at railways, drinking water reservoirs, etc.
4	On and under bridges where an accident can cause bridge collapse

Source: NPRA

12.5.4 Design and Performance Criteria

The performance criteria are shown in Table 12-5. The manufacturer must demonstrate or document by calculation that the performance criteria is fulfilled. With this impact criterion, the errant vehicle will be redirected safely to the road without crossing the line to the opposite direction of traffic. The vehicle shall remain in an upright position after the test to assure that the driver can control the vehicle. The safety barrier, or the bridge parapet, shall not be fractured into pieces which can harm pedestrians close to the vehicle.

Table 12-5: Requirements for the Containment Levels for Safety Barriers

Containment Level	Impact speed (km/h)	Impact angle (degree °)	Total mass (Kg)	Type of vehicle	Theoretical kinetic energy (KNm)
1	80	20	1,500	Car	43.3
2	110	20	1,500	Car	81.9
3	70	20	13,000	Bus	287.5
4	65	20	30,000	Rigid HGV	572

When a safety barrier is hit by a vehicle, it will deform (Figure 12-4). The 'deformation during impact test' is characterised by the working width. The barrier's working width (W) is the maximum horizontal distance between the front edge of the barrier before deformation and the rear edge after. If the vehicle body deforms around the road safety barrier, so that the latter cannot be used for the purpose of measuring the working width, the maximum lateral position of any part of the vehicle shall be taken as an alternative. The barrier's dynamic deformation or width of deformation, D, is the horizontal distance between the front edge of the barrier before deformation.

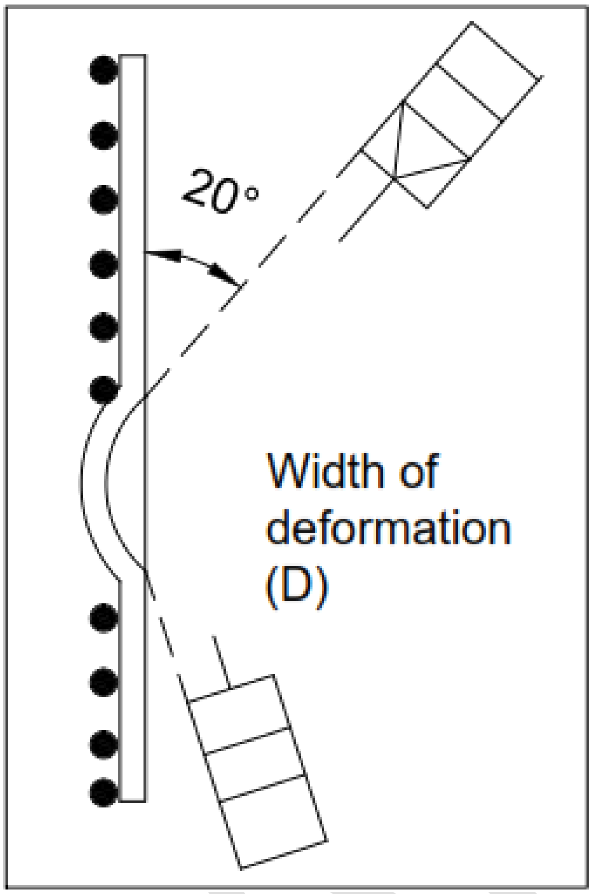


Figure 12-4: Deformation of Barriers

Source: NPRA

The safety barrier including parapet shall contain the vehicle without complete breakage of any of its principal longitudinal elements. Elements of the safety barrier including parapets shall not penetrate the passenger compartment of the vehicle. Deformations of the passenger compartment that can cause serious injuries shall not be permitted.

The vehicle must not spin and/or roll over (including rollover of the vehicle onto its side) during or after impact.

12.5.5 Requirement for Using Safety Barriers

On existing roads, the main consideration will be the crash history. If collisions with the hazard are occurring repeatedly and the hazard cannot be removed there may be justification for a safety barrier or guardrail. Cost-benefit analysis can help determine whether it is worthwhile installing a barrier.

Figure 12-5 and should be used as a general criteria and as a starting point for determining need for installation of guardrails.

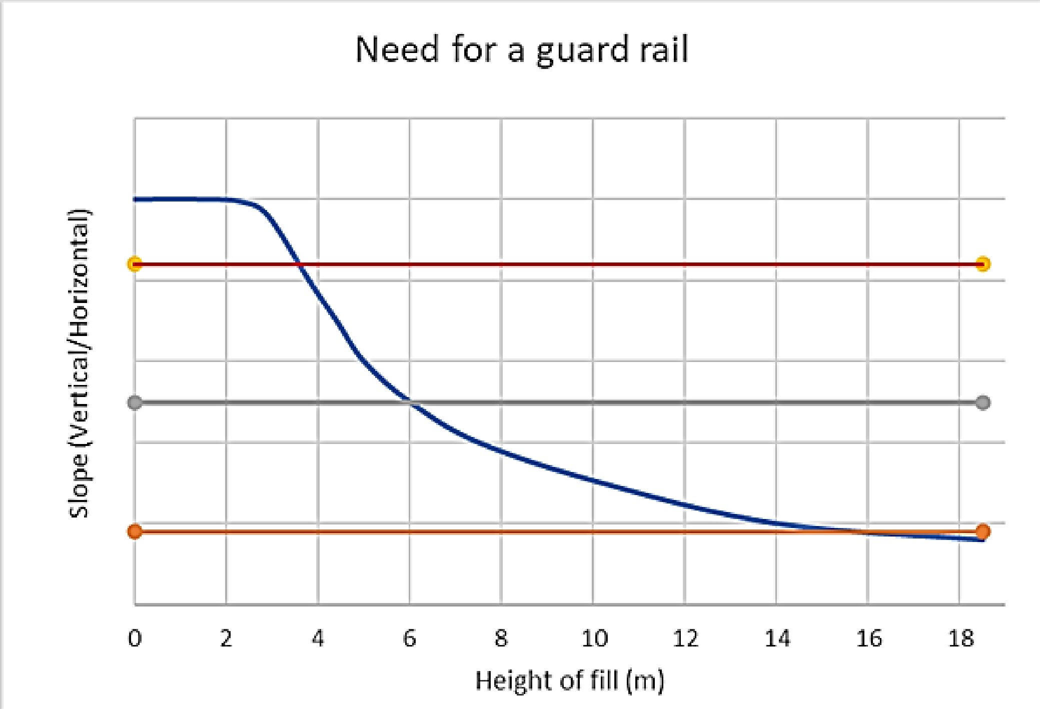


Figure 12-5: Guide for determining need for a guardrail.

Source: SATCC

In addition to installing safety barriers when the fill is high and the side slopes are steep, there is a substantial risk caused by obstacles within the clear zone area. Such obstacles include:

1) Bridge piers, abutments and culverts walls;
2) None yielding light columns, gantries and trees with diameter > 18 cm;
3) Concrete bumpers, solid concrete elements, culvert outlets, end of retaining walls;
4) Solid stones or rock outcrops which have a top more than 20 cm above the terrain or steep rock slopes where protruding parts should be less than 30 cm;
5) Solid installations.

Such dangerous roadside obstacles should be removed, replaced with yielding road furniture, or protected with a safety barrier or guardrail. If, however, a back slope is built in front of the obstacles in the clear zone, safety barriers are not necessary if the following criteria are satisfied, i.e.:

1. When the gradient of the back slope is 1:2, the height of the slope measured from the adjacent road surface is a minimum 1.8 m.
2. When the gradient of the back slope is 1:1.5 the height of the slope measured from the adjacent road surface can be reduced to 1.4 m.

12.5.6 Required Length of a Safety Barrier

To reduce costs, steel beam safety barriers are often installed in lengths that are too short to be effective. Generally, at least 30 m of steel beam strong post guardrail are needed for it to perform satisfactorily. Figure 12-6 and Table 12-6 give guidance on determining the length required. On a two-way single carriageway road, both directions of travel must be considered;

it cannot be assumed that vehicles will not hit the downstream end of a barrier. One of the common design 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. Any gaps through which vehicles may fall should always be closed.

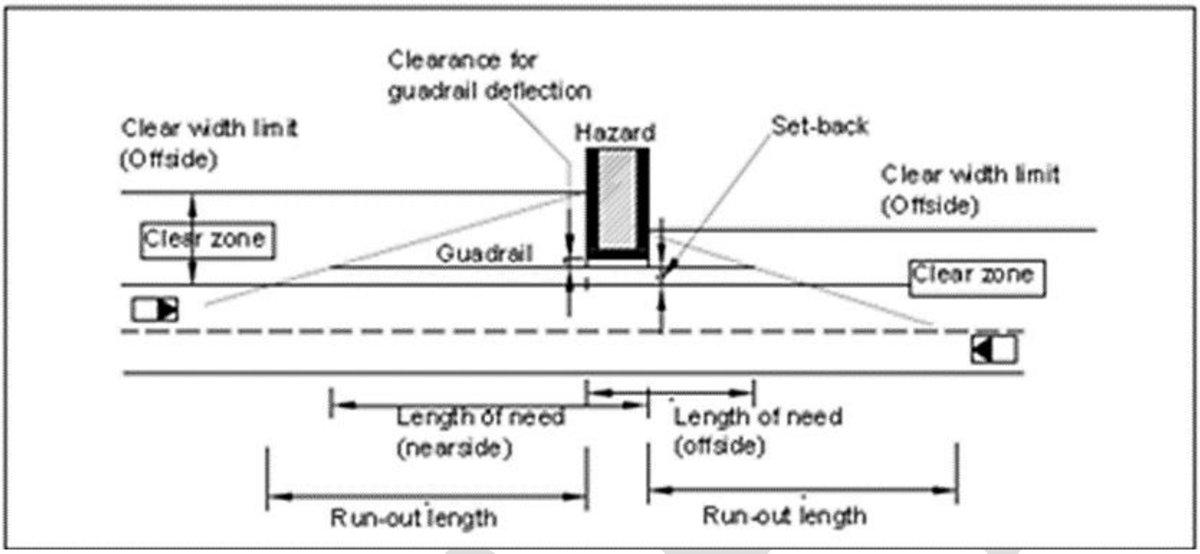


Figure 12-6: Length of Safety Barriers

The set-back should normally be 50 cm beyond the edge line of the road to the face of the barrier. With roads having AADT of 12,000 and above and a speed limit greater than 80km/h, the set-back should be 75cm.

Note that this is the starting point for estimating the need and Table 12-6 for estimating the length. Engineering judgement is required to determine the precise length for specific situations.

Table 12-6: Length of Safety Barriers

Speed km/h	Run out length (m)
60	50-60
80	80-90
100	100-110
110	120

12.5.7 Steel Beam Strong Post Guardrail

Steel beam, strong post guardrails are the most common type of safety barrier (Figure 12-7) Typical post details of the precise design varies, but the basic characteristics are that the steel beams are:

1) A W-shape (this is the part that comes into contact with the vehicle)
2) Over 4.130 m long
3) Mounted on steel posts that are set either 1.905 m or 3.810 m apart;

4) Mounted so that the centre of the beam is 600 mm above the height of the road surface; and

5) Incorporate a steel spacer block between the post and the beam to prevent the vehicle from hitting ("snagging") on the post ("snagging" will usually result in the vehicle spinning out of control).

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 to 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.

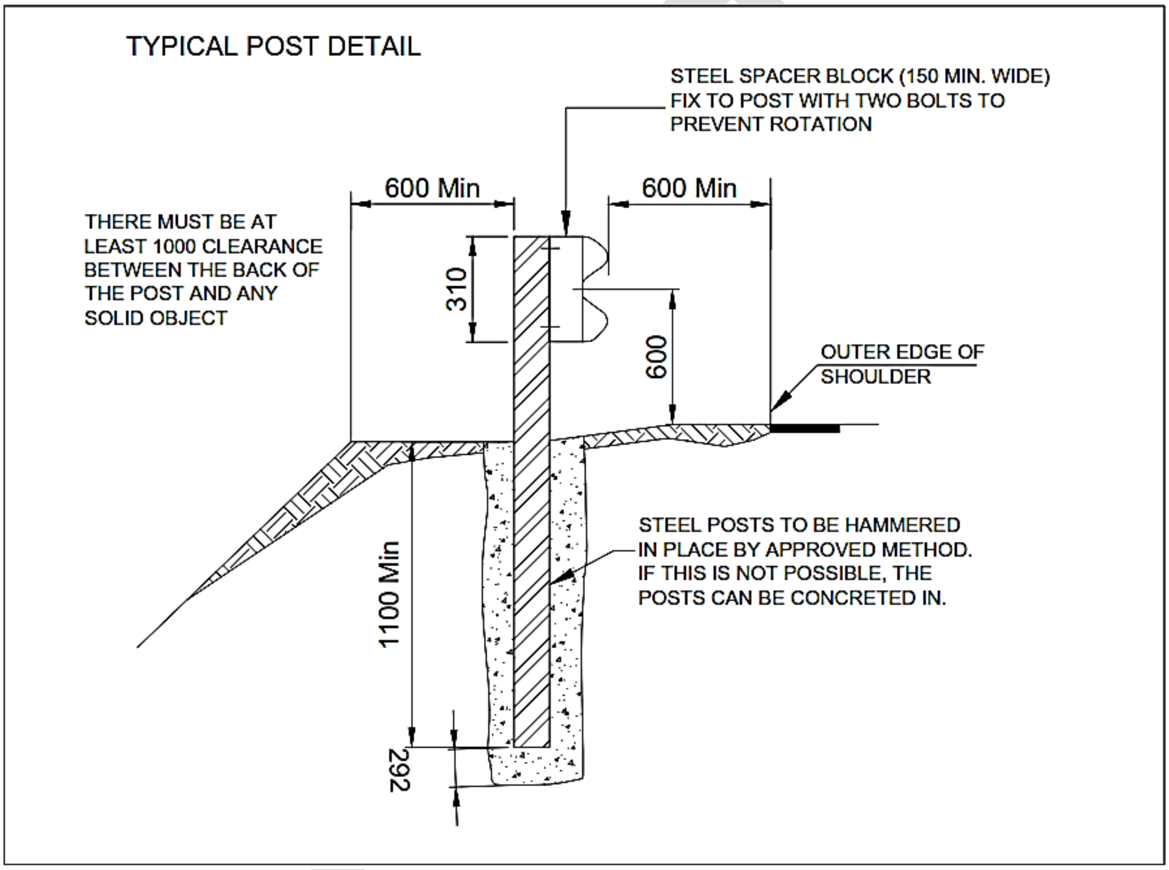


Figure 12-7: Typical Post Details, Steel Beam Safety Guardrail

12.5.8 Installation of Steel Beam Strong Post Guardrail

1) The beams must be overlapped in the direction of travel, so that if they come apart in an impact there is no end that can spear the vehicle.

2) The beams must be bolted together with sufficient bolts (typically eight) and the whole structure must be rigid.

3) The beam centre must be 600 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).

4) The spacer block must be fitted to the post with at least two bolts, otherwise it may rotate in a collision.

5) 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), a short section of beam between the main beam and the spacer block is necessary. This is often called a backup plate and it helps to prevent the beam hinging or tearing at this point.

6) If the posts and spacer 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.

7) 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 approximately 1000 mm centres), putting two beams together (one nested inside the other) or using extra-large concrete foundations.

8) 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, much longer posts must be used.

9) The guardrail should not be installed 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.

10) The guardrail must be set 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.

It can be appreciated that guardrails need to be carefully installed if they are to be worthwhile and the manufacturers expertise is probably essential.

12.5.9 Concrete Barriers

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 a New Jersey Barrier) tends to cause small vehicles to overturn, and the preferred shape is now a vertical or near-vertical wall (Figure 12-8). Concrete barriers generally require very little routine maintenance except after a very severe impact.

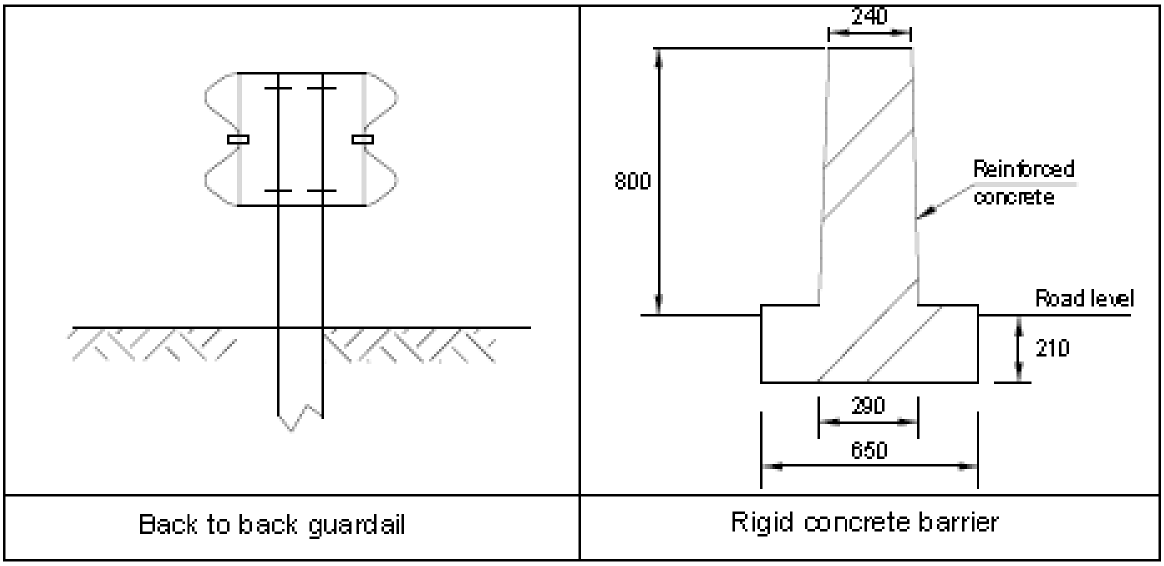


Figure 12-8: Various Barriers

12.5.10 Jersey Barriers

The Jersey barrier is the most well-known rigid barrier. It is constructed of concrete and has the best chance of preventing the vehicle from proceeding beyond the barrier.

However, the following problems have been noted:

1) Jersey barriers must be continuous, because an opening, in addition to providing no protection is a hazard.
2) The beginning and end of the barriers usually include no transition sections, and thus represent a hazard when struck head-on. A suitable metal barrier should be installed before the Jersey barrier with the adequate length and transition to deflect a potential head on collision with the Jersey barrier.
3) Jersey barriers deflect the vehicle, and the decelerations suffered by occupants of vehicles can be severe.

12.5.11 Wire-Rope Barriers

This type of barrier consists of two strands of cables fed through steel or concrete posts. These barriers are the least desirable configuration because:

1) If the cable is snapped due to an impact, the entire length of guardrail can become ineffective.
2) The cable can be stolen.

12.5.12 Grouted Rock Barrier

This rigid type of barrier makes economic sense because it employs materials available locally in its construction and provides labour-intensive employment. However, the barrier tends to be of a wider configuration than the alternatives, and therefore requires a larger construction width. It is of solid and substantial construction; therefore, it can also represent a hazard of itself. It suffers from the same problems as a Jersey barrier.

12.5.13 Median Barriers

High-speed dual carriageway roads with medians less than 1.5 x Minimum Clear Zone width may need to have median barriers to reduce the risk of cross-over crashes and/or to provide protection against collision with obstacles (e.g., lighting columns). Median barriers should not normally be used on urban dual carriageways with speed limits of less than 70 km/h. If such roads have a cross-over problem, it should be tackled through speed calming measures.

If the median is less than 9 m wide and when the design speed is greater than or equal to 90 km/h, or when the speed limit is 70 km/h or more, it should have a safety barrier or a soil embankment. In principle, there are four types of safety barriers that can be used as a median barrier (Figure-12-9).

1) One-sided steel beam/pipe barrier (one at each side of the median);
2) Double-sided steel beam/pipe barrier (placed as shown below);
3) Concrete barrier - either cast in situ or built by connecting pre-cast sections;
4) Or a soil embankment.

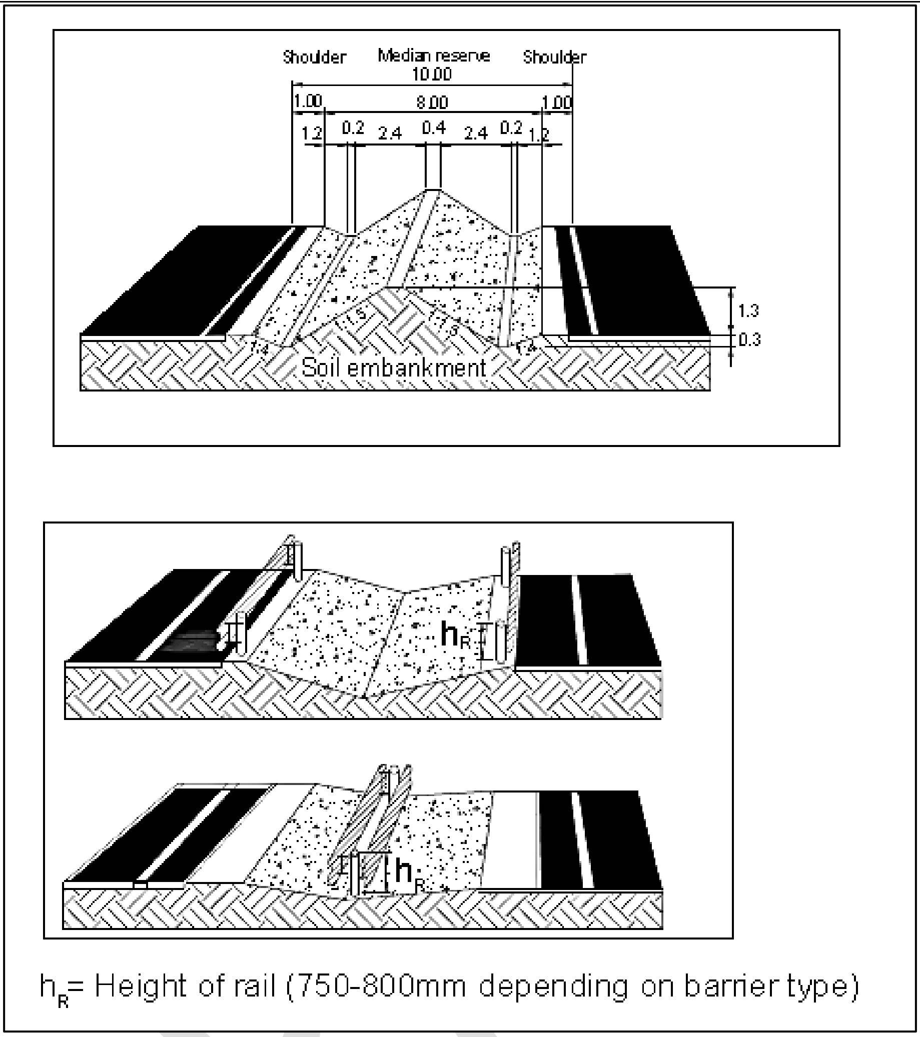


Figure-12-9: Different Types of Median Barrier

Median barriers often take the form of two guardrail beams mounted back-to-back on one post. These are not suitable where the median is narrower than 2.0 m because they deflect too much on impact. A monorail barrier with a box rail on top of the posts can sometimes be a good solution.

Concrete can be preferred in situations where higher performance is needed. Concrete barrier can be made in situ casted or by elements mounted together so it acts as a continuous barrier.

If possible, barriers 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 end treatment). Concrete barriers should be ramped down.

When constructing Median Barriers, the following must be observed:

1) Kerbs should never be used when median barriers are installed;
2) The consequences of no barrier are greater than barrier installations; and
3) Any clear zone problems relating to rigid objects and ditches in the median should also be resolved.

12.5.14 Terminals of Barriers, Guardrails and Bridge Parapets.

The ends of most barriers are very hazardous therefore, where possible, every effort should be made to terminate them where traffic speeds are low, and they must be constructed with leading and trailing terminal sections. In addition, short gaps (<80 m) should not be left in guardrails; they should be continuous.

There is no completely safe way of terminating barriers and guardrails, but the general advice is:

1) The end section should be stiffened by installing the support posts at closer spacing (e.g., half normal), and
2) The end section of the guardrail away from the edge of the shoulder should be flared until it is offset by at least 1m. A flare rate of at least 1 in 10 should be used. This reduces the risk of a direct impact; and
3) A special impact-absorbing terminal piece could be used.
4) Where possible the end of the barrier should be ramped down;
5) If approach speeds are unavoidably high, the end of the barrier should be protected by fitting a section (of at least 20 m) of semi-rigid guardrail.
6) Where space is restricted, such as at some bridge parapets, positive protection must be provided.

On a two-way road both the upstream and downstream ends of the guardrail/barrier need to be terminated in this way. The post spacing is normally halved for the first three to five lengths as shown in Figure 12-10.

One of the problems of ramped ends is that they can launch out-of-control vehicles into the air, with disastrous consequences. This problem can be reduced by ramping the beam down sharply and flaring. This 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.

12.5.15 Transitions from Guardrail to Bridge Parapets.

Collisions with the ends of bridge parapets can also be very severe. It is essential that these are shielded so that out-of-control vehicles are redirected along the face of the parapet. 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 (Figure 12-11). 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.

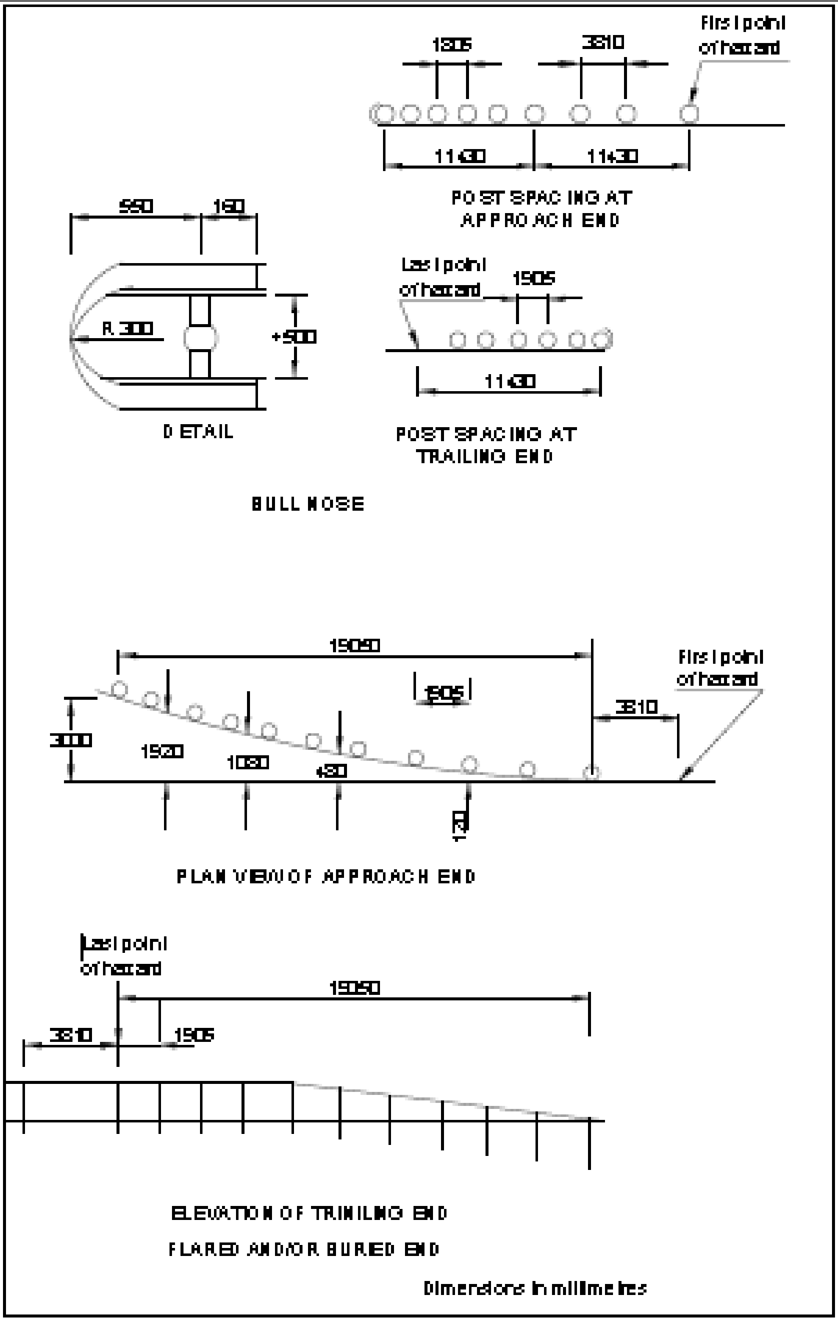


Figure 12-10: Guardrail End Treatment

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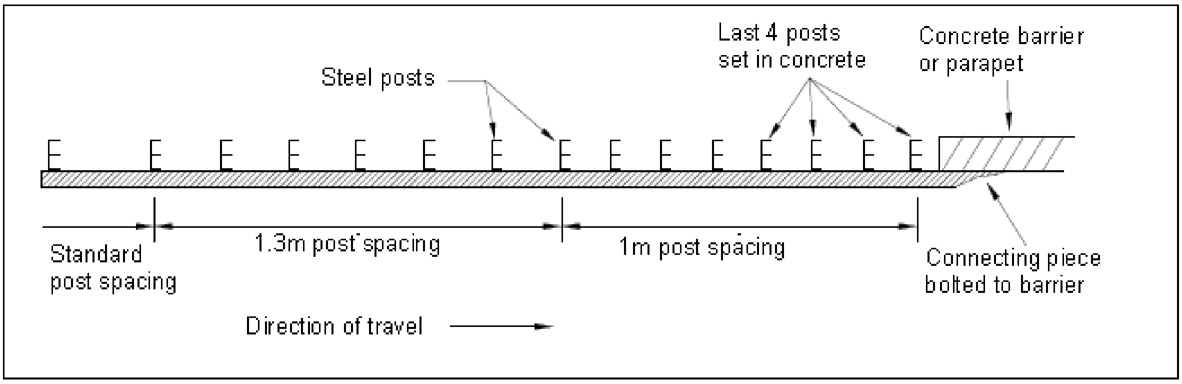


Figure 12-11: Typical Transition (W-beam Guard-rail to Rigid Object)

12.5.16 Pedestrian Barriers and Parapets

Uncontrolled pedestrian movements are a significant factor in urban traffic and safety problems. Pedestrian barrier can bring large improvements by segregating pedestrians from vehicular traffic. At intersections, a barrier can:

1) Reduce conflicts by channelling pedestrians to crossing points on the approaches.
2) Discourage buses, minibuses, and cyclists from stopping and parking within the intersection.
3) Discourage delivery vehicles from loading or unloading within the intersection; and
4) Discourage roadside vendors from occupying the road space in the intersection.

Other applications include:

1) Schools. A barrier can be used to prevent children from running into the road from the school gate.
2) Bus parks, cinemas, stadiums, etc – a barrier can channel pedestrian flows at areas of heavy pedestrian movement;
3) At ppedestrian crossings, underpasses', footbridges, a barrier helps to channel pedestrians to the crossing facility; and,
4) A 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.

Pedestrian barriers should:

1) Be strong and easily maintained.
2) Cause no serious damage to vehicles and the occupants when hit.
3) Not be hazardous to pedestrians, including the disabled.
4) Not interfere with visibility; and,
5) Look acceptable.

Pedestrian parapets 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. Figure 12-12 shows a typical design for a lightweight steel parapet. The minimum height for parapets is 1.2 m, but this should be increased to 1.4 m for cycleway, and 1.5 m 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. When the bridge has no facilities for pedestrians, the openings could be increased to 300 mm.

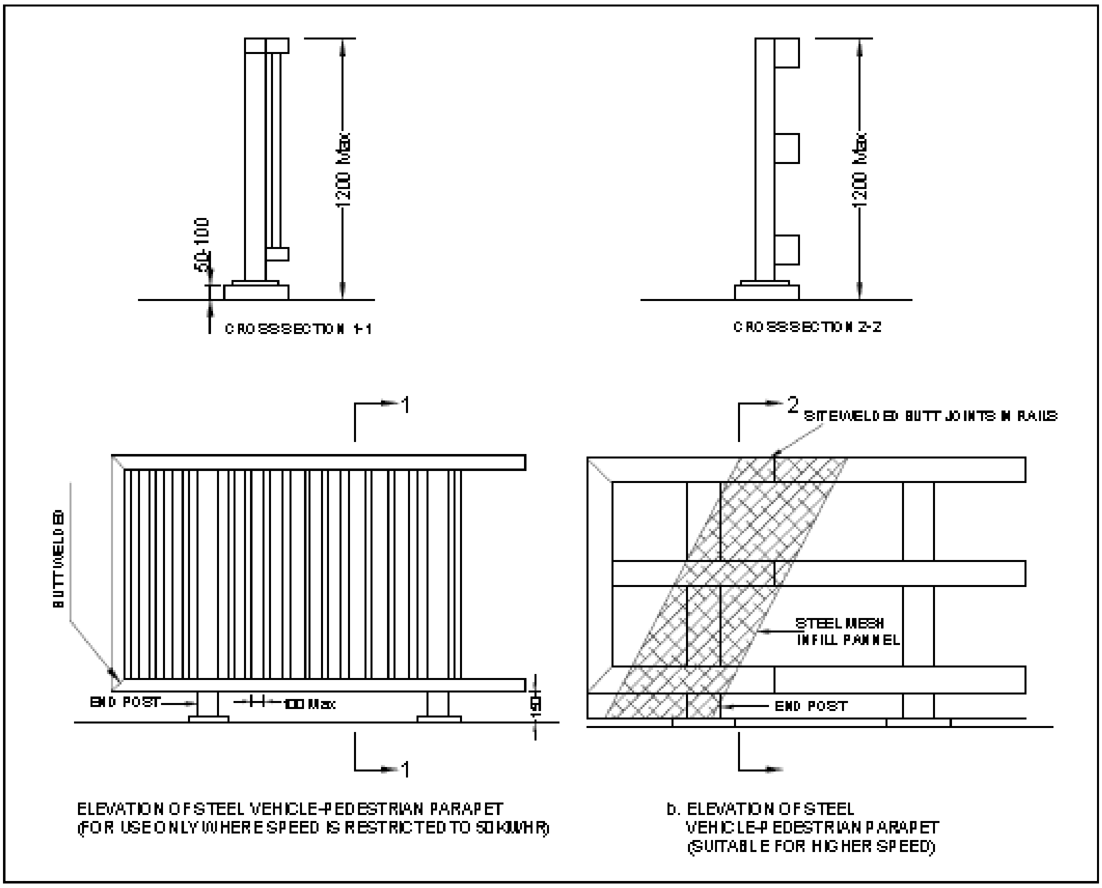


Figure 12-12: Details of a Vehicle/Pedestrian Parapet

12.5.17 Summary -of Use and Placement of Barriers

The potential use of barriers should be reviewed for several reasons:

1) In addition to the construction cost of the guardrail itself, there are other related costs. These include the need to construct a wider roadway to provide a platform for the construction of the barrier or guardrail. This is necessary, particularly in mountainous terrain and in rock cuts, and can add more to the construction costs than the cost of the barrier itself.

2) When traffic volumes are low, a cost/benefit analysis will often show that they are not cost effective.

3) Where mountainous terrain with steep side slopes is encountered, the conscientious driver will automatically adjust his behaviour to compensate for the safety hazards anticipated with the terrain, minimising the need for the guidance provided by a guardrail in some circumstances.

4) Guidelines rather than 'standards' usually govern the placement (or non-placement) of guardrails. Thus, they are not always an essential requirement.

5) The above factors can create problems with liability. Liability is minimised when guardrail placement is not a requirement. Conversely, if guardrails are placed but not maintained, the chances of problems associated with liability are much greater.

The conclusion from consideration of the above is that barriers/guardrails should not be constructed routinely where long and steep side slopes are encountered. However, a compromise in the interest of safety is to provide delineators at all such sections.

1) Short sections of guardrail should be employed on the approaches to all bridges. Without these, an errant driver can impact on the blunt end of the bridge rail, or proceed down the steep side slope into the river. Guardrails should be used at all four corners of the bridges, and should be of a parabolic end section configuration such that the guardrail begins a distance from the edge of the lane. The end treatment should not be blunt, but should be buried into the ground. Decreasing the spacing of the guardrail posts to provide a transition from the deformable rail section to the solid bridge railing should strengthen the section closest to the bridge railing. The end of the last rail should be doweled into the face of the bridge rail. Details are as indicated in the *Standard Detail Drawings*.

2) Where guardrails are employed, they should include reflectors to aid in the guidance of vehicles at night.

3) Safety barriers, or guardrails, are a compromise between the conflicting demands of construction costs and safety, and are themselves a hazard. To be warranted, guardrails should be a lesser hazard than that which they are intended to replace.

4) On existing roads an important warrant for guardrail installation is an adverse accident history. Another warrant for the installation of guardrails is to install these where the driver cannot anticipate the danger associated with the roadway segment. Prevention of an obvious fatal accident would come into this category such as on sharp bends in mountainous terrain to prevent running off the road into a deep gully.

5) In the case of new roads, it is necessary to consider whether an accident would be more likely with or without guardrails, and whether the outcome of such an accident is likely to be more serious without guardrails than with them. In certain areas where guardrails may be of benefit, for instance in mountainous terrain, it is probable that the additional width required for such an installation cannot be achieved without significant earthwork costs, often comprising rock materials.

6) Where guardrails are employed they need to be maintained. The responsible authority cannot be held liable for not installing guardrails, but could be held liable for an accident due to an un-maintained portion of guardrail.

12.6 Emergency Escape Ramps

12.6.1 Introduction

Where long, descending gradients exist, in addition to brake check areas (Section 12.4) the provision of an emergency escape ramp at an appropriate location is desirable for the purpose of stopping an out-of-control heavy vehicle away from the main traffic stream.

Highway alignment, gradient, length, and descent speed contribute to the potential for out-of-control vehicles. For existing highways, a field review of a problem grade may reveal damaged guardrail, gouged pavement surfaces or spilled oil, indicating locations where operators of heavy vehicles have had difficulty negotiating a downgrade.

While there are no universal guidelines available for new and existing facilities, a variety of factors are used in selecting the specific site for an escape ramp. Each location presents a different array of design needs requiring analysis of factors including topography, length and percent of grade, potential speed, economics, environmental impact, and accident experience. Ramps should be located to intercept the greatest number of runaway vehicles, such as at intermediate points along the grade.

Escape ramps may be built at any feasible location where the main road alignment is tangent. An escape ramp with an arrester bed should be built in advance of a singular point of the highway (interchange, curves that cannot be negotiated safely by a runaway vehicle, engineering structure, tunnel, service area, etc.) situated in a descending grade after a vertical gap of 130 m and in advance of populated areas.

12.6.2 Types

There are four types of emergency escape ramps. The first is a sand pile, the others are arrester beds, classified by grade (descending grade, horizontal grade, and ascending grade). They are illustrated in Figure 12-13. All function by application of the decelerating effect of loose material.

Sand piles, composed of loose, dry sand dumped at the ramp site are usually no more than 120 m in length. The influence of gravity is dependent on the slope of the surface. The increase in rolling resistance is supplied by the loose sand. Deceleration characteristics of sand piles are usually severe, but the sand can be affected by weather. Because of this characteristic, the sand pile is less desirable than the arrester bed. However, at locations where inadequate space exists for another type of ramp, the sand pile may be appropriate because of its compact dimensions.

Escape ramps are constructed adjacent to the carriageway. The use of loose material in the arrester bed increases the rolling resistance to slow the vehicle. Descending ramps can be rather lengthy because gravitational effects are not acting to help reduce the speed of the vehicle.

The preferred type of escape ramp is the ascending type with an arrester bed. Ramps of this type of use gradient resistance to advantage, supplementing the effects of the aggregate in the arrester bed, and generally reducing the length of ramp necessary to stop the vehicle. The loose material in the arresting bed increases the rolling resistance and serves to hold the vehicle in place on the ramp grade after it has come to a safe stop.

Each one of the ramp types is applicable to a particular situation and must be compatible with location and topographic controls at possible sites.

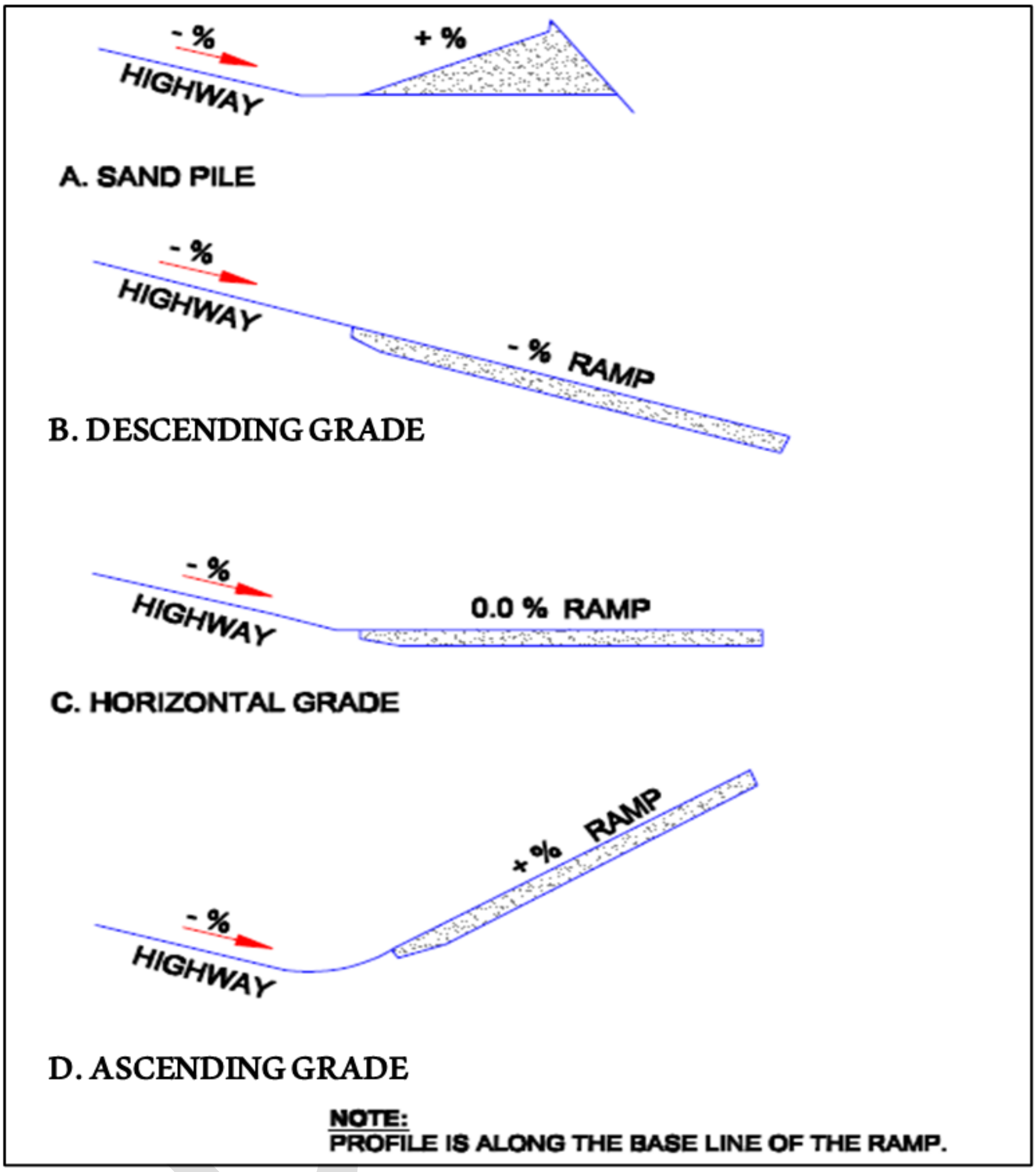


Figure 12-13: Basic Types of Emergency Escape Ramps

12.6.3 Design Considerations

The design and construction of effective e scape ramps requires the following considerations:

1) To safely stop an out-of-control truck, the length of the ramp must obviously be sufficient to dissipate the energy of the moving vehicle.

2) The alignment of the escape ramp should be tangential to the carriageway to relieve the driver of additional vehicle control problems.

3) The width of the ramp should be adequate to accommodate large heavy vehicles. Widths of ramps range from 3.6 to 12 m.

4) The in-fill material used in the arrester bed should be clean, not easily compacted, and have a high coefficient of rolling resistance. It should be single-sized natural or crushed coarse granular material or sand. Such material will maximize the percentage of voids, thereby providing optimum drainage and minimising compaction. The use of single-size aggregate also minimises maintenance, which must be performed by scarifying when the material is prone to compaction. Loose gravel or sand can also be used. A maximum particle size of 40 mm is recommended.

5) Contamination of in-fill material can reduce the effectiveness of the arrester bed by creating a hard surface layer at the bottom of the bed. Therefore, an aggregate depth up to 1.0m is recommended. To assist in decelerating the vehicle smoothly, the depth of the bed should be tapered from a minimum of 75 mm at the entry point to the full depth of aggregate in the initial 30 to 60 m of the bed.

6) A positive method of draining the arrester bed should be provided to avoid contamination of the arrester bed material. This can be accomplished by grading the base to drain, intercepting water prior to entering the bed or by edge drains. Geotextiles can be used between the sub-base and the bed materials to prevent infiltration of fines.

7) The entrance to the ramp must be designed so that a vehicle travelling at high speed can enter safely. Sight distance preceding the ramp should be provided and the full length of ramp should be visible. The angle of a departure for the ramp should be small. The main roadway surfacing should be extended to a point at the bed entrance such that both front wheels of the out-of-control vehicle will enter the arrester bed simultaneously.

8) Advance warning signs and markings are required to inform a driver of the existence of an escape ramp and to prepare well in advance so that there will be enough time to decide whether or not to use the escape ramp. It should indicate whether the ramp is occupied or not. Regulatory signs near the entrance should be used to discourage stopping or parking at the ramp.

To determine the distance required to bring a vehicle to a stop with consideration of the rolling resistance and gradient resistance, the following equation may be used:

$$L = \frac{100 * V^2}{254} * (R.G)$$

Equation 11-2: Distance required to bring a vehicle to a stop.

Where:

L = *distance to stop (i.e., the length of the arrester bed), m,*

V = *entering velocity, km/h,*

G = *% gradient of ramp,*

R = *rolling resistance expressed as equivalent % gradient*

For example, assume that topographic conditions at a site selected for an emergency escape ramp limit the gradient of an ascending ramp to 10 %. The arrester bed is to be constructed with loose gravel for an entering speed of 140 km/h. Using Table 12-7, and Equation 11-, R is determined to be 10 %. The length necessary is determined from the equation. For this example, the length of the arrester bed is about 385 m.

Table 12-7: Rolling Resistance of Roadway Surfacing Materials

Surfacing Material	Rolling Resistance (kg/100 kg GVM)	Equivalent Grade (%)[1]
Crushed aggregate, loose	50	5
Gravel, loose	100	10
Sand	150	15
Pea gravel	250	25

Note 1 Rolling resistance expressed as equivalent gradient.

A plan and profile of an emergency escape ramp with typical appurtenances is shown in the **Standard Detail Drawings.**

Where a full-length ramp is to be provided with full deceleration capability for the design speed, a 'last chance' device should be considered when the consequences of leaving the end of the ramp are serious. The use of a ramp end treatment should be designed with care to ensure that the advantages outweigh the disadvantages.

Mounds of in-fill material between 0.6 and 1.5 m high with 1:1.5 slopes have been used at the end of ramps in several instances as the 'last chance' device.

12.6.4 Maintenance

After each incident the in-fill materials should be reinstated. The arrester beds should be inspected periodically, and the in-fill materials replaced as necessary.

12.7 Railway Grade Crossings

The horizontal and vertical geometrics of a highway approaching an at-grade railway crossing should be constructed in a manner that does not require a driver to divert attention from roadway conditions. If possible, the highway should intersect the tracks at a right angle with no nearby intersections or driveways (Figure 12-14). This layout enhances the driver's view of the crossing and tracks and reduces conflicting vehicular movements.

Where this is not possible, the angle of skew must not be greater than 45º. Crossings should not be located on either highway or railway curves. Roadway curvature inhibits a driver's view of a crossing ahead and a driver's attention may be directed towards negotiating the curve rather than looking for a train. Railway curvature may inhibit a driver's view down the tracks from both a stopped position at the crossing and on the approach to the crossings.

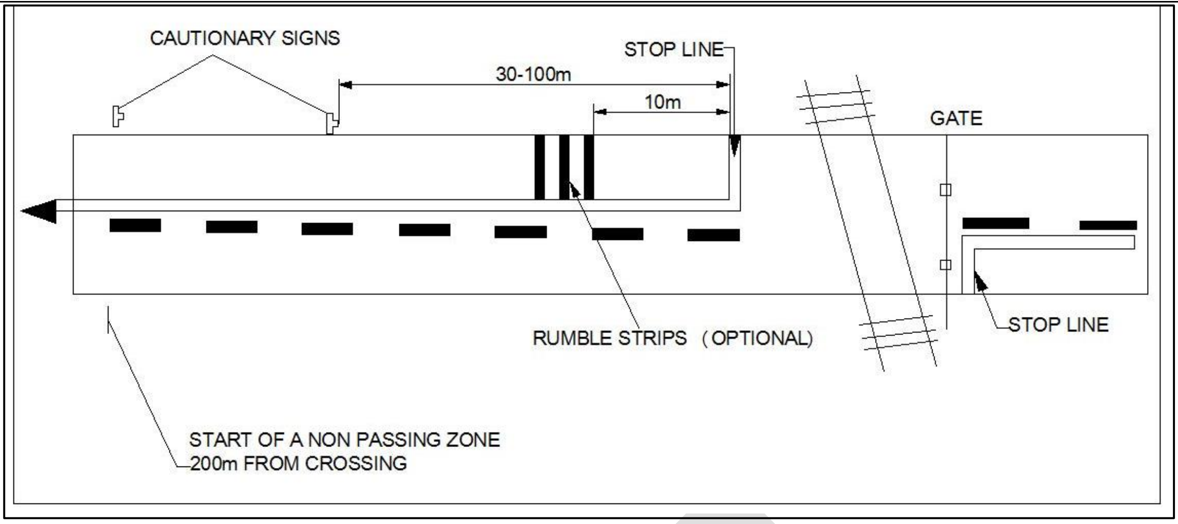


Figure 12-14: Railway Crossing Details with Rumble Strips

Where highways that are parallel with main tracks intersect highways that cross the tracks there should be sufficient distance between the tracks and the highway intersections to enable highway traffic in all directions to move expeditiously and safely.

It is desirable that the intersection of the highway and railroad be made as level as possible from the standpoint of sight distance, ride quality, braking and acceleration distances as in Figure 12-15. Vertical curves should be of sufficient length to ensure an adequate view of the crossing, and crest and sag curves are the same as for the roadway design. The sight distance requirements down the tracks are similar to those for a roadway junction.

It is necessary to install signing to provide a safe crossing. Traffic control devices for railroad-highway grade crossings consist of signs and pavement markings. Standards for design and placement of these devices are covered in the *Standard Detail Drawings*.

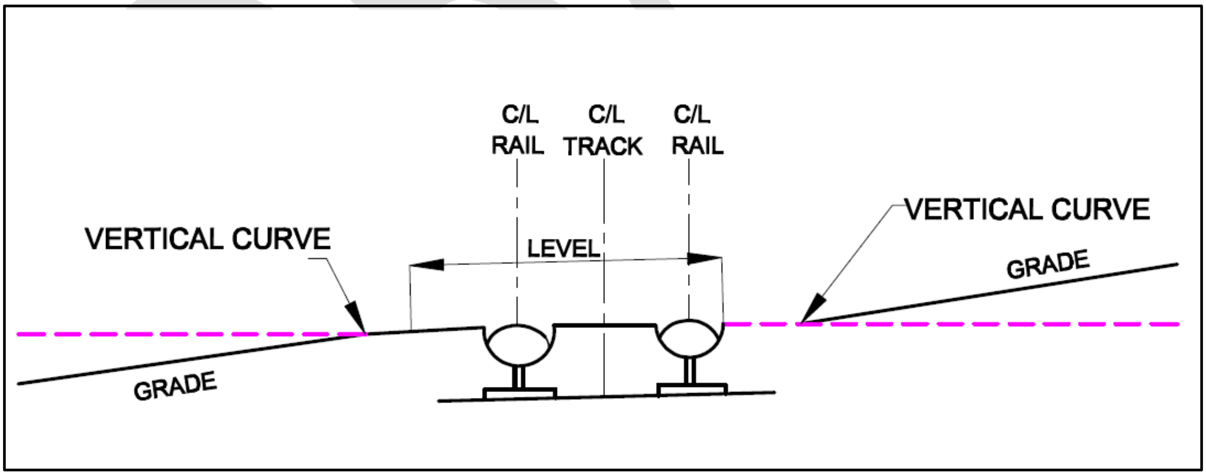


Figure 12-15: Railway Crossings, Details on Vertical Curve

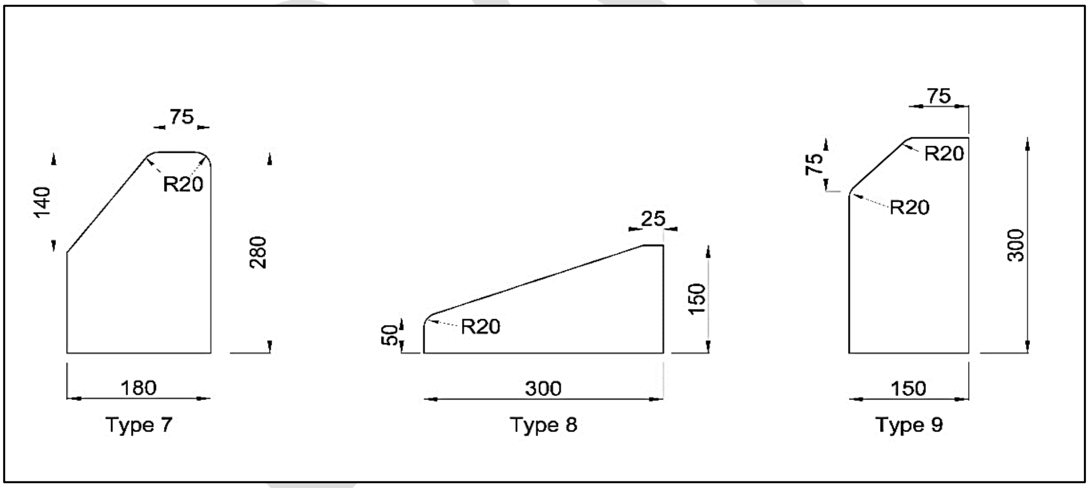
12.8 Kerbs

It is envisaged that all streets will be kerbed, whether surfaced or not. This is to facilitate drainage, but also to demarcate the people space, control traffic and ensure a neat amiable environment. Kerbing may be cast in situ, be precast concrete elements or be hewn from natural rock quarries. Kerbing along lower order mixed-use streets should preferably be of the mountable type, with semi-mountable kerbing at intersection bell-mouths and barrier kerbing at bus stops to ease embarking and disembarking. Precast or cast in-situ kerbing should preferably conform to the dimensional standards of SABS 927-1969 Figures 7 to 9 as illustrated in Figure 12-16.

Where any kerbing other than mountable kerbing is provided, ramps should also be provided for wheelchairs, other disabled persons, and prams. Barrier and semi-mountable kerbing are sometimes offset by a concrete channel of 150 mm width from the lane edge to ease construction, but a concrete channel is seldom provided with mountable kerbing.

Mountable kerbing normally comprises 300 mm wide concrete strips rising 100 mm over its width. If cast in situ, suitable expansion joints must be provided at intervals not exceeding 1.5 m and the sections cast alternately. Suitably shaped in-situ constructed transition sections are required between mountable and barrier or semi-mountable kerbing.

Kerbing hewn from natural rock should approach semi-mountable kerbing in shape and could be used to replace mountable kerbing.



Semi-mountable kerb Mountable kerb Barrier kerb

Figure 12-16: Dimensional standards of kerbing

Kerb types 7 and 9 should be provided with an in-situ concrete backing at every joint. Type 8 kerbing should be provided with an in-situ concrete bedding of 125 mm thickness.

13 Roadside Facilities and Amenities

13.1 Introduction

There are a number of amenities and facilities associated with the road network and/or road travel that that must be designed and built to standards compatible with those presented in this manual. This includes accommodating all the utilities essential to urban life as well as services to or for travellers themselves.

13.2 Public Utilities

13.2.1 General

All highway improvements, whether upgrading within the existing road reserve or an entirely new road reserve, generally entail new facilities and or adjustment of existing facilities. The costs vary considerably depending on the location of project. Table 12.1 indicates the utilities that are likely to be involved.

Table 13-1: Public Utilities

Surface Utilities	Underground Utilities
Sanitary sewers	Buried telephone lines
Water supply lines	Gas pipelines
Overhead power and communication lines	Power transmission cables
Drainage and irrigation lines	Storm drains and sewers
Street lighting	

The utility authorities responsible will have their own manuals and working practises but the following notes provide some general guidance.

13.2.2 General Considerations

The following factors should be considered in the location and design of utility installations.

1. Utility lines should be located to minimise the need for later adjustment, to accommodate future highway improvements, and to permit servicing such lines with minimum interference to traffic.
2. Longitudinal installation should be located on a uniform alignment as near as practicable to the road reserve to provide a safe environment for traffic operation and preserve space for future highway or street improvements of other utility installations.
3. To the extent feasible and practicable, utility line crossings of the highway should cross on a line generally at a right angle (90 degrees) to the highway alignment. Those utility crossings that are more likely to require future servicing should be encased or installed in tunnels to permit servicing without disrupting the traffic flow.
4. The horizontal and vertical location of utility lines within the highway road reserve should conform to the clear roadside policies and specific conditions for the

particular section involved. Safety of the travelling public should be a prime consideration in the location and design of utility facilities on the highway reserve.

5. Sometimes attachment of utility facilities to highway structures, such as bridges, is a practical arrangement and may be authorized. Electric and Telephone Cables and water main placing in one trench should be done according to Figure 13-1 unless otherwise stated by the concerned institutions.
6. All utility installations on, over, or under the highway reserve and attached structures should be of durable materials designed for long service-life, relatively free from routine servicing and maintenance, and meet or exceed the requirements of the applicable industry codes or specifications.
7. On new construction in road locations no utility should be situated under any part of the road, except where it must cross the highway.
8. Utility poles and other above ground utility appurtenances that would constitute hazards to errant vehicles should not be permitted within the highway clear zone. The only exceptions permitted would be where the appurtenance is breakaway or could be installed behind a traffic barrier erected to protect errant vehicles from some other hazard.
9. The placement of the required infrastructure for electricity, water, sewage and telecommunications is shown in Appendix E. Figure 12.1 is an example.

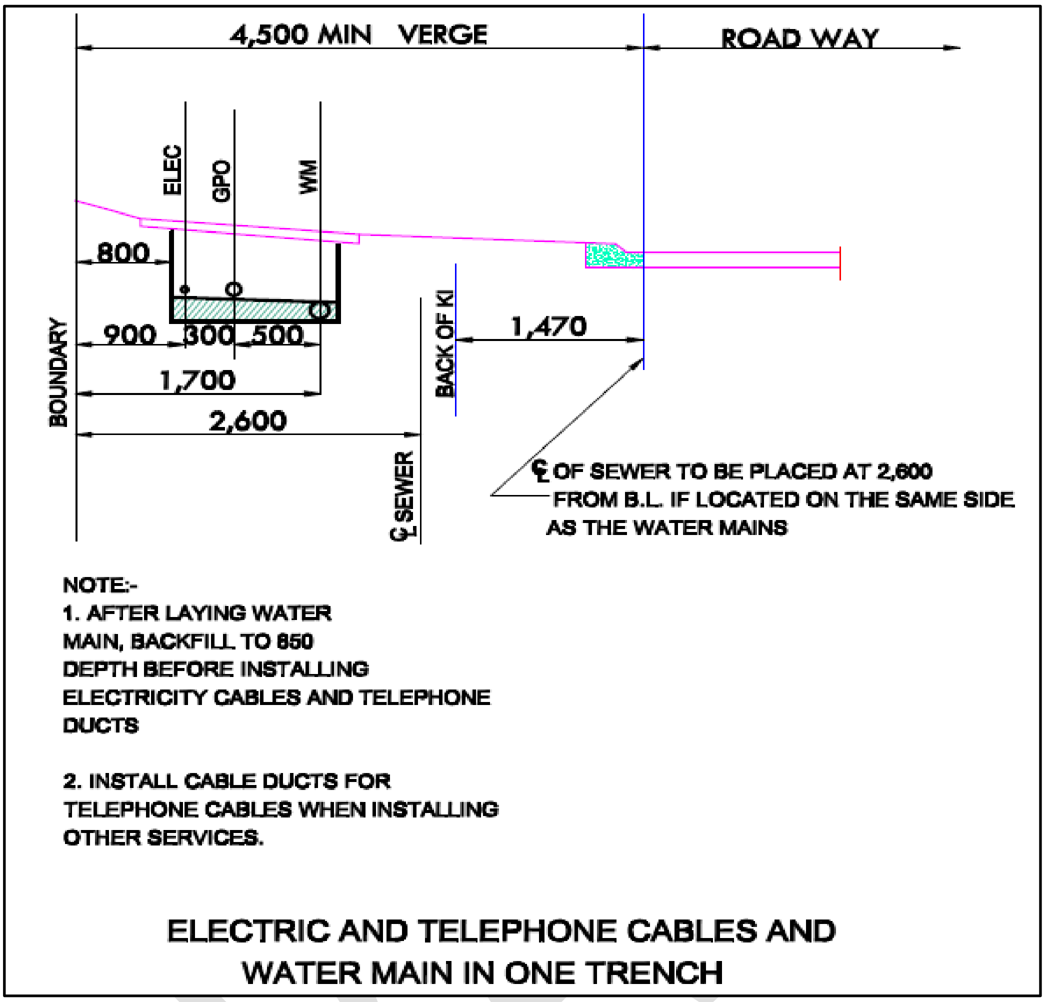


Figure 13-1: Utilities Placement Detail

13.3 Stopping Places

13.3.1 Roadside Vending

Roadside vending should be confined to official stopping and parking locations such as official rest areas and service centres.

13.3.2 Service Centres

The primary and usually the only service centres are official centres providing fuel, shopping facilities, restaurents and parking for short term and for longer term customers. They are designed and built to the normal standards of such trading areas and the exits and entrance areas are of a particularly high standard to cater safely for the wide range of vehicles and travel directions that the traffic requires.

13.3.3 Rest Areas and Scenic Overlooks

Safety rest areas are roadside areas with parking facilities separated from the roadway. They are provided for vehicle drivers to stop and rest for short periods in an atmosphere that affords a distinct change from the monotony of driving.

A scenic overlook is a roadside area provided for drivers to park their vehicles, beyond the shoulder, primarily for viewing the scenery or for taking photographs in safety. The attraction of such a facility depends upon the presence of scenic and historical points of interest. The facilities should be designed to avoid marring the landscape.

Site selection for safety rest areas and scenic overlooks should consider the scenic quality of the area, accessibility, and adaptability to development. Site plans should be developed that should include proper and safe location of entrances and exits, road signs and markings, acceleration and deceleration lanes as required, and parking areas for cars and trucks. They may also include certain types of rest facilities (benches, tables, shelters, drinking fountains, rest rooms).

Where such facilities are specified, the average distance between rest areas should be 15 to 25 km. As far as possible, such facilities should be avoided where adjacent roadway gradients are in excess of 4%.

14 Modelling and Earthworks Computation

14.1 Introduction

In modelling the road prism, the horizontal alignment and vertical alignment and the selected cross-sectional elements are combined and superimposed on the existing ground profile. This then forms the Digital Terrain Models (DTM) from which design drawings and quantities can be derived. Several software are available on the market for performing this task and some of the standard input parameters are described below.

14.2 Cross-section Template

The cross-section template is a graphical representation of the road typical section superimposed on the existing ground terrain model. The template establishes the basis for computing the earthwork quantities. A project can have several templates applied on different sections.

14.3 Typical Section Details

The proposed typical sections for a project define the basic geometric criteria of the template and include:

1) No of lanes and widths
2) Cross-falls
3) Super-elevation development parameters
4) Side cut and fill slopes
5) Location of ditches and type
6) Edge kerbing where specified
7) Pavement details

14.4 Earthworks

14.4.1 Cut-fill balance

In urban projects, or rural works on flat terrain, it is seldom possible to balance cut and fill. In rolling terrain, earthwork costs generally are minimised by balancing cut and fill quantities, after adjusting for:

1) stripping
2) removal of materials unsuitable for construction
3) compaction or bulking factors
4) depth of topsoil removal.

The first option for achieving earthworks balance is to adjust the vertical alignment. The second option can be to shift the alignment. Minor changes can be achieved by altering verge widths or batter slopes. The optimum solution usually involves adjustments both to the horizontal and the vertical alignment.

The limits of the balance area have to be chosen with care to ensure that total project costs are minimised. For example, earthworks from the hills could be used as fill within the flat plains, but the limit will depend on the on-site costs of fill material from other sources, including the costs of cartage. If material won from cuts is suitable for pavement material, it must be deducted from common earthworks and added to the pavement material quantity.

For large road projects a comprehensive geotechnical investigation should be undertaken before the grade line is fixed. The materials report includes advice on the suitability of fill materials, batter stability and possible sources of imported fill.

Designers should seek to gain agreement to the proposed use of fill materials from the Road Authority or responsible agency.

14.4.2 Earthwork Quantities

Earthworks quantities are usually extracted using the average end area method. Designers should be aware of the intrinsic limits of accuracy due to variations of the terrain between surveyed cross-sections, and variations from a truly prismoidal shape. The calculated volume may vary from the actual volume by 5 to 10% from these factors alone.

Where the alignment lies on a small radius curve, further quantity adjustments are required because the ends of the prism are not parallel. No adjustments are required for computer programs that extract volumes using vertical triangular prismoids.

Factors that affect earthwork quantities include:

1) Depth of topsoil to be removed prior to placement of fill (the stripping depth).
2) Quantity of material that has to be removed to provide a stable base for the pavement or embankment.
3) The compaction factor, that is, the ratio in volume between one cubic metre of in situ material and the same material after placement and compaction. Cohesive materials commonly occupy less volume after compaction, while rock may occupy more volume after excavation and placement.
4) Material which is unsuitable for embankment construction including:
    i) topsoil and other material with organic content
    ii) large boulders
    iii) excavated hard rock which may be uneconomical to crush to a size which can be compacted.
    iv) any unstable or expansive material to be carted to waste.
5) Flattening of embankment batter slopes outside stability limits to provide for safety and maintenance requirements.
6) It is preferable that ground survey be used for detailed design, in order to eliminate this factor and improve accuracy.
7) Subject to the construction specification, possible use of material otherwise classed as unsuitable material in noise attenuation mounds or other land forming.

When calculating earthwork quantities, designers should document all factors that have been taken into account in the computations.

15 Design Quality Assurance

15.1 General

In the execution of the design brief judicious design control must be exercised and controls must be exercised on all the design outputs which consists of the following:

1. Design calculations;
2. Drawings;
3. Schedules of Quantities;
4. Project Specifications.

For each of the above specific design controls are exercised and hold points are established before the design is completed.

15.2 Design Calculations

All design calculations for each specialist field shall be reviewed and checked by the designer and a senior design engineer or project leader. These checks shall be to ensure that the correct design standards have been adhered to and that overall design requirements have been attained in the design. Design checklists shall be placed on record on the design calculation file and the project file for audit of the project quality assurance plan. Checklists with regard to the design shall be drawn up.

Upon completion of the design the check lists shall be made available to the Design Manager and Project Director at the final Design Review meeting. This must be carried out before the design drawings and documentation are submitted for approval.

15.3 Drawings

The designer shall prepare drawings which are clear, accurate and with enough details to meet the intended purposes. Depending on their purpose the drawings can be classified as Preliminary Design Drawings for feasibility studies, Detailed Engineering Drawings for tendering and construction purposes and As-built Drawings for Archive purposes.

Intermediate drawings submitted as Preliminary Drawings and Draft Final Drawings in the Detailed Engineering Design Projects shall be stamped to indicate that they are yet not ready for implementation stages.

For the purpose of easy identification and storage, the drawings should be provided with standard numbering.

All drawings shall be checked in a systematic way starting with the person responsible for the production of the specific drawing, the designer and finally the Specialised Engineer or responsible Director.

The checks shall be carried out to ensure that the design has been implemented correctly onto the drawings and that the drawings meet the requirements of the Employer. All drawings shall further be checked to ensure that they have been prepared in accordance with the set requirements

Drawing checklists shall be prepared and placed on record on the design calculation file and the project file for audit. The following drawing checklists have been drawn up for the drawings of each of the following elements of the design: [Insert list]

Upon completion and finalisation of the drawings and checklists shall be made available to the Design Manager and Project Director at the Final Review Meeting. This must be carried out before the design drawings and documentation are submitted for approval.

15.4 Project Specifications

The project specifications shall be drawn up by the various specialised engineers for each project and then compiled by the Project Leader who shall check each section against the requirements for each design. The Project Specifications shall be checked by the Project Document Co-ordinator and the applicable checklists shall then be made available to the Design Manager and Project Director at the Final Review Meeting. The checklists shall be handed in for record keeping along with the other design checklists.

15.5 Schedule of Quantities

All Schedules of Quantities are to be prepared by the specialised designers and after checking, forwarded to the Project Leader for inclusion in the project documents. After compilation by the Project Leader, the contents of the schedules shall be scrutinised and checked by the Project Document Co-ordinator who shall table the final draft of these schedules to the Detail Design Manager and Project Director at the Final Review Meeting. A single check list listing all the sections of the standard specification is to be used for this purpose and shall be handed in for record keeping along with the design and drawing check lists.

15.6 Design Changes

Design changes initiated either at the time of review by the Project Director, Design Manager and the Employer shall be made and noted on the Design or Drawing checklists and signed off when completed. Drawing revisions shall be noted in the “revisions” block on the drawing title block. The nature of the revision after approval by Employer shall be annotated in the revision block and the information to be revised shall be neatly crossed out and the new dimension, detail or information shall be added above or alongside the original. Where substantial changes have to be carried out to a drawing the complete drawing shall be amended and re-issued with a note in the amendment block to state that the drawing has been amended and replaced.

15.7 Advisory Notes

Advisory Notes shall be prepared and issued by the Project Director or Design manager for the approval of the Project Director in order to establish and manage all relevant design information, design programme matters, pro-formas for Project documents and any other relevant information required by Designers and Sub-designers for the execution of the Design. The information contained in the Advisory Notes shall be mandatory to designers who shall implement this information implicitly and in all respects.

15.8 Design Reviews

Design review meetings will be convened by the Design Manager and will be chaired by the Project Director. The meetings shall be held on a regular basis during the design of each project package and programme progress and design review will be addressed. Interfacing between the project leader and specialised fields will also be reviewed and the Project Director will address co-ordination of these fields.

A final design review meeting will be held 3 or 4 days before the completion of the detail design and the design, drawings, schedules of quantities and project specifications will be reviewed by the Project Director and Design Manager. The Project Director will issue instructions for changes, amendments or revisions, which he may deem necessary before the handing in of the detail design documentation.

Programme and progress review meetings will be held with all sub-designers on a regular basis and shall be convened by the Design Manager. The Project Director or in his absence, the Design Manager, shall chair these meetings.

15.9 Audits

Internal audits of designers and sub-designers shall be carried out in accordance with the policies and guidance provided by the road agency.

16 Departures from Standards

Where the designer departs from a standard, written approval must be obtained from the Chief Engineer- Roads. The Designer should submit the following information:

1) The number, name, and description of the road section.
2) The design parameter for which a Departure from Standards is desired.
3) A description of the standard, including normal value, and the value of the Departure from Standards.
4) The reason for the Departure from Standards, and
5) Any mitigation to be applied in the interests of safety.
6) Justification for departure

The Designer must submit all major and minor Departures from Standards to the respective regional directorate for evaluation. If the proposed Departures from Standards are acceptable, the Departures from Standards will be submitted to the Quality Assurance, Road Inspection and Safety Directorate for final approval.

An appropriate proforma is shown below based on the proposed modifications to the appropriate part of the manual or specifications that, if approved, will allow the project to proceed. At this stage this is not a permanent change but merely a temporary one for the affected project.

DEPARTURE FROM STANDARDS – Approval form

Project Name:

17 Bibliography

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Giummarra G (2001). ***Road Classifications, Geometric Designs and Maintenance Standards for Low Volume Roads***. Research Report AR 354, ARRB Transport Research Ltd, Vermont, South, Victoria, Australia.

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