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Road Design Manual

Volume 1: Geometric Design

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

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Foreword

This manual was developed by the Ministry pursuant to The Fourth Schedule of the Constitution which assigns to the National Government the functions and powers of setting standards for the construction and maintenance of all public roads including those under the County Governments.

It is part of a series of manuals that replace the first generation of road manuals developed in the first and second decades after independence. The second-generation road manuals were developed to cover the entire road project cycle covering planning, appraisal, design, contracts, construction, maintenance, operations and monitoring. The series incorporates best practices, climate change considerations, and recent technologies to enable safe, secure, and efficient road infrastructure.

Under the Kenya Vision 2030, infrastructure expansion and modernisation are one of the foundations for the realisation of Kenya's economic, social and political transformation into a rapidly industrialising middle-income country. The plan envisages an integrated, safe and efficient transport and communication infrastructure network consisting of roads, railways, ports, airports, waterways, and telecommunications infrastructure.

The strategies to be pursued under the Vision 2030 plan to improve infrastructure services and to maximise the economic and social impacts of infrastructure development and management include strengthening the institutional framework for infrastructure development and maintenance; raising efficiency and quality of infrastructure projects; Enhancing local content of identified infrastructure projects to minimise import content; Benchmarking infrastructure facilities and services provision with globally acceptable performance standards; and, Implementing infrastructure projects that will stimulate demand in hitherto marginalised areas.

The first three 5-year Medium Term Plan (MTP) under the Vision 2030 from 2008 to 2022 targeted construction of 1,950 km, 5,500 km and 10,000 km of new paved roads totalling 17,450 km. This was a massive infrastructure development program intended to double the paved road network in 10 years compared to the 8,600 km developed from independence in 1963 to 2008.

Implementing MTP I to III resulted in the construction of 14,000 km of paved roads, which was the phenomenal expansion of the paved road network that extended the paved road coverage to the Arid and Semi-Arid regions previously neglected. However, some key milestones of the Vision 2030 goals have not been realised. This has been due to internal and external challenges. External challenges included: climate change - prolonged droughts and floods; the emergence of the COVID-19 pandemic; global supply chain disruptions; exchange rate volatility; and rising interest rates in the leading economies.

The internal challenges included: inadequate road maintenance equipment; pavement overloading by heavy goods vehicles; huge maintenance backlog of the road network; low contracting and supervision capacity particularly in the Counties; poor quality control and assurance of works; congestion in urban areas; encroachment on road reserves; high cost and delays in payments of land acquisition; lack of harmonisation of cross-border transport regulation and operational procedures; rapid urbanisation; increased traffic volume with the exponential growth of motorcycle traffic; high cost/delays in relocation of utilities and services along and across road reserves; inadequate funding of projects and programs; and, delay or default in payments for goods, services and works.

The inability to address some of the above challenges is largely due to intrinsic systemic challenges which include: inadequate funding of research on roads and road construction materials; poor planning; lack of internalization of policies and processes; lack of respect for professionals and standard practice; ineffective coordination in the implementation of programs and projects; lack of inclusivity in engagement of manpower and procurement of services and works; and, lack of unity of purpose and synergy in development and delivery of projects.

The infrastructure expansion from 2008 – 2022 did not build the local contracting capacity (Micro, Small and Medium Enterprises) rather it destroyed them due to delays or defaults in payments of invoices at both national and county levels.

The implementation of MTP III came to an end on 30th June 2023, ushering in the implementation of the Fourth Medium Term Plan (MTP IV), which has been aligned with the aspirations of the Kenya Vision 2030 and the Bottom-Up Economic Transformation Agenda (BETA) planning approach and its key priorities.

BETA is the Government's transformation agenda geared towards economic turnaround through a value chain approach. BETA has targeted sectors with the highest impact to drive economic recovery and growth. This will be achieved by bringing down the cost of living; eradicating hunger; creating jobs; expanding the tax base; improving foreign exchange balances; and inclusive growth. BETA ensures rational resource allocation by eliminating wastage of resources occasioned by duplication, overlaps, fragmentation and ineffective coordination in the implementation of programmes and projects.

The Fourth Medium Term Plan key priorities are clustered under five key sectors, namely: Finance and Production; Infrastructure; Social; Environment and Natural Resources; and Governance and Public Administration. The infrastructure sector seeks to: enhance transport connectivity by constructing 6,000km of new roads, maintaining rural and urban roads, rail, air and seaport facilities and services; expanding communication and broadcasting systems; and promoting the development of energy generation and distribution by increasing investments in green energy (geothermal, wind, solar and hydro). The infrastructure gap is expected to be bridged by promoting economic participation of the private sector through public-private partnerships in the financing, construction, development, operation and maintenance of infrastructure.

BETA entails a shift of focus to fundamentals in project planning and implementation which include: respect for technical input, regulations and standard practices; adherence to project life cycle i.e., planning, feasibility studies and design before procurement of works; public and stakeholder consultation; procurement within budgetary ceilings; shifting focus during project implementation from the finished product ‘black top’ to the construction of the foundation; building local capacity particularly MSMEs by ensuring prompt payments; and capacity building at all levels to enable internalization of policies and processes.

The first generation of the road manuals were used for 35 to 45 years. It is my sincere hope that the second generation of the road standards which have been developed in alignment to the BETA approach will guide in solving most of the above challenges and those expected to emerge in the next 50 years. Implementation of the manuals will enable achievement of the BETA aspirations which include inclusive growth; creation of sustainable employment; building of MSMEs; climate change adaptation and realisation of the UN SDGs; enhanced efficiency in management of infrastructure and transport system; and, laying the foundation for the next national long-term plan at the end of the Vision 2030.

The second generation of the road manuals and specifications was prepared through an extensive consultative process involving review of existing standards and consultation with stakeholders, Ministerial Departments and Agencies, the Technical Task Force, public consultation and stakeholders’ workshops at review and drafting phases, and the National Steering Committee.

On behalf of the Government of Kenya, I would like to thank the African Development Bank for its support in 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 Road manuals updating exercise.

Hon. Davis K. Chirchir, E.G.H
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.

Eng. Joseph M. Mbugua, CBS
Principal Secretary, State Department for Roads

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:
The Principal Secretary, State Department for Roads, Ministry of Roads and Transport, Works Building, Ngong Road, P.O. Box 30260 - 00100, NAIROBI Email: ps@road.go.ke

A secured PDF copy maybe downloaded from: www.roads.go.ke/downloads

Notification of Errors and Requests for Amendments

While all care and consideration has been applied in the compilation of this document, the Ministry accepts no responsibility for failure in any way related to the application of this manual or any reference documents cited in it.

Requests for edits and corrections can be freely sent to the following address:
The Principal Secretary, State Department for Roads, Ministry of Roads and Transport, Works Building, Ngong Road, P.O. Box 30260 - 00100, NAIROBI Email: ps@road.go.ke

Amendments Request Form:

Request No.	Name	Organisation	Chapter	Page	Section/ Clause	Ref. to: Figure/ Table/	Type of Request	Request

Type of request: General – G; Editorial – E; Technical - T

Amendments to Date:

Amendment No.	Description	Amendment Effective Date	Amendment Approved by:

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A National Steering Committee was set up and chaired by the Permanent Secretary, Ministry of Roads and Transport, with the following membership: Principal Secretary for Devolution, Office of the Deputy President; Chief Executive Officer, Inter-Governmental Relations Technical Committee; Chief Executive Officer, Council of Governors; Managing Director and Council Secretary, Kenya Bureau of Standards; Director, National Transport and Safety Authority; Director General, Kenya Roads Board; Director General, Kenya Wildlife Services; Chief Executive Officer, Engineers Board of Kenya; Director General, Kenya Rural Roads Authority; Director General, Kenya Urban Roads Authority; President, Institution of Engineers Kenya; Director Policy, Strategy and Compliance; Kenya National Highways Authority; Chief Engineer, Roads Division, State Department for Roads; Chief Engineer, Materials Testing and Research Division, State Department for Roads.

The technical work was undertaken under the guidance of a Technical Task Force, chaired by Eng. David Maganda, with the following gazetted members: Francis Gichaga (Prof.) (Eng.), Andrew Gitonga (Eng.), Timothy Nyomboi (Dr.) (Eng.), Rosemary Kungu (Eng.), Charles Obuon (Eng.), Sylvester Abuodha (Prof.) (Eng.), Samuel Kathindai (Eng.), Nicholas Musuni (Eng.), Charles Muriuki (Eng.), Tom Opiyo (Eng.), John Maina (Eng.), Fidelis Sakwa (Eng.), Daniel Cherono (Eng.), Maurice Ndeda (Eng.), Theo Uwamba (Eng.).

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Abbreviations

AADT	Annual Average Daily Traffic
AASHTO	The American Association of State Highway and Transportation Officials
ADT	Average Daily Traffic
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 Hourly 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
KSI	Killed or Seriously Injured
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
PRC	Point of Reverse Curvature
PSD	Passing Sight Distance
PT	Point of Tangency. Where a Circular Curve ends. Also designated EC
PT	Pedestrian Traffic
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 road.
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	The part of the road which is trafficked by road users under normal operation. This includes vehicular traffic lanes, pedal cycle lanes, auxiliary traffic lanes passing places, lay-bys and bus-bays. The carriageway excludes shoulders and hard strips.
Catchwater Drain	A longitudinal 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.

Glossary of Terms *(continued)*

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 road	A collective term for interchange links, link roads, ramps and loops designed as part of full grade separated junction.
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.
Cross-section	The assembly of the various components of the road between the road boundaries, measured at right angles to the direction of travel.
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.

Glossary of Terms *(continued)*

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 85th 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.
Expressway	A multi-lane, 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.
Eye Height	Assumed height of a driver's eyes above the surface of the roadway used for the purpose of determining sight distances.
Free Haul	Maximum distance through which excavated material may be transported without added cost above the unit bid price.
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.

Glossary of Terms *(continued)*

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.

Glossary of Terms *(continued)*

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.

Glossary of Terms *(continued)*

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.

Glossary of Terms *(continued)*

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 Authority	Means the Road Authorities in terms of the Kenya Roads Act, 2007 and Urban Areas and Cities Act, 2011.
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	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.

Glossary of Terms *(continued)*

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 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 5 m contours crossed per km **Rolling** 11-25 5 m contours crossed per km **Mountainous** >25 5 m 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 straight 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.

Glossary of Terms *(continued)*

Trunk Road Arterial 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 Channelised 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 i.e. a town, municipality or a city as defined in the national statutes. May also be a link traversing an unbuilt up area between two adjacent urban areas, hence displaying urban operational characteristics.

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.

Walkway The rural equivalent of the urban sidewalk.

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 to enable 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
A. General	**Procedures and Standards Manual**	**PSM**
	1. General	
	2. Policies	
	3. Procedures Guidance	
	4. Codes of Practice	
	5. Guidelines	
	6. Product/Testing Standards	
B. Planning	**Network and Project Planning Manual**	**NPM**
	1. Road Classification	
	2. Route/Corridor Planning	
	3. Route/Corridor Planning	
	4. Highway Capacity	
	5. Project Planning	
C. Appraisal	**Project Appraisal Manual**	**PAM**
	1. Environmental Impact Assessment and Audit	
	2. Social Impact Assessment	
	3. Traffic Impact Assessment	
	4. Road Safety Audits	
	5. Project Appraisal	
	6. Feasibility Studies	
D. Design	**Road Design Manual**	**RDM**
	1. Geometric Design	
	2. Hydrology & Drainage Design	
	3. Materials and Pavement Design for New Roads	
	4. Bridges and Retaining Structures Design	
	5. Pavement Maintenance, Rehabilitation and Overlay Design	
	6. Traffic Control Facilities and Communication Systems Design	
	7. Road Lighting Design	
E. Contracts	**Works and Services Contracts Manual**	**WSCM**
	1. Forms of contracts	
	2. Standard Specification for Road and Bridge Construction	
	3. Bills of Quantities	
	4. Standard/Typical Drawings	
F. Construction	**Road Construction Manual**	**RCM**
	1. Construction Management	
	2. Project Management	
	3. Site Supervision	
	4. Quality Assurance	
	5. Quality Control	

This table continues onto the next page...

Project Cycle Stage	Manual: Volume or Part/Chapter	Code
G. Maintenance	**Road Asset Management Manual**	**RAAM**
	1. Maintenance Management	
	2. General Maintenance	
	3. Pavement Maintenance	
	4. Bridges & Structures Maintenance	
H. Operations	**Road Operation Manual**	**ROM**
	1. Traffic Management	
	2. Vehicle Load Control	
	3. Emergency Services	
	4. Tolling	
I. Monitoring & Evaluation	**Road Design Manual**	**MEM**
	1. Performance Monitoring Manual	
	2. Technical Audits	
	3. Poverty, Gender Equality and Social Inclusion Monitoring	

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:

Table 1.1 Road Design Manual Coding Structure

Vol.	Manual Title	Part Name	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 and Urban Roads	RDM 1.3
2	**Road Design Manual: Vol. 2** Hydrology & Drainage Design	**Part 1** – Hydrological Surveys	RDM 2.1
		Part 2 – Drainage Design	RDM 2.2
3	**Road Design Manual: Vol. 3** Materials & Pavement Design for New Roads	**Part 1** – Ground Investigations and Material 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 & Retaining Structures Design	**Part 1** – Geotechnical Investigation and Design	RDM 4.1
		Part 2 – Bridge and Culvert Design	RDM 4.2
		Part 3 – Retaining Structures Design	RDM 4.3
		Part 4 – Reinforced Fill Structures Design	RDM 4.4
		Part 5 – Bridges and Structures Condition Survey	RDM 4.5
		Part 6 – Bridge Maintenance Design	RDM 4.6
5	**Road Design Manual: Vol. 5** Pavement Maintenance, Rehabilitation & 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 & 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
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 Objective of this Part

The general objectives of geometric design of a road project should be to achieve the desired balance between the level of traffic service provided, safety, amenity, whole-life costs and flexibility for future upgrading or rehabilitation and environmental impact.

The specific objectives of geometric design under this part include:

1. Optimisation of safety by providing a road and roadside that is designed to minimise fatalities and serious injuries for all road users;
2. Optimisation of 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;
3. Uniformity of design parameters along a route and/or within a network, particularly across administrative boundaries, to provide a consistent and operationally effective travel experience relative to the functional class of the road;
4. Minimise costs associated with construction, maintenance, and operation of the road whilst meeting all other objectives;
5. Provision 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 and future expansion of the road;
6. Minimise adverse environmental and social impacts (during construction and operation) and enhancement of the environment where possible both in the immediate vicinity of the road and over a wider area including integration of the road design into the surrounding environment to achieve a visually pleasing outcome;
7. Optimisation of opportunities to cater for the needs of all road user groups including local residents, businesses, and community groups; and,
8. Provision of safe and attractive walking and cycling environments within communities to promote primary healthcare through physical activities.

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 motorised and non-motorised 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:

1. Cross sections and head-rooms;
2. Alignment curvature, both horizontally and vertically;
3. At-grade junctions;
4. Grade separated junctions;
5. Road safety facilities; and
6. Roadside amenities.

1.4 Organisation of this Part

This Manual is divided into 18 chapters as follows:

Chapter 1 attends to 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 and requesting, and effecting departures from standards.

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. The chapter also sets out the 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 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 is on the modelling of the design and calculation of earthworks and pavement quantities.

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 are essentially an 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.

Chapters 14 to **17** provide design parameters specific to interurban roads, rural roads, urban arterial and collector roads, and urban streets respectively.

Chapter 18 is a bibliography of references.

1.5 Terminologies

Figure 1.1 Illustrations Of Standard Terms Used for the Cross-Sections for Single Carriageway Roads

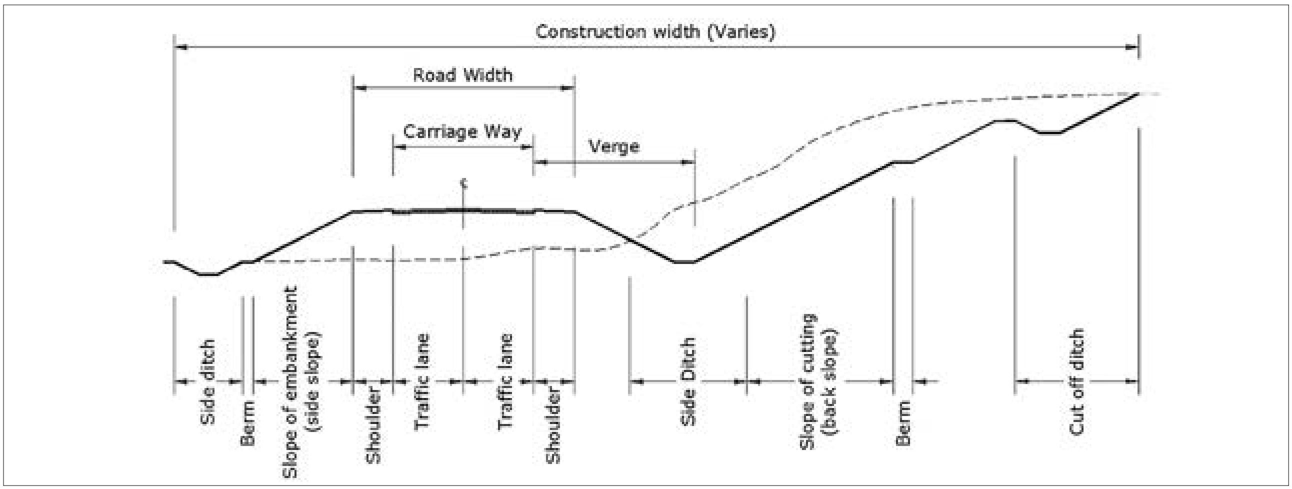


Figure 1.2 Illustrations of Standard Terms Used for the Cross-sections for Dual Carriageway Roads

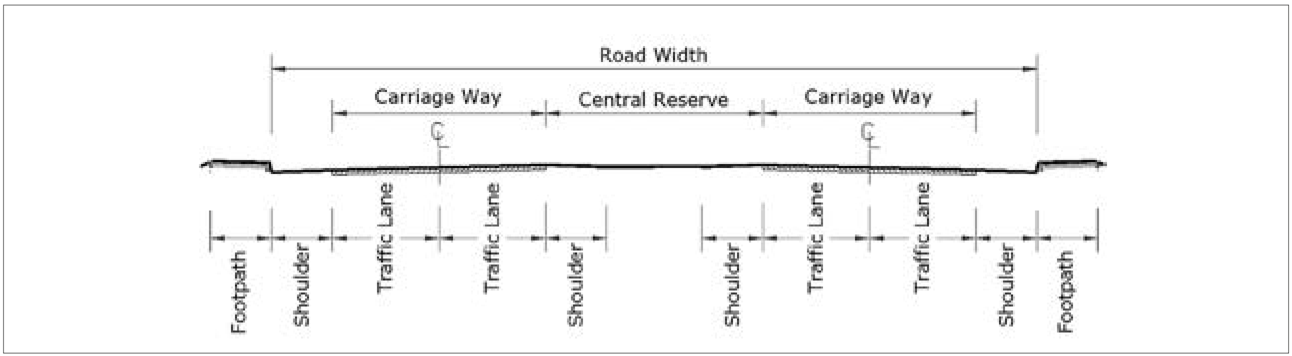
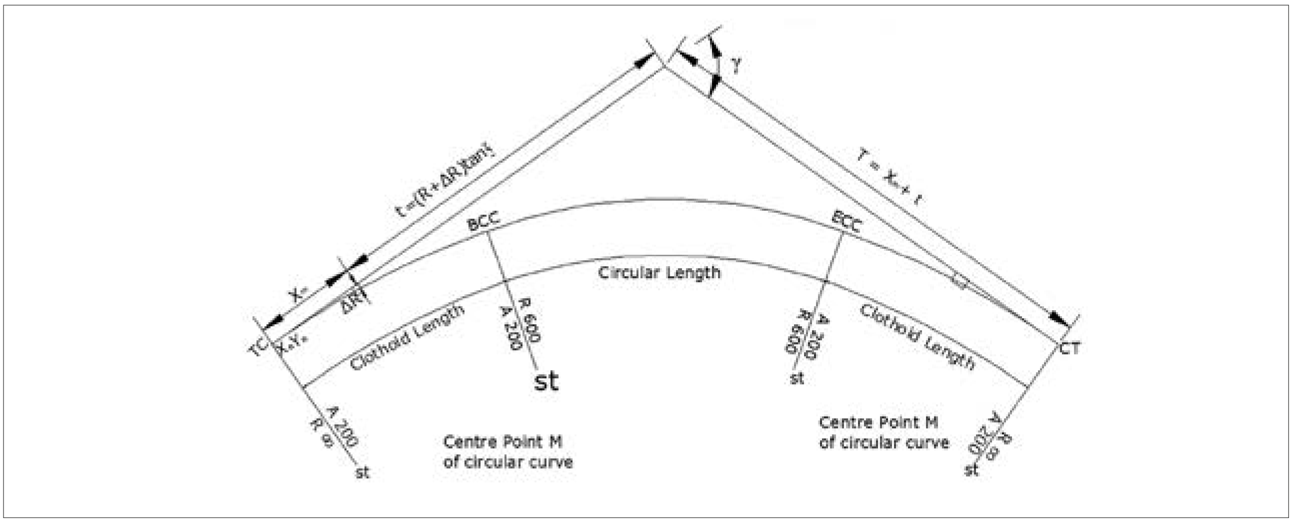
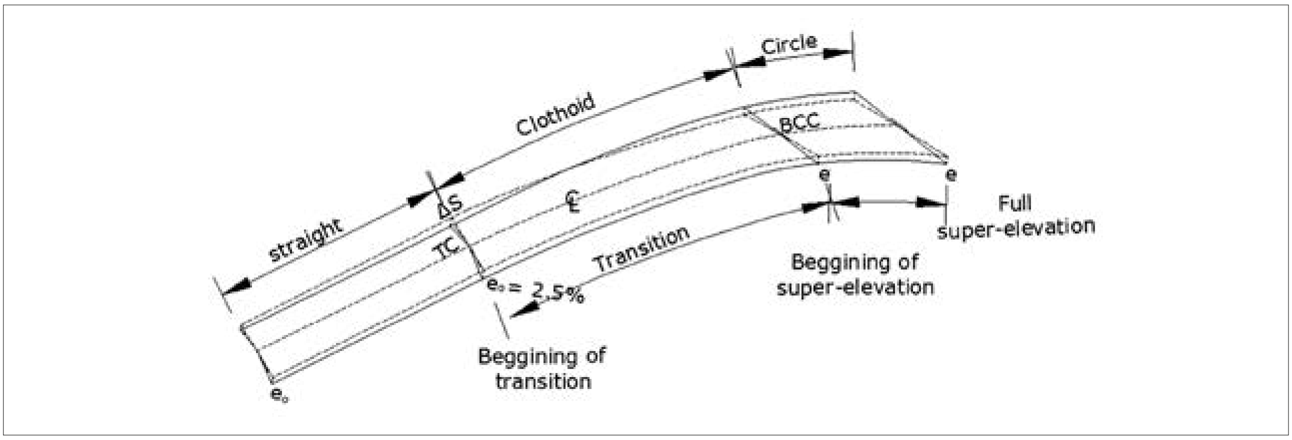


Figure 1.3 Illustrations for Horizontal Curves Characteristics



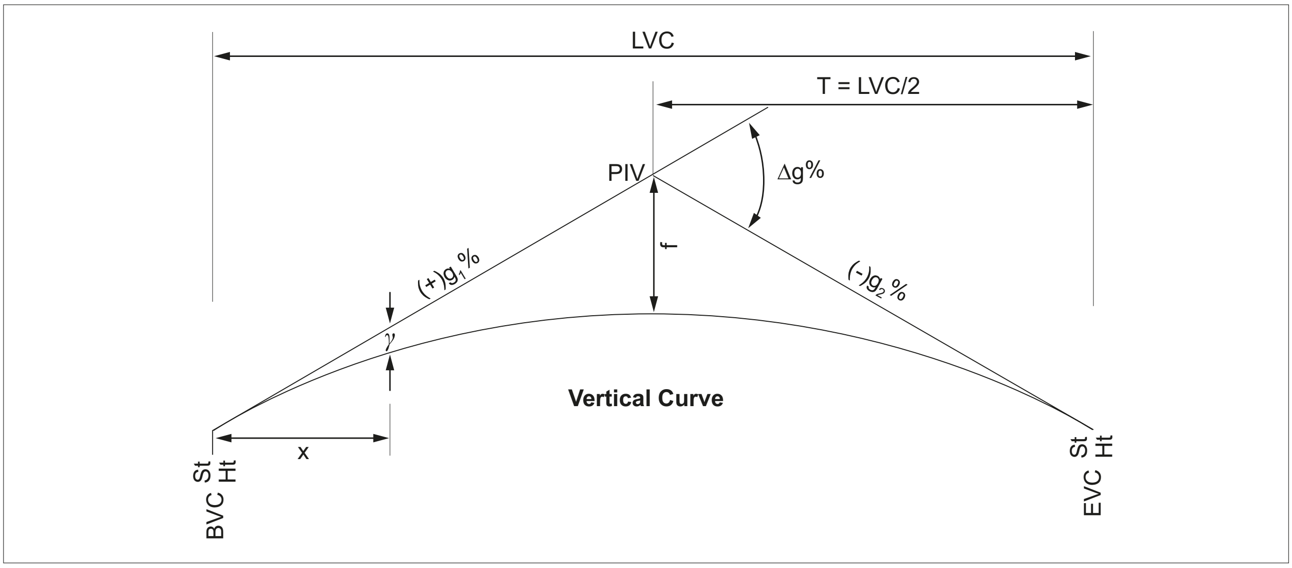
__R__ – Radius of circular curve; __A__ – Parameter of clothoid; __TC__ – Tangent to clothoid (clothoid origin); __BCC__ – Beginning of circle from clothoid end; __ECC__ – End of circle to clothoid; __CT__ – Clothoid to tangent; __st__ – Station - chainage; __Lcl__ – Length of clothoid; __Lci__ – Length of circle; __Lcc__ – Total curve length (circle + clothoids).

Figure 1.4 Illustrations for Super-elevation Vertical Curves Characteristics



ΔR *– Shift;* ***Xm*** *– Abscissa of centre point;* ***T*** *– Tangent Length;* ***PI*** *– Point of Intersection (horizontal);* ***γ*** *– Deviation angle (gamma);* ***Xn Yn*** *– Co-ordinates of station n;* $\boldsymbol{e_o}$ *– Normal crossfall;* ***e*** *– Super-elevation;* ***ΔS*** *– Rate of change of super-elevation;* ***℄*** *– Centre - line of road.*

Figure 1.5 Illustrations for Vertical Curves Characteristics



g *– Gradient (%);* ***PIV*** *– Point of intersection of vertical curve;* ***St Ht*** *– Station chainage;* ***Rv*** *– Height above sea level (m);*
LVC *– Equivalent radius of vertical curve (m);* ***Δg*** *– Length of vertical curve (m), Algebraic difference in gradients (*/.);*
BVC *– Beginning of vertical curve;* ***EVC*** *– End of vertical curve;* ***T*** *– Tangent length of vertical curve;* ***γ*** *– Tangent offset (vertical);*
x *– Horizontal length in plan;* ***f*** *– Centre correction;* ***Ls*** *– Stopping sight distance;*

1.6 Design Process

1.6.1 Overview

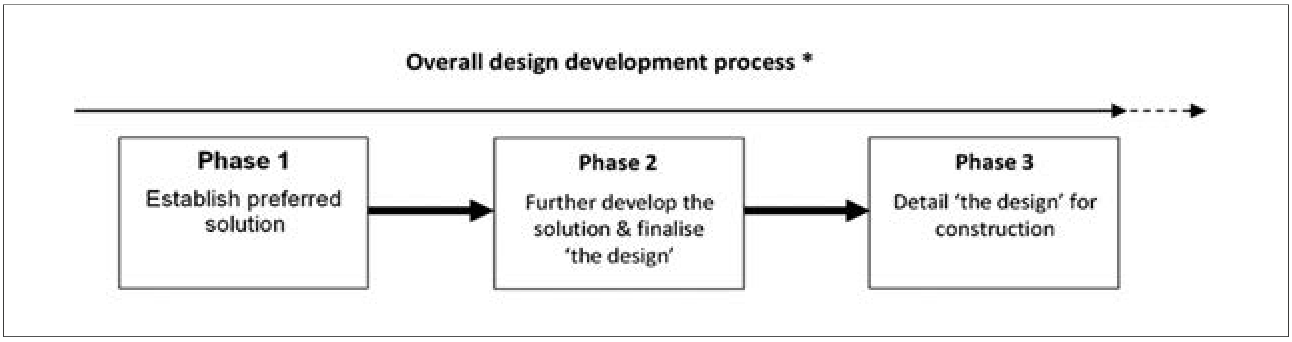
The design process considers three general types of projects:

1. New construction: construction on new alignments where no existing roadway is present. May involve new construction at sites with no existing development or removal of existing structures in built up areas.
2. Reconstruction: projects that utilise an existing roadway alignment or make only minor changes to an existing alignment involving 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.
3. Improvement: projects undertaken on the existing roadway alignment to address specific needs without changing the basic roadway type except for minor changes. The needs include reduction of current or anticipated traffic operational congestion or improvement of safety.

The development of any of the three types of projects above will typically follow a process similar to that shown in Figure 1.6.

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 1.6 Geometric Design Process



1.6.2 Phase 1 – Establishing the Preferred Solution

Establishing the preferred solution includes the following steps:

1. Project conceptualisation: Consulting relevant agencies and stakeholders to understand challenges and potential solutions.
2. Data collection:
   a. Primary surveys: of the route corridor to collect information on the current status, alignment characteristics, traffic, road user demographics, etc.
   b. Desktop studies: Reviewing of masterplans, census data, transport plans, and land use plans to understand goals and planned investments and incorporate them in the preliminary designs.
3. Preliminary design: Preparation of design alternatives incorporating adequate facilities for NMT and pedestrians.

4. Stakeholder engagement: Presenting the concept designs to stakeholders and getting feedback from stakeholders, including utility providers. The designer will coordinate the:
    a. Informing of stakeholders of planned developments and seek their feedback.
    b. Engaging relevant stakeholders, including government agencies, transport operators, vendors, business associations, disability groups, and NGOs.
    c. Posting the concept designs on a website for review by the public.
    d. Revising preliminary designs based on stakeholder feedback.
    e. Holding additional stakeholder meetings during the design process as needed.
5. Analysing the different options (including a Safe System assessment and road safety audit).
6. Recommending a preferred option for further development.

1.6.3 Phase 2 – Detailed Design

The detailed design phase involves review the design brief to confirm the preferred option, especially in cases where there is a significant time lapse since completion of phase 1. Typical flow of activities for this phase are shown in Figure 1.7.

This phase will entail:

1. Undertaking any supplementary studies that will yield information and data required for the refinement of the preferred option.
2. Detailed design of horizontal and vertical alignment, junctions, drainage facilities, safety elements, roadside amenities, and ancillaries.
3. Preparation of detailed design reports and drawings.
4. Design reviews by internal review panels for quality control and updating to incorporate the feedback. Based on the guidance in the procedure manual or policies on the public consultations, the updated designs may be posted on the web for review by the public.

1.6.4 Detailing the Design for Construction

Having developed the preferred solution, the last phase in the process is the production of sufficient detail to allow the subject project to be constructed and meet set requirements and specifications.

Using modern CAD methods detailed construction drawings for all elements of the design will be prepared and such drawings will be for contractual use at construction phase.

Contribution will then be made the preparation of bid documents, specifications, and bills of quantities for project execution and maintenance provisions.

1.7 Departures from Standards

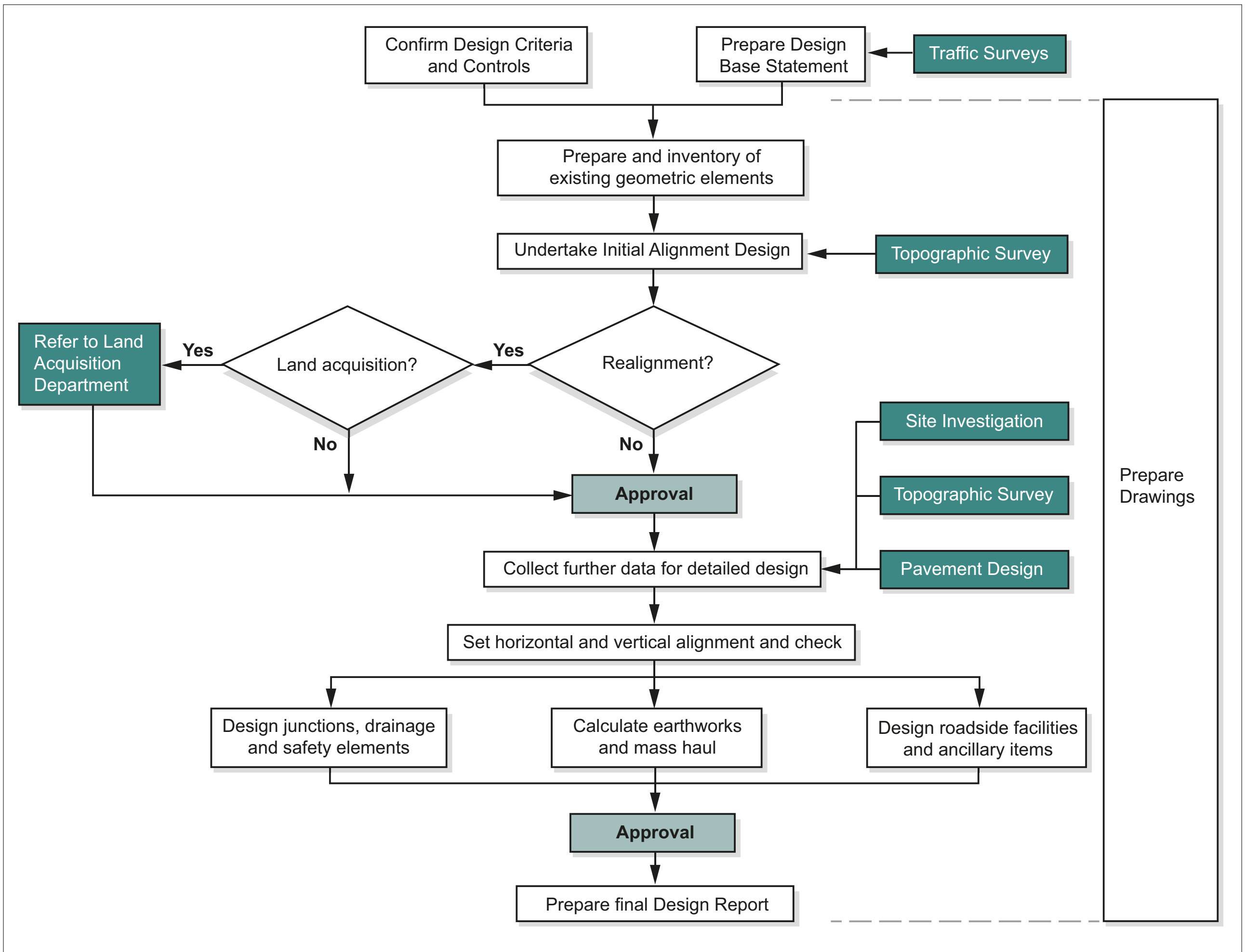
Where the designer departs from a standard, written approval must be obtained from the Chief Engineer for Roads.

In seeking approval of a departure from standards, a designer should submit the following information according to the standard format prescribed in the preliminaries to this manual:

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;
5. Any mitigation to be applied in the interests of safety; and,
6. Justification for the departure.

The approval of departures by the Chief Engineer for Roads may involve a consultative process in accordance with the guidance in the procedures and guidelines manual, or statutes governing public consultations.

Figure 1.7 Detailed Design Process



2 The Road System

2.1 General

2.1.1 Road Classification System

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

1. To provide traffic mobility between centres and areas; and
2. To provide access to land and properties adjoining the roads.

For roads whose major function is to provide mobility, i.e., to cater for through and long-distance traffic, high and uniform speeds and uninterrupted traffic flows are desirable.

For roads whose major function is to provide land access, high speeds are unnecessary and, for safety reasons, undesirable.

Each road provides varying levels of these two functions. In the functional classification scheme, the overall objective is that the road system, when viewed in its entirety, yields an optimum balance between its access and mobility purposes.

For nomenclature purposes, those roads that provide a high level of mobility are called 'Arterials'; those that provide a high level of accessibility are called 'Locals'; and those that provide a more balanced blend of mobility and access are called 'Collectors.'

Arterials provide mostly mobility; Locals provide mostly land access; and Collectors strike a balance between the two. While most roads offer both 'access to property' and 'travel mobility' services, it is the road’s primary purpose that defines the classification category to which a given road belongs.

Thus, the function of a particular road in the national, regional, and local road network has a significant impact on the design criteria to be chosen, and the design engineer has to give careful consideration to this aspect in the early stages of the design process. The following steps are required:

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

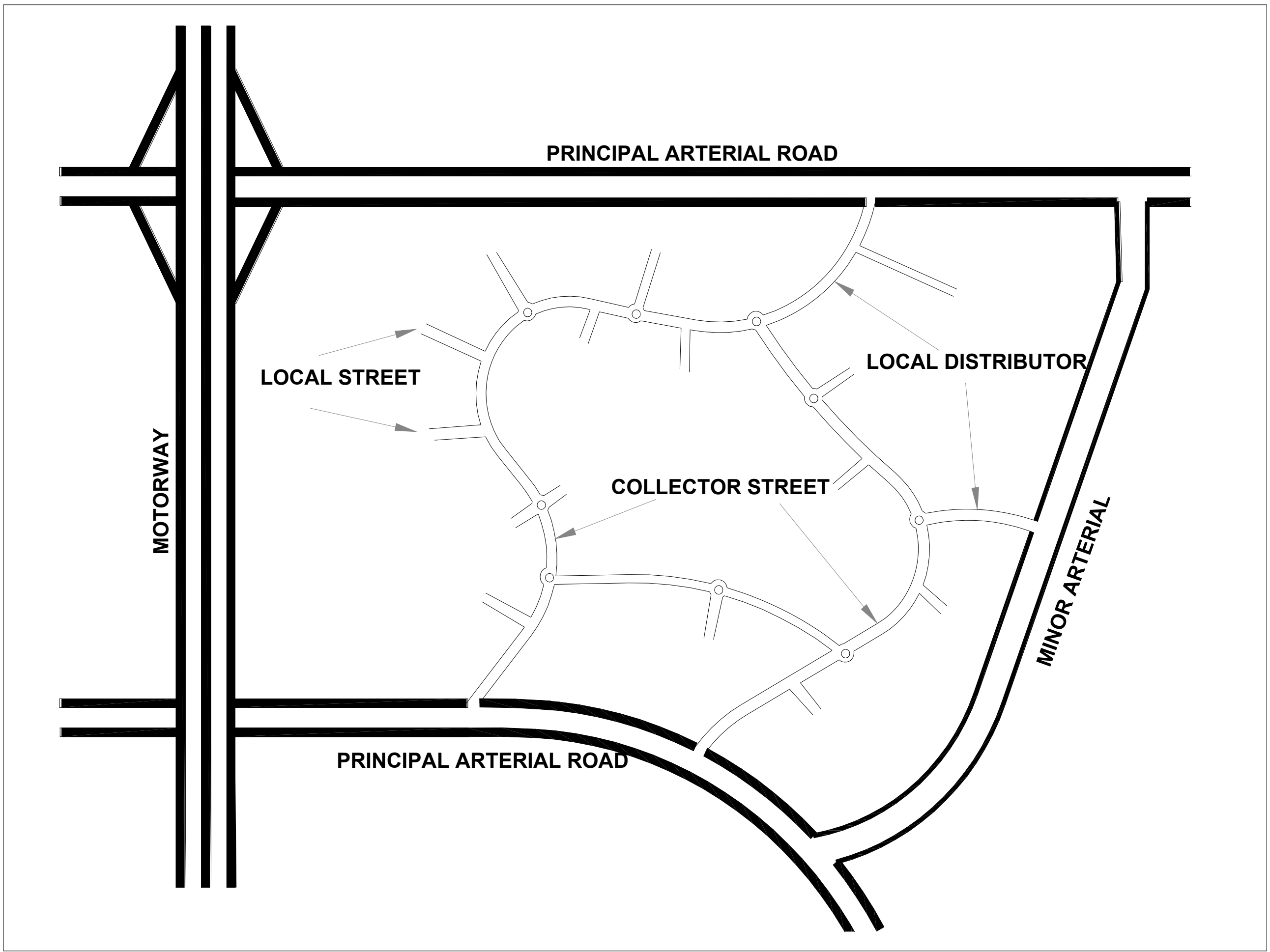
When the functional classification and level of access control are given, design standards can be applied which will encourage the use of the road as intended. Design features that can convey the level of functional classification to the driver include carriageway width, continuity of alignment, spacing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

2.2 Hierarchy of Functional Classification

2.2.1 General

The hierarchy of functional classification in is depicted by Figure 2.1.

Figure 2.1 Hierarchy Essentials



The above road networks serve two important types of traffic movement:

1. Through traffic that has no relation or not need to enter the area – served by principal arterial and minor arterial roads.
2. Traffic that has direct relationship and need to access the area or circulate within an area – served by minor arterial, collector and local streets.

The types of movements that are served by the above classified roads have several functions such as provision of through traffic, access to properties, bus routes and stops, cycle tracks, and pedestrian facilities. Some roads serve more than one functions to varying degree, however, mixing incompatible functions may lead to traffic conflict. Therefore, the concept of road hierarchy is to define the main purpose of each road class which will then be used as a basis for planning of land use and traffic management.

2.2.2 Arterials

Ideally, arterial roadways are characterised by limited access to abutting properties and a capacity to quickly move relatively large volumes of traffic. Rural arterials provide connections between major urban areas and provide a Level of Service (LOS) suitable for statewide or interstate travel. Urban arterials serve the major centres of activity of urbanised areas, the highest traffic volume corridors, and the longest desired trips. Rural and urban arterial systems are connected to provide continuous through movements at approximately the same LOS.

For design, arterial roadways are divided into the following categories:

- **Expressways** – The expressway is the highest level of arterial. These facilities are characterised by full control of access, high design speeds, and a high level of driver comfort and safety.

  Expressways are considered a special type of roadway within the functional classification system, and separate design criteria have been developed for these facilities.

- **Principal and Minor Arterials** – In both rural and urban areas, principal arterials accommodate higher traffic volumes and greater trip lengths. Many of these are divided facilities, which may have partial control of access. Minor arterials provide a mix of interstate and inter-county travel service in rural areas and provide intra-community connections in urban areas. Minor arterials, as compared to principal arterials, accommodate relatively lower travel speeds, trip lengths, and traffic volumes, but provide more access to property than the principal arterial system.

2.2.3 Collectors

Collector routes are characterised by an approximately even distribution of their access and mobility functions. Traffic volumes and speeds are typically lower than those of arterials. In rural areas, collectors serve intra-county travel needs and provide connections to the arterial system.

In urban areas, collectors act as intermediate links between the arterial system and points of origin and destination.

2.2.4 Local Roads

Any public road not classified as an arterial or collector is classified as a local road. Local roads are characterised by their many points of direct access to adjacent properties and their relatively minor value in accommodating mobility. Speeds and volumes are usually low and trip lengths short.

2.2.5 Identification of Functions

The road hierarchy is essential in the identification of the function proposed for all the elements of the road system, and it will assist in considering activities such as:

1. Network planning.
2. Traffic management.
3. Safety.
4. Route signing.
5. Parking facilities.
6. Cycle and pedestrian facilities.
7. Environmental and land use consideration.
8. Economic consideration.

Government policies and the master planning process for the operational and management needs of the above elements must also be related to the functional classification.

2.3 Kenyan Road Classification

2.3.1 Functional Classification and Link-role Categories

The roads in Kenya are classified based on their functional and link-role categories as shown in Table 2.1.

Table 2.1 Kenya Road Classification System

Road Class	Link Role	Functional Category	Network Description
Interurban Roads			
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.
B	**National Highways**	Major arterial roads through urban & rural areas.	Roads forming the strategic national highways routes linking centres of national importance to each other or to Class A roads.
C	**Primary Roads/ Inter County Roads**	Minor arterial roads through urban and rural areas.	Inter-county roads linking regionally important centres in at least two counties or two higher class 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.
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.
F	**Forest Roads**	Local rural road.	Roads in Government gazetted forests.
G	**Government Institutions Access Roads**	Local rural road.	Access road to Government facilities in rural and remote areas.
K	**Coffee Roads**	Local rural road.	Roads accessing coffee (Kahawa) growing areas.
L	**Land Access Roads**	Local rural road.	Roads accessing settlement scheme areas.
P	**Game Park Roads**	Local rural road.	Roads in National Parks and Game Reserves.
R	**Rural Access Roads**	Local rural road.	Roads accessing rural communities or villages.
S	**Sugar Roads**	Local rural road.	Roads accessing sugar growing areas.
T	**Tea Roads**	Local rural road.	Roads accessing tea growing areas.
U	**Unclassified Rural Roads**	Local rural road.	Unclassified roads including minor roads.
W	**Wheat Roads**	Local rural road.	Roads accessing wheat growing areas.
Urban Roads			
UA	**Urban Arterial Roads**	Arterial roads within municipalities and cities	Major and minor arterial roads located within municipalities and cities.
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.
UL	**Local Urban Streets**	Local urban streets in urban areas.	Local urban streets in residential, industrial, and commercial areas.

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 and Transport will maintain a central register of all road numbers in Kenya.

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, a number can re-emerge 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 any agency 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

Network / Road Class	Numbering System
Inter-Urban Roads	
Trans Africa Highways, Class A	**1. Expressways:** 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.
	2. Main route: Designated with alphanumeric number having letter 'A' and three-digit number i.e. A104 and A109.
	3. 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.
	4. 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.
Eastern Africa Regional Highways, Class A	**1. Expressways:** 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.
	2. Main route: Designated with alphanumeric number having letter "A" and one to two-digit number i.e. A1 and A23.
	3. 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.
	4. 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.

Network / Road Class	Numbering System
National Highways, Class B	**1. 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.
	2. 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.
	3. 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.
	4. Supplemental radial & spur 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., 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.
Primary Roads or Inter County Roads, Class C	Designated with alphanumeric number having letter 'C' and a number from 100 to 999 progressively from West to East e.g., C100.
Rural Roads	
Secondary Roads or Inter Sub-County Roads, Class D	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.
Sub-County Roads, Class E	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	Designated with alphanumeric number having letter 'F', 'G', 'P', or 'R' as applicable, 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 100to 999 e.g., R16-999 for a road in Machakos County.
Urban Roads	
Urban Arterials, Class UA	Designated with road name and an alphanumeric number having letter 'UA', the County Number, a dash, a town/municipality/borough code in accordance with national statutes and one to two-digit number suffix preceded by a dash ranging from 1 to 99 progressively from West to East e.g., UA47-01-19 for arterial in Nairobi CBD.
Urban Collectors, Class UC	Designated with road name and an alphanumeric number having letter 'UC', the county number, a dash, a town/municipality/borough code in accordance with national statutes and three-digit number suffix preceded by a dash ranging from 100 to 999 progressively from West to East e.g., UC47-01-200 for a collector in greater Nairobi CBD.
Urban Local Streets, Class UL	Designated with road name and an alphanumeric number having letter 'UL', the county number, a dash, a town/municipality/borough code in accordance with national statutes and four-digit number suffix preceded by a dash ranging from 1000 to 9999 progressively from West to East e.g., UL47-01-1600 for a street in Nairobi CBD.

2.4 Access Control

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.

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 Roads 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 Roads Authority in order to ensure adequate standards of visibility, surfacing, drainage etc.

Road function determines the level of access control needed. 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
UC	Full	Partial
UL	Unrestricted	Unrestricted

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

1
2
The Road System

2.5 Road Reserve

Road reserves are provided to accommodate current and future road connections or changes in alignment, road width or junction layout to enhance the safety.

The applicable road reserve width for interurban and rural roads are shown in Table 2.5 and for urban roads in Table 2.6.

Table 2.5 Road Reserve for Interurban and Rural (m)

Functional Class	Road Reserve	
	Desirable	Reduced
A	110	80
B	60	40
C	40	40
D	25	25
E	20	20
F,K,G,T,S,W,U	15	9

Table 2.6 Road Reserve Urban (m)

Functional Class	Road Reserve	
	Desirable	Reduced
UA (multi-lane)	110	80
UA	30 - 40	30
UC	18 - 25	18
UL	12	6
UL (NMT only)	9 - 12	6

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.

5. Refinement of selection of standard based on anticipated traffic volumes at the end of the designated design life.

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 road reserve width, carriageway width, continuity of alignment, standards of alignment and grades, spacing of junctions, frequency of accesses, traffic controls and provisions for non-motorised traffic and pedestrians.

The Kenyan 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 are divided into three main classes (Table 2.1). Each functional class can be built in most terrains with the design speed appropriate to the terrain and their location (urban or rural).

Typical design standards have been developed for Kenya and the application to the functional classes is as given in Table 2.7 and Table 2.8 below for rural and urban roads respectively. Roads with heavier traffic will be provided with a higher standard for both rural and urban cases. Roads whose function is to provide long distance travel or rapid throughput will require higher design speeds and standards, while roads which serve local traffic and the effect of speed is less significant, will require lower or relaxed standards.

Chapters 14 to 17, give the details of each typical design standard as applicable to inter-urban, rural, urban arterial and collector roads, and urban streets. These are in descending order of design hierarchy, DR1 to DR7 for interurban and rural roads, and DU1 to DU6 for urban roads. The standards were set taking into account sound technical principles and best practices in geometric design as outlined in Chapters 1 to 13. Departures from these standards can be sought from the Chief Engineer for Roads following the procedures given in Section 1.6.

Table 2.7 Design Standards for Rural Road Classes

Design Standard	Functional Class	Design Traffic	Cross Section	Description of Design Standard
DR1	A	DHV > 8000	R1, R2	Geometric design standard for rural 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.
DR2	A, B	AADT > 4000 or 500 < DHV < 8000	R3, R4	Geometric standards suitable for long to intermediate trip lengths with high to medium travelling speeds. The road is usually designed with partial access control.
DR3	B, C	2000 < AADT < 4000 or 250 < DHV < 500	R4, R5	Geometric standard road suitable for intermediate trip lengths with medium travelling speeds and partial access control.
DR4	B, C	500 < AADT < 2000	R5, R6	Geometric standards serving mainly local traffic. Partial or no access control.
DR5	D	150 < AADT < 500	R6, R7	Geometric standard road with two-way flow. This standard is used for roads accommodating local traffic with low volumes of commercial traffic.
DR6	E	50 < AADT < 150	R7, R8	Geometric standards and is applied to roads with very low traffic volumes.
DR7	F, G, K, L, P, R, S, T, U, W	AADT < 50	R8, R9	Geometric standards with a gravel surfacing carrying low traffic volumes. The need for provision for two-way traffic is very low.

Notes: ***a)*** *An A 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 or B road will still be of significance to traffic (including through traffic), but less so than an A road.*
c) *C and D roads 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 Standards for Urban Road Classes

Design Standard	Functional Class	Design Traffic ADT	Cross Section	Description of Design Standard
DU1	UA	>12 000 per lane	U1, U2	Geometric design standard for 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.
DU2	UA	6 000 - 12 000 per lane	U2, U3	Geometric standards suitable for long to intermediate trip lengths with high to medium travelling speeds. The road is usually designed with partial access control.
DU3	UC	4 000 -6 000 per lane	U3, U4	Geometric standard road suitable for intermediate trip lengths with medium travelling speeds and partial access control.
DU4	UC	2 000-4 000 per lane	U4, U5	Geometric standards serving mainly local traffic. Partial or no access control.
DU5	UL	500 – 2000 per lane	U5, U6	Geometric standard road with two-way flow. This standard is used for roads accommodating local traffic with low volumes of commercial traffic.
DU6	UL	<500 per lane	U6	Geometric standards and is applied to very low traffic volume. It is used where two-way traffic is not required.

Notes: ***a)*** *A UA 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 UC road will still be of significance to traffic (including through traffic), but less so than an UA.*
c) *A UL road will be of lower significance and be of primarily local importance but will perform a more important function than local roads.*

3 Design Controls Criteria

3.1 General

Roads need to provide for the safe, convenient, effective, and efficient movement of persons and goods. The design of roads should be based on the capabilities and behaviour of all road users, including pedestrians, cyclists, motorcyclists, and on the performance and physical characteristics of vehicles (including public transport). At the same time, consideration must also be given to the whole range of economic, social, environmental, and other factors that may be involved.

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 and combinations of these elements should be determined based on the following controls and criteria:

1. Road function, environment, and access control.
2. Land use, physical features, and terrain.
3. Design life.
4. Design vehicle.
5. Motorcycles.
6. Driver performance and human factors.
7. Non-motorised traffic.
8. Speed and speed controls.
9. Traffic volume.
10. Road capacity and level of service.
11. Sight distances.
12. Safety considerations.
13. Environmental and social considerations.
14. Economic and financial considerations.

All these controls and criteria should be considered 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 flexibility of the standards.

3.2 Road Function, Environment and Access Control

The function of a particular road, as defined by the majority of the road users (long-distance traffic, through traffic, local traffic, etc.) has to be taken into account in the determination of design standards for the project and in particular in the selection of the design speed and cross-section.

Careful consideration must be given to the choice of design standards for roads whose major function is to cater for long distance regional traffic (generally International and national trunk and, major and minor urban arterials). Due to the long distances involved, traffic tends to move at high speeds on some of these roads and it may therefore be necessary to adopt higher standards than are warranted by traffic volumes alone to provide an acceptable level of road safety.

Guidelines for the selection of design standards in relation to road function are given in subsequent chapters for a number of design elements.

Depending on the function of a road, various levels of access control should be imposed as described in Section 2.4. 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.

3.3 Land Use, Physical Features and Terrain

3.3.1 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 Authority, in order to co-ordinate the project with existing and proposed land uses and to protect the selected route from conflicting developments.

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.

3.3.2 Route Location

The first fundamental consideration in the design process is route location. An appropriate route shall be located that considers the intended functional classification, public opinion, environment, costs, 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 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 to obtain the best economic solution and the best possible service to the traversed area with the least detrimental effects.

3.3.3 Land Use Integration

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

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.

3.3.4 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.

3.3.5 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, when 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

3.3.6 Terrain

3.3.6.1 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 general slopes between the two ends of the road section in question and classifying the terrain according to the following definitions in Table 3.1 and illustrated in Figure 3.1 to Figure 3.4.

Table 3.1 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%.

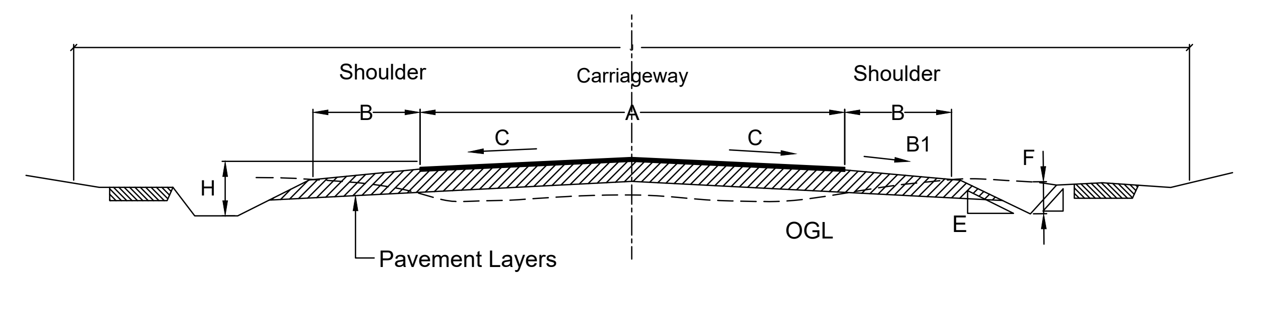
In mountainous areas, the geometric standard takes account of the constraints imposed by the difficulty and stability of the terrain. The design standard may need to be adjusted locally to cope with exceptionally difficult terrain conditions, but every effort should be made to design the road alignment 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.

3.3.6.2 Flat Terrain

This is 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 %.

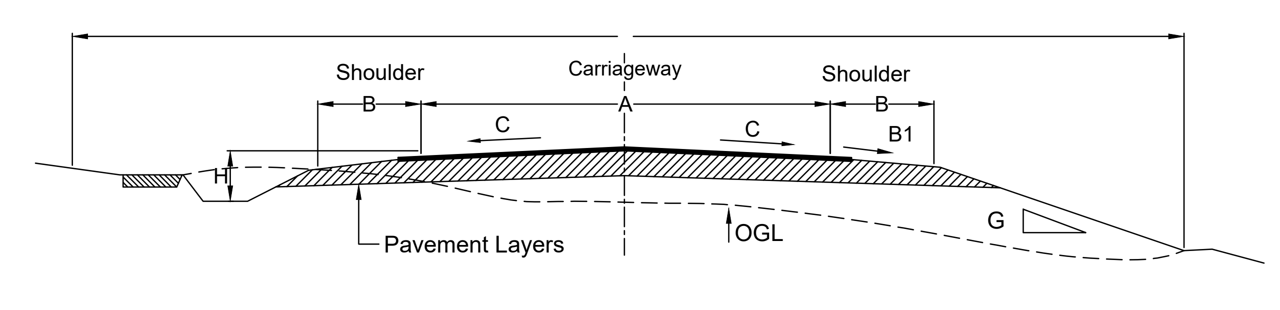
Figure 3.1 Typical Cross Section in Flat Terrain



3.3.6.3 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 %.

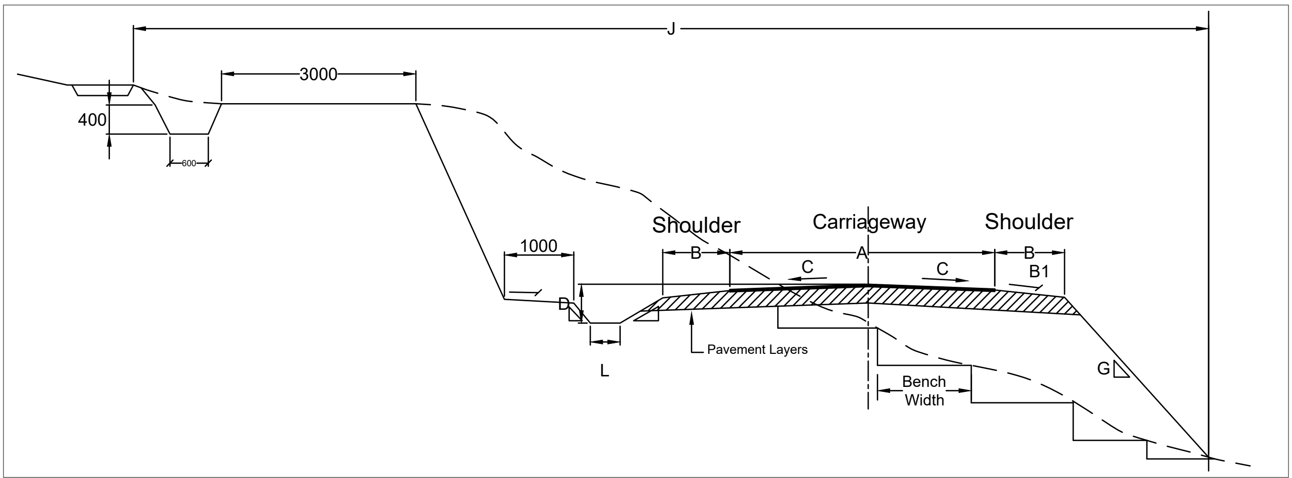
Figure 3.2 Typical Cross Section in Rolling Terrain



3.3.6.4 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 between 25-50 %.

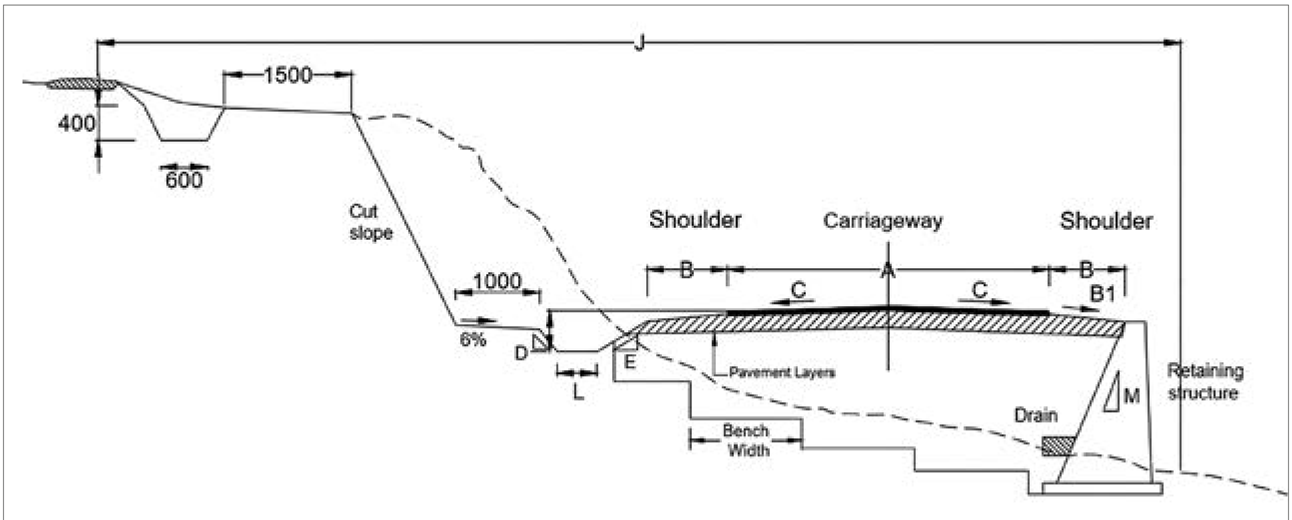
Figure 3.3 Typical Cross Section in Mountainous Terrain



3.3.6.5 *Escarpment Terrain*

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

Figure 3.4 Typical Cross Section in Escarpment Terrain



3.4 Design Life

Design year is the 'forecast' year used for design. The 'year of opening' is when the construction will be completed, and the project location is fully operational. Design year selection is dependent on a decision to design for the year of opening, or for a future year based on forecast or planned conditions.

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 etc. 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.

The design year can therefore be any interim year selected between the project year of opening year and the end of the design life. The selection of the design year will depend on the nature of the project for example, for some safety projects the year of opening may be selected as the design year. However, the design year shall generally be the year at the end of the design life as suggested in Table 3.2.

Table 3.2 Geometric Design Life

Project Type	Design Life (Years)
Gravel roads	10
New low volume sealed roads	10
New high-volume roads with flexible pavement	20
Rehabilitation of flexible pavement	20
Reconstruction of flexible pavement	20
Concrete pavements	20

3.5 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 walkway.

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 harmonisation of the design vehicles for EAC Partner States has been adopted for this Manual. The design vehicles indicated in Table 3.3 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 3.3 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 3.3 Design Vehicle Characteristics

Design Vehicle	Code	Height (m)	Width (m)	Length (m)	Front over-hang (m)	Rear over-hang (m)	Wheelbase (m)	Minimum turning radius (m)
Passenger Cars								
4x4 utility	DV1	1.3	2.1	5.8	0.9	1.5	3.4	7.3
Buses								
Single unit bus	DV2	4.1	2.6	12.3	2.1	2.6	7.6	12.8
Articulated bus	DV3	3.4	2.6	18.3	2.1	2.6	6.7 + 5.9 = 12.6	12.1
Trucks								
Single unit truck	DV4	4.3	2.6	11.0	1.5	3.0	6.5	12.8
Truck + semi-trailer	DV5	4.3	2.6	15	1.2	0.6	4.8 + 8.4 = 13.2	13.7
Typical 5-axle truck	DV6	4.6	2.6	18.5	1.2	1.8	5.5 + 10.0 = 15.5	13.7
Rigid truck and drawbar trailer	DV7	4.6	2.6	22	1.2	1.8	6.1 + 12.9 = 19	12.0
Interlink (with a short truck tractor)	DV8	4.6	2.6	22	1.2	1.8	4.1 + 6.52 + 7.94 = 18.9	12.0
Rigid truck and drawbar trailer	DV9	4.6	2.6	22	1.2	1.8	6.1 + 12.8 = 18.9	13.7

Figures below illustrated the turning paths of the various design vehicles.

Figure 3.5 Dimensions and Turning Radius Path for Single Unit Truck (DV4)

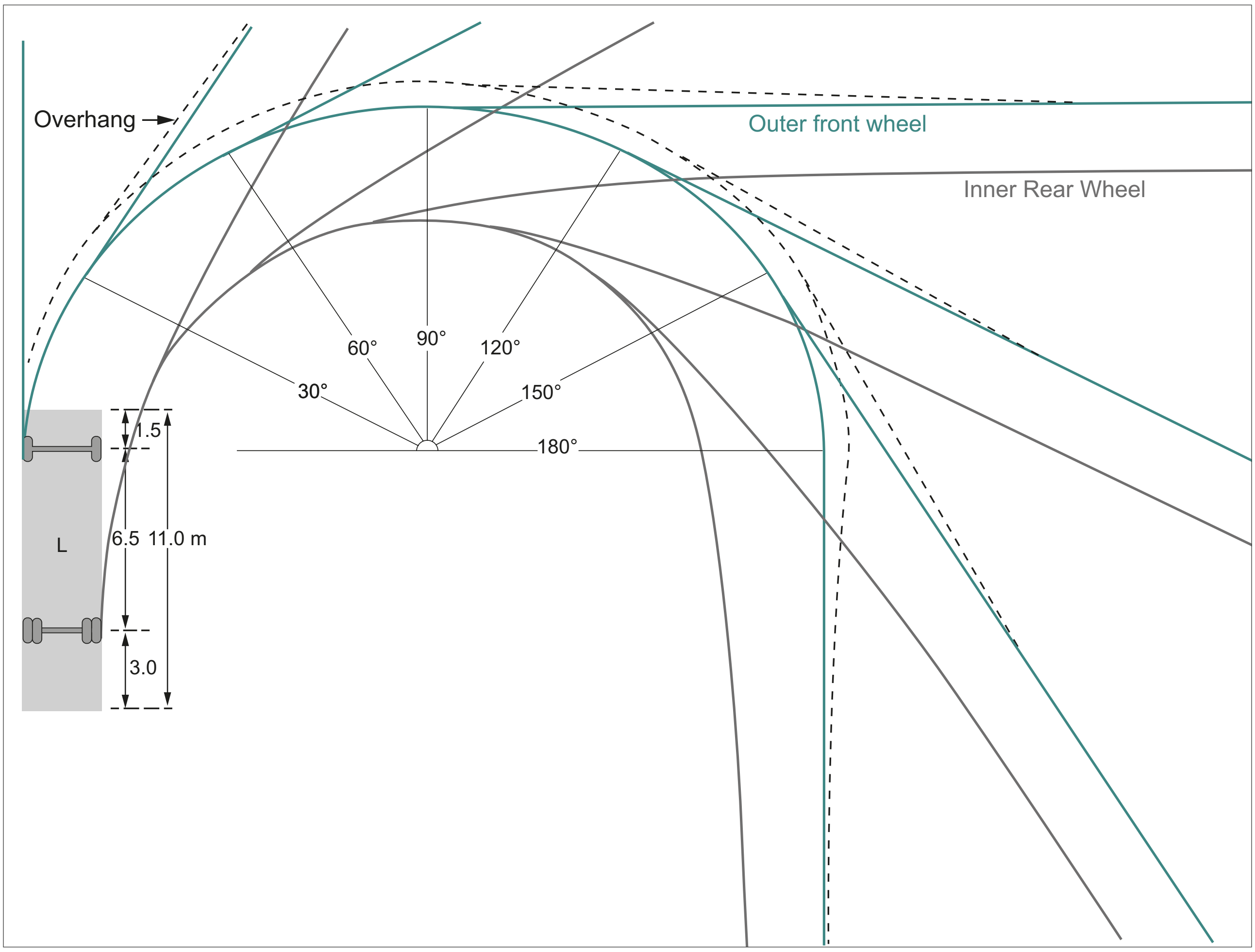


Figure 3.6 Dimensions and Turning Radius Path for Single Unit Bus (DV2)

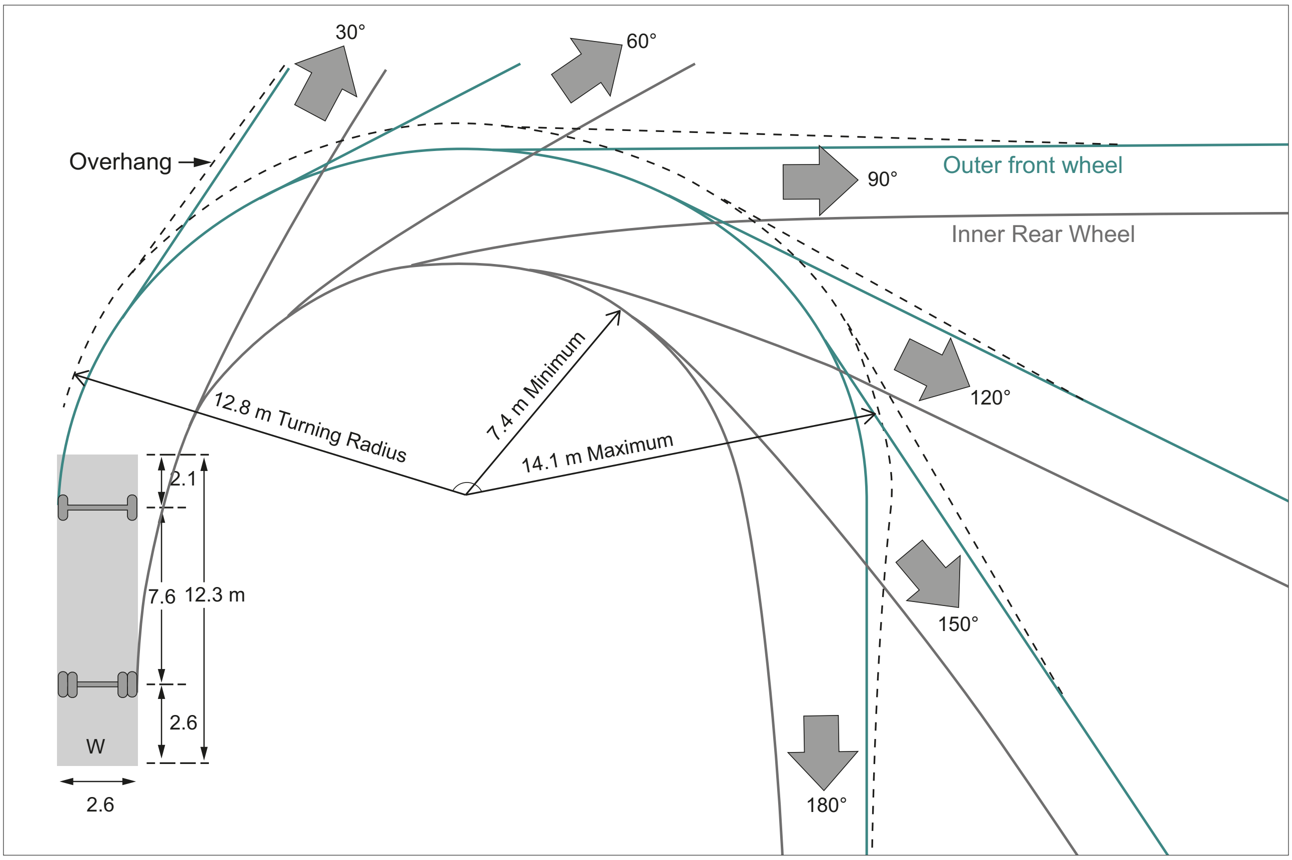
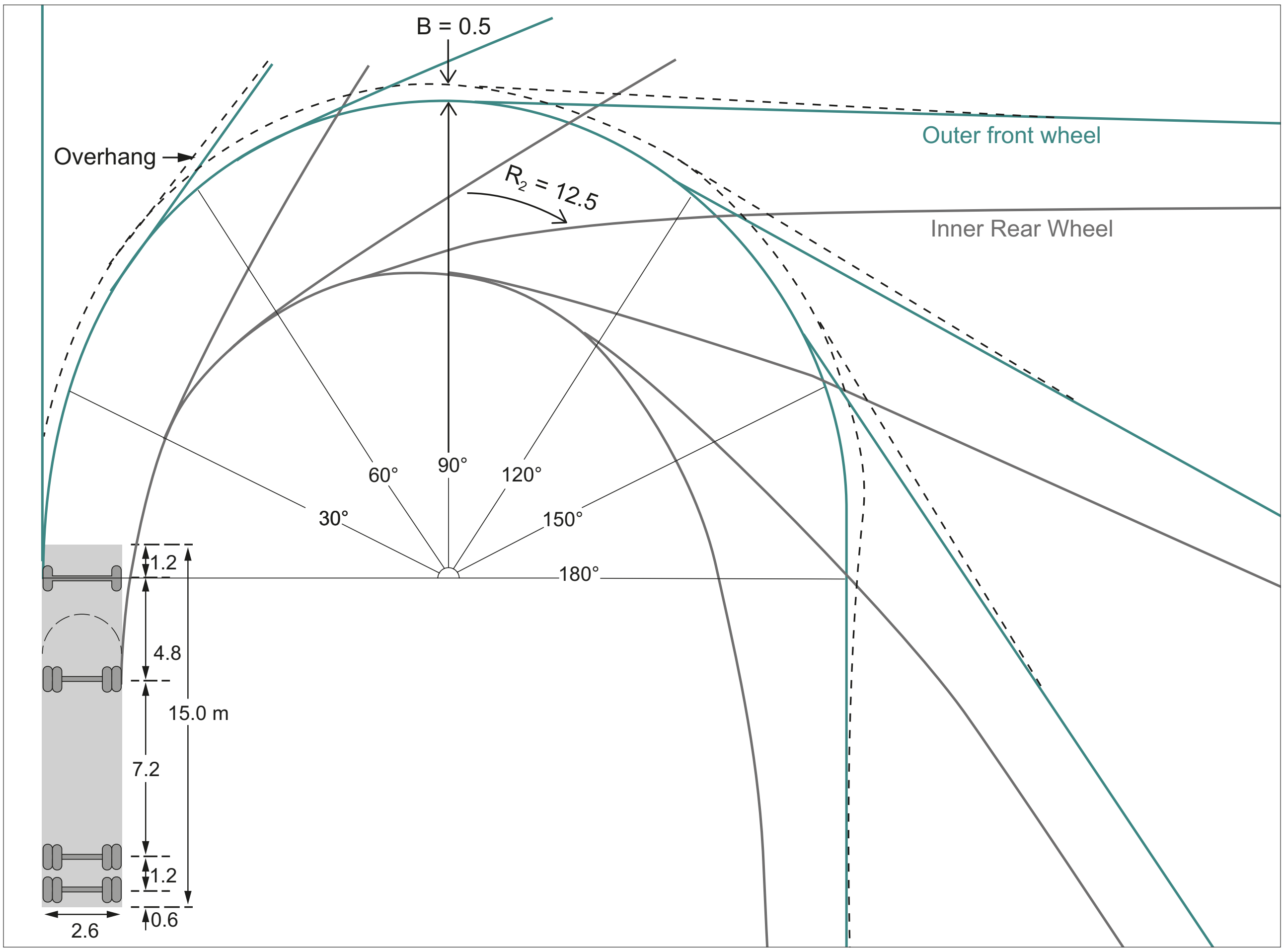


Figure 3.7 Dimensions and Turning Radius for a Semi-Trailer Combination (DV5)



3.6 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. At an impact of 60 km/hr, 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.

3.7 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. Providing bicycle-only lanes;
3. Bicycle-safe drainage grates;
4. Adjusting manhole covers (if present) to the grade; and,
5. Maintaining a smooth and clean riding surface.

3.8 Pedestrian Traffic

In general, pedestrian safety is enhanced by the provision of median refuge islands of sufficient width at road junctions, and separate pedestrian walkways (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 13. 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 minimise 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.

3.9 Driver Performance and Human Factors

3.9.1 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'. 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 85^{th} percentile and the design speed on an element such as a horizontal curve should be less than 20 km/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.

3.9.2 The Six 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 3.13 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).

3.9.3 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.

3.9.4 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.

3.10 Speed and Speed Controls

3.10.1 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 the 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.

3.10.2 Speed classification

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

1. **Desired Speed** – the speed at which a driver wishes to travel, determined by a combination of motivation and comfort.
2. **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.
3. **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.
4. **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 enroute when the vehicle is at rest are not considered 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.
5. **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.

3.10.3 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 road 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 recognises 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 desirable ranges of design speeds for the rural and urban roads in Kenya are shown in Table 3.4 and in Table 3.5 respectively.

Table 3.4 Design Speeds for Rural Roads

Functional Class	Terain							
	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 3.5 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.

3.11 Traffic Volume

3.11.1 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. 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.

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.

3.11.2 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.

3.11.3 Design Volume - High traffic volume roads

On major roads carrying relatively heavy traffic volumes throughout the year (current AADT > 1000) hourly traffic should 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. Generally, heavily trafficked roads should be designed to accommodate the 30^{th} to 50^{th} highest hourly volume in year ten after opening (DHV = Design Hourly Volume), depending on economic considerations.

3.11.4 Conversion of AADT to DHV

In design, peak-hour volumes are 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:

$$DDHV = AADT * K * D$$

Equation 3.1

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 30^{th} peak hour flow. This means if the 365 peak hour volumes of the year at a given location are listed in descending order, the 30^{th} peak hour is the 30^{th} 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 infrequently. 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 3.6. The values are illustrative, and specific data on these characteristics should be available for the project road from the local highway authorities.

Table 3.6 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.

3.11.5 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:

1. **Normal traffic:** Traffic which would travel along the same road even if no improvement were provided.
2. **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.
3. **Generated traffic:** Additional traffic that occurs in response to the provision or improvement of the road.

3.12 Road Capacity and Level of Service

3.12.1 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 utilised 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.

3.12.2 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 3.7

Table 3.7 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 motorised 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

3.12.3 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 3.8. These should not be confused with Passenger Car Units (PCU) that are used for capacity and congestion estimates for heavily trafficked roads.

Table 3.8 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	

3.12.4 Variation of PCU by Environment

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. Details of these variations are given in Chapters 14 to 17 for the gazetted road classes in Kenya.

3.12.5 Variation of PCU by Terrain Type

Passenger Car Unit values differ in rolling and mountainous terrain as shown in Table 3.9 but precise values cannot be calculated so a degree of interpolation and engineering judgement is required.

Table 3.9 Variation in PCU Values by Terrain Type

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

3.12.6 Design capacities

Typical design capacities of roads in Kenya are given in Chapters 14 to 17 for inter-urban, rural, and urban roads respectively.

The design of main traffic routes in built-up areas are based on peak-hour demands and on the average daily traffic in rural areas.

In urban settings, 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.

3.12.7 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.

Level of Service requirements for roads in Kenya are given in Chapters 14 to 17 for inter-urban, rural, urban arterial and collector roads, and urban streets, respectively.

Table 3.10 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 3.10.

Table 3.10 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.
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 3.11.

Table 3.11 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 3.12 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 3.12 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 3.12 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).

3.13 Sight Distances

3.13.1 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:

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

Stopping Sight Distance is applicable to all types of roads. 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 3.13

Table 3.13 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 3.14
The efficiency of the brakes of the vehicle	–
Gradient of the road	Varies

3.13.2 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 3.14 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 3.14 also illustrates some values obtained by various studies and shows the values used.

Table 3.14 Friction Factors

Friction Type	Road Type	Design Speed (km/h)									
		25	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	0.09
	Unpaved	0.16	0.15	0.13	0.12	0.12	0.11	0.10	0.09	0.09	0.09

3.13.3 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 roads 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 t x V), where t is perception reaction time in seconds.

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

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

Equation 3.1

Where,

d = Stopping distance (m).

t = Perception reaction time (s).

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 3.15 and Table 3.16 respectively. Local feedback from accident investigations can be used to refine the values quoted in Table 3.16 to derive 'desirable' values for enhanced safety.

Table 3.15 Desirable Stopping Sight Distance

Design Speed (km/h)	Desirable Stopping Sight Distance (m) for Gradients						
	Up			Level	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	-	-
140			350	375	400		

Table 3.16 Minimum Stopping Sight Distance

Design Speed (km/h)	Minimum Stopping Sight Distance (m) for Gradients						
	Up			Level	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
90	-	110	115	120	125	135	145
100	-	-	135	140	145	155	170
110	-	-	155	165	170	185	-
120	-	-	175	190	200	-	-
140			210	220	230		

Table 3.17 Minimum Stopping Sight Distance for Different Gradients

Design Speed (km/h)	Friction	Minimum Stopping Sight Distance (m) for Different Gradients						
		Up			Level	Down		
		9%	6%	3%	0	-3%	-6%	-9%
40	0.4	41	41	42	44	45	46	48
50	0.39	55	57	58	60	62	65	68
60	0.38	72	74	76	79	82	86	91
70	0.37	91	94	97	101	105	111	120
80	0.36	112	116	120	126	132	140	149
90	0.35	135	140	146	154	162	173	185
100	0.34	161	168	176	185	197	210	227
110	0.35	190	199	209	221	235	253	275
120	0.35	212	222	233	245	261	356	301
130	0.31	257	270	286	305	328	356	393
140	0.30	295	312	331	355	383	419	465

3.13.4 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 3.15, 3.16 and 3.17 plus a 10 m 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 5.0 m. 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.

3.13.5 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.

The intersection sight distance along the major road is determined by:

$$ISD = 0.278\ V_{major}\ t_g$$

Where,

ISD = Intersection sight distance (length of the leg of sight triangle along the major road) (m).

V_{major} = Design speed of major road (km/h).

t_g = Time gap for minor road vehicle to enter the major road.

Typical ISD values for a range of design speeds are given in Table 3.18 below.

Table 3.18 Intersection Sight Distances (m)

Design Speed of Major Road (km/hr)	120	100	85	70	60	50
Distance (m)	295	215	160	120	90	70

3.13.6 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 3.6).

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. Recognise 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 3.19. 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 0.10 m object height have been adopted.

Table 3.19 Decision Sight Distances for Various Situations (m)

Design Speed (km/h)	Situations				
	Interchanges		Lane Drop	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.*

3.13.7 Passing Sight Distance (PSD)

Passing Sight Distance is the minimum distance required by a vehicle to safely overtake or pass another slower vehicle on a two-way single carriageway roads without causing an oncoming vehicle that comes into view after the overtaking manoeuvre has started to slow below its normal speed.

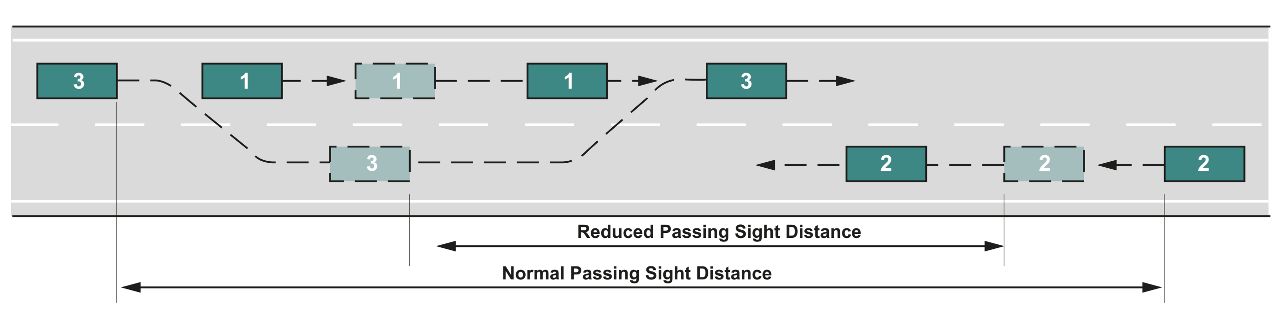
The manoeuvre is one of the most complex but important driving tasks. It is also relatively difficult to quantify for design purposes because of the various stages involved, the large number of relative speeds of vehicles that are possible, and the lengthy section of road needed to complete the manoeuvre.

Calculating the passing sight distance required for a given roadway is based on the following six assumptions:

1. The vehicle being passed travels at a constant speed throughout the passing manoeuvre.
2. The passing vehicle follows the slow vehicle into the passing section.
3. Upon entering the passing section, the passing vehicle requires some time to perceive that the opposing lane is clear and to begin accelerating.
4. While in the right-hand lane, the passing vehicle travels at an average speed that is 15 km/hr faster than the vehicle being passed.
5. An opposing vehicle is coming toward the passing vehicle.
6. There is an adequate clearance distance between the passing vehicle and the opposing vehicle when the passing vehicle returns to the left lane.

Figure 3.4 shows a schematic representation of normal and reduced passing sight distances. A driver finding that he has insufficient distance after initiating the passing manoeuvre can choose to abort the manoeuvre. The reduced passing sight distance is then the sight distance required on a two-lane road to enable the passing manoeuvre to be aborted.

Figure 3.8 Provision of Safe Passing Sight Distance



Values for minimum Passing Sight Distance at various design speeds are given in Table 3.20.

In general it is sufficient to provide the Reduced Passing Sight Distance as given in Table 3.20 in constrained situation.

Table 3.21 gives guide values for the extent to which Passing Sight Distance should be provided relative to Design Speed.

Table 3.20 Passing Sight Distances (m)

Design Speed (km/h)	**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 3.21 Guide Values for Provision of Minimum Passing Sight Distances (m)

Design Speed (km/h)	**Minimum Proportion of Road with Reduced Passing Sight Distance**		
	ADT < 1000 pcu	ADT = 1000 - 3000 pcu	ADT > 3000 pcu
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

3.13.8 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 one (1) degree is usually used for calculation purposes. At speeds above 80 km/hr, only large, light-coloured objects can be perceived at the generally accepted stopping sight distances.

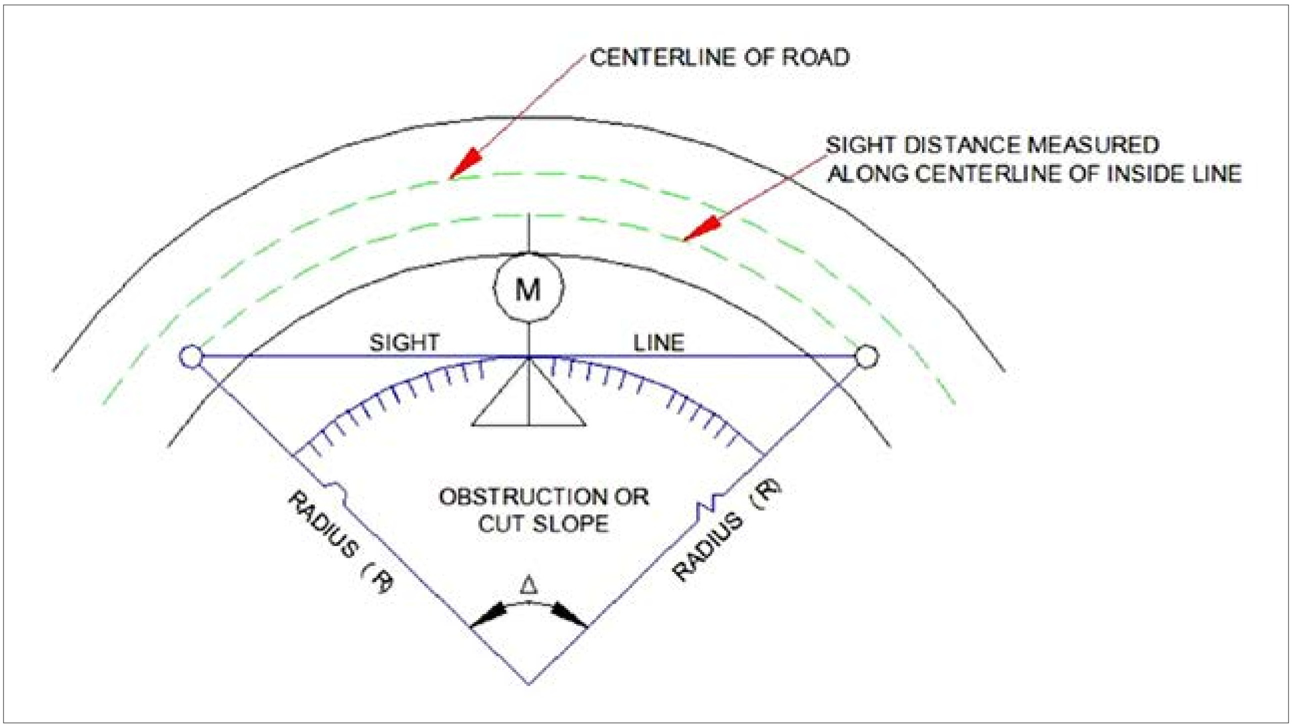
3.13.9 Control of Sight Distance

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 - 1.05 m.
   b. Object height for Stopping Sight Distance - 0.10 m.
   c. Object height for Meeting and Passing Sight Distance – 1.10 m.
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 3.5 and Figure 3.6. 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 earthworks 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 3.9 Sight Distance for Horizontal Curves



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

Length of Sight Line (S) = $2R\ sin\ (\Delta/2)$

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

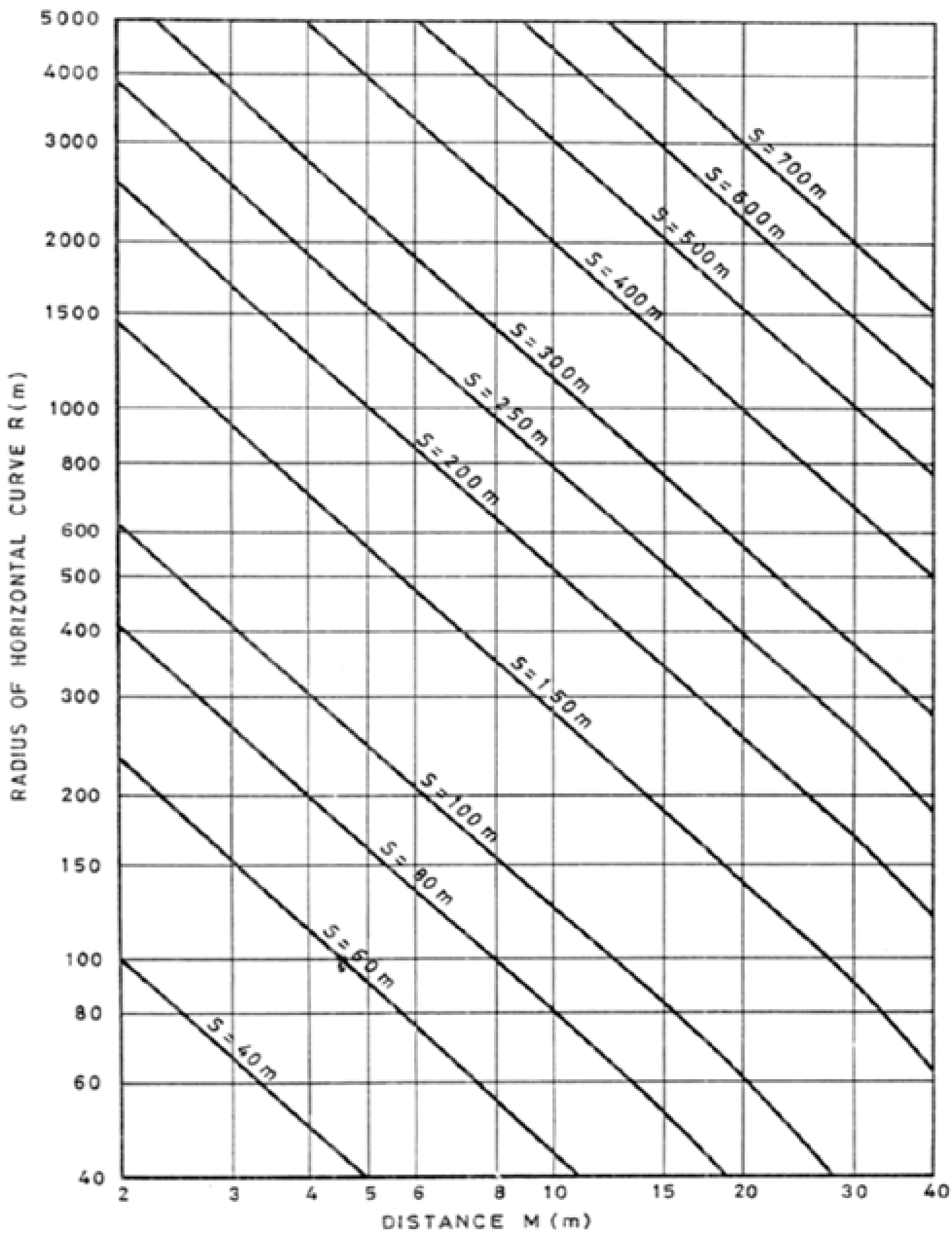
Where, Δ = Deflection angle (°).

Example:

Radius = 1000 m, Δ = 200;

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

Figure 3.10 Sight Distance for Horizontal Curves



3.13.10 Sight Distances for Pedestrians and Cyclists

At intersections, drivers of motorised traffic and cyclists and pedestrians must be afforded adequate sight distances to allow for making of correct judgements and taking safe movement actions.

The required sight distances are mainly dependent of the speed of the vehicles on the road and the distance to be crossed.

Crossing speeds for pedestrians are dependent on age, health, loads carried and hindrances by other pedestrians. Crossing speeds for pedestrians are usually between 3 km/hr (slow) and 6 km/hr (fast, running).

Cyclists' crossing speeds are usually approximately 4.5 km/hr on average.

Sight distances for pedestrians and cyclists at crossings are given in Table 3.22.

Table 3.22 Sight Distances for Pedestrians and Cyclists for Crossing for Average Speed of 3 km/hr

Crossing Distance (m)	**Time Needed for Crossing** (s)	**Average Speed of Motorised Traffic** (Km/Hr)	**Sight Distance** (m)
5.0	4.5	30	40
		50	65
		70	100
6.0	4.9	30	44
		50	70
		70	110
7.0	5.3	30	47
		50	75
		70	120
8.0	5.6	30	50
		50	80
		70	125

3.13.11 Safe Stopping Distances for Cyclists

Cyclists must continuously be aware of traffic around them as well as the physical road environment to be able to perceive and react to dangerous situations. The perception-reaction time of a cyclist is approximately 2 seconds. For a deceleration rate of 1.5 m/sec the safe stopping distances are given in Table 3.23.

Table 3.23 Safe Stopping Distances for Cyclists

Bicycle Speed (km/hr)	**Safe Stopping Distance** (m)
10	8
15	14
20	21
25	30

3.14 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 recognises:

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-motorised traffic.
5. Adequate provision for other non-motorised 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 Audit**.

3.15 Environmental and Social Considerations

3.15.1 General

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.

3.15.2 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.

3.16 Economic and Financial Considerations

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 Management, 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.

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 walkways.

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 walkways should be included in the cross-section where appropriate (at the cost of the width of the shoulders).

The basic nomenclature of cross-section components of a road in a rural setting is given in Figure 4.1 and Figure 4.2, and largely applies to the urban setting as well.

Figure 4.1 Basic Cross Section Components Nomenclature – Single Carriageway

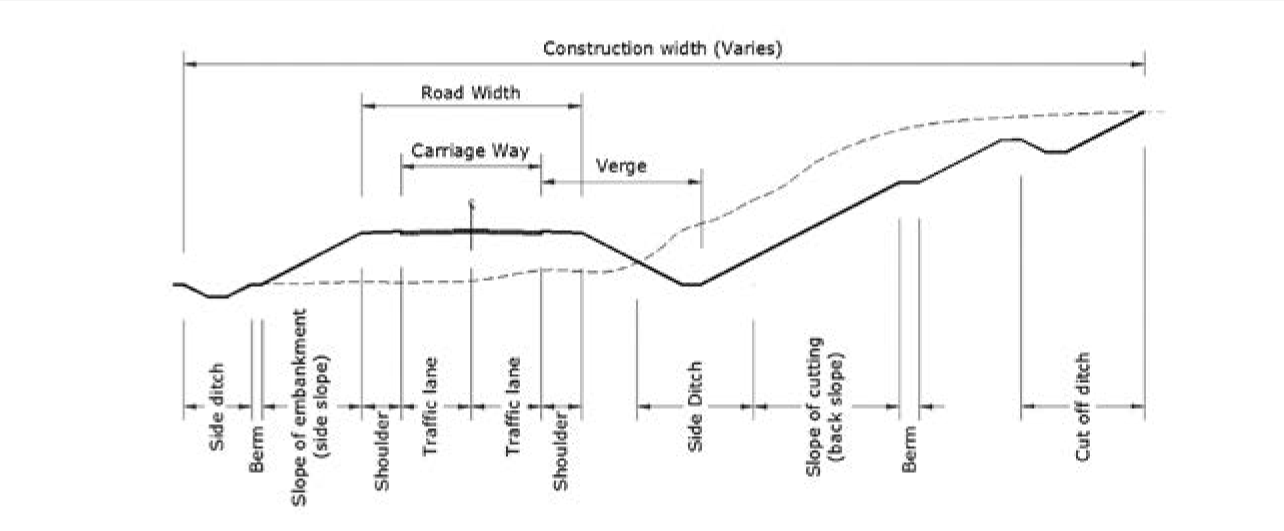
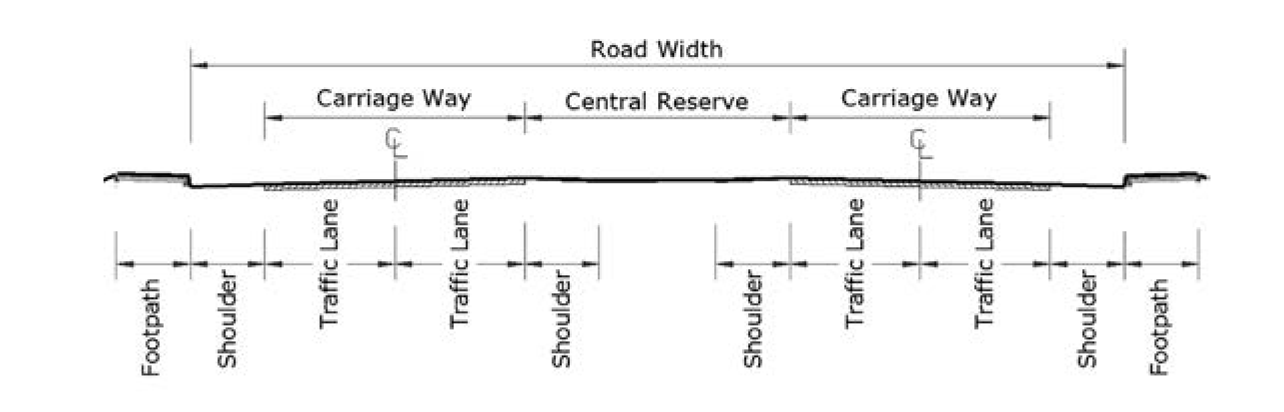


Figure 4.2 Basic Cross Section Components Nomenclature – Dual Carriageway



1 2 3 4 Cross Section

4.2 Selection of Cross Section

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 basic 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 walkways 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 and 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 optimised allocation of space to provide the most appropriate road 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. Handcarts.
4. Bicycles.
5. Motorcycles.
6. Public transport.
7. Cars.
8. Good vehicles.

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.

4.3 Traffic Lanes

4.3.1 General

There are many types of lanes that apply to cross sections, 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 Traffic Control Facilities and Communication Systems: 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-Second Rule, Section 3.9.2).

4.3.2 Typical Lane Width for Paved Roads

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.

Allowable through lane widths in Kenya are as given in Table 4.1.

Table 4.1 Typical Lane Widths For Paved Roads

<table>
<tr><th>Road Category</th><th>Road Class</th><th>Functional Class/Link Role</th><th>Lane Width Range (m)</th></tr>
<tr><td rowspan="3">Inter-urban roads</td><td>A</td><td>International highways</td><td>3.65</td></tr>
<tr><td>B</td><td>National highways</td><td>3.65</td></tr>
<tr><td>C</td><td>Primary roads/ inter-county roads</td><td>3.50</td></tr>
<tr><td rowspan="3">Rural roads</td><td>D</td><td>Secondary roads/ inter-sub-county roads</td><td>3.50</td></tr>
<tr><td>E</td><td>Minor roads/ sub-county roads</td><td>3.25 - 3.50</td></tr>
<tr><td>F, G, P, R, S, W, T, U</td><td>Local roads</td><td>3.00 – 3.50</td></tr>
<tr><td rowspan="3">Urban roads</td><td>UA</td><td>Urban arterial roads</td><td>3.75</td></tr>
<tr><td>UC</td><td>Urban collector roads</td><td>3.75</td></tr>
<tr><td>UL</td><td>Local urban streets</td><td>3.00 – 3.50</td></tr>
</table>

4.3.3 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 multi-lane 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 3.0 m should be provided.

4.3.4 Trafficked Lanes Crossfall

Undivided pavements with two or more lanes on tangents or flat curves shall have a high point, or crown, along the centreline with uniform downward slopes towards each edge in order to facilitate surface water run-off and to prevent mud from the verge from spreading over the carriageway. This downward slope is termed crossfall.

For rural roads with bituminous pavements the minimum crossfall shall be 2.5 % and 4 % for rural roads with gravel pavements. A minimum crossfall of 5 % shall be used for earth roads.

Auxiliary lanes shall have a crossfall of the same direction and rate as the adjacent lane.

The crossfall on gravel or grass shoulders shall in all cases be 4.0 %.

1
2
3
4
Cross Section

4.4 Shoulders

4.4.1 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 stabilised shoulders are required to minimise 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.

4.4.2 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 4.2 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 4.2 Recommended Widths of Shoulders

Road Category	Most Common Functional Class and Type	Shoulder Width (m)
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 UA and UC 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.

4.4.3 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 Roads as guided in Chapter 1.

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

Table 4.3 Shoulder Crossfall

Material in Shoulder	Crossfall
Earth/Grassed	4%
Gravel or crushed rock	4%
Full depth pavement with bitumen seal or asphalt wearing course	4%
Concrete	4%*

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

4.4.4 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.

4.5 Walkways

4.5.1 General

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

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

The decision as to whether a walkway 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.

Separate walkways shall be provided on all arterial and collector roads. The required widths depend on the peak volume of pedestrians anticipated per hour. On average a width of 1 m is required for each 1000 pedestrians per hour.

The width of a walkway for pedestrians is dependent on its location, purpose, and the anticipated demand on the facility. Minimum widths are as tabulated in Table 4.4 . If the walkway is immediately adjacent to the kerbing, the minimum width should be increased by 0.6 m. 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 walkway 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).

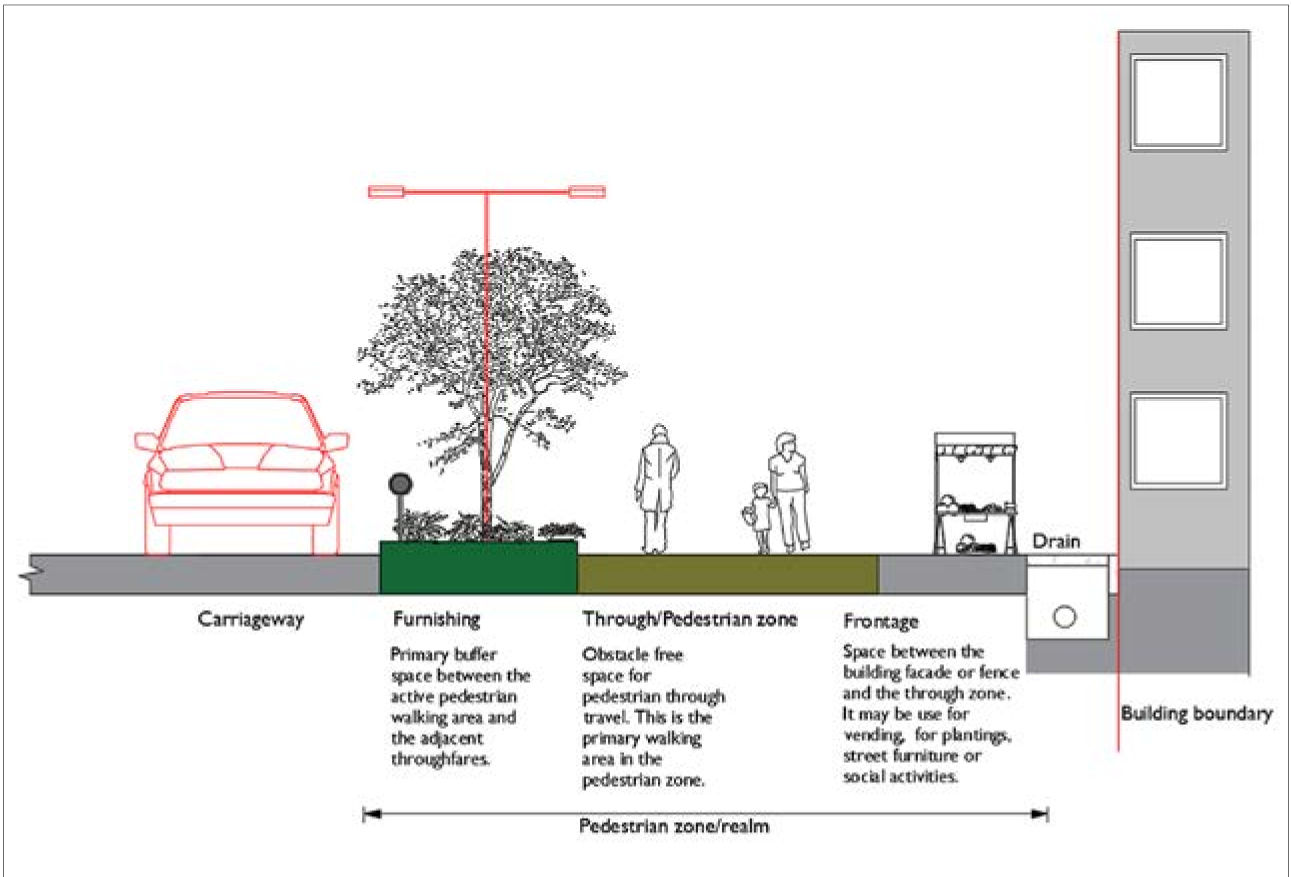
Walkways must be continuous, not too sinuous and be unimpeded by obstructions.

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 1 in 16, where this is not possible, they should be no steeper than 1 in 10. A kerb height of 150 mm would thus require a ramp length of 2.4 m. Ideally there should be a clear walkway width of 1.5 m beyond the top end of the ramp so that, where a ramp is provided, the overall walkway 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.

4.5.2 Walkway Zones

There are three distinct walkway zones as shown in Figure 4.3 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.

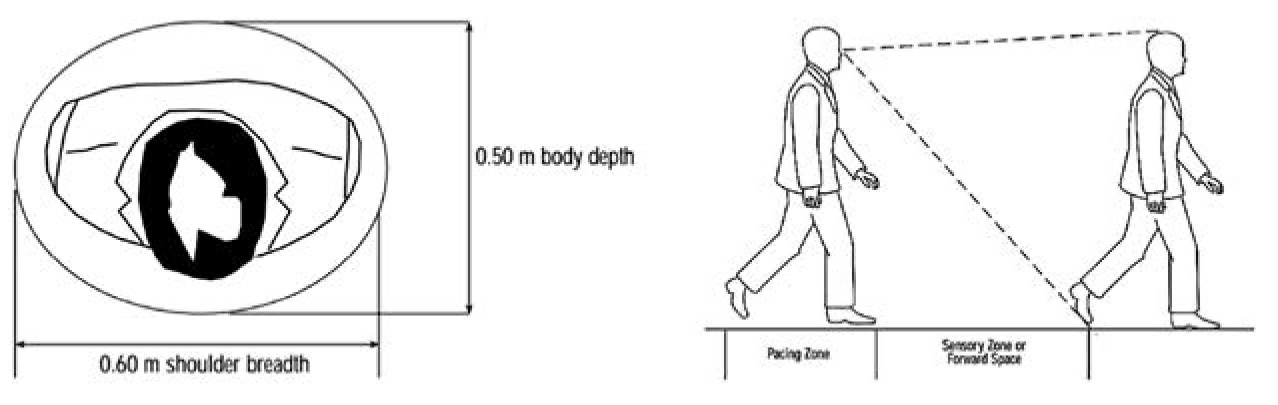
Figure 4.3 Distinct Zones of Walkway



4.5.3 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 square metres. In evaluating a pedestrian facility, an area of 0.75 sqm is used as 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.

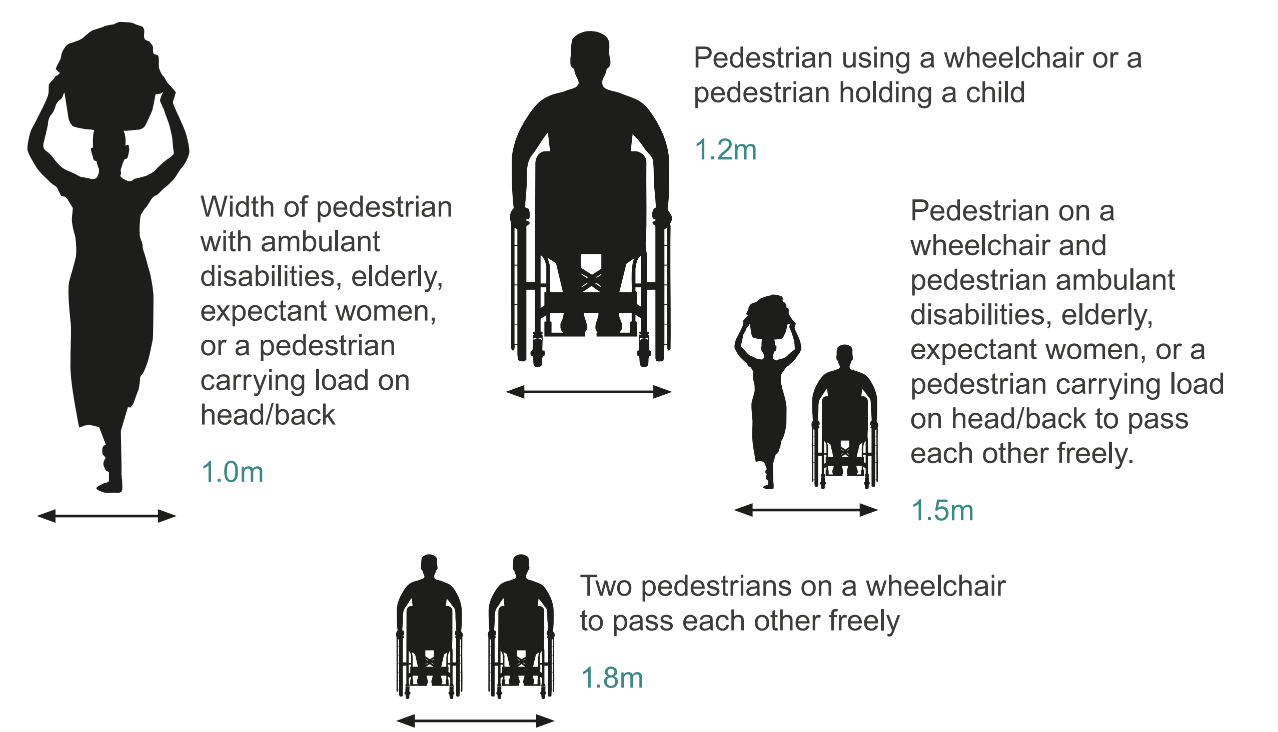
Figure 4.4 Pedestrian Eclipse And Pedestrian Walking Space Requirement



4.5.4 Width of Walkways

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 walkway 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.

Figure 4.5 Pedestrian Path Requirements



To avoid interference when two pedestrians pass each other, each pedestrian should have at least 0.8 m of walkway width. Minimum requirement for other pedestrians is as shown in Figure 4.5.

Table 4.4 Recommended Walkway 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 0.15 m allowance both sides. • Not adequate for commercial or shopping
High pedestrian volumes	3.0 (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 walkway 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 organised 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.

4.5.5 Walkways in Rural Areas

Walkways are not normally provided in rural areas. Table 4.5 indicates conditions where paved walkways are recommended in rural areas. Walkway 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 walkways is at the edge of the road reserve. In rolling or mountainous terrain through cuts and fills, such a walkway is not comfortable for walking and so pedestrians often prefer to walk on the more level surface of the shoulder. In level terrain, the walkways 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 walkways 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 graded 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 4.5 Recommended for Paved Walkways in Rural Areas

Walkway	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

4.5.6 Walkway Crossfall

The crossfall of a paved walkway may vary from flat (for example with slotted drain tops) to 2.5%. Walkways 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.

4.6 Cycle Lanes

4.6.1 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.

4.6.2 Cycle Lane Capacity

For design purposes a peak capacity of single lane (1.2 m wide) bicycle lane separated from other traffic shall be 1000 cyclists/hr.

For a 2.0m wide track, one-way flow the capacity shall be taken as 2000 cyclists/hr.

4.6.3 Widths of Cycle Lanes

When the volumes of cyclists are not able to be determined, the widths shown in Table 4.6 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 4.6 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 given in Table 4.7.

Table 4.7 Shared Paths for Cyclists and Pedestrians

Situation	Suggested Minimum Width (m)		
	UL Road Class	UA & UC Road Class	Recreational Paths
Shared Path	2.0	3.0	3.0

Figure 4.6 Shared Paths for Cyclists and Pedestrians

4.6.4 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.

4.6.5 Grades

Excessive gradients on hills can be unpleasant to cyclists and act as a deterrent to bicycle riding, designers should strive to minimise 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.

4.6.6 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.

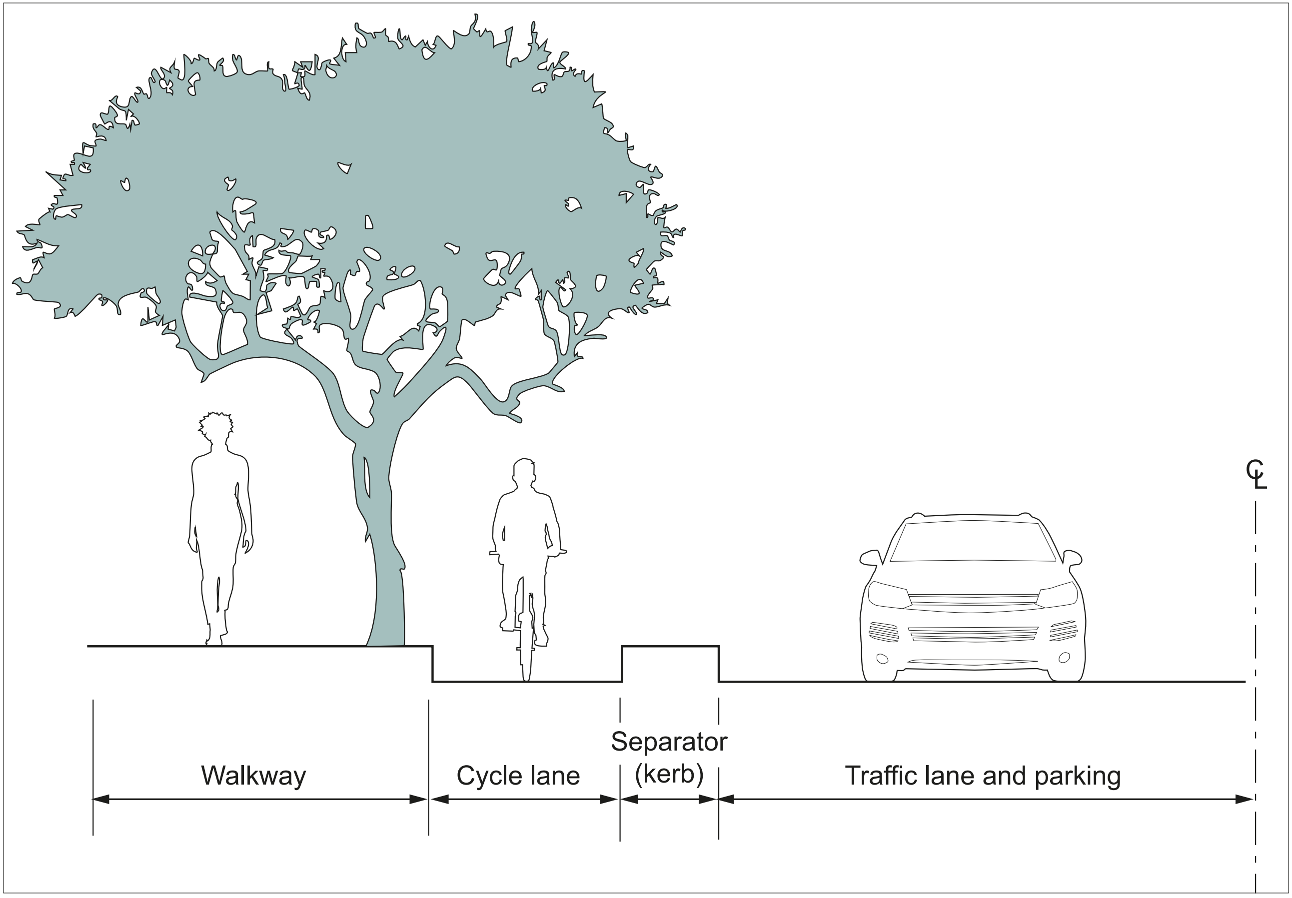
4.6.7 Cycle Lane Crossfall

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-motorised traffic facilities beyond the drainage ditch and away from the carriageway.

Figure 4.7 Cross-section of a Protected Bicycle Lane



4.7 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. with characteristics as given in Table 4.8.

Table 4.8 Motorcycle Physical Characteristics

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

4.8 Medians

4.8.1 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 serve to:

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.

4.8.2 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. Figure 4.8 shows pictorial views of typical median treatments whilst Table 4.9 provides detailed descriptions.

Table 4.9 Descriptive Details of Typical Medians

Length	Width
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 (grassed) for aesthetics and to restrict headlight glare.

Figure 4.8 Typical Median Treatments

4.8.3 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 4.10).

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, 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 4.10 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 0.75 m 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

4.8.4 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 4.11. 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 4.11 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

4.8.5 Median Slope Treatments

Figure 4.9 Median Slope Treatment (30-50 m Corridor)

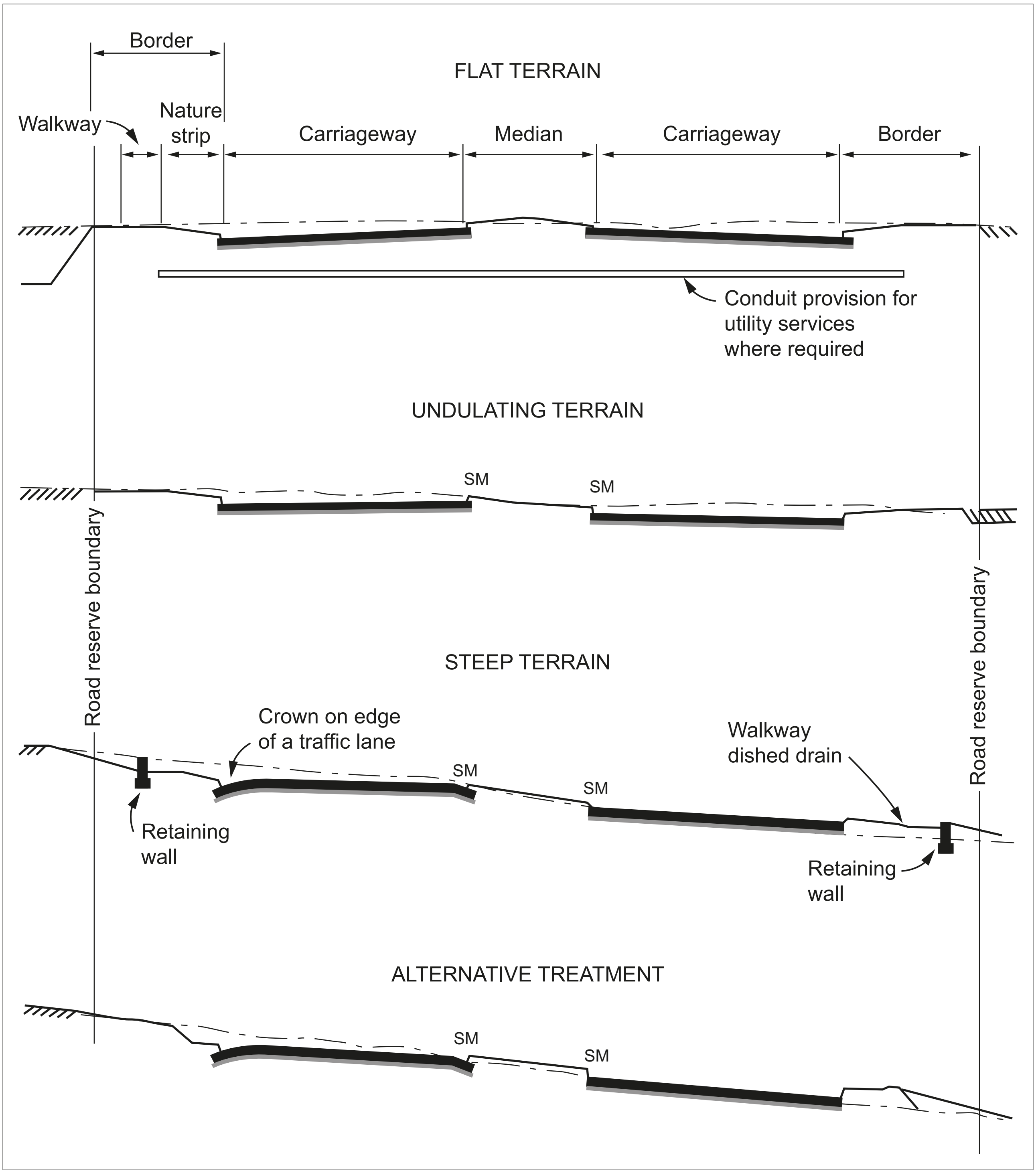
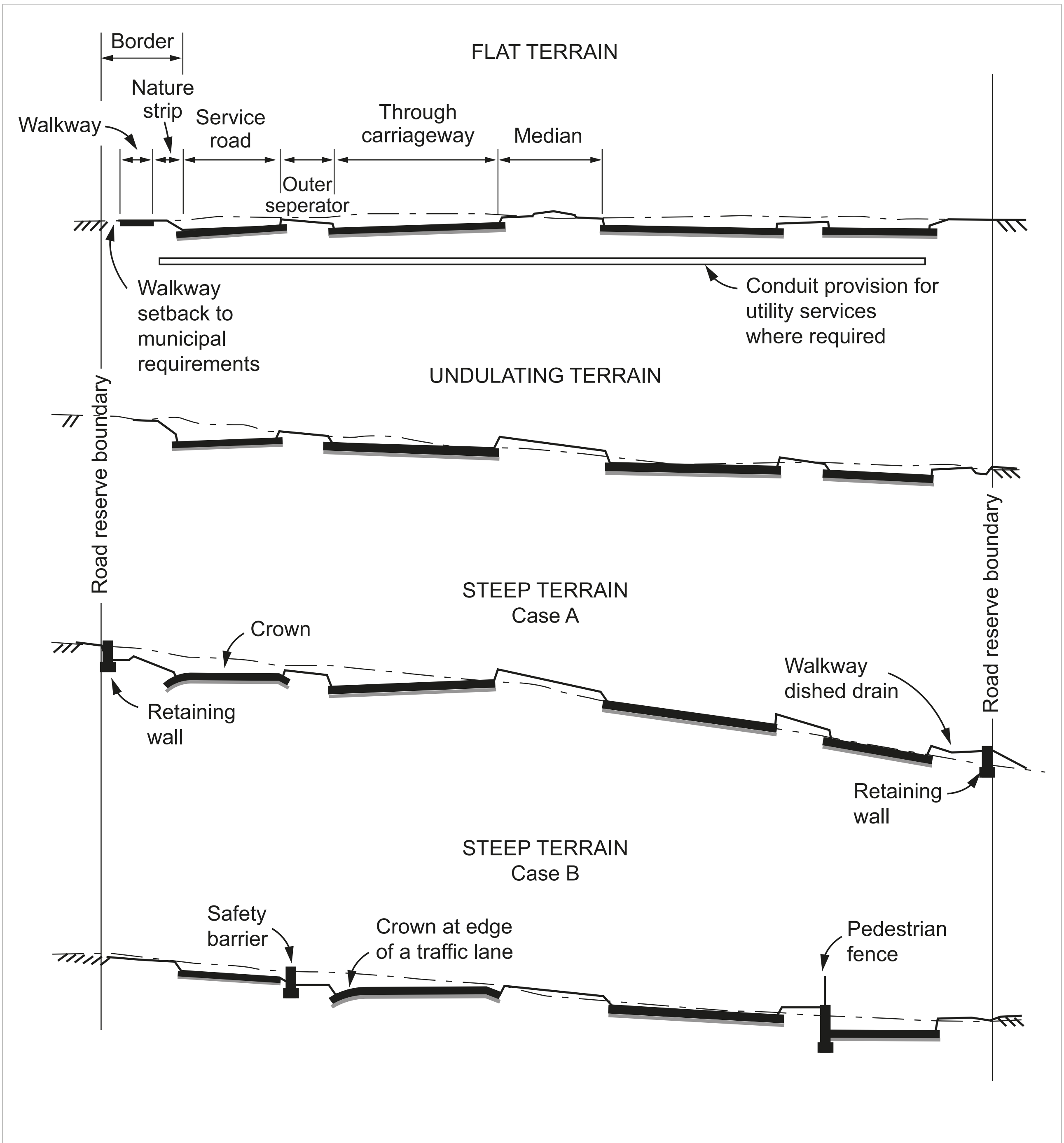


Figure 4.10 Median Slope Treatment (>50 m Corridor)



4.8.6 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.

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3
4
Cross Section

4.9 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.

4.10 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 4.12.

Table 4.12 Recommended Width 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] With staggered parking, the width can be reduced to 5.5m*

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.

4.11 Transit Lanes

4.11.1 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 multi-lane 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.

4.11.2 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 dedicated 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.

4.12 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.

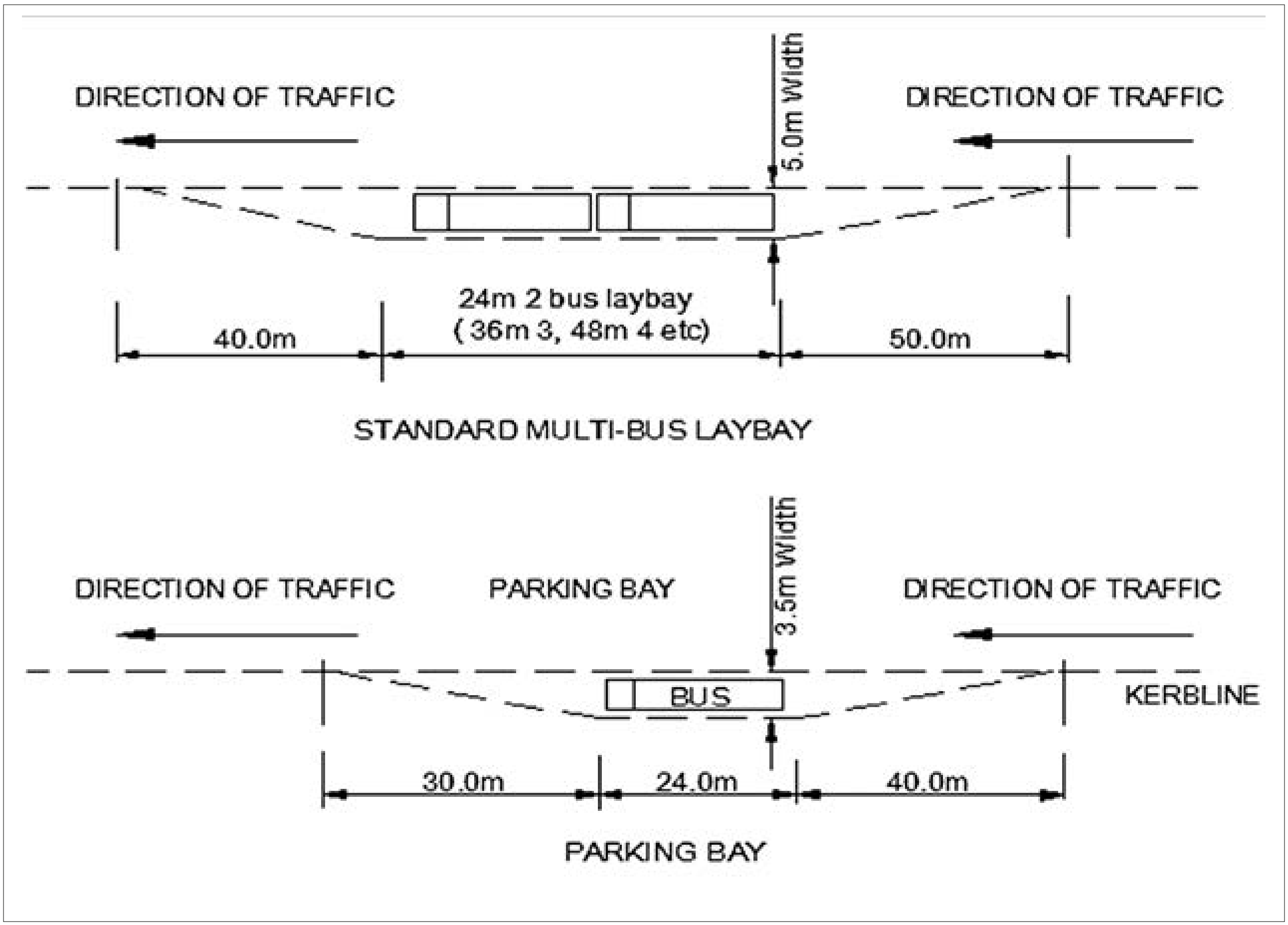
The deceleration lane should be tapered at an angle flat enough to encourage the bus or taxi operator to pull completely clear of the through lane as in Figure 4.11. A taper of 10:1, longitudinal to transverse, is a desirable minimum.

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 4.11 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 walkways should also be provided at bus lay-bys.

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.

Figure 4.11 Bus Lay-bys and Parking Bays



4.13 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 4.13 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 4.13 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

4.14 Pedestrian Bridges

The minimum clear width of a pedestrian footbridge should be 1.8 m. 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 3.0 m. 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.

4.15 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 and of areas as indicated in Table 4.14.

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.

1
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Cross Section

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 undersise remnants should also be considered. Where cars occupy motorcycle spaces, installation of kerbing may be required. The minimum size of a motorcycle space is as given in Table 4.8.

Table 4.14 On-Street Parking Space Dimensions

Typical Parking	Space Requirement	
	Width	Length
Parallel parking	2.4 m	6 m
Angled parking at 45° [a]	2.4 m to 2.7 m	5.4 m

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.

4.16 Widening

4.16.1 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 centreline 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 4.15 shows the values to be used.

Table 4.15 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]	Widening (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:

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

- ***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.*
- ***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 4.16). For larger vehicles, there are several swept path analysis programs that are available.*

Table 4.16 Widening on Buses and Trucks

Radius of curve (m)	Single Unit Bus or Truck, Widening (m)	19 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

4.16.2 Fill Widening

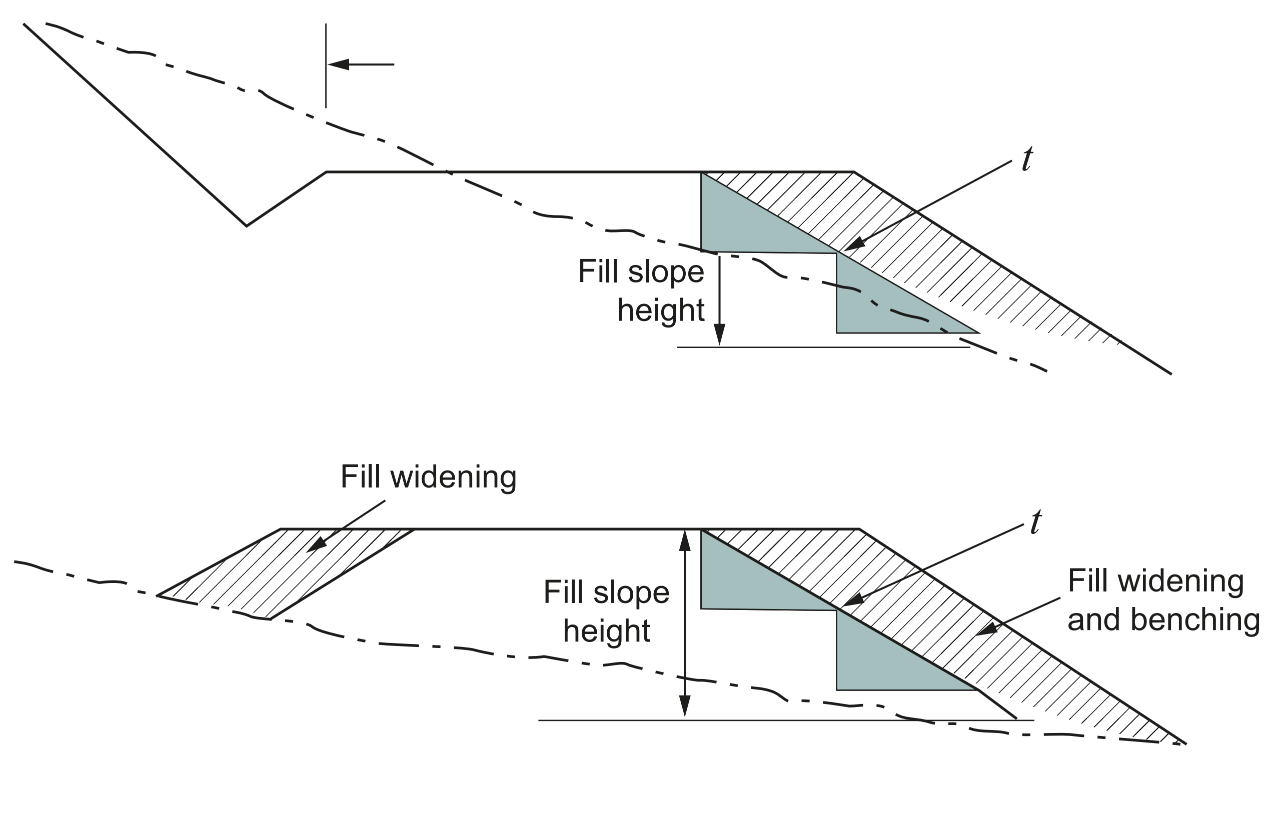
If fill widening is required, benching should be applied (Figure 4.12) which modifies the road width independent of travelled road width.

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 4.12 Fill Widening and Benching



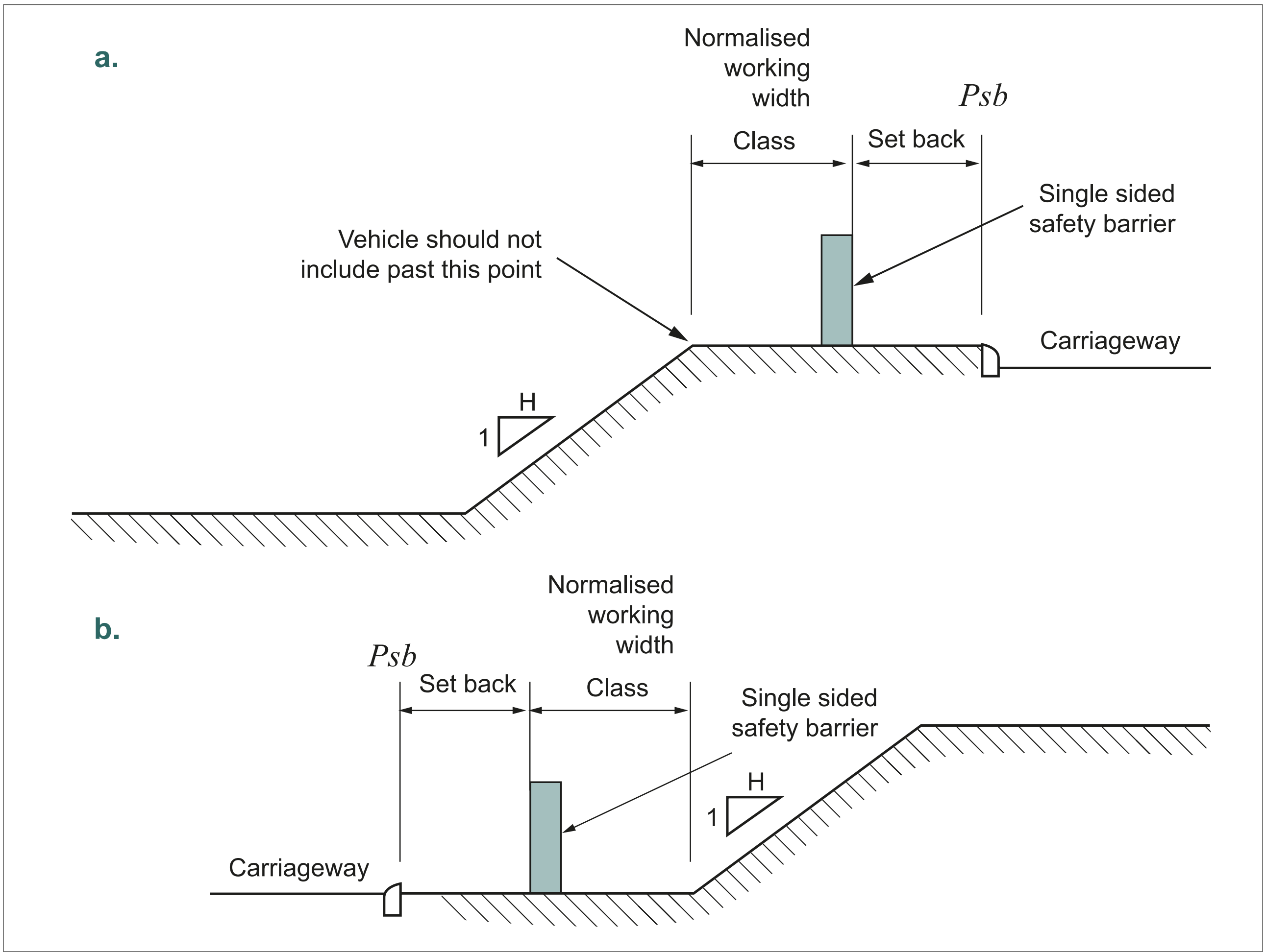
Cut slopes are inherently more stable than fill slopes. The designer should try to minimise fill slope length by 'pushing' the alignment into the hill side to minimise erosion. If economical, this will result in longer cut slopes and add slight to moderate cuts at the centreline. The result will be a moderate fill slope as shown in Figure 4.12 with no additional fill widening required.

4.16.3 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 within the clear zone is not reasonably practicable in terms of engineering, economic, environmental or sustainability considerations (see Sections 3.16 and 3.15).

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 4.13 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. 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 4.17 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 0.6 m. 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 4.17 but also no less than the working width of the safety barrier minus the actual width of the safety barrier.

4.16.3.1 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.

4.16.4 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 3.12.3 and the modifications to specifications are summarised in Table 4.18 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 4.18 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

4.16.5 Additional Width Based on Surrounding Land Use

The more populated areas in urban 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 walkway 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 12.

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

4.16.6 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.

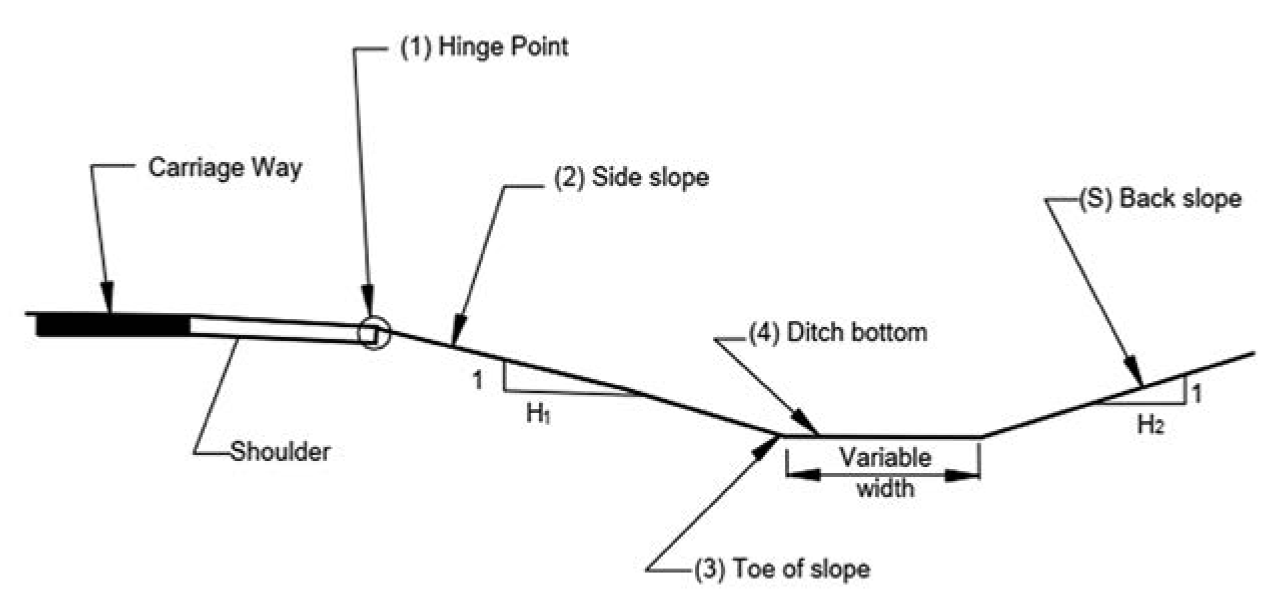
Edge marking is a convenient method of indicating the boundary between the lane and the shoulder. Rumble strips (Refer to **RDM Volume 6: Traffic Control and Communication Systems - Part 4 Other Traffic Control Facilities**) can also be used and have been shown to reduce the rate of run-off-road incidents by up to 25%.

4.17 Side Slopes and Back Slopes

Three regions as shown in 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 4.19.
3. Toe of the slope (intersection of the fore slope with level ground or with a back slope, forming a ditch).

Figure 4.14 Details of the Road Edge



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Cross Section

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 3 – Retaining Structures Design).**

Table 4.19 Recoverable and Non-recoverable Slopes

Standard	Modification
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 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 4.20 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.

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.

Table 4.20 Slope Ratio Table - Vertical to Horizontal

Material	Height of Slope (m)	Side Slope (V:H)		Back Slope V:H
		Fill	Cut	
Earth Soil	0.0 – 1.0	1:4	1:2	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		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		

4.18 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.

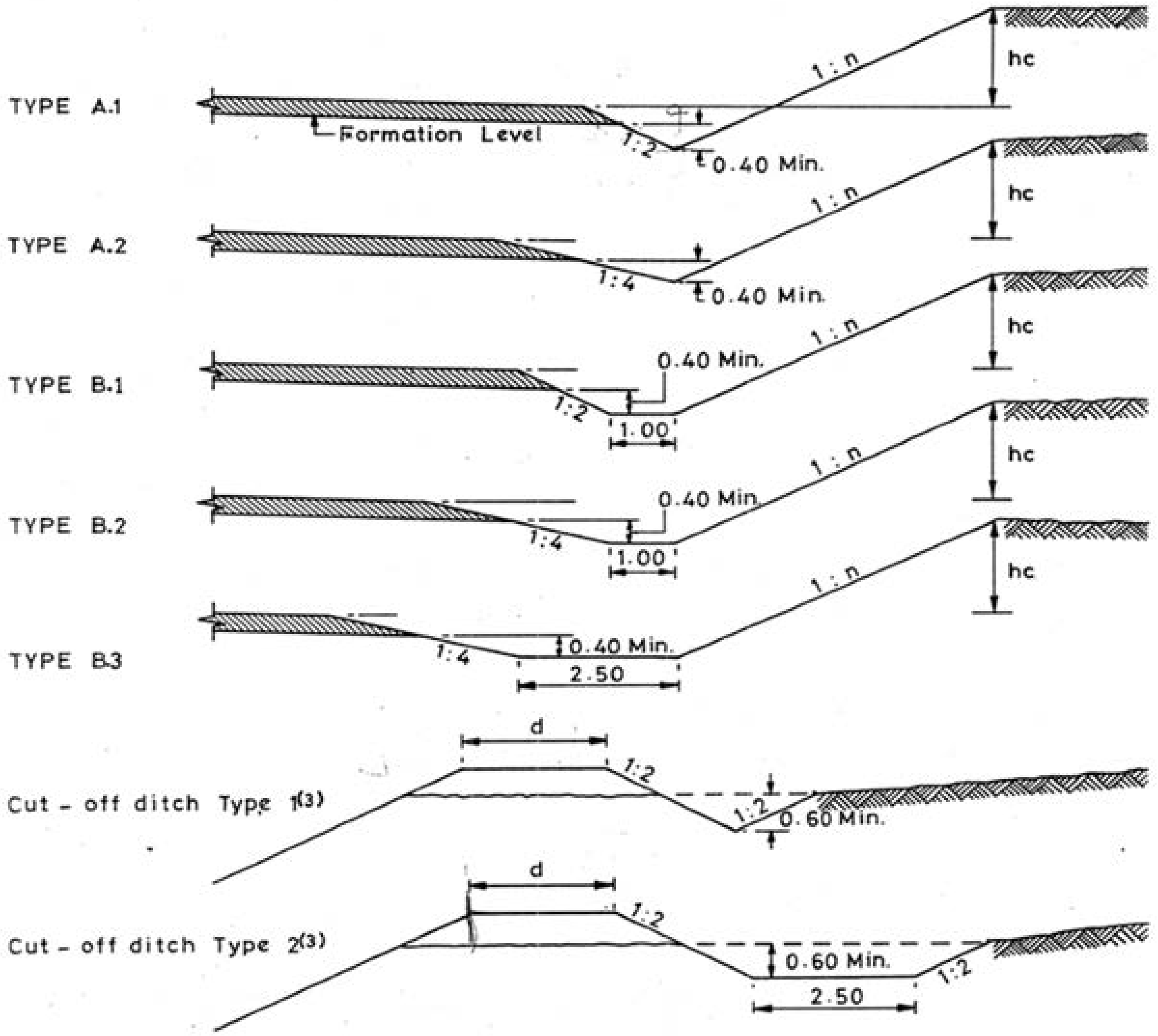
The types of side ditches and cut-off ditches which will generally be used are shown on Figure 4.13 whilst Table 4.21 and Table 4.22 gives guidelines regarding the choice of each particular type.

The guidelines given in the afore-mentioned tables are based upon general economic and aesthetic considerations. However, the type of side ditch selected must be checked to ensure that it will carry the expected flow without running so deep as to wet the road pavement nor so fast as to cause scour.

Due to their location, cut-off ditches are usually difficult to maintain and should therefore, whenever possible, be constructed as "natural permanent depressions" with as gentle side slopes as possible.

For detailed design of side drains and drains, reference should be made to **Road Design Manual: Volume 2 - Hydrology and Drainage Design - Part 2: Drainage Design of Highways, Rural and Urban Roads.**

Figure 4.15 Typical Side Ditches and Cut-off Ditches



Height of cut hc	Slope (1:n) of cutting in earth (1)	Distance, d, between edge of cutting and cut - off ditch.
hc ≤ 1.00	1 : 3	d = 1.0
1.00 < hc ≤ 3.00	1 : 2	d = 4.0
hc > 3.00	1 : 1.5 (2)	d ≥ 5·0

(1) In rock the slope 1 : n will vary from 1 : 0.33 to 1 : 0.10
(2) If earth not stable at 1 : 1.5, the slope maybe decreased to 1 : 2
If natural slope steeper than 1:1·5, then the natural slope may be used as the cut slope.
(3) The slopes into the ditch should be as indicated or flatter.

Table 4.21 Guideline for Selection of Side Ditch Types

Standard	Modification	
A1	Hilly to mountainous terrain with heavy earthwork.	Back slope to be varied according to stability of cut material. Slope should be stable and enable vegetation to establish.
A2	Rolling terrain with moderate earth work. Hilly to mountainous terrain where flatter ditch than Al is required due to capacity and/or velocity limitations.	As for A1
B1	Hilly to mountainous terrain where flatter ditch than Al is required due to capacity and/or velocity limitations.	As for A1 Width may be increased if fill material is required
B2	Rolling terrain with moderate earthwork where a flatter ditch than A2 is required due to capacity and/or velocity limitation.	As for B1
B3	Flat terrain with little earthwork. Rolling terrain with moderate earthwork where a flatter ditch than B2 is required to capacity and/or velocity limitations.	As for B2

Table 4.22 Guideline for Selection of Cut-off Ditch Type

Cut-off Ditch	To Be Used Under the Following Conditions	Remarks
1	Hilly to mountainous terrain with heavy earthwork.	
2	Large catchment area and in areas liable to silting and/or damage to the ditch profile by pedestrians, etc.	

4.19 Bridge Widths and Vertical Clearances

The minimum horizontal clearance between the carriageway edge and the face of an abutment or pier shall generally be 1.50 m. In exceptional cases this standard may be reduced to 1.0 m.

The minimum horizontal clearance between the edge of a cycle track or walkway and the face of a structure shall be 0.25 m.

No reduction in cross-sectional standard shall be made to the carriage way or shoulders over pipe or box culverts.

The standard minimum headroom or clearance under bridges or tunnels shall be 5.5 m for all classes of road . This clearance should be maintained over the carriageway(s) and shoulders. Where future maintenance of the carriageway is likely to lead to a raising of the road level, then an additional clearance of up to 0.1 m may be provided. Where the existing headroom exceeds the standard minimum, and a reduction would affect local industry, then a greater clearance may exceptionally be justified. Headroom for road over standard-gauge rail bridge shall be 7.2 m to allow for double stacking.

The minimum headroom or clearance over cycle tracks and walkways shall be 2.4 m.

To determine the standard bridge cross-sections refer to **RDM Volume 4, Part 2 Bridge and Culvert Design.**

The design of the bridge shall allow for the removal, if required at a later date, of the walkway and its conversion into a traffic lane.

4.20 Recommended Cross-sections

Dimensional details of recommended cross-sections for inter-urban, rural and urban roads are given in Table 4.23 and Table 4.24.

In any rural or 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 highway or street approach will determine the selection of the elements required in a particular cross section.

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.

Chapters 14 to 17 provides the designer with geometric design details applicable to the design standards set for inter-urban, rural roads and urban roads.

These standards are to be used as guide by the designer.

Table 4.23 Dimensional Details of Standard Cross Sections For Inter-Urban And Rural Roads

Cross Section Type	Surface Type	No. of Carriageway	One Direction Cross-section elements / minimum widths (m)							Central Median (m)	Main Features	Access Control
			Frontage Road			Outer Separator	Roadway					
			Shoulder	MT Lane	Shoulder		Outer Shoulder	MT Lane	Inner Shoulder			
R1	Paved	4	1.50	2 x 3.25	1.50	2.00	2.70	2 x 3.65	1.50	2.00	4-carriageway road with 2-way, 2-lane frontage road, outer separator and 2 to 6-lane MT expressway with outer and inner shoulders in each direction and central median.	Full
R2	Paved	4	1.50	1 x 3.25	–	2.00	2.70	2 x 3.65	1.50	2.00	4-carriageway road with 1-way frontage road, outer separator and 2 to 6-lane MT expressway with outer and inner shoulders in each direction and central median.	Full
R3	Paved	2	–	–		–	2.70	2 x 3.65	1.00	2.00	2-carriageway road with 2 to 6-lane MT expressway with outer and inner shoulders in each direction and central median.	Full/Partial
R4	Paved	1	–	–		–	2.50	1 x 3.65	–	–	Single carriageway road with 2-way 3.65 m MT lane and 2.50 m shoulders in each direction.	Partial
R5	Paved	1	–	–		–	2.00	1 x 3.50	–	–	Single carriageway road with 2-way 3.50 m MT lane and 2.00 m shoulders in each direction.	Partial
R6	Paved	1	–	–		–	1.50	1 x 3.25	–	–	Single carriageway road with 2-way 3.25 m MT lane and 1.50 m shoulders in each direction.	Partial/ Unrestricted
R7	Paved	1	–	–		–	1.00	1 x 3.00	–	–	Single carriageway road with 2-way 3.00 m MT lane and 1.00 m shoulders in each direction.	Unrestricted
R8	Gravel	1	–	–		–	–	1 x 3.00	–	–	Single carriageway road with 2.70 to 3.00 m MT lane in each direction.	Unrestricted
R9	Earth	1	–	–		–	–	1 x 3.00	–	–	Single carriageway road with 2.70 to 3.00 m MT lane.	Unrestricted

Table 4.24 Dimensional Details of Standard Cross Sections for Urban Roads

Cross Section Type	Surface Type	No. of Carriageway	Cross-section elements / minimum widths (m)								Central Median (m)	Main Features	Access Control
			NMT Lanes		Parking Bay	Service Road	Outer Median	Roadway					
			Walkway	Pedal / Cycle Track				Outer Shoulder	Traffic Lane	Inner Shoulder			
U1	Paved	4	2.00	2.00	3.50	2 x 3.50	2.00	2.00	3 x 3.65	1.00	2.00	4-carriageway road with 2-way, 2-lane service road, outer separator and 2 to 6-lane MT expressway with outer and inner shoulders in each direction and central median.	Full
U2	Paved	4	2.00	2.00	3.50	1 x 3.50	2.00	2.00	2 x 3.65	1.00	2.00	4-carriageway road with 1-way service road, outer separator and 2 to 6-lane MT expressway with outer and inner shoulders in each direction and central median.	Full
U3	Paved	3	3.00	–	3.50	1 x 3.50	2.00	2.00	1 x 3.65	–	–	3-carriageway road with 1-way service road with packing bay and 2-way expressway. Pedal-cycle to use the service road.	Full
U4	Paved	2	2.00	1.50	–	–	–	2.00	2 x 3.65	1.00	2.00	2-carriageway road with 2 to 3-lane expressway and raised walkway and pedal-cycle track in each direction and central median.	Full
U5	Paved	1	3.00	2.00		–		1.50	1 x 3.50	–	–	Single carriageway road with 2-way 3.50 m MT lane and 2.00 m shoulders in each direction.	Partial
U6	Paved	1	3.00	–	3.50	–	–	–	1 x 3.65	–	–	2 or 1-way single carriageway with 1 to 2-MT lanes, parking bay and at least 3 m width raised walkway on both sides. Pedal-cycle to use the MT lane.	Unrestricted
U7	Paved	1	3.00	3.00	–	–	–	–	–	–	–	Pedal-cycle and walkways only.	Unrestricted
U8	Paved	1	–	3.00	–	–	–	–	–	–	–	Pedal-cycle only.	Unrestricted
U9	Paved	1	3.00	–	–	–	–	–	–	–	–	Walkways only.	Unrestricted

Cross Section

5 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 steady 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 should be avoided, as some conditions might force the sharp curves which can be mitigated with speed regulation and adequate warning signs.

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 Circular Curve

The basic equation for a circular curve is as follows:

$$R_{min} = \frac{V_D^2}{127\,(e + f)} \qquad \text{Equation 5.1}$$

Where,

R_{min} = Radius (m).

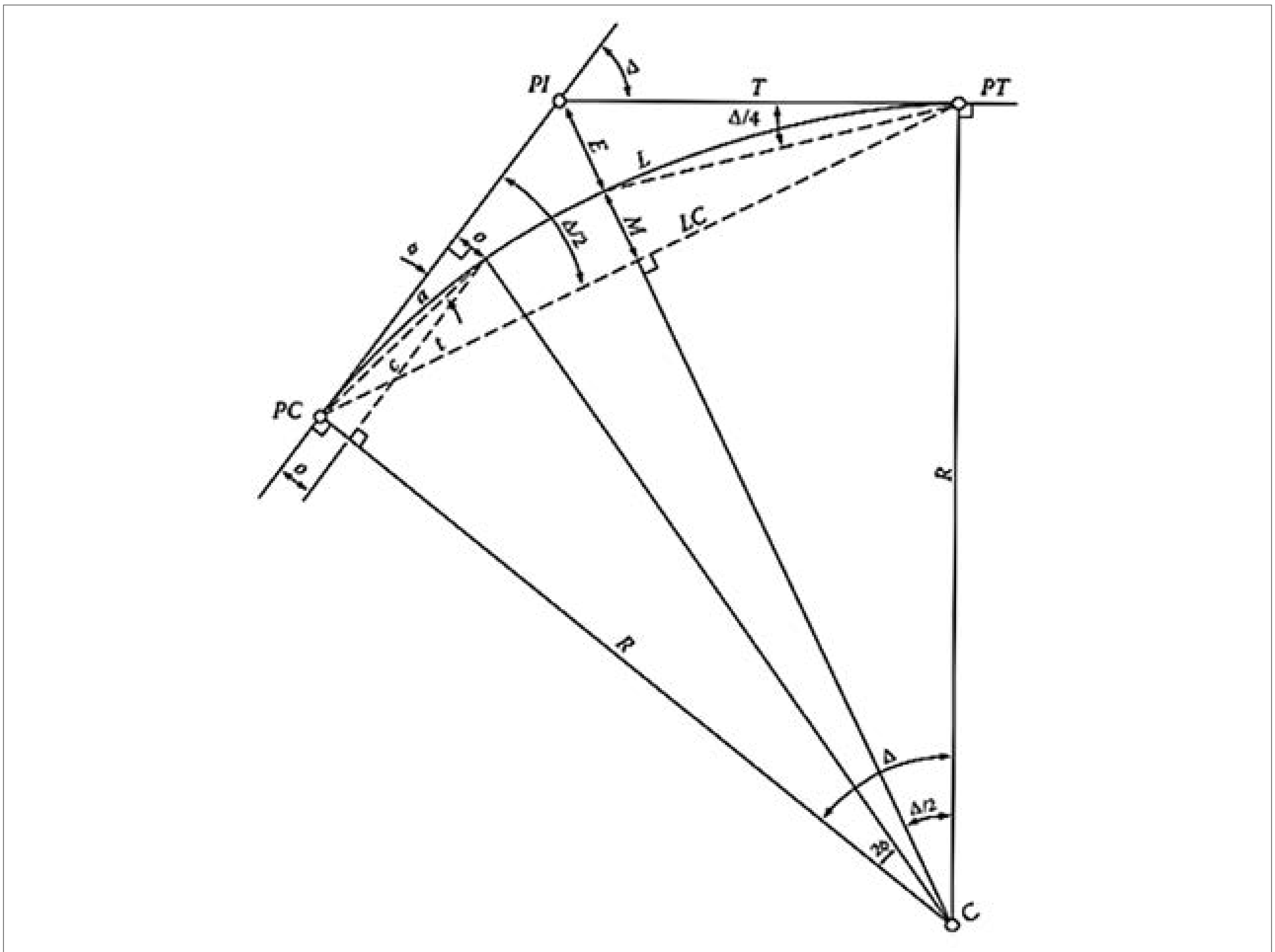
V_D = Design speed (km/h).

e = Super-elevation rate (decimal).

f = Side friction factor (decimal) (Section 3.13).

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.

Figure 5.2 Curve Formulae

$$T = R\tan(\Delta/2)$$

$$c = 2R\left(\sin\frac{90a}{\pi R}\right)$$

$$L = \frac{\Delta}{360}2\pi$$

$$\cos\phi = \frac{R-o}{2R}$$

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

$$t = R\sin 2\phi$$

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

$$o = c\sin\phi$$

$$M = R\left(1-\cos\left(\frac{\Delta}{2}\right)\right) = E\cos\left(\frac{\Delta}{2}\right)$$

$$o = R - \sqrt{R^2 - t^2}$$

$$a = \frac{\Delta\pi R}{90}$$

$$o = R(1-\cos 2\phi)$$

5.3 Minimum Horizontal Radius of Curvature

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

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 straight roadway segments joined by an isolated curve designed at or near the minimum radius are unsafe because 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.

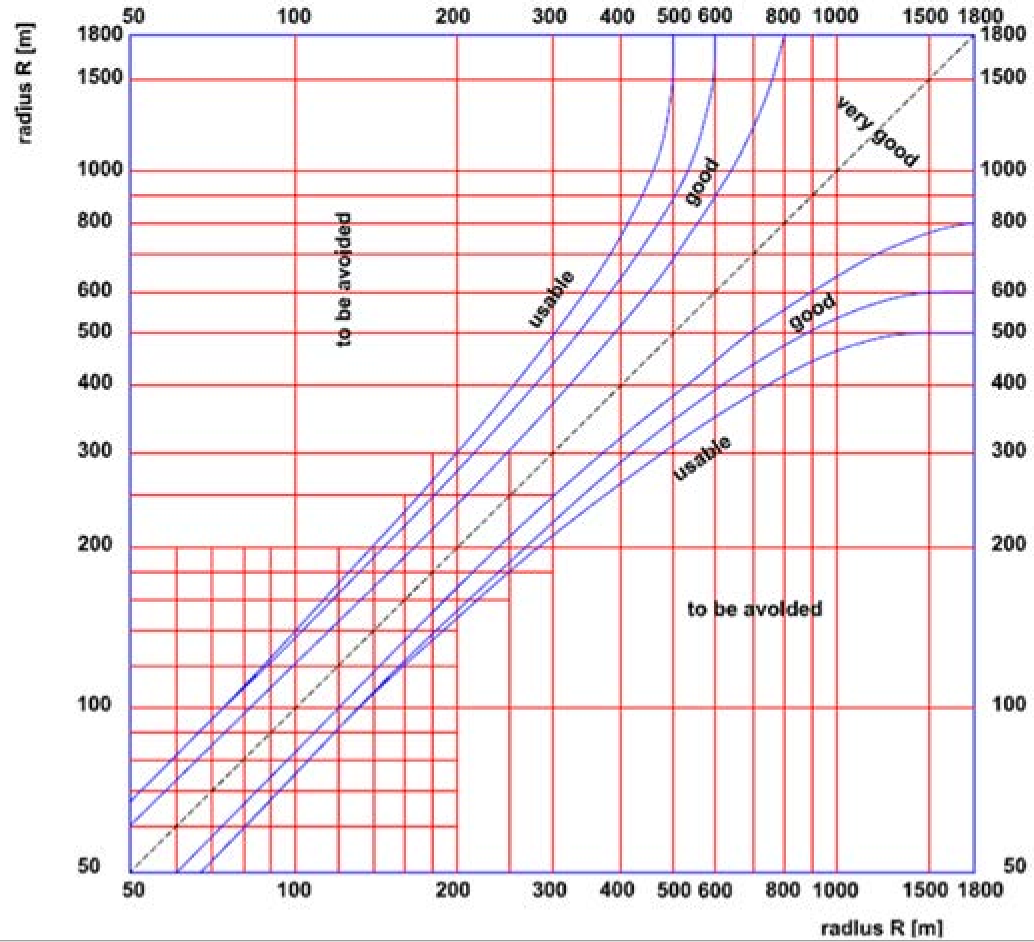
5.6 Straight Sections

5.6.1 Design Speeds Greater than 120 km/hr

Long straights of twenty kilometres or more have crash rates similar to those on minimum length straights. The lowest crash rate occurs in a range of lengths from 8 to 12 kilometres. This range is recommended for the maximum length of straight on any route where the design speed is 120 km/h or more. However, a long straights can cause serious problems of dazzle from approaching headlights which can be extremely dangerous and therefore, if there are large volumes of night traffic, straight 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 straights in which a winding alignment is used with straights 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

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.*

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 straight 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 straights should, ideally, not be longer than 1.5 – 2.0 km.

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

5.6.3 Minimum and Maximum Length of Straight Road Sections

From an aesthetic point of view, the straight may often be beneficial in flat country but rarely in rolling or mountainous terrain. However, long straights increase the danger from headlight glare and usually lead to excessive speeding. Overtaking opportunities must however be provided at reasonable intervals and straights are often the most appropriate solution.

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Short straights between curves in the same sense should be avoided ('broken back' effect). If such straights have to be used, the unfavourable appearance may be improved by the introduction of a sag curve.

The minimum allowable length of straight is that which accommodates the rollover of Super-elevation in a reverse curve situation. It has been found that extremely long straights, e.g. lengths of twenty kilometres or more, have accident rates similar to those on minimum length straights, the lowest accident rate occurring in a range of eight to twelve kilometres. This range is recommended for consideration in fixing the maximum length of straight on any route. This maximum is based on the assumption of a design speed of 120 km/h or more. At lower design speeds it is necessary to consider maximum lengths considerably shorter than eight to twelve kilometres, as discussed below.

As used in current practices (rule of thumb), the maximum length of straight in metres should not exceed ten to twenty times the design speed in km/h for design speeds less than 100 km/hr. If the achievable maximum length of straight across the length of the route is regularly greater than this guideline value, thought should be given to consideration of a higher design speed.

At a design speed of 120 km/h or higher, a maximum straight length of 1200 to 2400 metres would clearly be meaningless.

This oscillation in speed is inherently dangerous, thus consistency in design dictates that the difference between design speed and 85th percentile speed, and variations in 85th percentile speeds between successive elements should be limited as far as possible. These differences and variations should be less than 10 km/h but ensure that an acceptable design still results if they are less than 20 km/h. Differences in excess of 20 km/h constitute poor design so that, at a greater level of precision than the proposed rule of thumb.

The 85th percentile value of speed has been used as the basis of design in this guide. Thus, 15 per cent of the vehicles could be considered to be exceeding the design speed on any section of road.

Based on the above discussion, the following guidelines apply for the lengths of straights:

1. Straights should not have lengths greater than (20 x V_D) metres (V_D in km/h) for V_D <100km/hr
2. Straights should not have lengths greater than 4km metres (V_D in km/h) for V_D > = 100km/hr
3. Straights between circular curves following the same direction should not have lengths less than 200 m to allow Super-elevation run-off.

Table 5.4 Minimum and Maximum Lengths of Straights (m)

Design Speed V_D (km/h)	Minimum Length of Straights (m) = ($6V_D$)	Maximum Length of Straights (m) = ($20V_D$)
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: *The minimum and maximum lengths of straights 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]

Notes:

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 limit 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 Chapter 3.

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 straights (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

5.9.1 General

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, super-elevation 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, super-elevation 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 super-elevation above 6% should be with approval of the Chief Engineer Roads.

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.

Super-elevation deficiency (inadequate super elevation compared to what is specified) has potential impacts on safety as shown in Table 5.6. Inadequate super-elevation 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 super-elevation.

Table 5.6 Potential Safety Impacts of Super-elevation 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.

5.9.2 Super-elevation Development

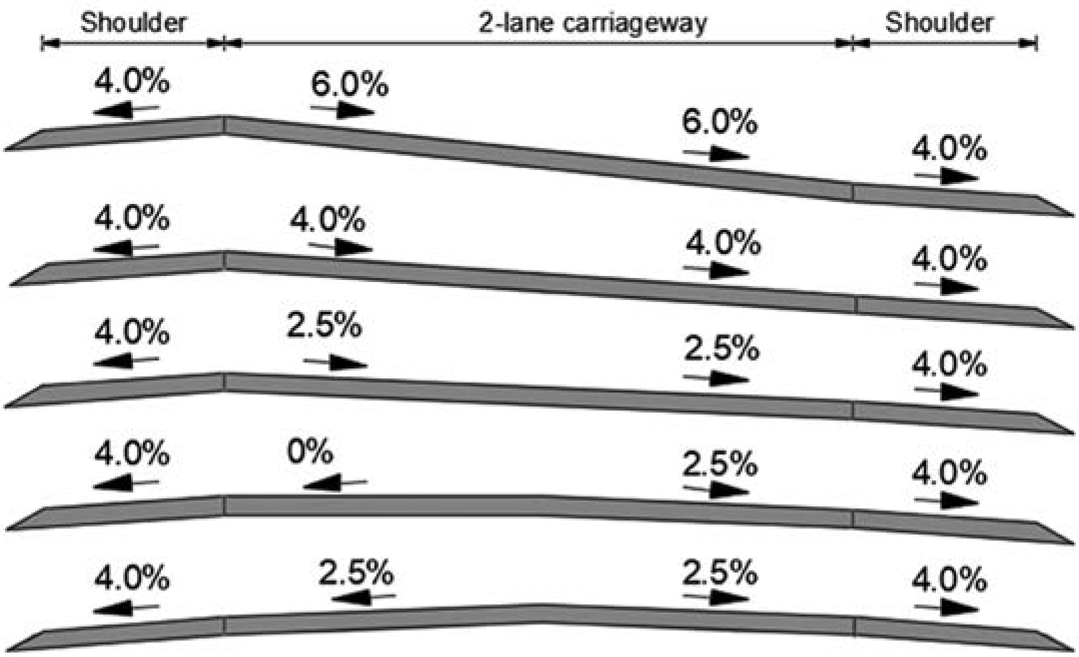
Super-elevation development has two components: tangent runout and super-elevation runoff. Tangent runout involves the rotation of the outside lane(s) of the cross-section from the normal camber, usually 2.5 %, to a zero cross-fall. Super-elevation runoff then continues this rotation until a cross-fall 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 super-elevation 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 super-elevation 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.

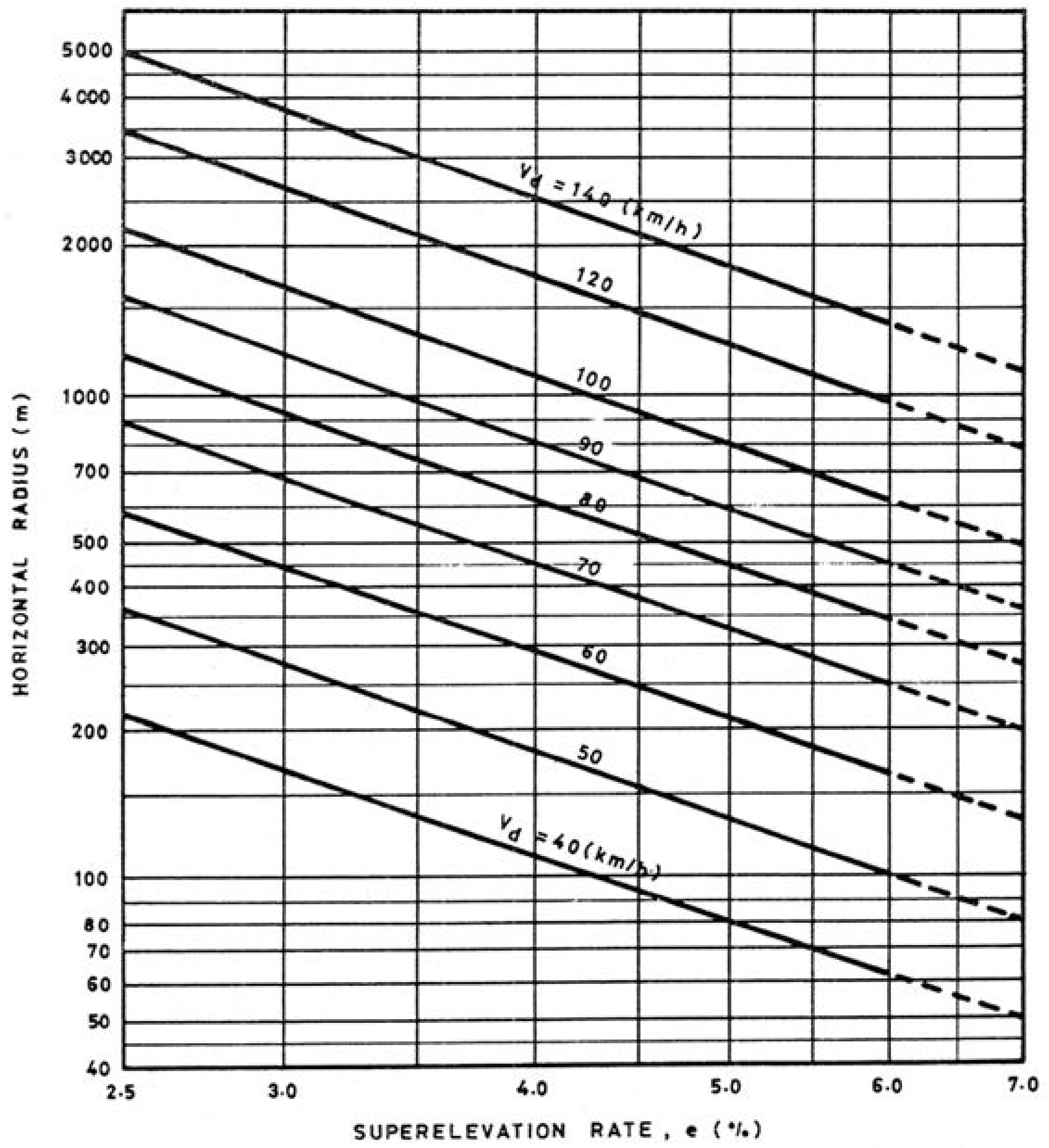
However, as in the case of the two-way road, no hard and fast rules can be laid down concerning the selection of the location of the axis of rotation. It could be at the centreline of the median, at the edge of the median or even having the entire cross-section rotating as a unit around one of other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

Figure 5.4 Super-elevation of a Two-Lane Carriageway About the Centreline



The normal maximum permissible super-elevation rate, e_{max}, is 6 % and is applicable for the minimum horizontal radius at the selected design speed, where major constraints exist e_{max} = 7 % may in particular cases be applied. Super-elevation rates for radii larger than R min are given in Figure 5.5 for different design speeds.

Figure 5.5 Super-elevation Rates for Different Design Speeds and Horizontal Curve Radii



5.9.3 *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 5.7). 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.

5.9.4 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 affected 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 5.7, 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.

The length required to accommodate the super-elevation run-off, Lo is calculated from the following equation:-

$$Lo_{min}\text{ (m)} = \frac{e - e_O}{\Delta S_{max}} \times \frac{w}{2}$$

Where,

e = Super-elevation on the circular curve.

e_O = Camber or crossfall on straight (negative if opposite to e).

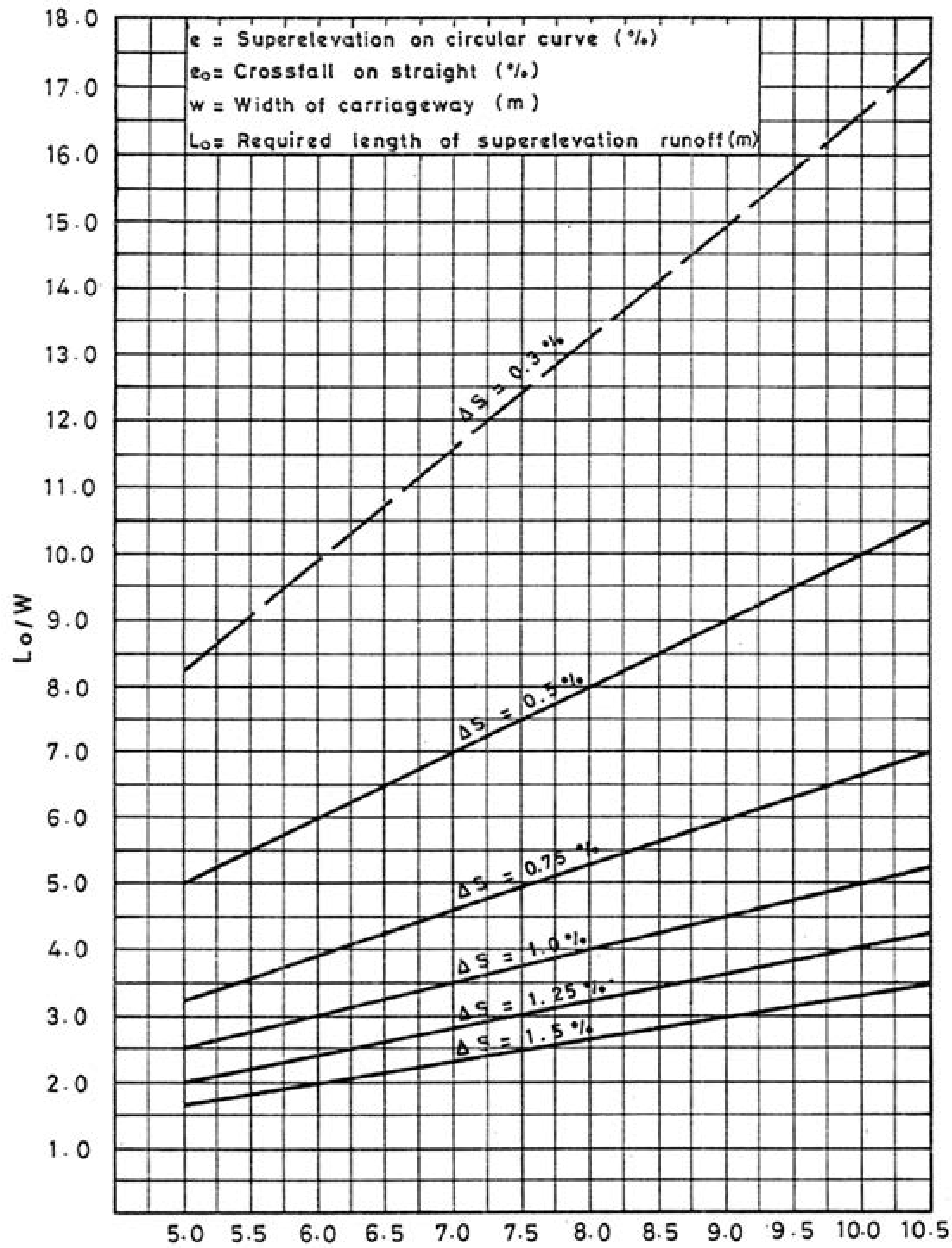
w = Carriageway width.

Values for Lo_{min} can be easily calculated by reading Lo/w from the applicable Δ line in Figure 5.6, where ΔS_{max} is decided by 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

Figure 5.6 Lo as a Function of Carriageway Width and Super-elevation



5.10 Transition Curves

All the components of the horizontal alignment are shown in Figure 5.7. 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 straight (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/hr.
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.

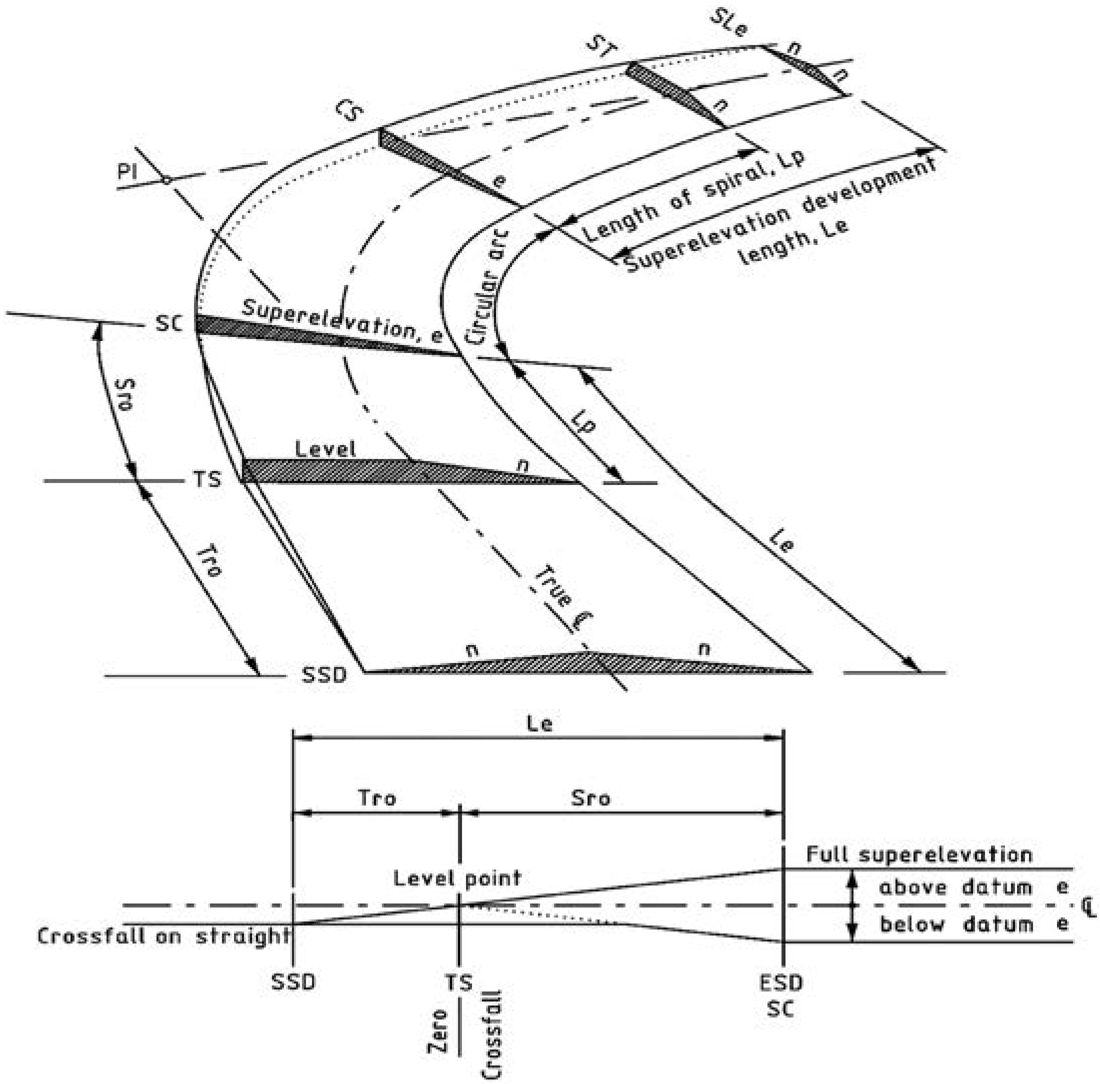
The purpose of introducing transition curves is to:

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.

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

Notwithstanding the above, if transition curves are applied the super-elevation run-off should take place within the length of the transition curve. Where transition curves are not used, half the super-elevation run-off should take place on the straight and the other half in the curve. Examples of super elevation run-off are given in Figure 5.8.

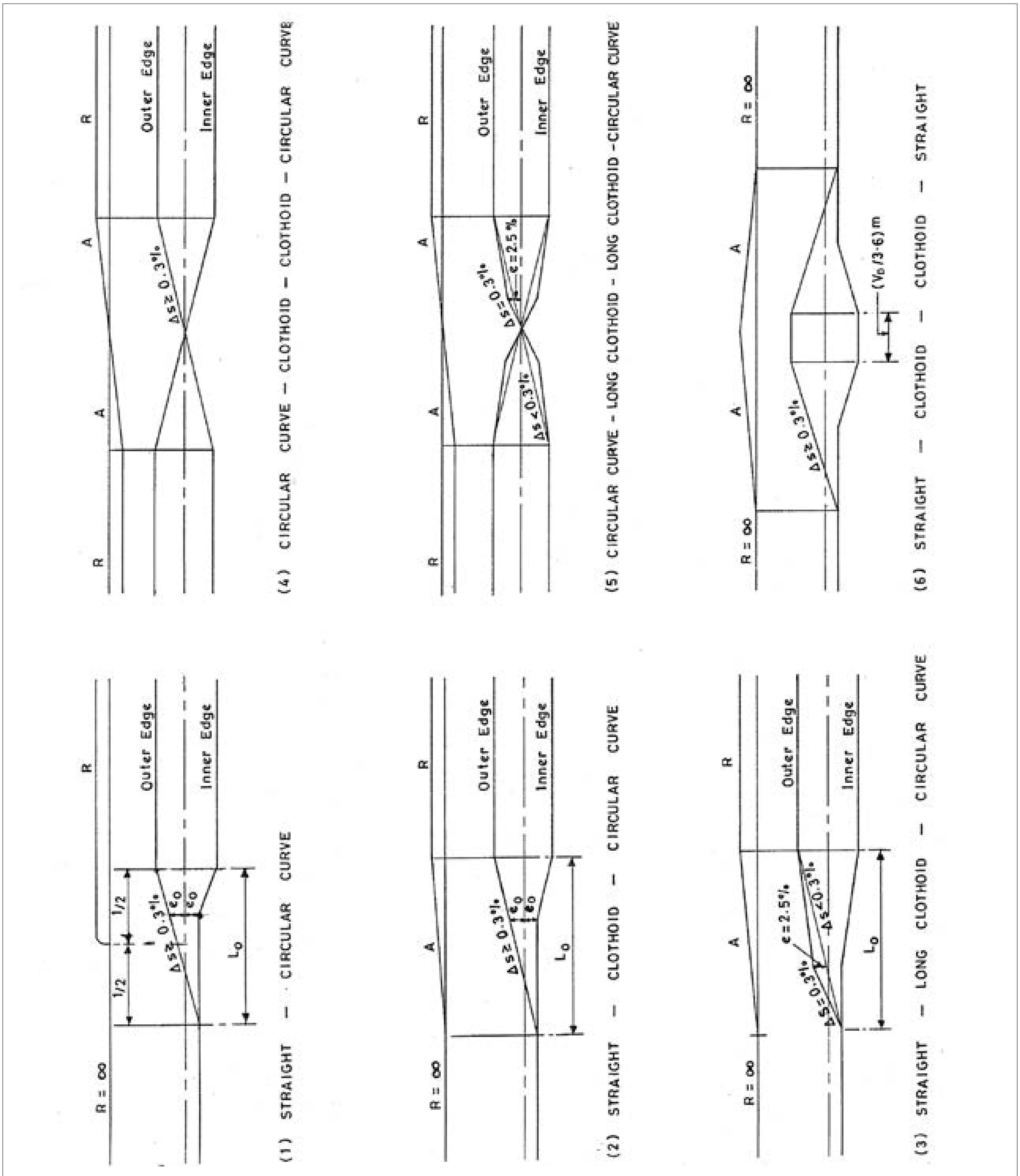
Figure 5.7 Elements of Super-Elevation And Transition Curve



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 5.8 Examples of Super-elevation Runoff



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.

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

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

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:

$$R = \frac{\left[1 + \left(\frac{dy}{dx}\right)^2\right]^{3/2}}{\frac{d^2y}{dx^2}} = 0.86$$

Equation 5.3

The equation for the applicable clothoid transition curve is:

$$A^2 = R.L$$

Equation 5.4

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:

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

Equation 5.4

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:

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

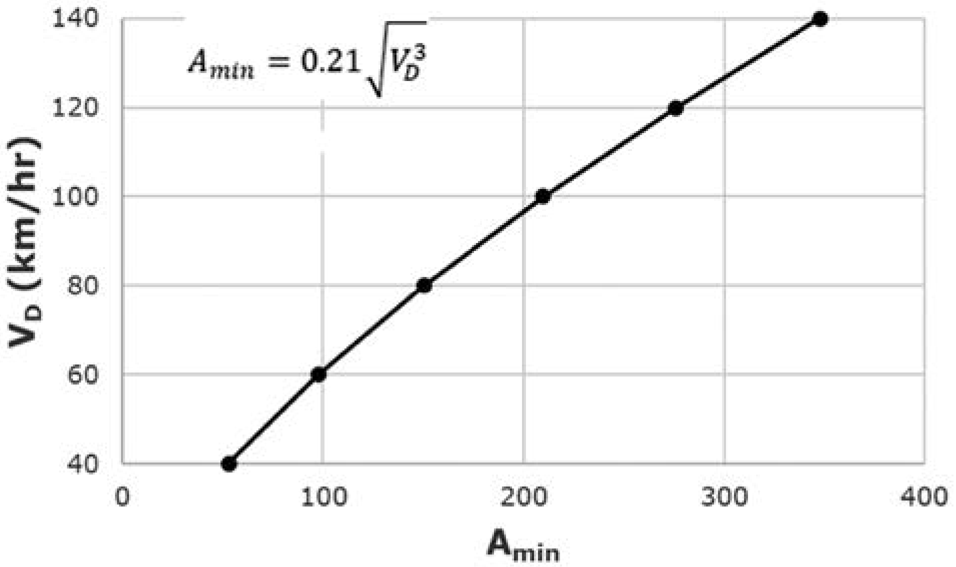
and for aesthetic reasons the shape of the curve should be clearly visible therefore:

$$A_{min} = \frac{R}{3}$$

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.

The value of A_{min} may be read directly from Figure 5.9.

Figure 5.9 A_{min} as a Function of Design Speed



5.11 Compound Curves

Compound curves, are curves in the same direction but of different radii, and without any intervening straight 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 straight should be at least 150 m long. The straight 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 straight (tangent) to a sharper curve, although a spiral clothoid transition curve is preferred.

As a guideline, the ratio between the length of the clothoid and the length of the curve should be such that the clothoid length is not less than 1/3 of the curve length. Consequently for short circular curves the minimum value of A will generally be adopted whilst for long circular curves values of A well above A_{min} should be adopted.

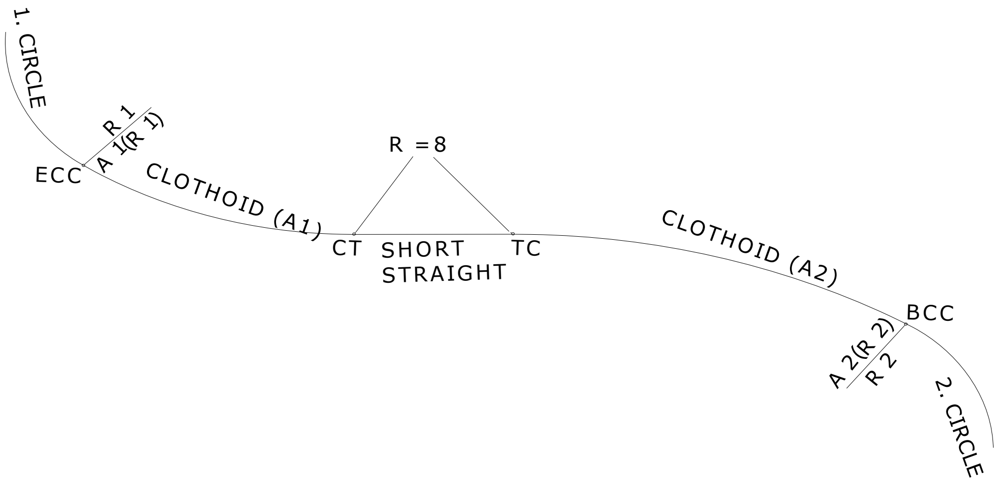
The different uses of clothoids as transition curves are as follows:-

a. **Transition Curve:** Straight to Circular Curve

b. Transition Curve: Circular Curve to Circular Curve, S-Compound Curve
To ensure a harmonic alignment and uniform rate of change of super-elevation, both clothoid branches should have approximately the same parameters. The following condition should be fulfilled:

$$\frac{2}{3} > \frac{A_1}{A_2} < \frac{3}{2}$$

For practical reasons (e.g. small errors in surveying) it is advisable to incorporate a short straight of 10-20m between the two clothoids. This straight shall not be considered as an independent design element, and thus the super-elevation run-off shall be carried through the short straight.



c. 'O' - Compound Curve: The 'O' - compound curve comprises two circular curves in the same sense with a clothoid connecting section. One of the circular curves must lie completely inside the other, they must not intersect each other and they must not have a common centre-point.

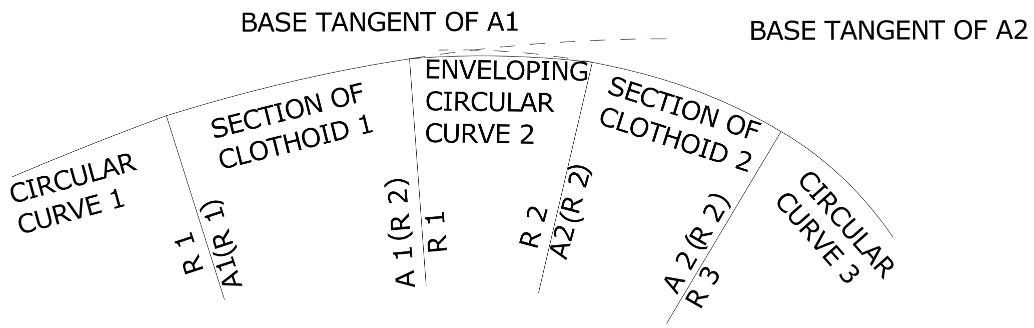
Particular conditions for 'O' Compound Curves:

$$\frac{R_2}{2} < A > R_2$$

Where,

R_2 = The smaller of the two radii.

The ratio between the two radii shall comply with the requirements of Figure 5.3. If the ratio is with.in the 'Very Good' area of Figure 5.3. then no transition curve is required.



d. Double 'O' - Compound Curve: If the circular curves lie outside of each other, if they intersect each other or if they have a common centre point, then the connection can be achieved with an enveloping circular curve. In this case, two successive clothoid constructions are necessary as shown below:

The ratio between the two successive radii shall comply with the requirements of Figure 5.3. If the ratio is within the 'Very Good' area of Figure 5.3 then no transition curve is required.

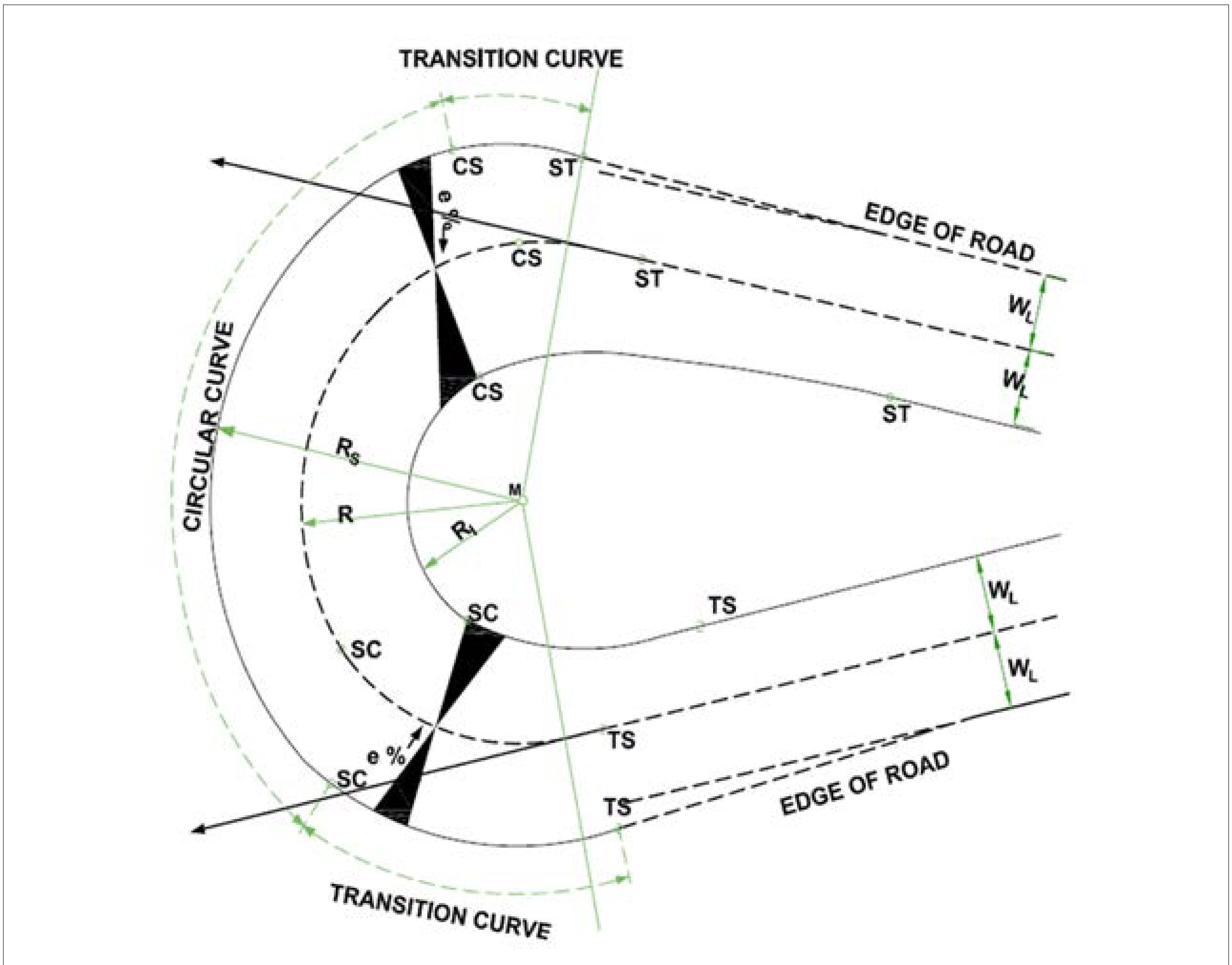
5.12 Hairpin Curves

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 1.

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 to allow for:

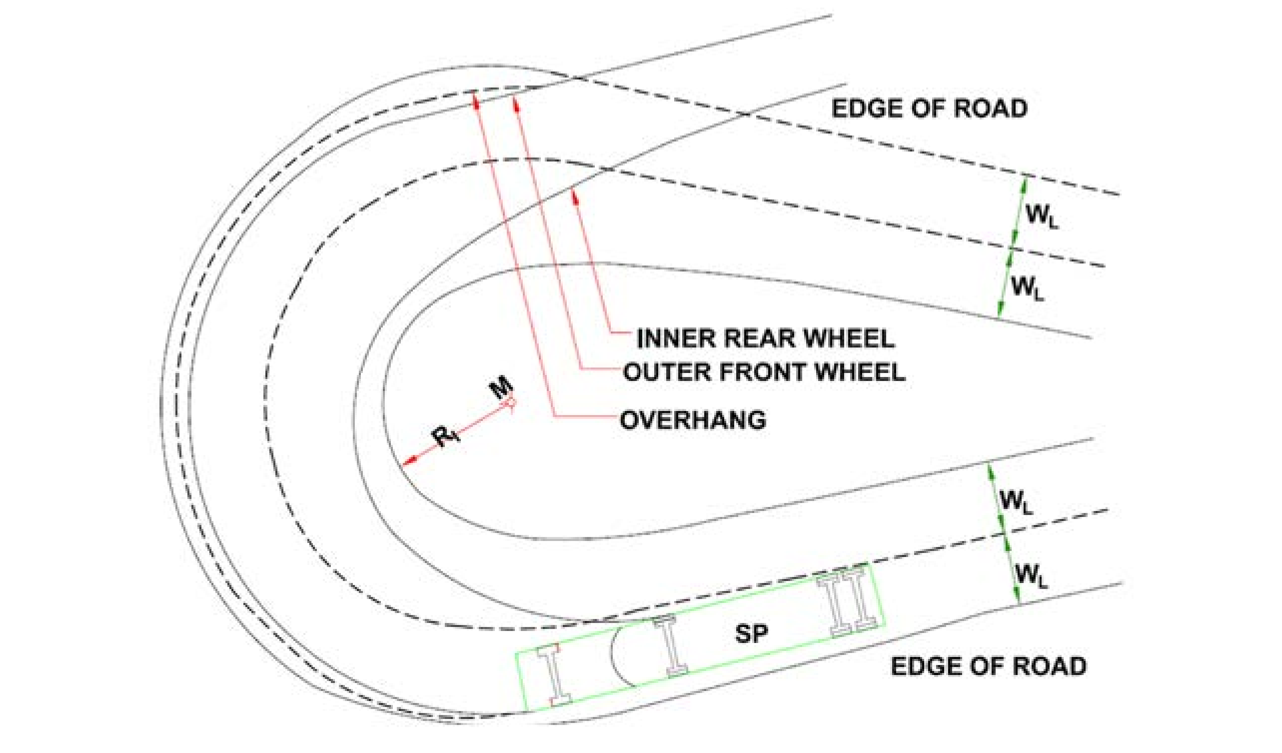
1. Passage of two opposing DV4 vehicles. This is recommended for all interurban roads and urban arterials and collectors.
2. Passage of a single DV4 and a DV1. This is recommended for Class C interurban roads.

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 3.1 or Figure 3.2 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

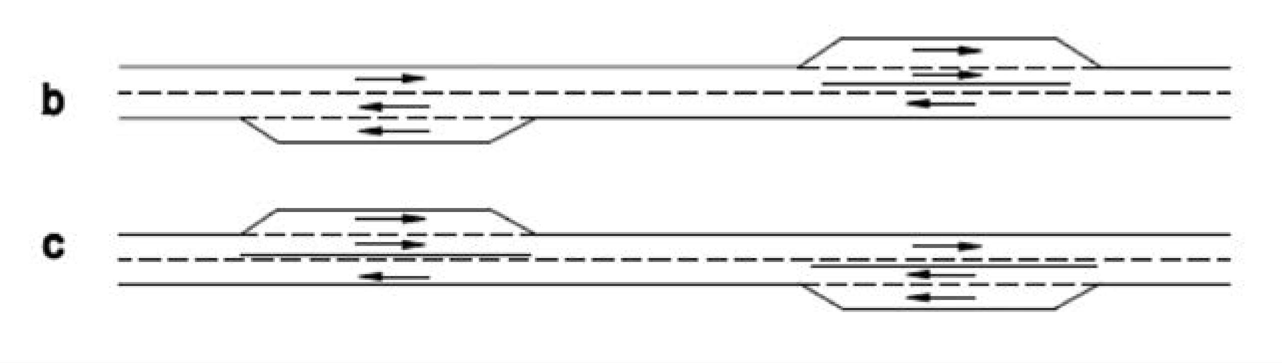


It is important to provide relief from a severe gradient through the switchback. Where switchback curves are unavoidable in mountainous terrain, there is a need to reduce the maximum allowable gradient at any point through the curve. The maximum allowable gradient through a switchback curve is 4 percent.

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 1.

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 200 m is recommended. Since both the entry and the exit tapers indicate a change in operating conditions it is recommended that decision sight distance 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

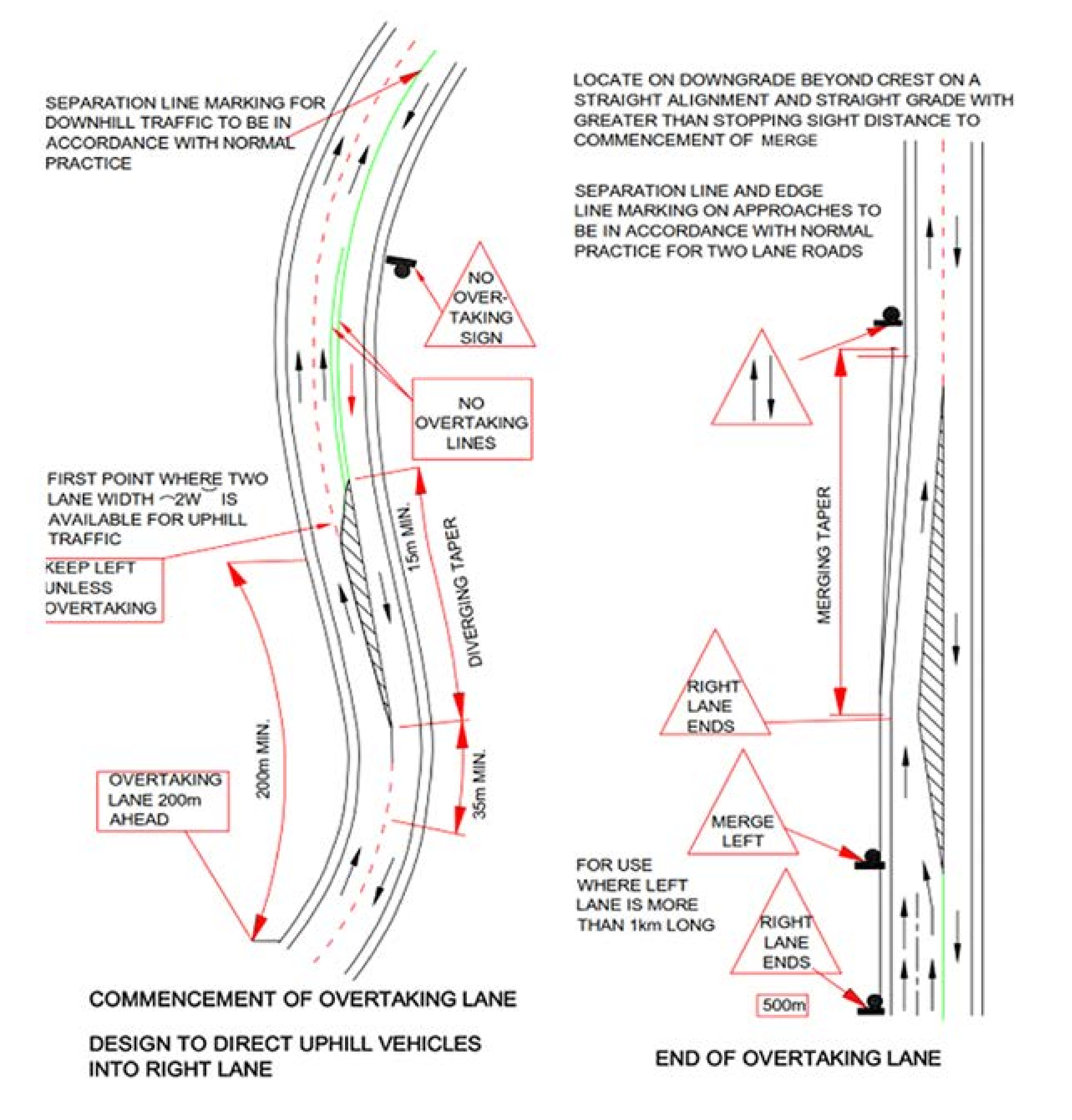
Passing lanes can be used instead of climbing lanes on hilly sections of road. Unlike climbing lanes, passing lanes tend to operate at the speeds prevailing on the rest of the road. Reductions in lane width are thus not recommended and such passing lanes should have the same width as the basic lanes. 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 e_{max} = 4.0%

Radius (m)	40	50	60	70	80	90	100	110	120
7000	n	n	n	n	n	n	n	Design speeds above 100 km/h are not suitable	
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		
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 e_{max} = 6.0%

Radius (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*

1 2 3 4 5 Horizontal Alignment

Table 5.11 Super-elevation Rates for e_{max} = 8.0%

Radius (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 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 respectively.

Figure 6.1 Crest Curve

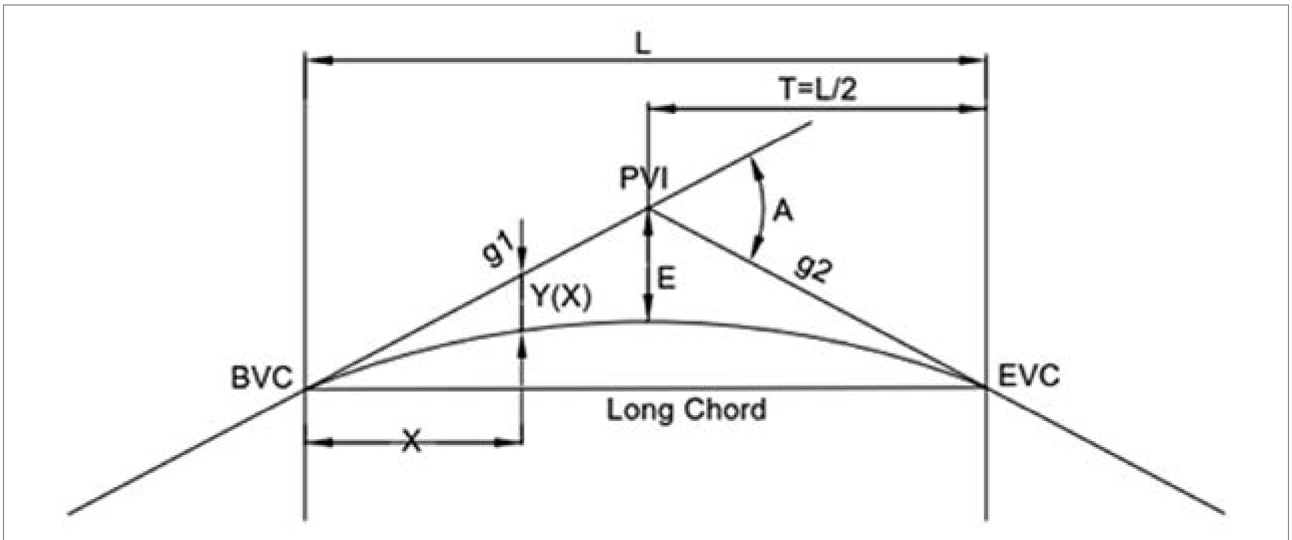
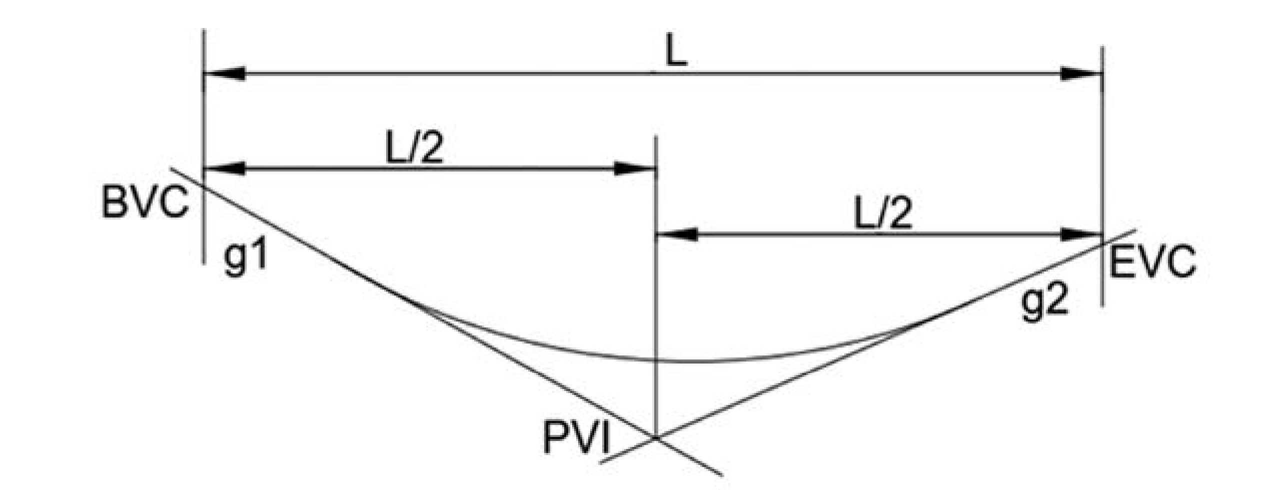


Figure 6.2 Sag Curve



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:

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

Equation 6.1

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;

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

Equation 6.2

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

Equation 6.3

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

Equation 6.4

The Point of Vertical Intersection (PVI) always occurs at an x coordinate of 0.5L hence, from Equation 6.1, the elevation is always;

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

Equation 6.5

Example:

For the crest curve shown in Figure 6.1, the two tangent grade lines are +6% and -3%. The Beginning of the Vertical Curve is at chainage 0.000 and its elevation 100.0 m. The length of the vertical curve is 400 m. Compute the End of the Vertical Curve and the coordinates of the Intersection Point.

The y coordinate of the EVC is $\quad Y(L) = (g_1 + g_2)\dfrac{L}{200} + Y(0)$

$$= (6 - 3)\frac{*400}{200} + 100.0 = 106.0$$

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

The coordinates of the VPI are $\quad X(IP) = \dfrac{L}{2} = 200.0$ and

$Y(PVI) = Y(0) + 6.400/200 \quad = Y(0) + 12 = 112\text{m}$

6.3 Crest Curves

6.3.1 General

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

and therefore:

$$L_m = K.G$$

Where,

L_m = Minimum length of vertical crest curve (m) and large K corresponds with long curves.

S = Required sight distance (m).

h_1 = Driver eye height (m).

h_2 = Object height (m).

K = Is a constant for given values of h_1 and h_2 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

Eye height (h_1) has been taken as 1.05 m, and object height h_2 of 0.2 m above the road surface. Minimum values of K for crest curves are shown in Table 6.1 and Table 6.2 for paved and unpaved roads respectively.

Table 6.1 Minimum Values of K for Crest Vertical Curves (Paved Roads)

Design Speed (km/h)	K for stopping sight distance (g = 0%) h_2 = 0.2 m	K for minimum passing sight distance
25	1	30
30	2	50
40	5	90
50	10	130
60	17	180
70	30	245
80	45	315
85	55	350
90	67	390
100	100	480
110	140	580
120	185	680

Table 6.2 Minimum Values for Crest Vertical Curves (Unpaved Roads)

Design Speed (km/h)	K for stopping sight distance (g = 0%) h_2 = 0.2 m	K for minimum passing sight distance
25	1	30
30	2	50
40	6	90
50	11	135
60	20	185
70	35	245
80	58	315
85	72	350
90	90	390
100	130	480

6.3.2 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.

6.3.3 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.

6.4 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

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 0.6 m and a beam divergence of 1°, 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/100 m. In the direction of increasing chainage, up-grades are taken as positive and down-grades as negative.

6.5.2 Maximum Gradients and Lengths

The absolute gradient and the maximum desirable gradients are shown in Table 6.5 for different classes of terrain. Table 6.6 gives the maximum length for maximum desirable grades.

If gradients on which the truck speed reduction is less than 20 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.

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 6.5 Suggested Maximum Gradients for Paved Sections

Terrain	Maximum Gradient (%), for Paved Sections			
	DR1 – DR7		DU1 – DU6	
	Desirable	Absolute	Desirable	Absolute
Flat	3	5	3	5
Rolling	4[1]	6	6	8
Mountainous	6[1]	8	8	10
Escarpment	7[1]	9	8	10

Notes: *[1] On expressways a maximum gradient should not exceed 4%*

Table 6.6 Maximum Lengths of Gradient

Gradient %	Maximum Length of Gradient (m)
2	No limit
3	No limit
4	900
5	800
6	700
7	600
8	500

Source: *SANRAL and SATCC Geometric Design Guidelines*

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 authorised through a formal Departure from Standard as described in Chapter 1.

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 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.

6.5.5 Gradients Through Villages

In many cases the natural grade level is flat through villages. The adjacent walkways 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 walkway may be required in some areas.

6.6 Climbing 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.

1
2
3
4
5
6
Vertical Alignment

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 6.6 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 pcu 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 pcu to 6000 pcu.
3. Climbing lanes will normally be required on roads with AADT > 6000 pcu 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 30 m 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.

1 2 3 4 5 6 Vertical Alignment

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.

The widths of the through traffic lanes of 3.25 m or higher shall be reduced by 0.25 m.

The width of the climbing lane shall be equal to that of the adjacent reduced single lane so as to give three traffic lanes of equal width.

Reduction of the shoulder widths on both sides of the road – shoulder adjacent to the climbing lane by 0.5 m and the shoulder adjacent to down-hill lane by 0.5 to 1.0 m depending on the cross-section type.

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.

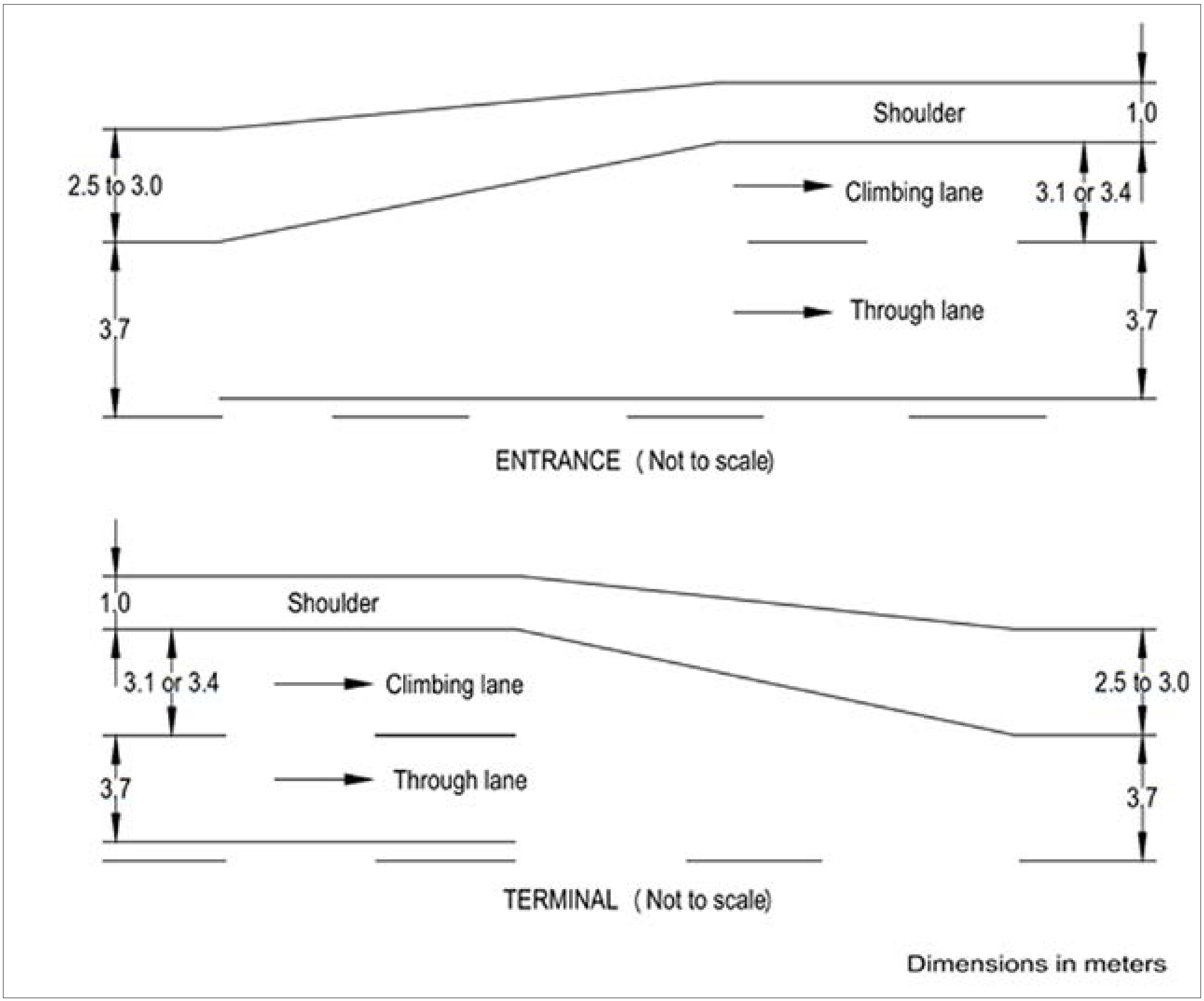
The position of the lane-drop must allow the slower vehicles to gain enough speed to merge with the faster vehicles. Lane-drops should not be situated on curves. Figure 6.3 illustrates the recommended layout for climbing lanes.

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.

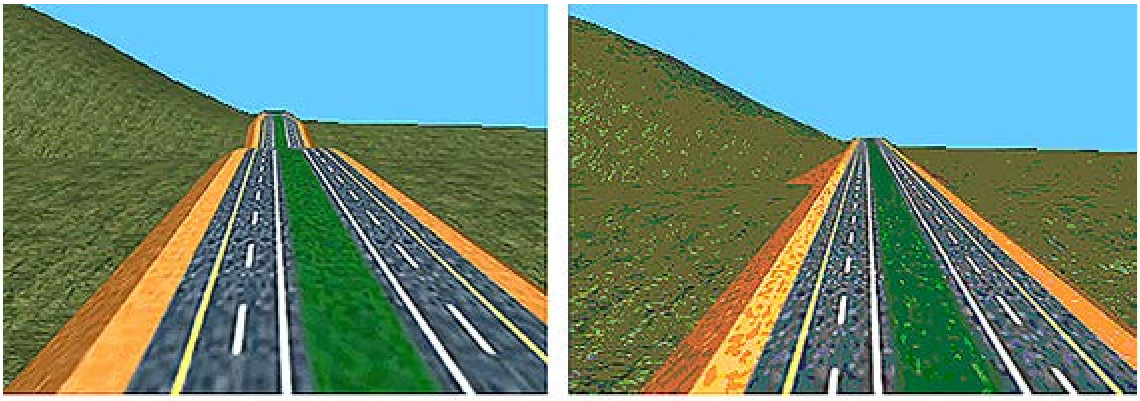
As a first consideration, the traffic engineer should obtain data from a fixed weighbridge if one exists near the study road or if there is a virtual weigh-in motion station along the study corridor (see Section 2.1). If not, or in case additional data is required, then a portable static weigh pad should be used to measure the axle loads.

Static scales have high accuracy, with a typical precision of 3 to 5 percent, but they suffer the risk of safety, delays, and avoidance by motorists. Avoidance of weigh stations by trucks, which either take alternative routes or avoid driving while the weigh station is in operation, can be exacerbated by the delays and the high visibility of the static weighing operation.

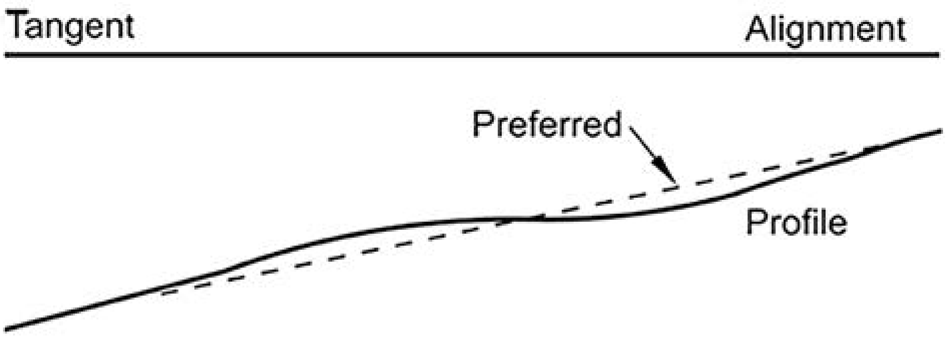
It is recommended that portable static scales be regularly and accurately calibrated by the manufacturer using calibrations provided by an accredited facility.

Other versions of WIM are in portable form and have increasingly become popular. However, they are less accurate and therefore, they should not be used for obtaining axle load data for design purposes. Research evidence indicates that portable WIM scales underestimate the weights for steering axles (which are lighter) and overestimate the drive and trailer axle weights (which are heavier).

Figure 7.1 Hidden Dip (Roller Coaster) Profile



A. Hidden Dip (Roller Coaster) **B.** Hidden Dips Eliminated



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.

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 7.2 Short Humps on Long Horizontal Curve

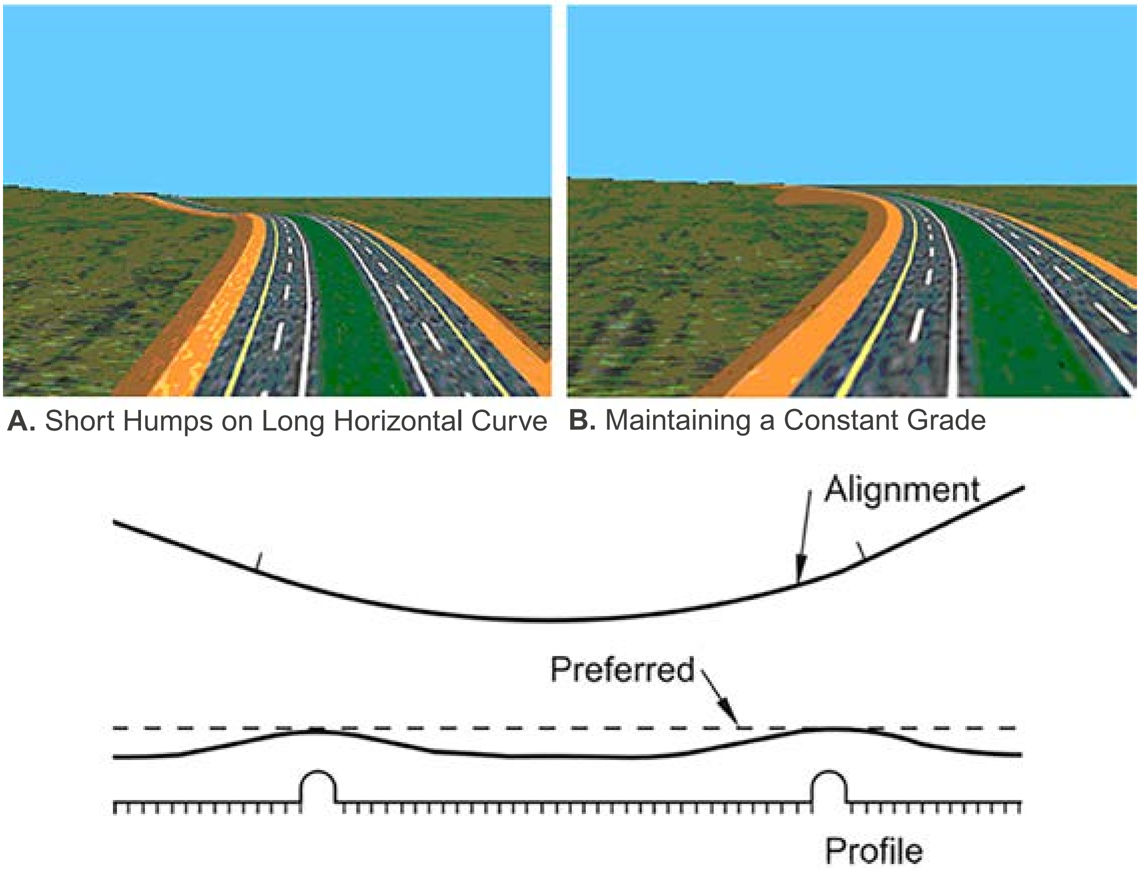


Figure 7.3 Out-of-phase Vertical and Horizontal Alignments

7.6 A Short Dip in the Alignment Preceding a Horizontal Curve

Similar to Section 7.4, 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.

Figure 7.4 Short Dip in the Alignment Preceding a Horizontal Curve

A. Short Hump and Dip Preceding Horizontal Curve

B. Eliminating the Crest Curve in Advance and the Following Sag Curve

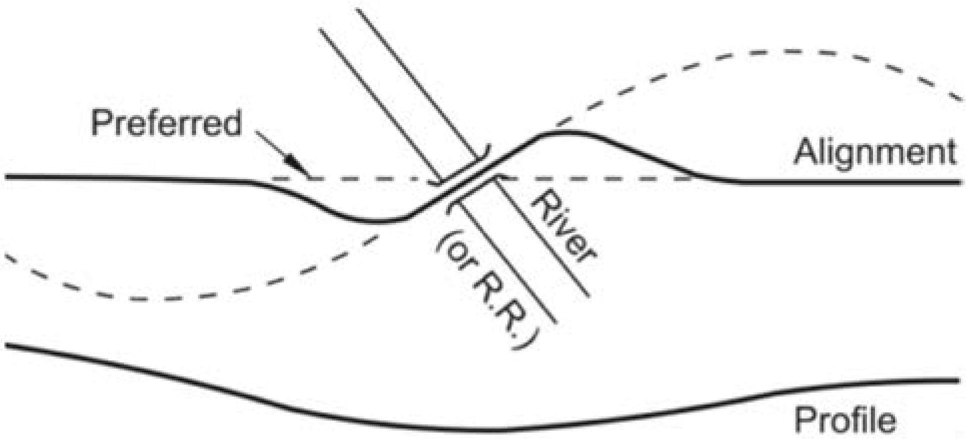
7.7 Distorted Alignment

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.

Figure 7.5 Distorted Alignment to Create a Square River Crossing

A. Distorted Alignment to Create a Square River Crossing

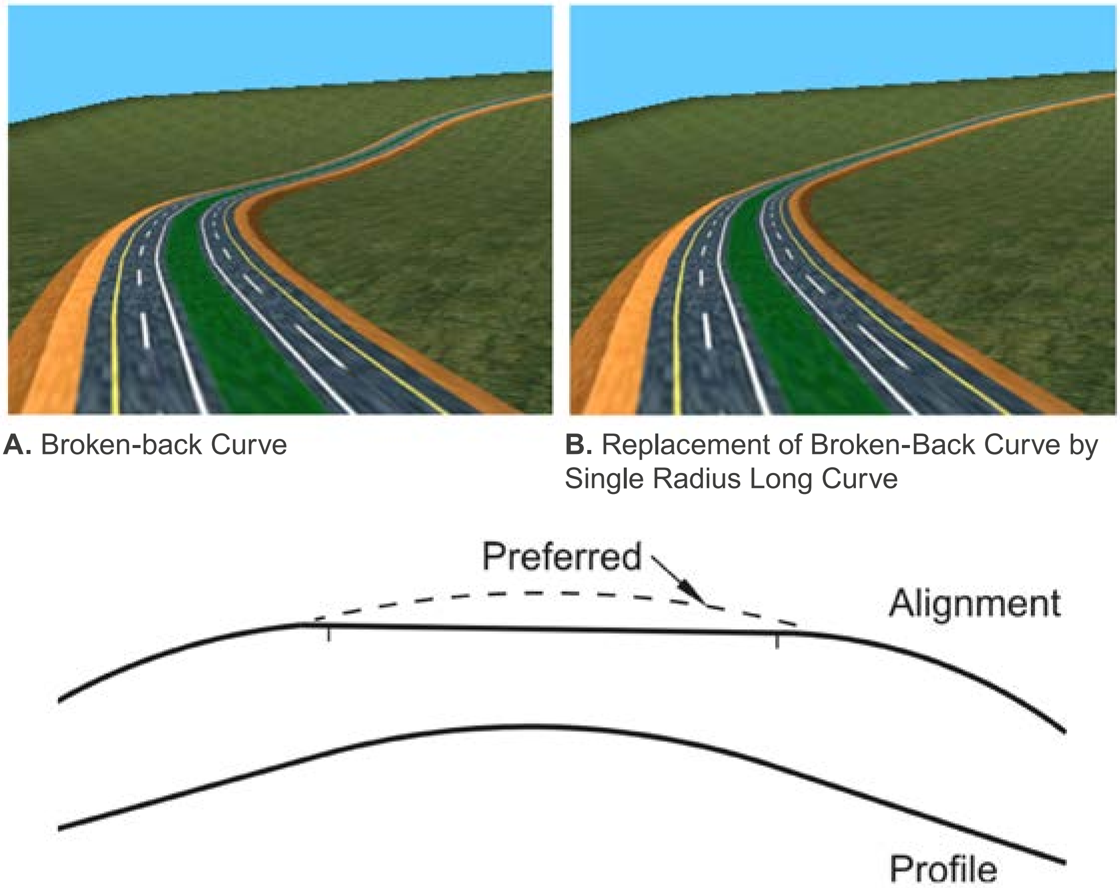
B. Skew Crossing Improves Horizontal Alignment



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.

Figure 7.6 Broken-back Curve



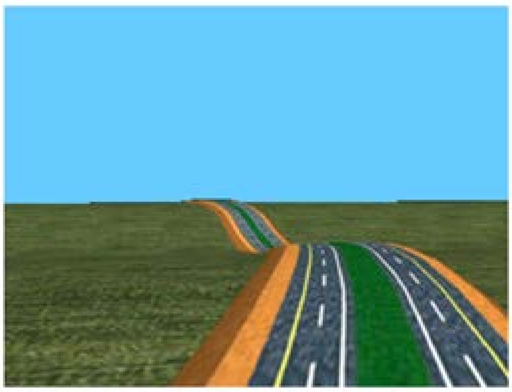
A. Broken-back Curve

B. Replacement of Broken-Back Curve by Single Radius Long 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)



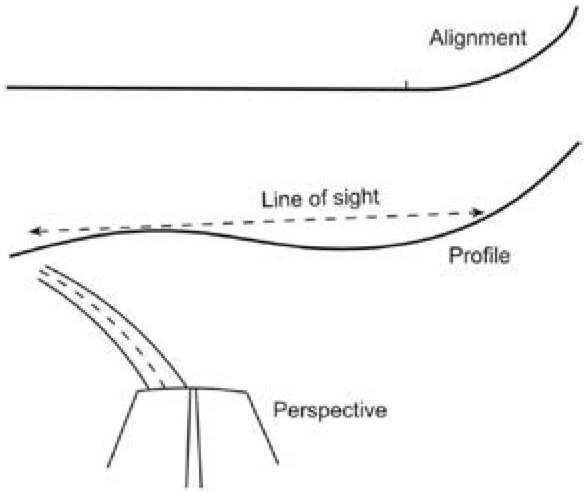
7.10 Both Ends of the Vertical Curve Lie on the Horizontal Curve

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

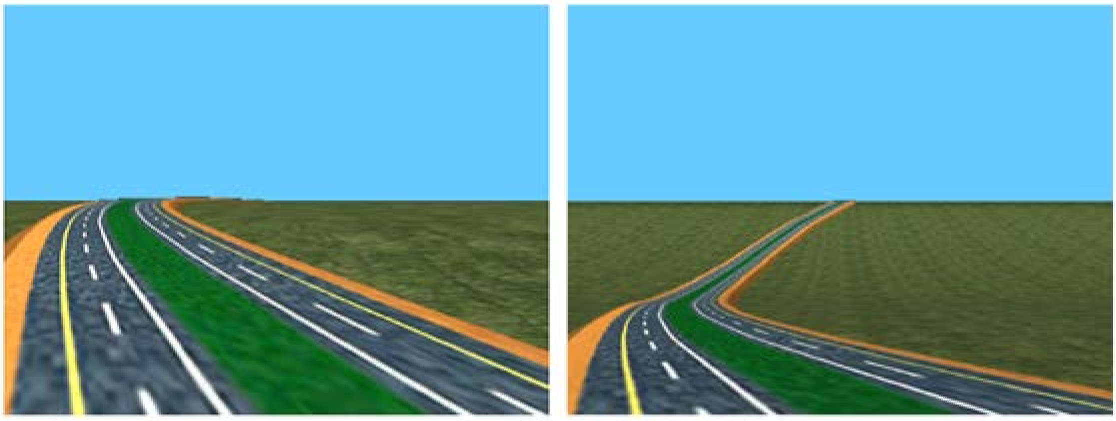
If a vertical crest curve overlaps both ends of a sharp horizontal curve, a hazard may be created because a vehicle must undergo a sudden change of direction during the passage of the vertical curve while sight distance is reduced.

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.

Figure 7.9 Well-coordinated Horizontal And Vertical Alignment



A. Well-coordinated Crest and Horizontal Curves

B. Well-coordinated Sag and Horizontal Curves

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 7.1 unless a refined hydraulic analysis has been made. The standard minimum headroom or clearance under major bridges or tunnels is 5.5 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 5.5 m. (RDM Volume 4; Part 2 – Bridge and Culvert Design) for further reference.

Table 7.1 Vertical Clearance from Superstructure to Design Flood Level

Design Flow at Bridge (m^3/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 Modelling and Earthworks Computation

8.1 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.

8.2 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.

8.3 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.

8.4 Earthworks

8.4.1 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 haulage. 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.

8.4.2 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.

9 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.

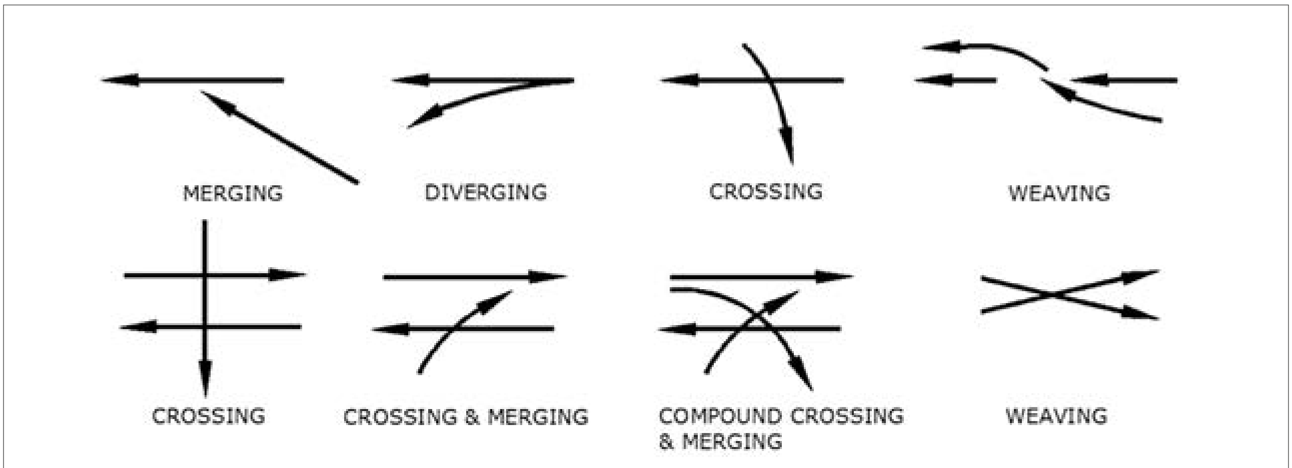
9.1.2 Vehicle Characteristics

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 3.5 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 3.5.

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

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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.

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.

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, channelising islands and/or ghost islands. For unsignalised 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 unsignalised T junctions, adjoining traffic should give way to traffic on the 'major' road in both directions.
	Staggered Junction	Staggered junction with channelising 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.
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.

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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 Channelised T intersection **C** Channelised T intersection
Roundabouts	Priority to traffic already on the roundabout but can also be traffic signal controlled		**D** Priority **E** Signalised
Traffic controlled	Traffic signals or give way signs		**E** Signalised Intersection

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 9.6) 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.

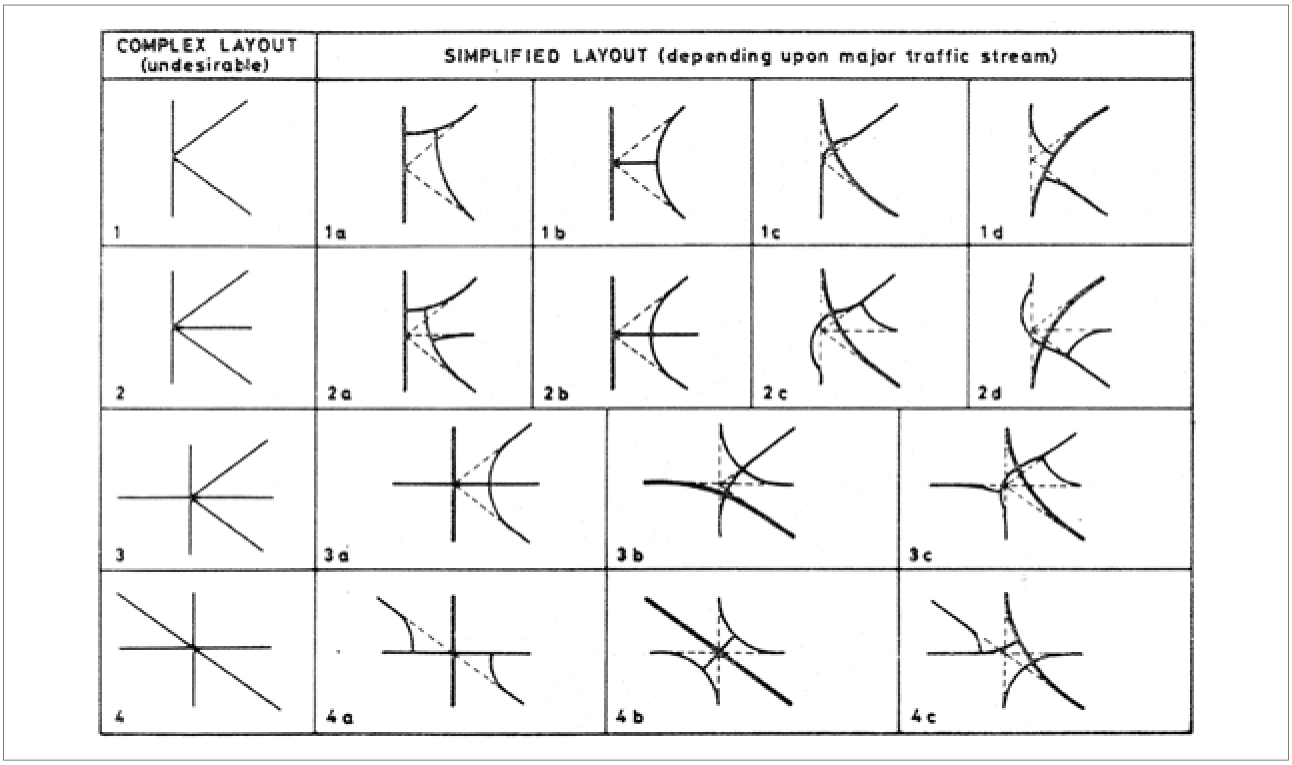
At existing rural and urban junctions, upgrading a simple junction to provide a right turning facility from the major road should always be considered because vehicles waiting on the major road to turn right inhibit the through flow and create a hazard.

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 9.4, and Figure 9.7.

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 junction layouts involving the intersection of four or more roads are encountered, these should be simplified by realigning the approaches, to safer, more comprehensible and manoeuvrable layouts. Examples of such simplifications are given in Figure 6.4.2. Considerations should also be given to using roundabouts. Experience in some countries has shown that converting crossroads into roundabouts can reduce crashes by more than 80 %.

Figure 9.2 Examples of Simplification of Complex Intersections

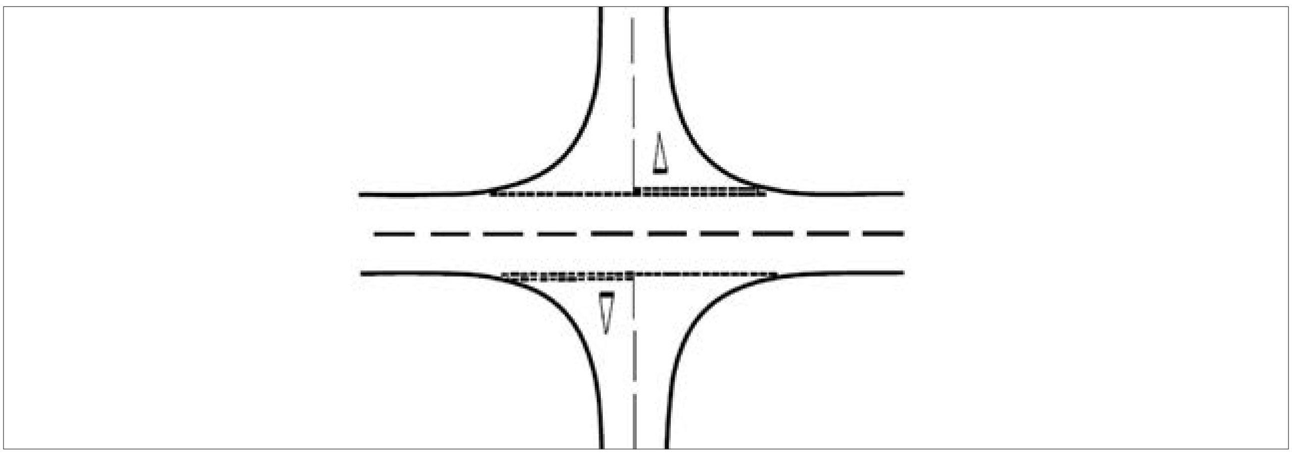


T-intersections include the staggered T-intersection (Figure 9.6 and Figure 9.7), which cater for cross-traffic. Staggered T-intersections are often the result of a realignment of the minor route to improve the angle of the skew of the crossing.

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 9.4 and Figure 9.5) and/or traffic signals.

Figure 9.3 Crossroads with Stop Lines



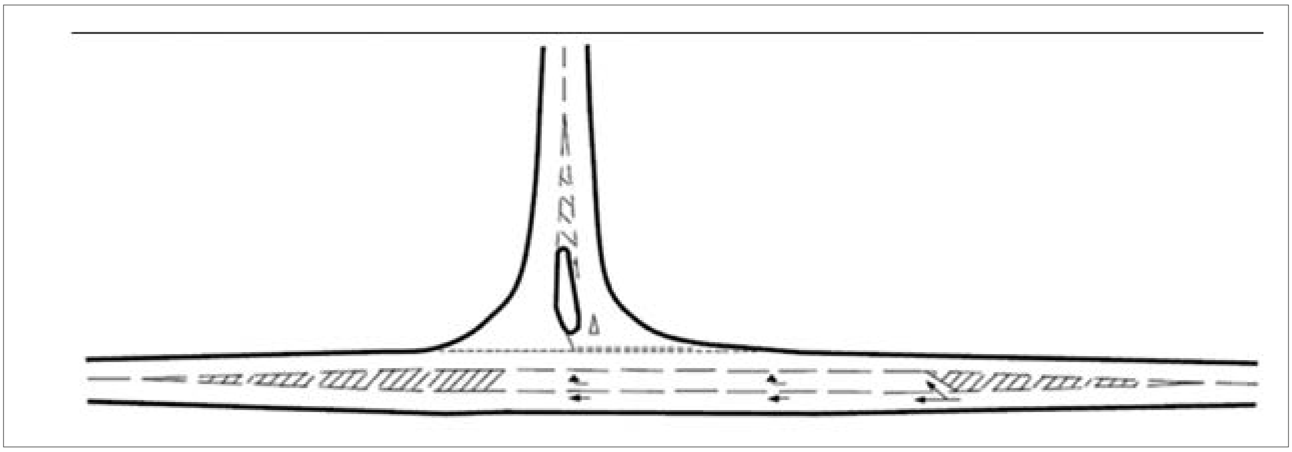


Figure 9.4 Partly Channelised T-Intersection with Ghost Islands

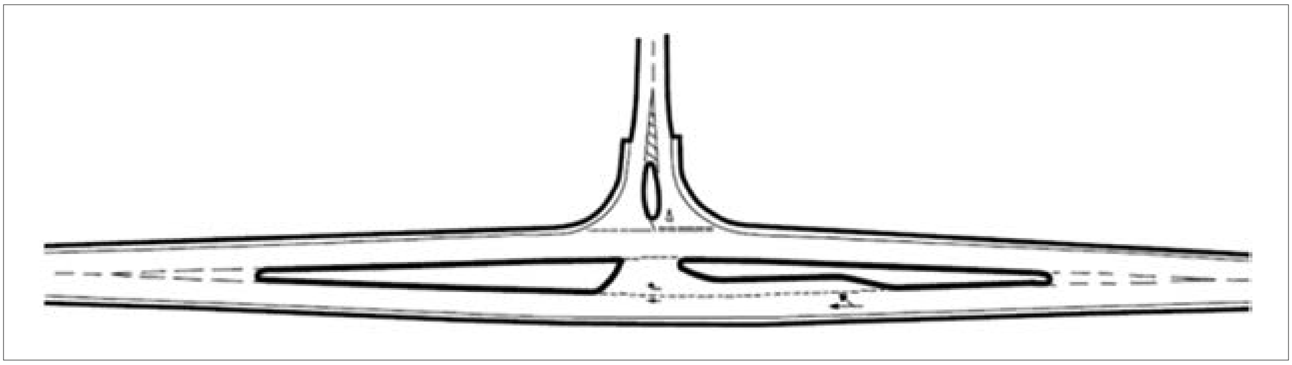


Figure 9.5 T-Intersections with Channelisation

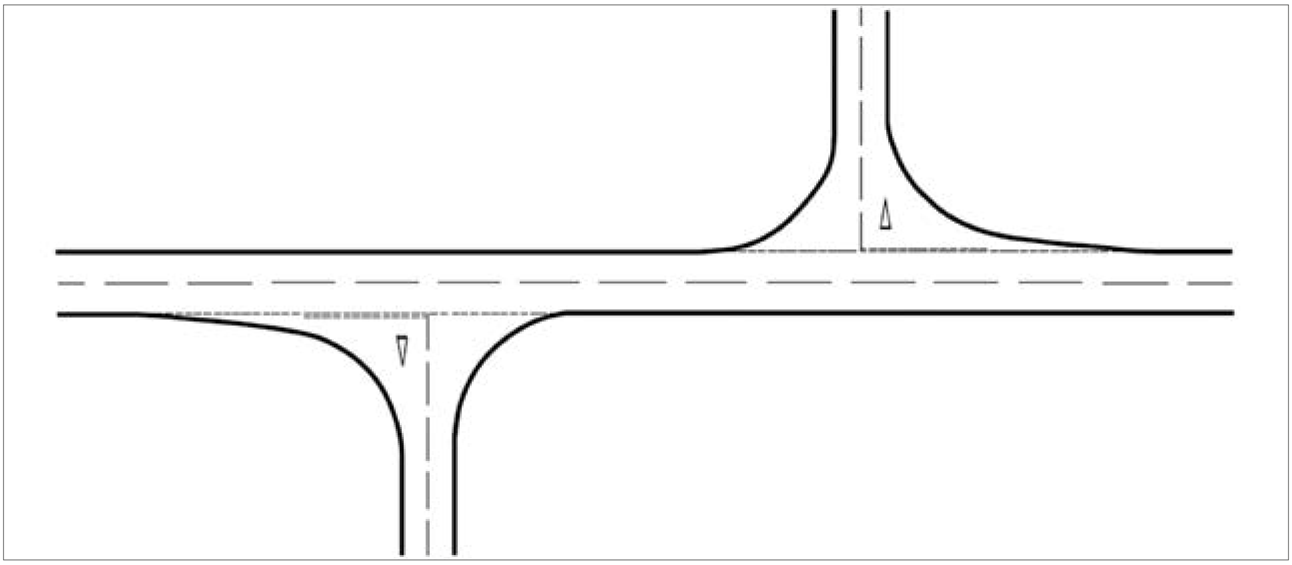
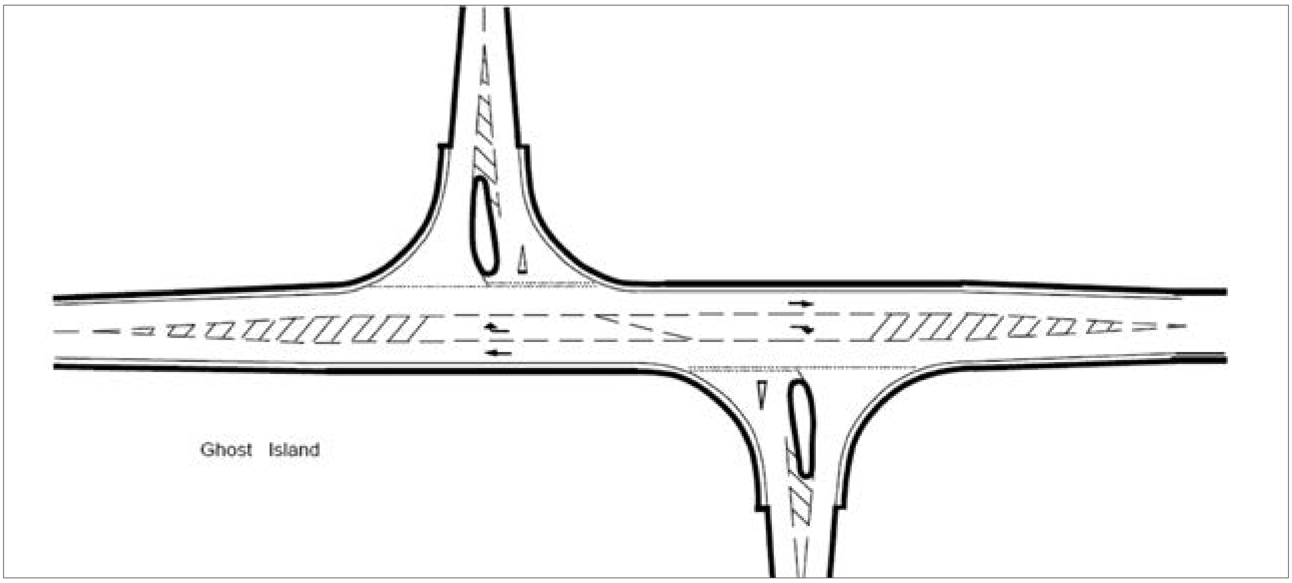


Figure 9.6 Staggered Intersection

Figure 9.7 Partly Channelised Staggered Intersection with Ghost Islands



9.2.3 Skew Intersections

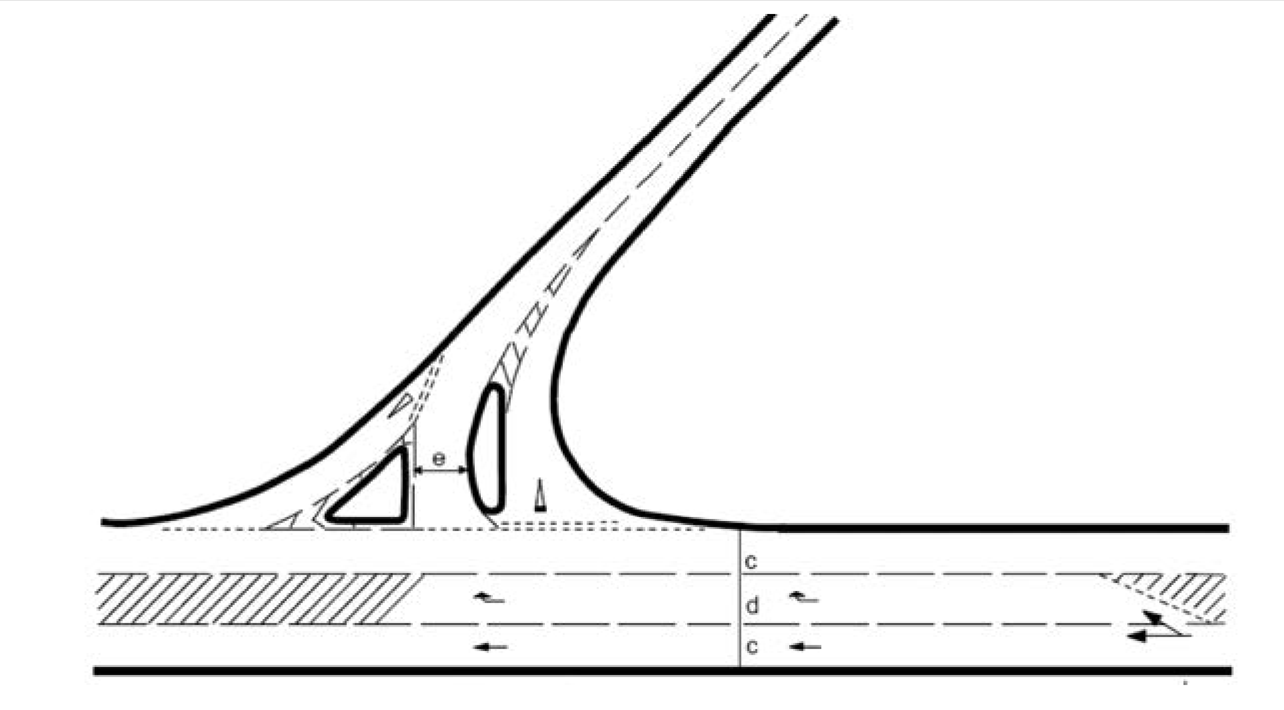
The angle of intersection of two roadways influences both the operation and safety of an intersection (Figure 9.8). Large skews are undesirable because:

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 channelisation 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°, the intersection can usually be replaced by two relatively closely spaced T-intersections.

Figure 9.8 Skew or Y Intersection with Ghost and Channelising Islands.



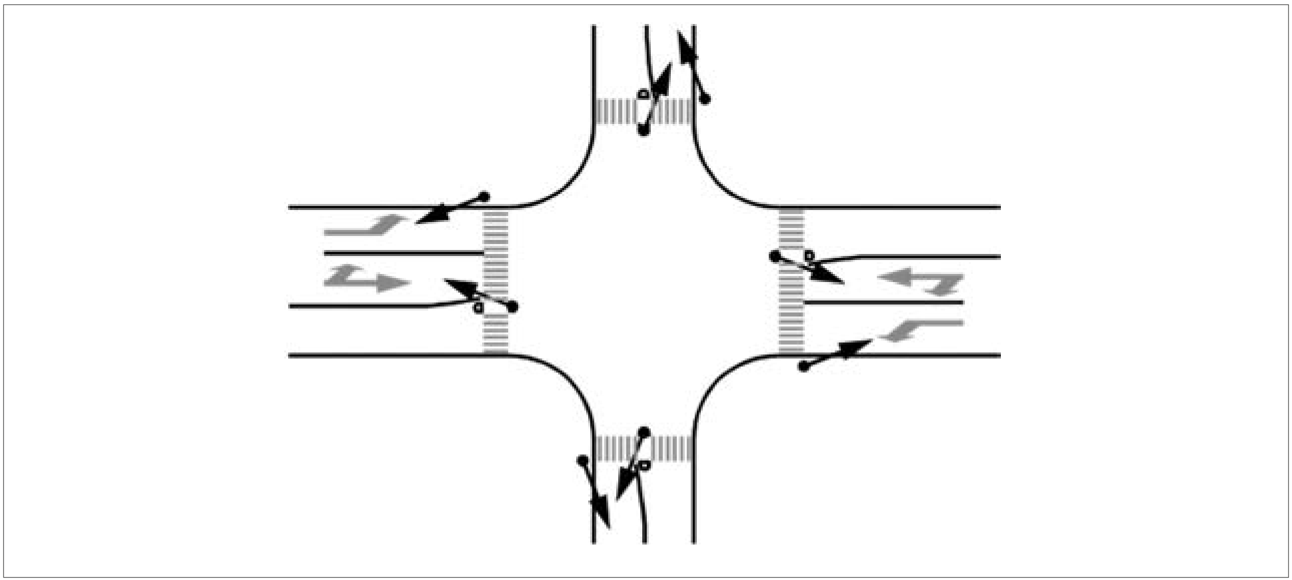
9.2.4 Roundabouts

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 10.

9.2.5 Controlled Intersections

Controlled or signalised intersections are interchanges where the traffic is controlled by traffic lights, Figure 9.9. 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 9.9 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.
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.

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9.4 Factors Affecting Selection of Intersection Type and Design

9.4.1 Functionality

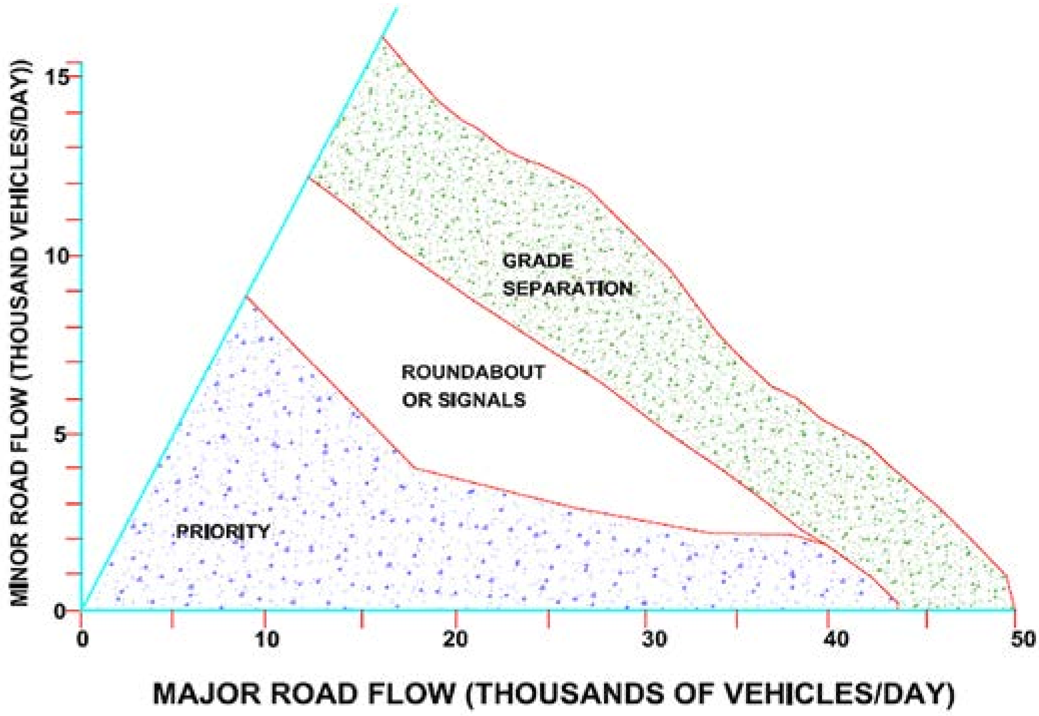
The functionality of the road is the key to identifying the most appropriate designs. In Chapter 2 over 60 design standards covering many aspects of functionality are introduced. Intersections are required on all roads hence their design must be flexible, and compromises must be made in many circumstances.

Table 9.4 and Figure 9.10 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)

Traffic Flow on Major Road (1000)	Type of Intersection			
	Traffic Flow on Minor Road			
	< 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 9.10 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 3.13.6).

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 Speed** (km/h)	**Minimum Radius** (m)
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

9.5.4.1 General

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:

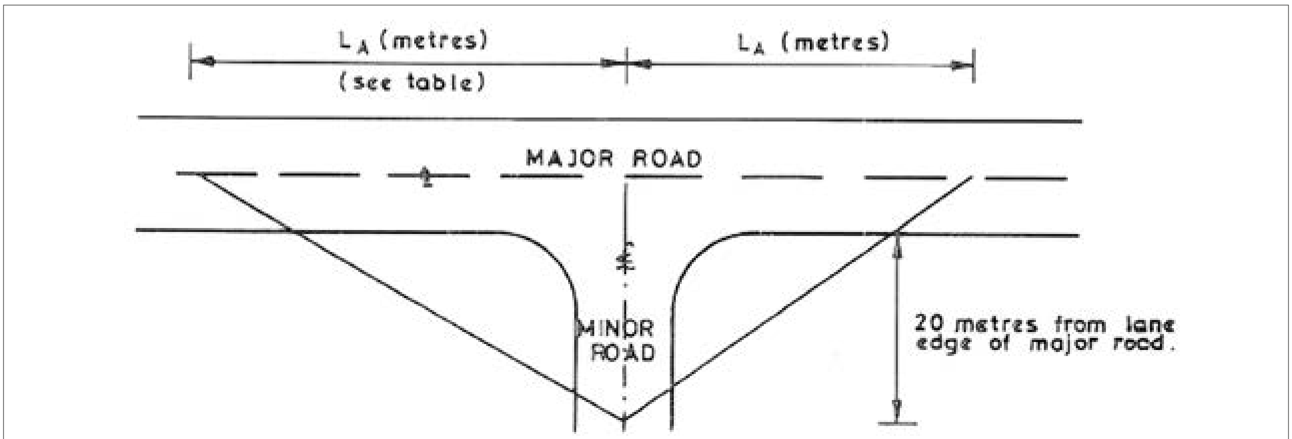
1. Whether the vehicle is in motion and at what speed when it reaches the intersection (yield control or approach control) or whether the vehicle begins from a stopped position (stop or departure control).
2. The type and power of the vehicle.
3. The length of the vehicle.
4. The distance the vehicle 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.

9.5.4.2 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 as depicted in Figure 9.11 and Figure 9.12 below.

Figure 9.11 Visibility Triangle for 'Approach Conditions'



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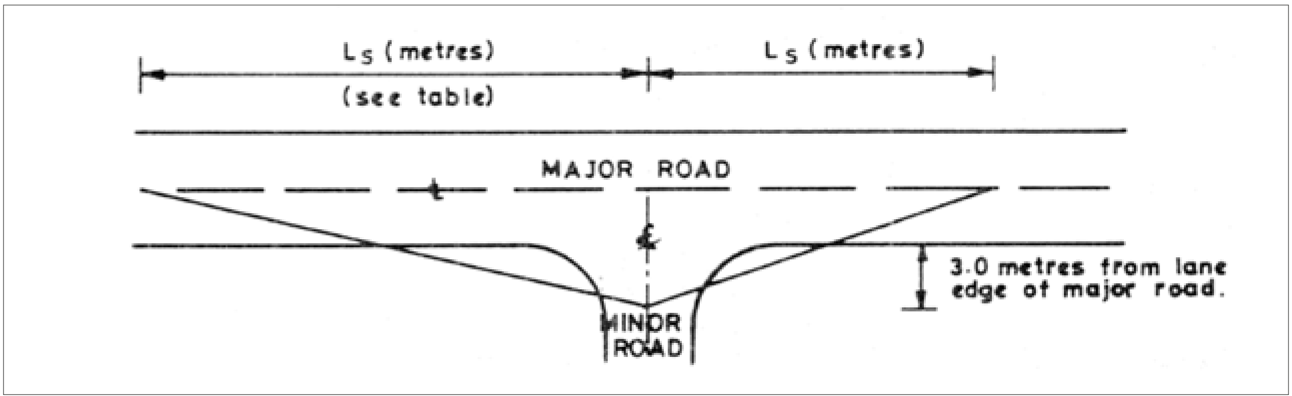
At-Grade Intersections

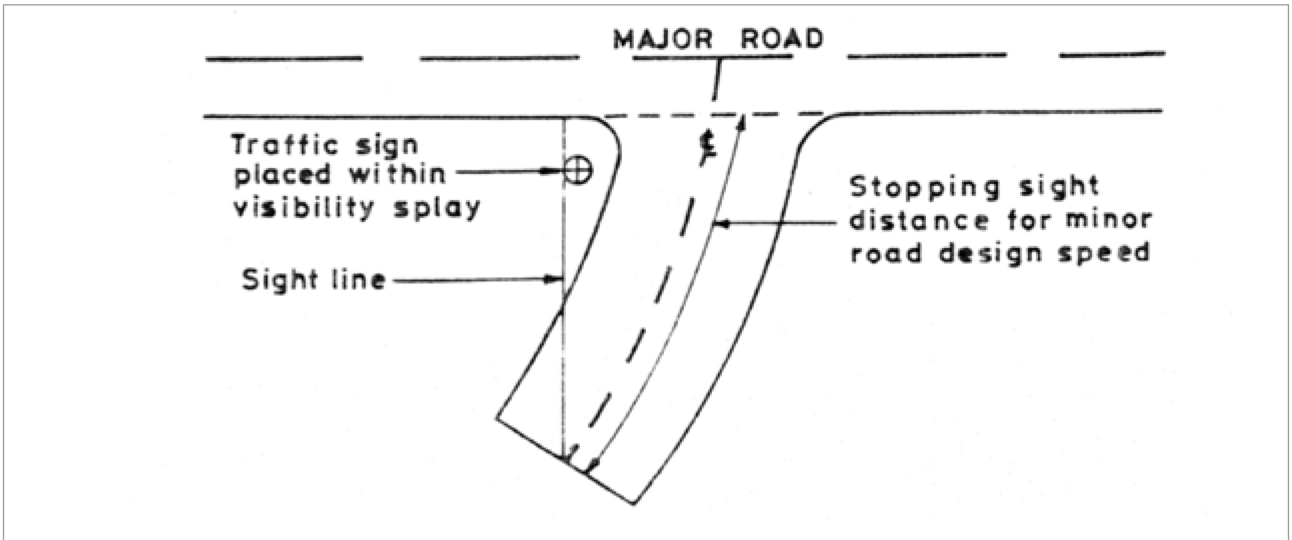
Figure 9.12 Visibility Triangle for 'Stop Conditions'



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.

On the minor road, particularly where the approach to the junction is on a horizontal curve, the visibility of traffic signs is essential and a visibility splay in accordance with Figure 9.13 must be provided.

Figure 9.13 Minor Road Approach Visibility Requirements



Where site conditions make it impossible to improve an existing junction to these standards, at least the visibility splays for the 'stop condition' must always be provided.

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, L_A (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 emphasise 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 3.6 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	Improve of sight distances
Poor corner 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 channelisation
Poor drainage	Improved drainage paths
Low surface friction	Improved surface skid resistance
Steep grades at intersections	Avoid building on a gradient greater than 3 % (stopping sight distances increase quickly as down gradient increases)
Number of adjacent driveways or access points.	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 Manoeuvrability

To provide for ease of manoeuvrability:

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 12 m;
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.

Having selected the basic junction layout, it is necessary to check that it offers sufficient capacity for both the major and minor road turning manoeuvres. The capacity of a major/minor priority junction depends upon the traffic flow and turning proportions from the different approaches.

Because the basis of control means that traffic on the major road is not delayed, capacity is expressed in terms of the number of minor road pcu's that can enter the junction at a particular level of major road flow.

The method given below is based on the gap (measured in seconds) in the major road traffic stream that the minor road vehicle requires to enter the main road safely. The gap required varies according to the particular manoeuvre and the Junction Design Speed. Figure 9.14 shows diagrammatically the various different manoeuvres at junctions for both single and dual carriageway roads and gives the appropriate gap required for each manoeuvre. On this figure the manoeuvring traffic is depicted by the symbol 'q' and the major road traffic by the symbols 'Q1' or 'Q2'. The major road flow Q, the calculation of which is given in the left hand column of Figure 9.14, is that flow in which the appropriate gap must occur. For each gap there is given a capacity curve reference (A, B, C, D, E, F or G); these capacity curves are shown on Figure 9.15. The calculated value of 'Q' is entered on Figure 9.15 and for the appropriate capacity curve, the capacity of the minor road manoeuvre, q, may be read off.

The capacity, q, of the minor road manoeuvre is then compared with the actual or predicted volume for this manoeuvre. To ensure that delays are kept to an acceptable level, the actual or predicted volume of a manoeuvre should not normally exceed 85% of the capacity value. Each manoeuvre, i.e., merging, cutting or cutting and merging should be checked in this way.

If for any manoeuvre, the predicted volume exceeds 85% of the capacity then either a staged design or an alternative form of control must be considered.

Figure 9.14 Junction Manoeuvres Major Road Flows and Gaps

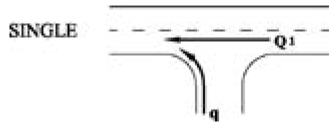
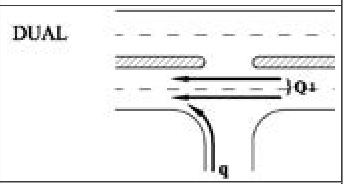
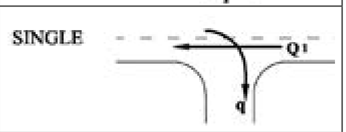
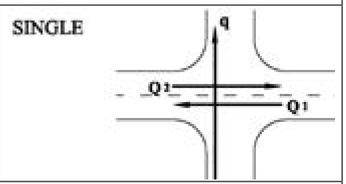
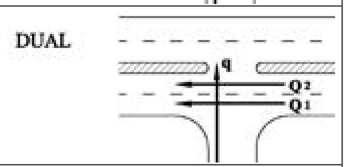
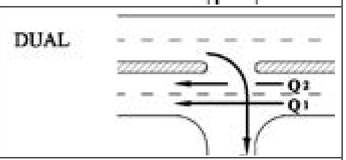
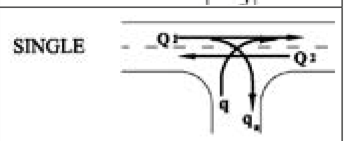
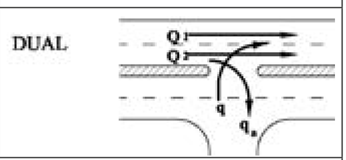
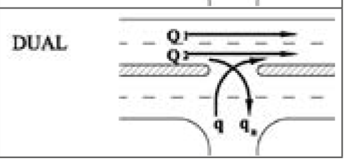
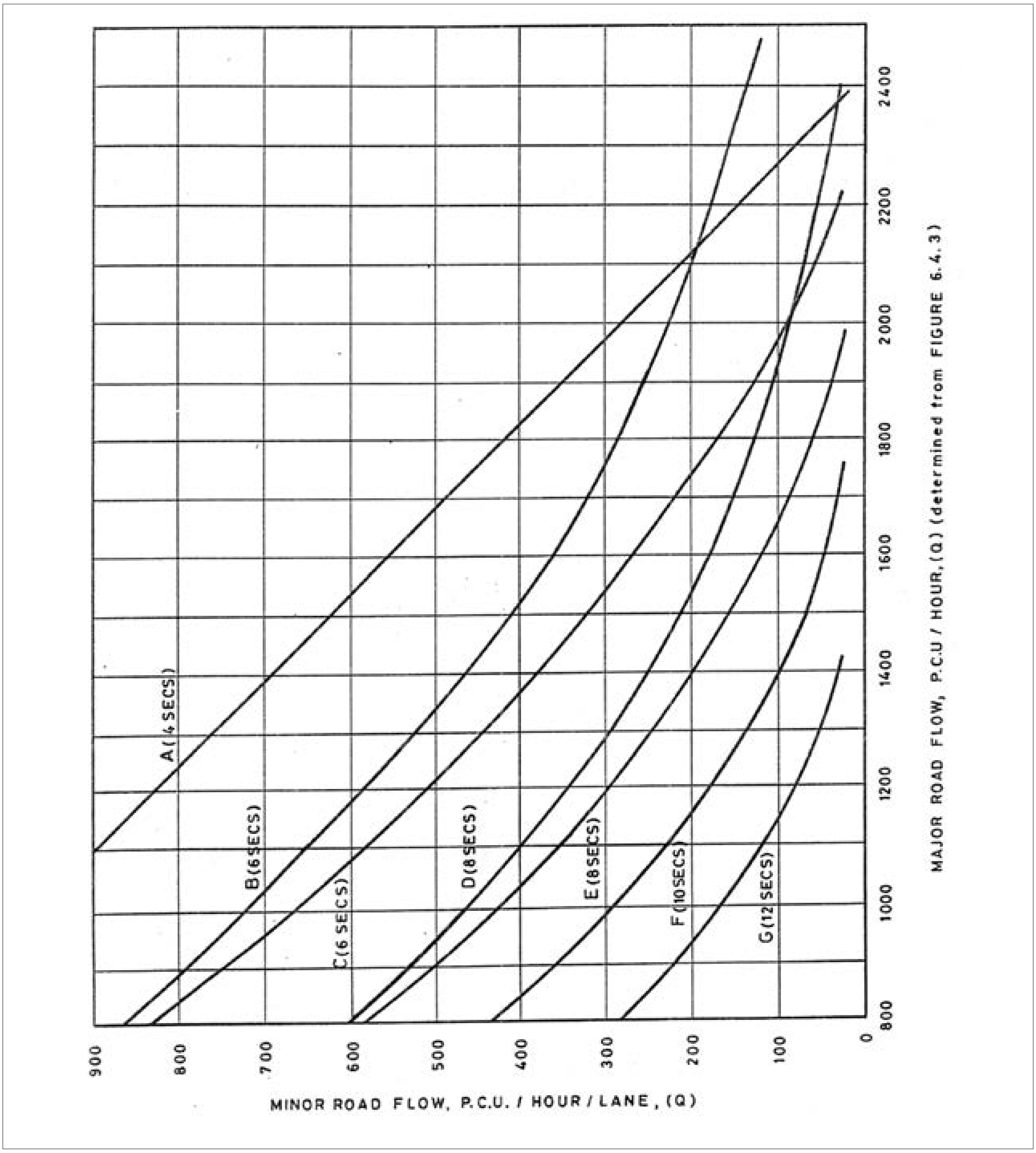
Mano-euvre	Major Road Flow Q P.C.U/Hr	Capacity Curve and Gap for Junction Design Speed		Pictorial & Basic Representation of Manoeuvres Layouts
		< 65 km/h	> 65 km/h	
Merging	Q =Q1	**Curve A** (4 secs)	**Curve C** (6 secs)	
	Q = 40% Q1	**Curve A** (4 secs)	**Curve C** (6 secs)	
Cutting	Q = Q1	**Curve A** (4 secs)	**Curve C** (6 secs)	
	Q = Q1 + Q2	**Curve B** (6 secs)	**Curve D** (8 secs)	
	Q = Q1 + Q2	**Curve B** (6 secs)	**Curve D** (8 secs)	
	Q = Q1 ♦ Q2	**Curve B** (6 secs)	**Curve D** (8 secs)	
Cutting & Merging	Q = Q1*Q2*1⅓q_R	**Curve D** (8 secs)	**Curve F** (10 secs)	
	Q = Q,*Q2»1^q_R	**Curve D** (8 secs)	**Curve G** (12 secs)	
	Q = 60%(Q1 + Q2)+ 1⅔q_R	**Curve C** (6 secs)	**Curve E** (8 secs)	

Figure 9.15 Capacity of Minor Road Flows



9.7 Intersections

9.7.1 Priority and Controlled Intersections

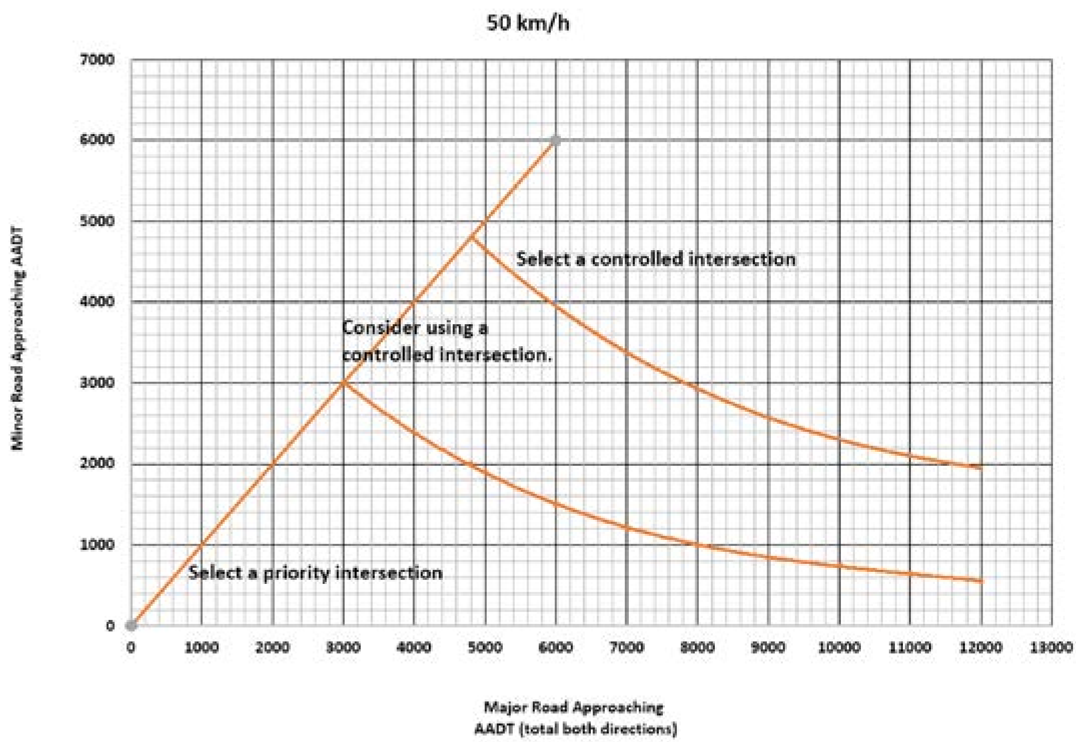
Priority intersections are introduced in Section 9.2.2 and examples are shown in Figure 9.3 through to Figure 9.9. The selection of a priority intersection should be based mainly on safety. The selection can be made by using known relationships between safety levels and the average daily approaching traffic volumes (AADT in vehicles/day) based on accident statistics. Figure 9.17 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.

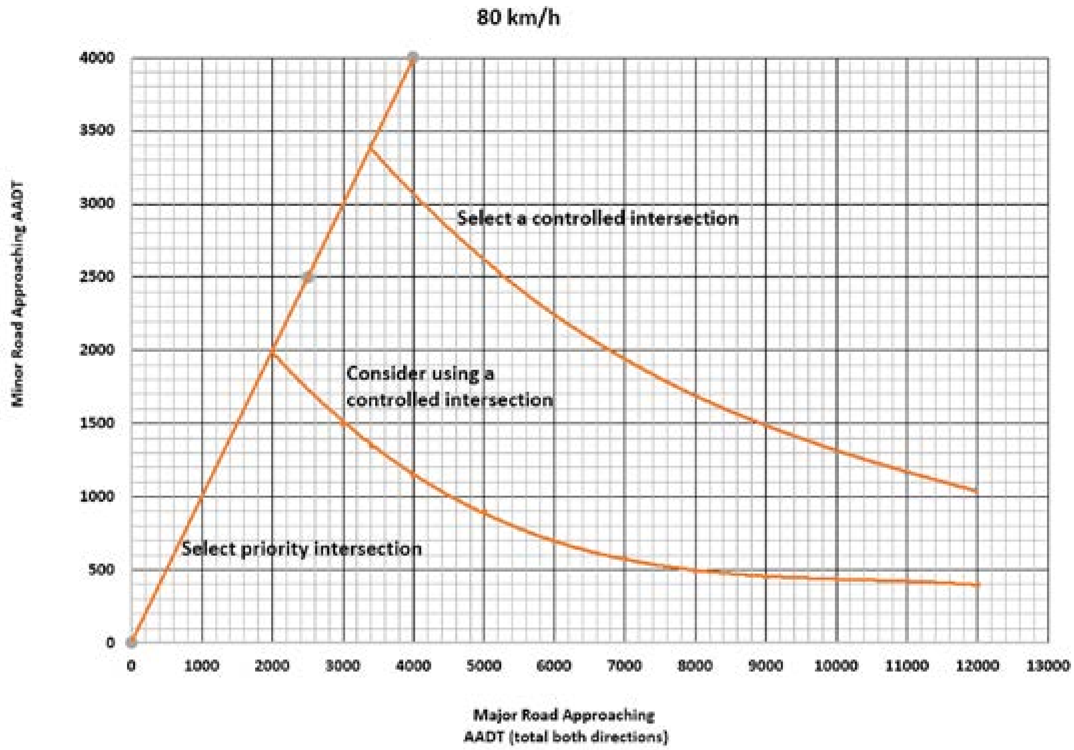
Many intersections must cope with peak hour flows for a short period of the day and much lower traffic flows at other times. Thus, sometimes just a priority interchange is required but at others a controlled interchange is necessary. In these circumstances a signalised interchange is required. Figure 9.17 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 9.16a (design speed 50 km/h) is primarily for urban interchanges, Figure 9.16b (80 km/h) is for metropolitan or rural interchanges and Figure 9.16c (100 km/h) is for rural interchanges only.

Figure 9.16 Capacity of Minor Road Flows

a)



b)



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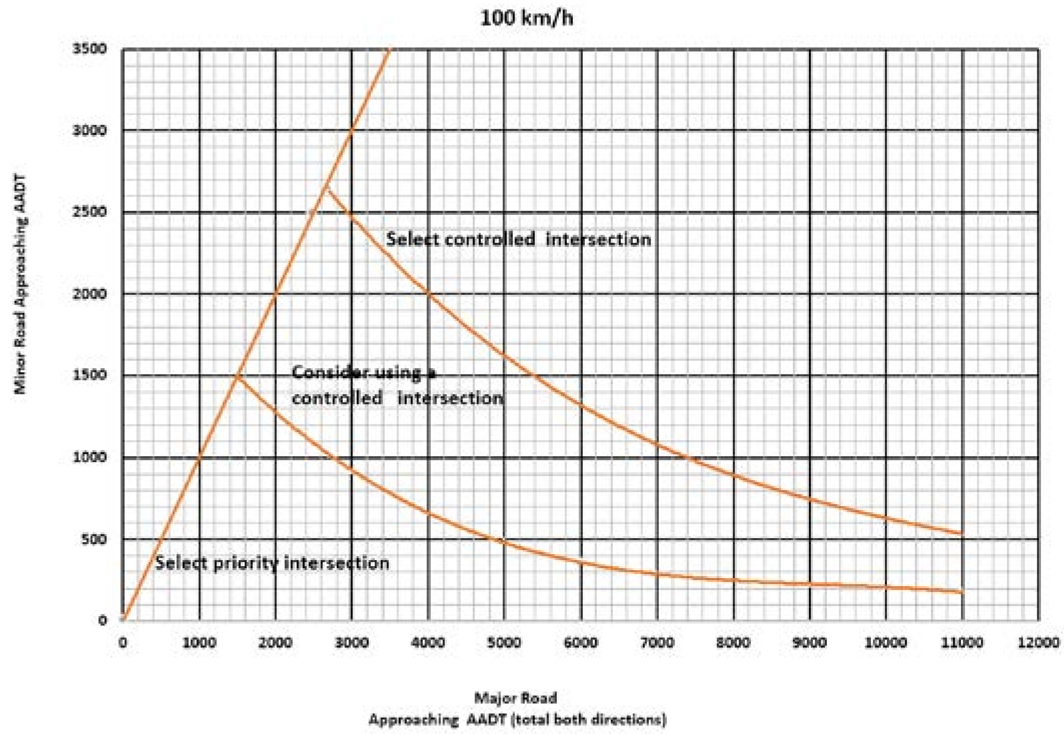

Note: *The lines in these figures cannot be precise. They are for guidance.*

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At-Grade Intersections

9.7.2 Priority Intersections Based On Traffic In The Peak Hour

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.

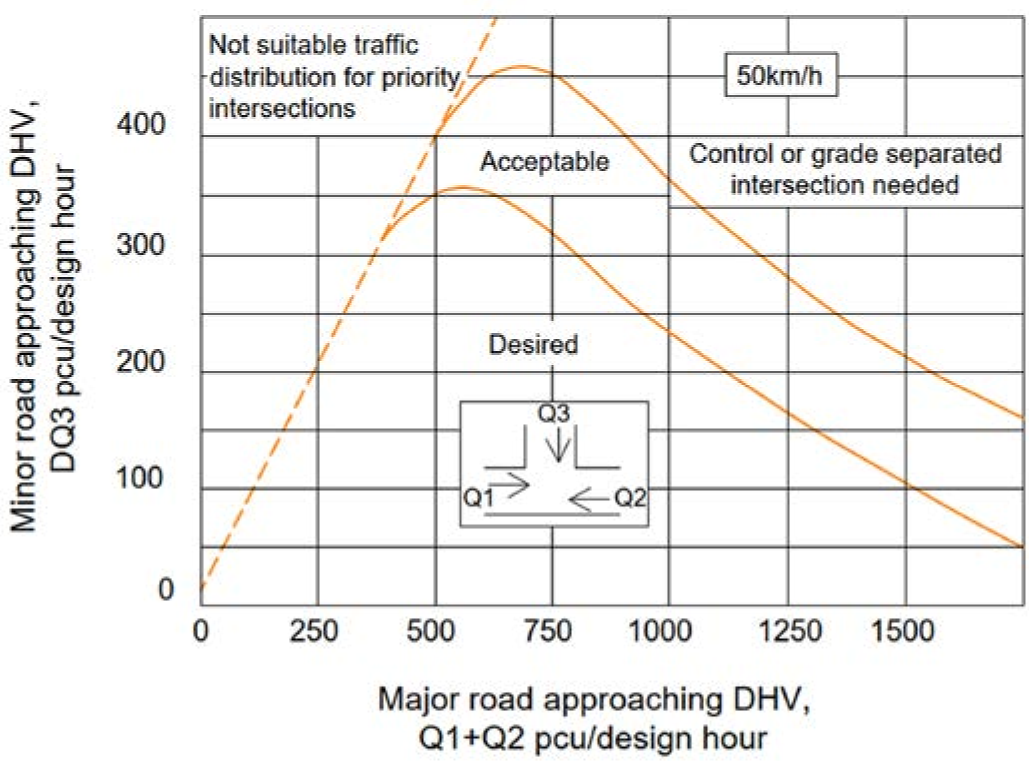
Figure 9.17 shows the relationships between capacity and traffic volumes in PCU/h approaching the interchange during the 'design hour'. The diagrams are for T-intersections on 2-lane roads with speed limits of 50, 80 and 100 km/h. Table 9.11 summarises the important traffic levels. The 'desired' level 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.

Table 9.11 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 next Sections.

Figure 9.17 Capacity and Traffic Volumes in PCU/h During the 'Design Hour'



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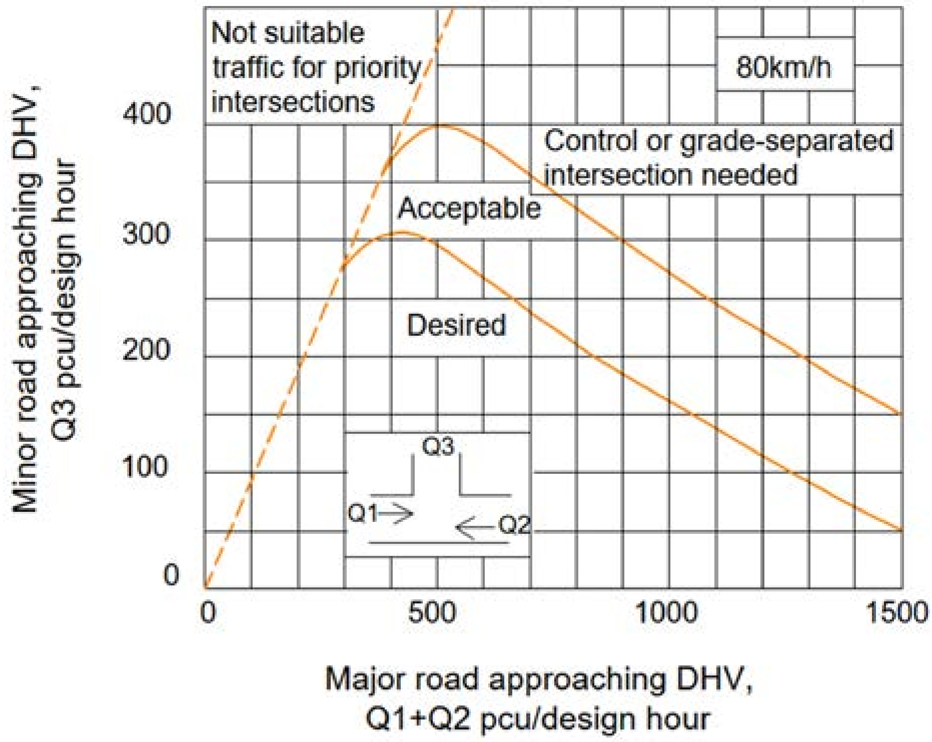

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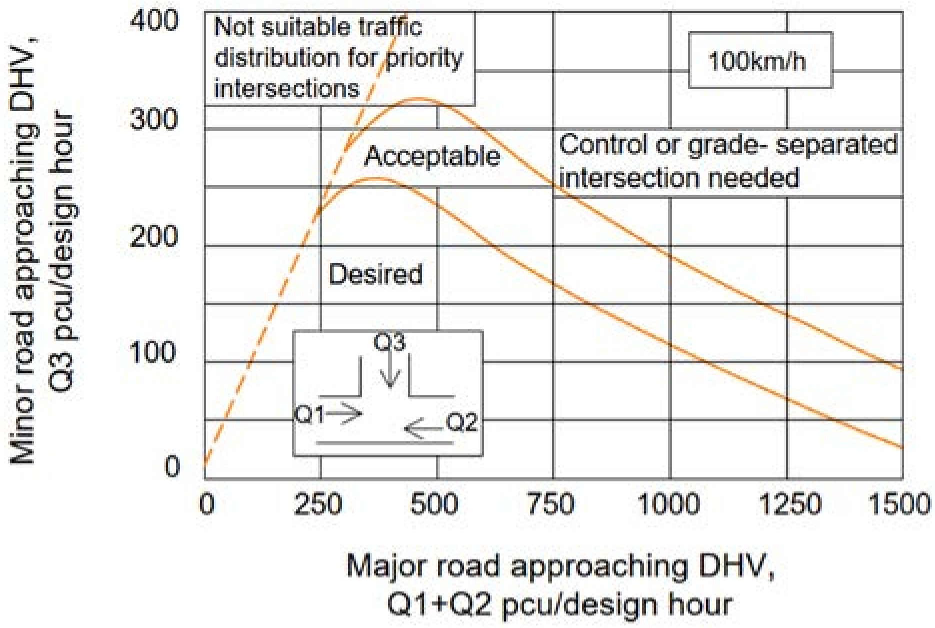


Note: *The lines in these figures cannot be precise. They are for guidance.*

9.8 Signal Controlled Intersections

For higher traffic levels Table 9.4 and Figure 9.18 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

Where,

C_a = 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 9.18 and Figure 9.19. Note the locations of the pedestrian crossings.

Figure 9.18 Example of a Signal-Controlled Crossroads

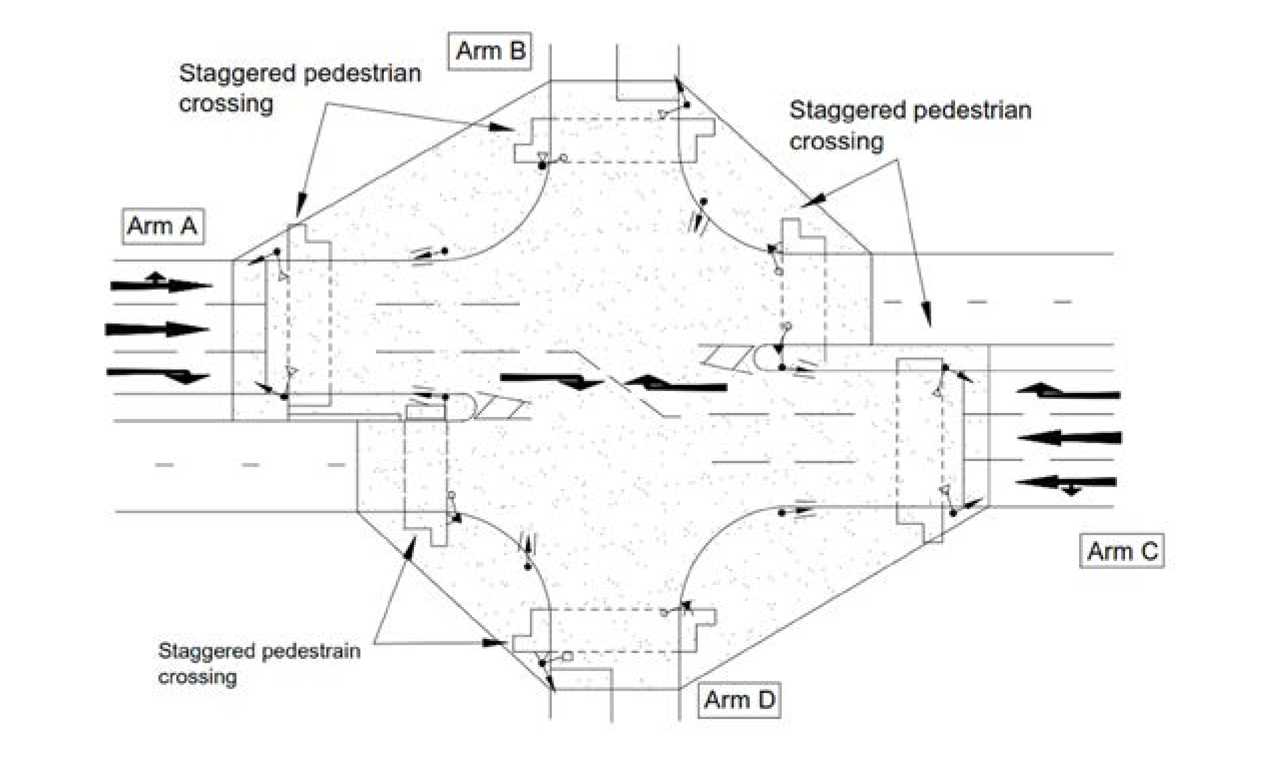
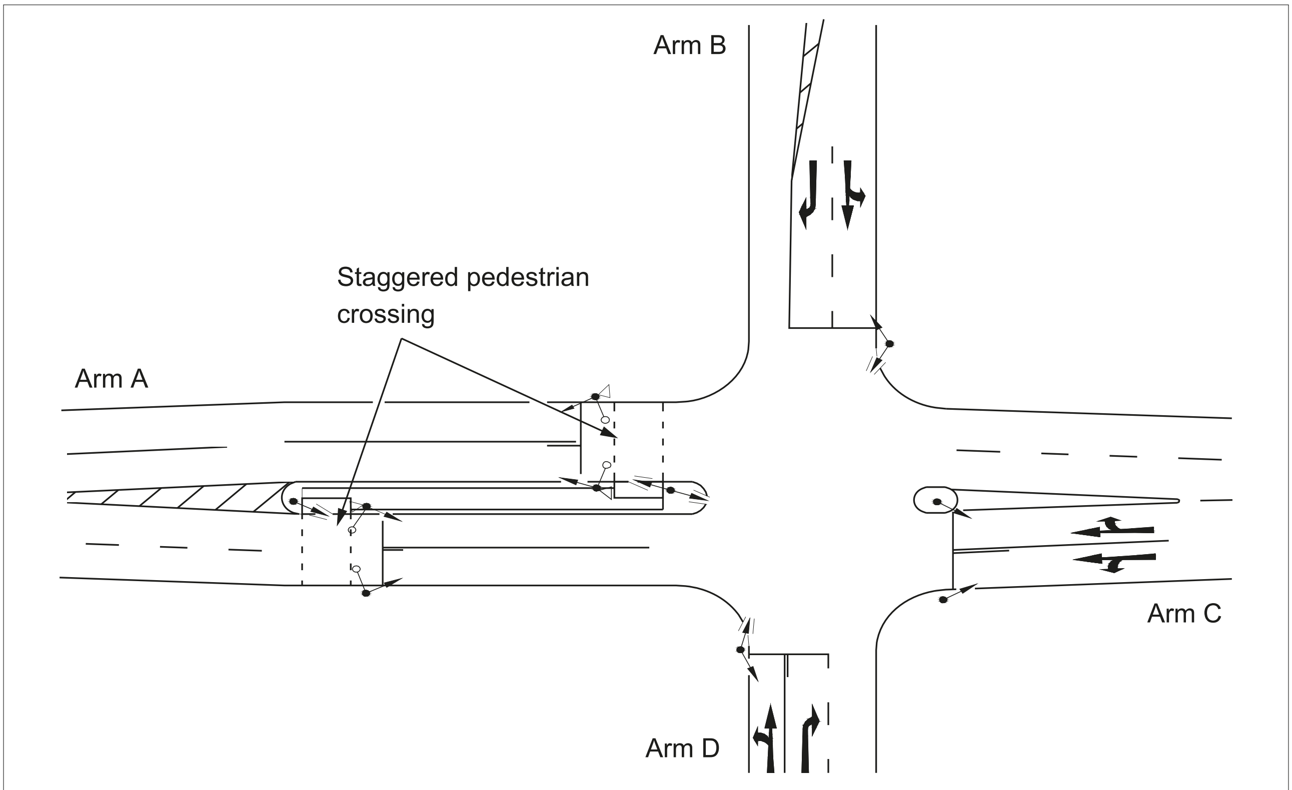


Figure 9.19 Signal-Controlled Crossroads with a Staggered Pedestrian Crossing



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At-Grade Intersections

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 9.12 shows the recommended minimum spacing between successive uncontrolled intersections.

Table 9.12 Minimum Spacings (m) Between Uncontrolled Intersections

Design Speed	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 9.13 lists the steps required for intersection design. In practice this process will also be dependent on current methods in use.

Table 9.13 Steps in the Selection and Design Procedure

Step	Access Class
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:

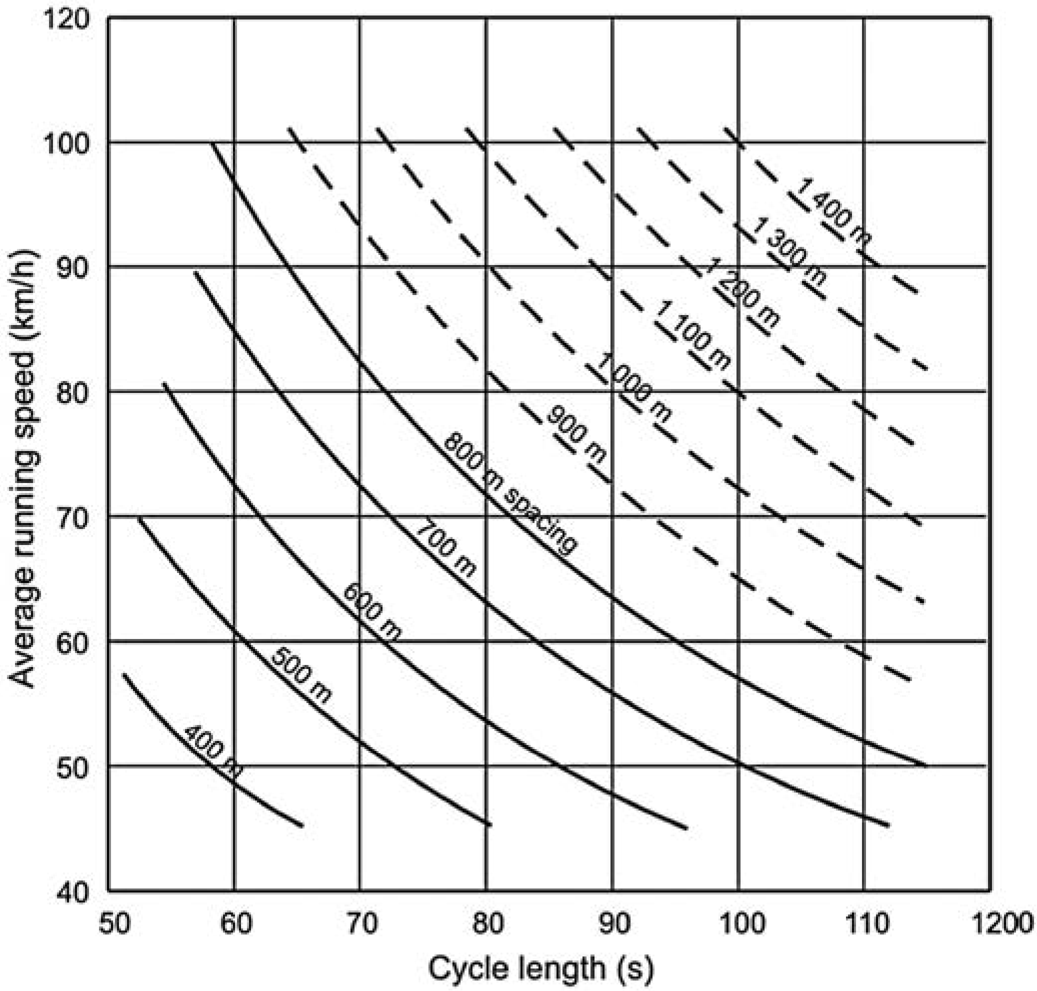
1. A distance between minor road centre lines equal to the passing sight distance appropriate for the design speed of the junction plus half the length of the widened major road sections at each junction, or
2. A grouping of minor road junctions into pairs to form staggered T-junctions and a distance between pairs as in (1) above.

For signalised roads, the traffic signal cycle lengths and traffic speed should be consistent with the intersection spacings as indicated in Figure 9.19. 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 9.20 Desirable Spacings of Controlled Intersections



Note: *Dashed lines indicate reduced platooning and signal progression benefits for signal spacing greater than 800 m*
Source: *SANRAL, (Geometric Design Guidelines)*

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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 Walkways 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).

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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 9.4, Figure 9.7 and Figure 9.8.

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 9.5 and Figure 9.8 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 9.5) 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 9.21 (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.

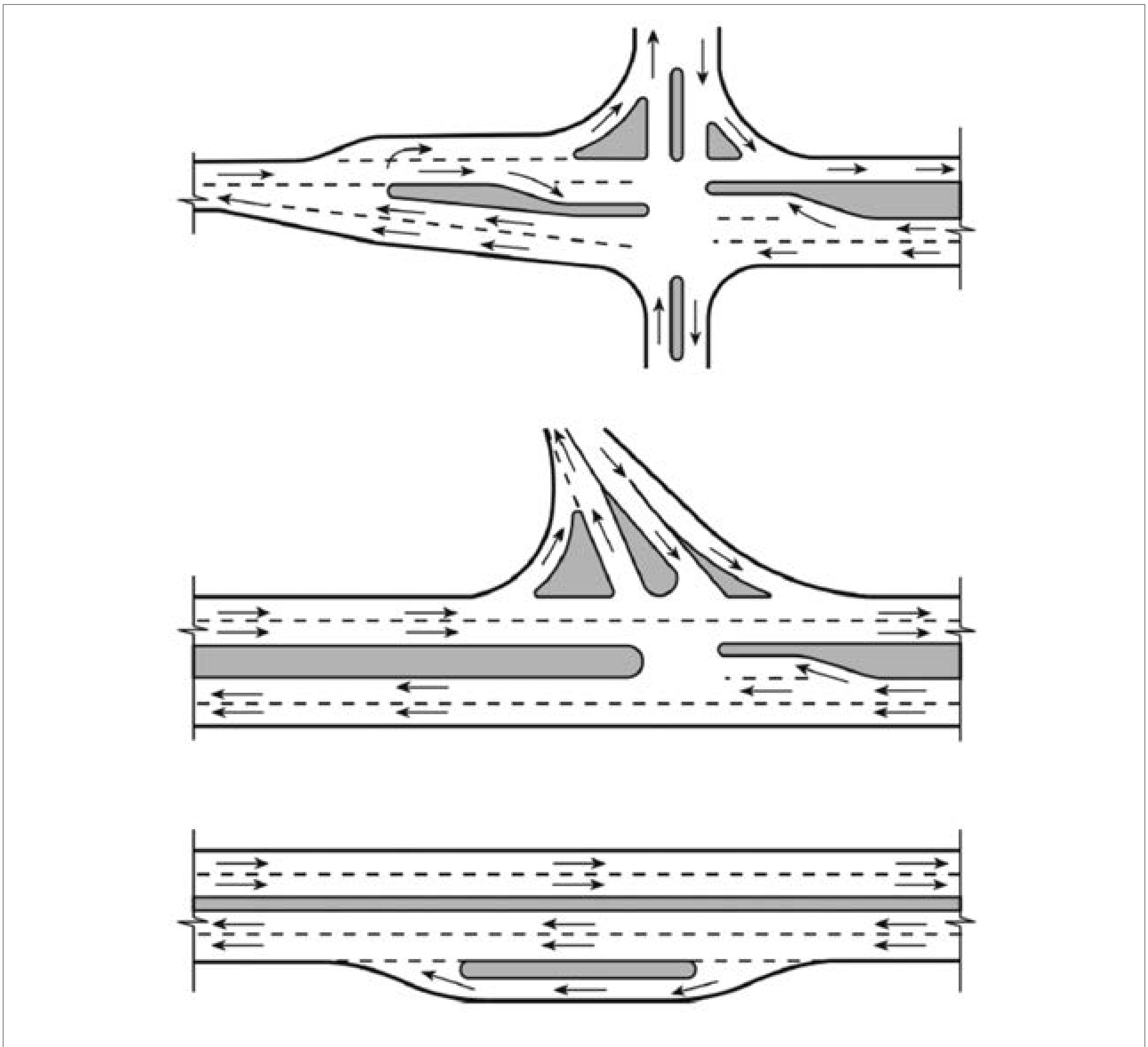
Islands may be kerbed, painted or simply non-paved. Kerbed islands provide the most positive traffic delineation and are normally used in urban areas to provide some degree of protection to pedestrians and traffic control devices. The island kerbs should be offset a minimum of 0.3 m from the edge of 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.

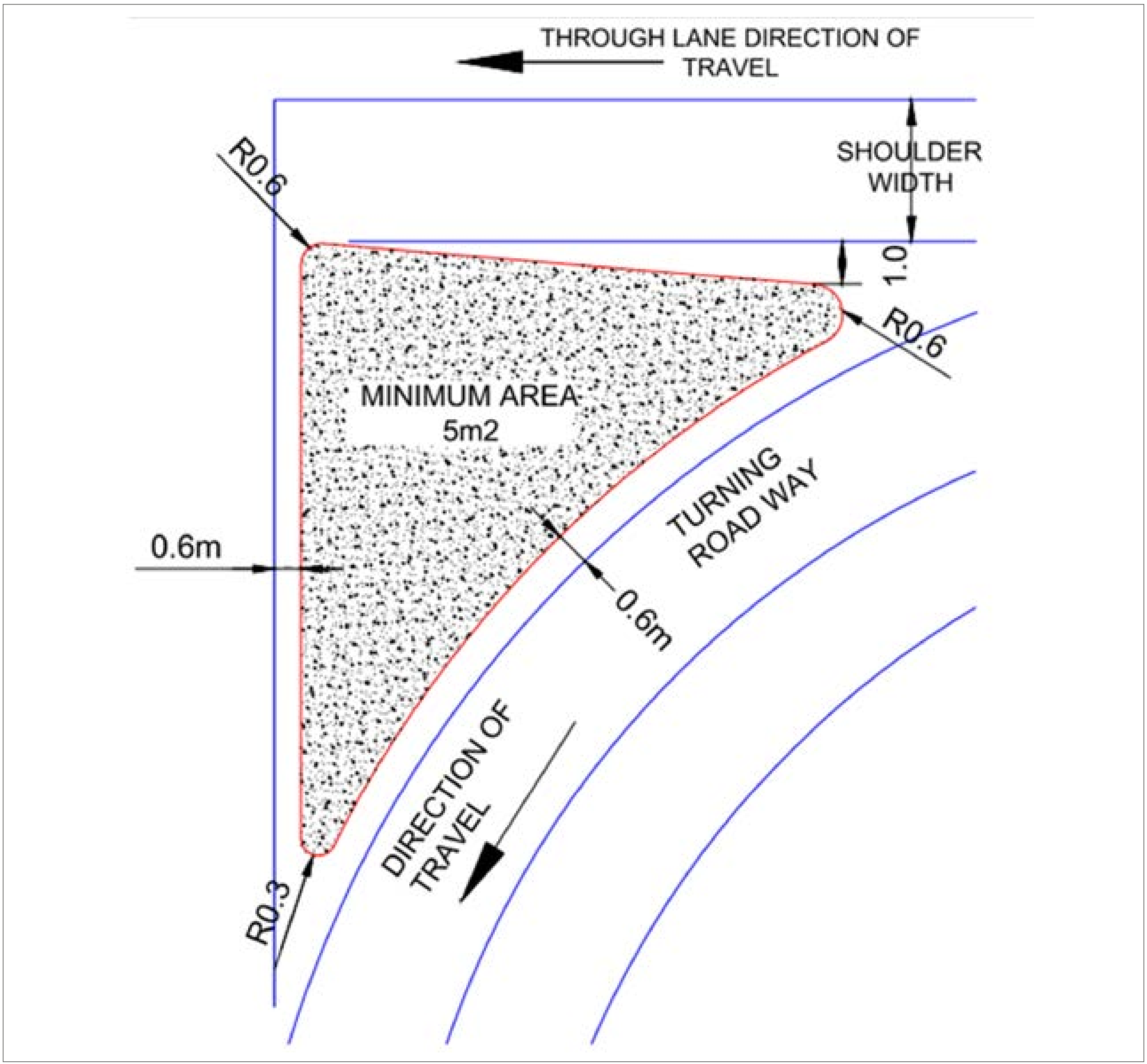
Figure 9.21 Typical Traffic Islands



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 9.22. 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 9.22 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.

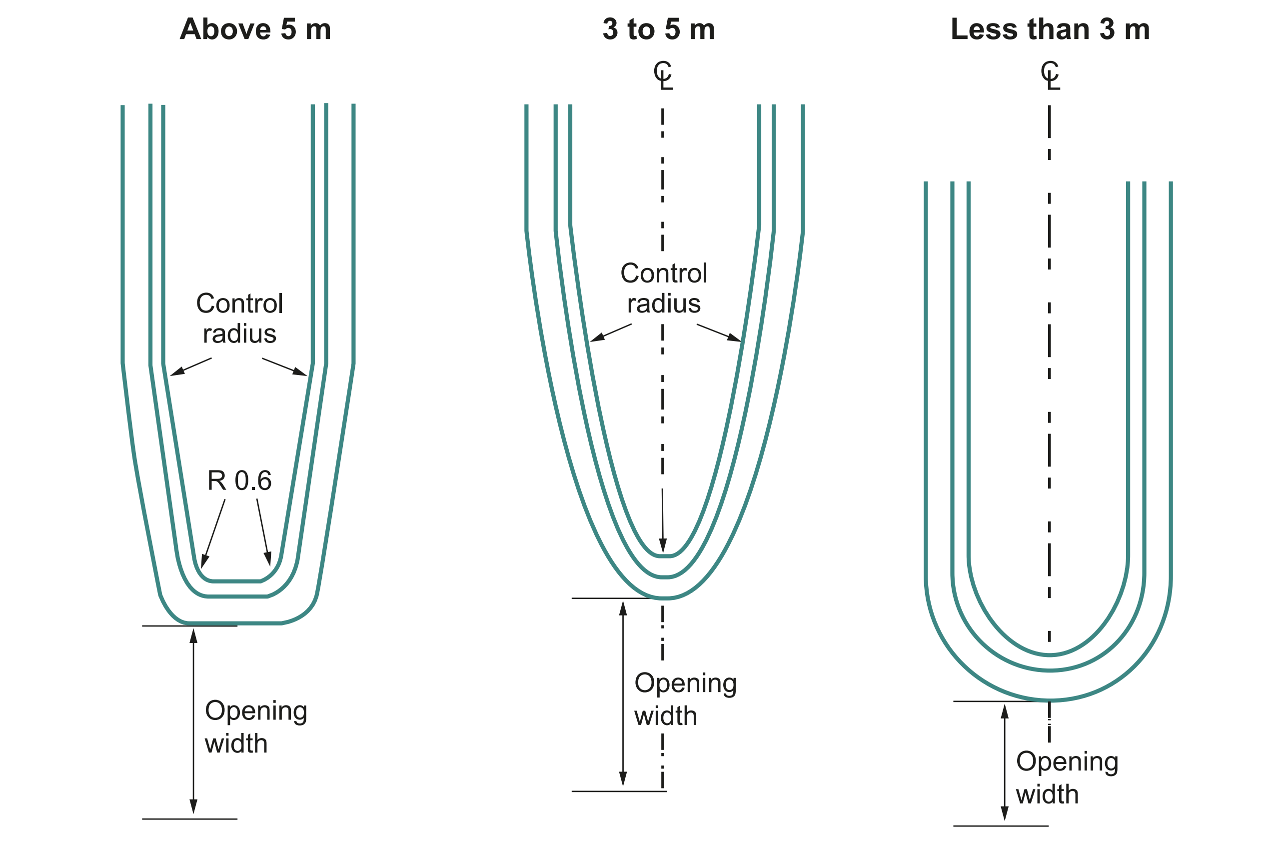
A further control on the layout of the median opening is the volume and distribution of traffic passing through the intersection area. If the median is wide enough to accommodate them, it may be advisable to make provision for speed-change and storage lanes. The additional lanes reduce the width of the 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 9.23.

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 9.23 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 9.24 and Figure 9.25. 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.

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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.

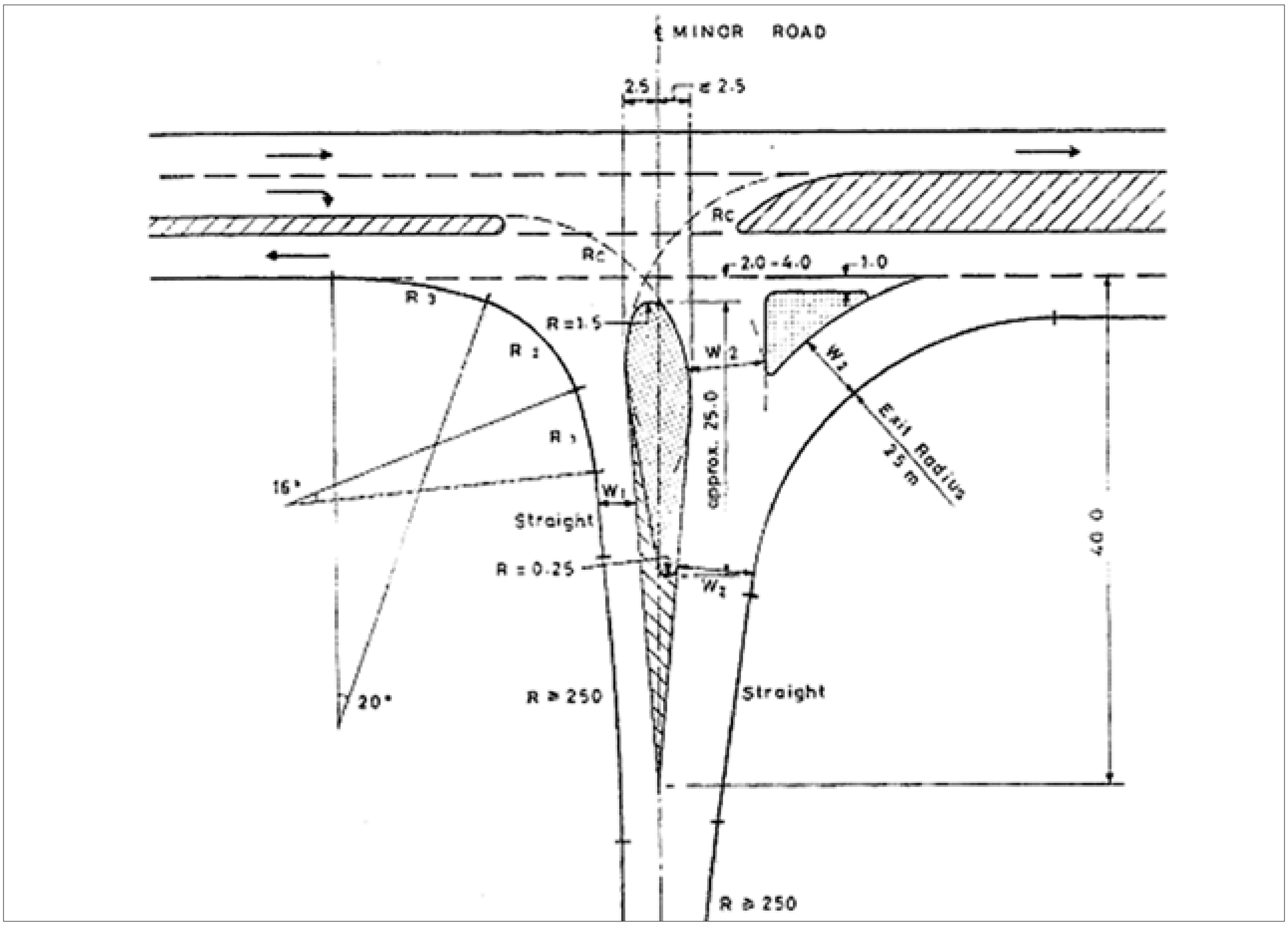
9.11.8 Channelised Intersection Layout

A channelised intersection, as shown in Figure 9.4 is to be used whenever a separate right turn lane is required. The layout in Figure 9.7 also includes a left turn lane.

The partly channelised intersection shown in Figure 9.5 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 9.7.

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 9.24 Layout of a Channelised Intersection



Notes:

R_C = Central radius dependent upon vehicle turning characteristics (minimum turning radius) recommended value: 15 m.

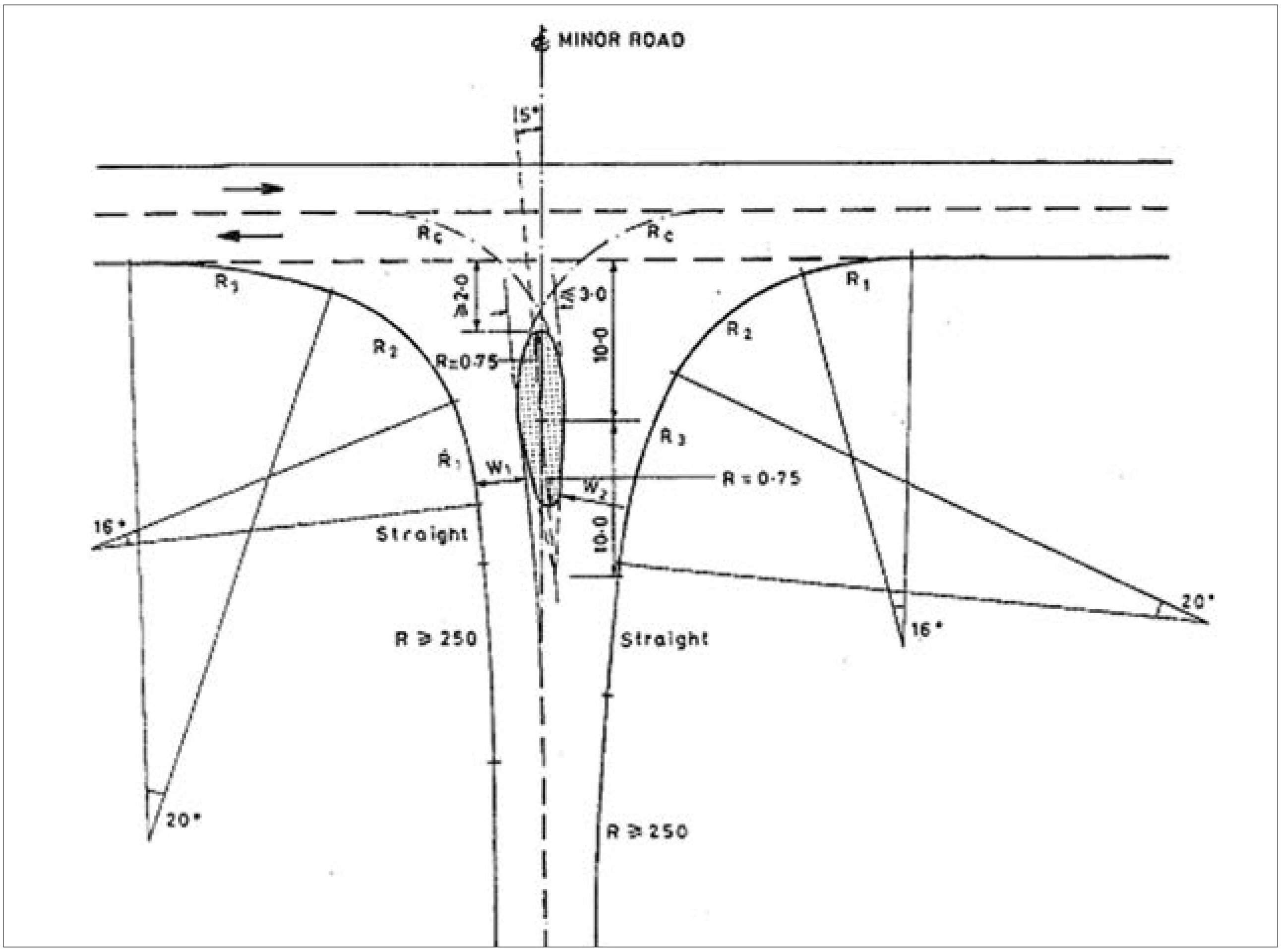
The ratio R_1: R_2: R_3 to be 2:1:3 and the recommended value for R_2 is 12.0 m.

W_1 is equal to the minor road lane width but shall not be less than 3.0 m.

W_2 is 5.5m (excluding offsets to raise kerbs)

For detail of major road widening, see Section 9.11.9

Figure 9.25 Layout of a Partially Channelised Intersection



Notes:

RC = 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 10,000 m 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.

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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.

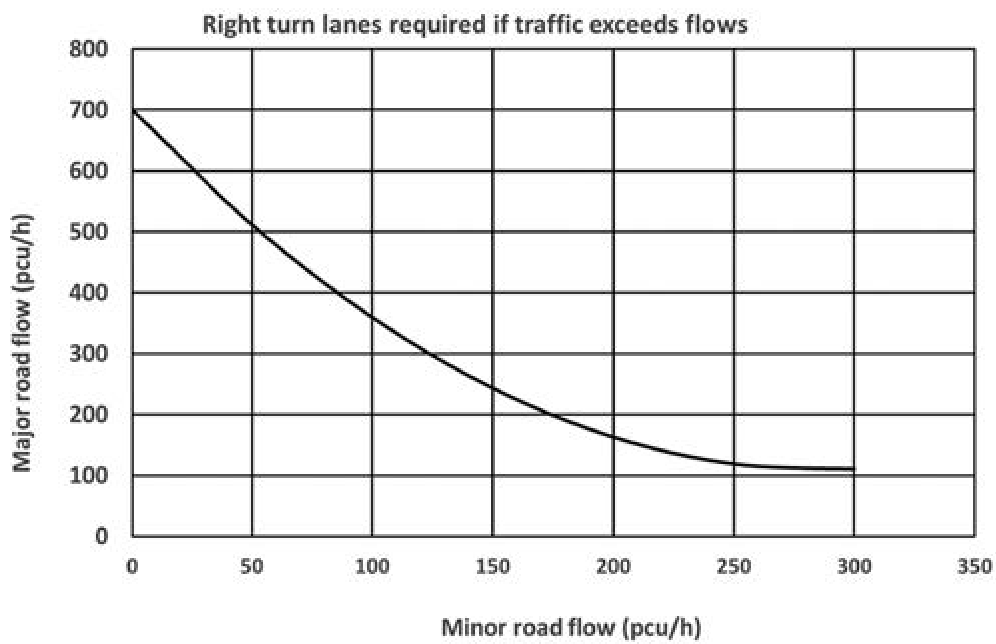
9.11.10 Speed Change Lanes

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 9.26.
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 9.26 Conditions Requiring a Right Turn Lane



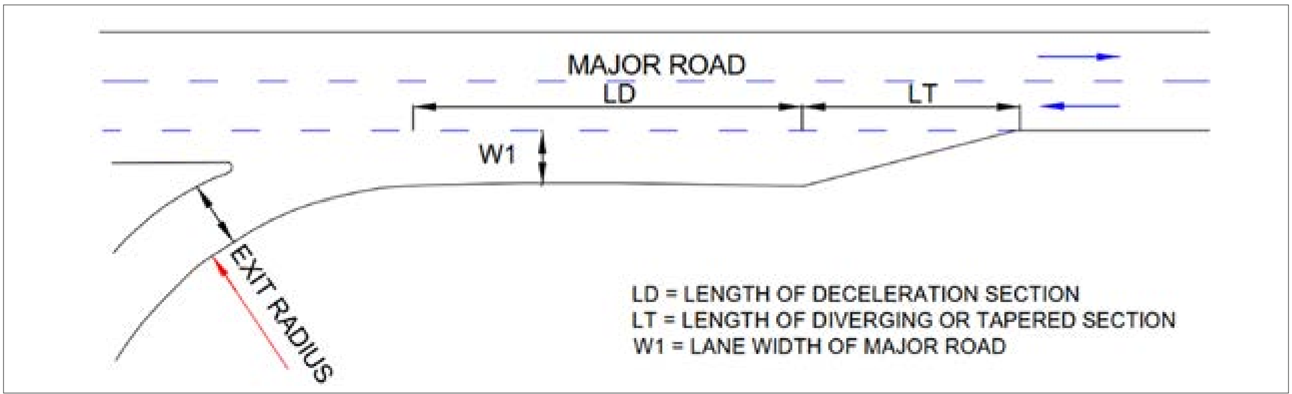
9.11.11 Decelerating Lane: Left Turn

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 9.27.

Figure 9.26 Conditions Requiring a Right Turn Lane



The length of the left turn lane including the taper, measured as shown in the Figure 9.27, is related to design speed as indicated in Table 9.14. 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 9.14 Length of Left Turn Lane

Main Road Design Speed (km/hr)	Length 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

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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 9.15 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 9.15 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 9.16. Acceleration also takes place on the taper, which is thus included in the overall length of the acceleration lane.

Table 9.16 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

A separate lane for right turning traffic (i.e. traffic turning right from the major road into the minor road) shall be provided under any of the following conditions:-

1. On dual carriageway roads.
2. When the Junction Design Speed is 100 km/h or greater and the A.A.D.T. on the major road in Design Year 10 is greater than 1500 pcu
3. When the ratio of the major road flow being cut to the right turning flow exceeds the values given on Figure 9.28.
4. On four, or more lane undivided highways.

Right turn lanes consist of a taper section, a deceleration section, and a storage section. The minimum lengths are as for left turn lanes and shown in Table 9.17.

Details of the layout for a right turn lane are shown in Figure 9.29 for a single carriageway and in Figure 9.30 for a dual carriageway. Both figures also show additional details of a right turn lane.

Table 9.17 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 9.28 Criteria For Determining The Provision Of Right Hand Turn Lanes

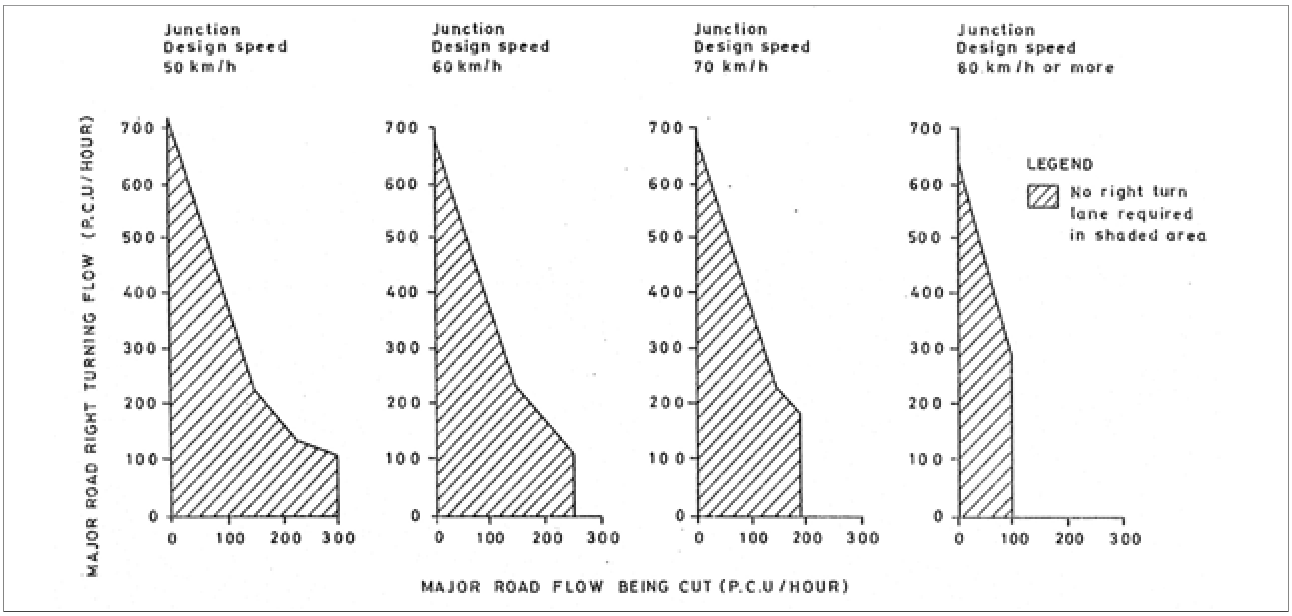


Figure 9.29 Layout for Right Turn Lane, Single Carriageway

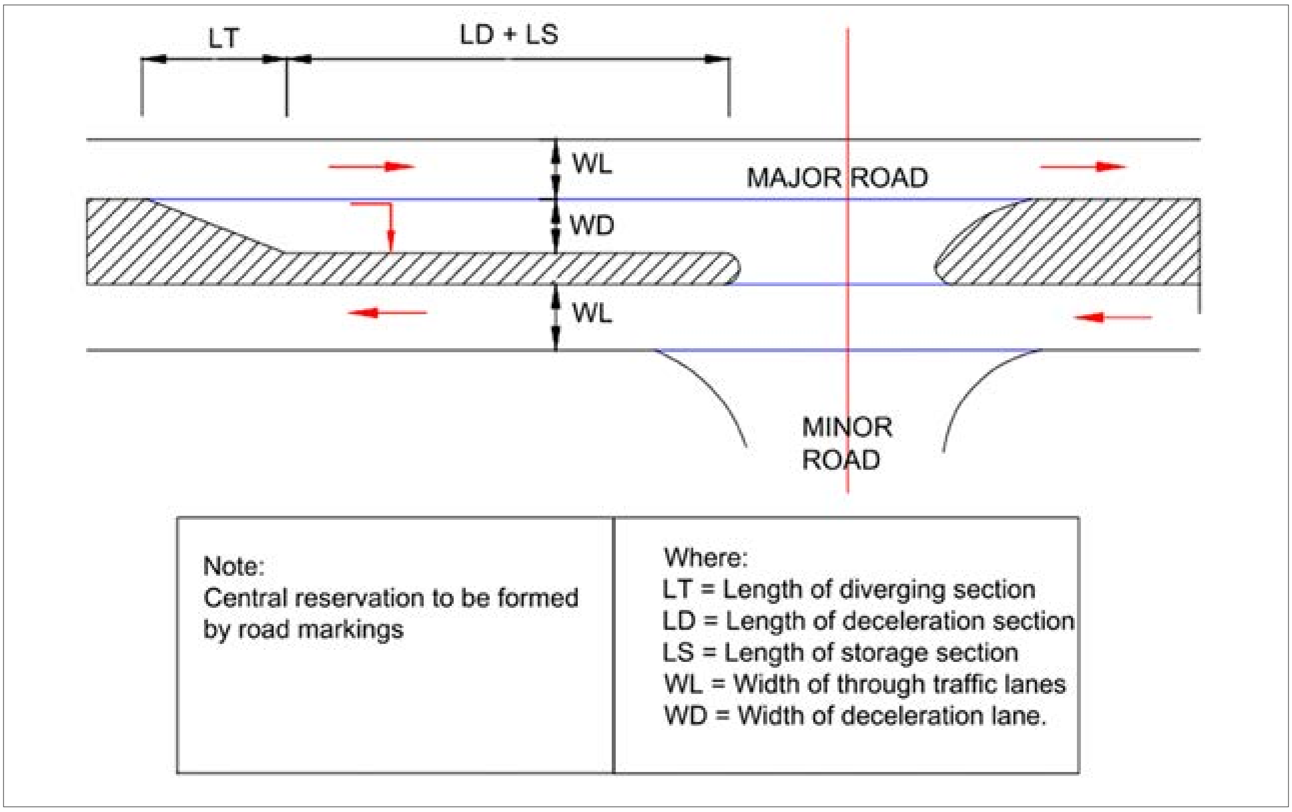
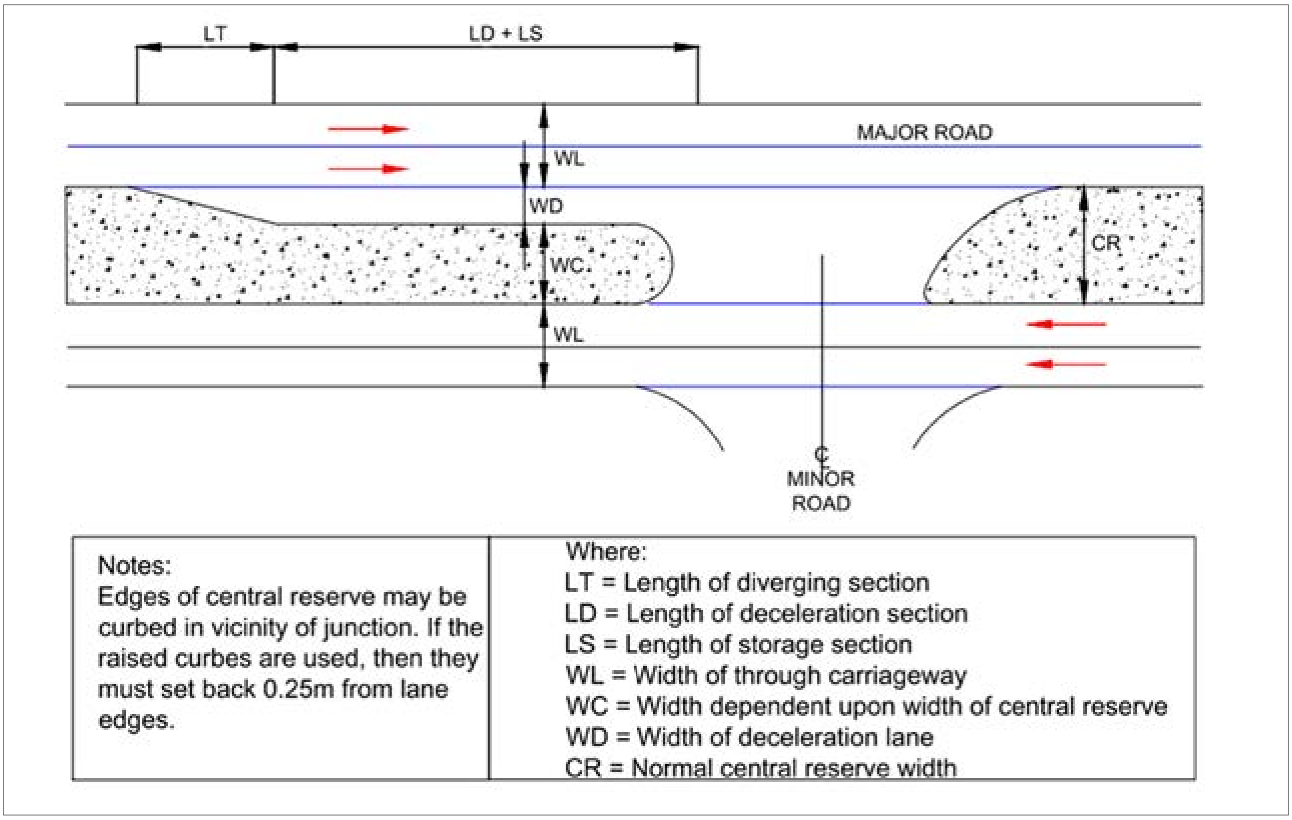


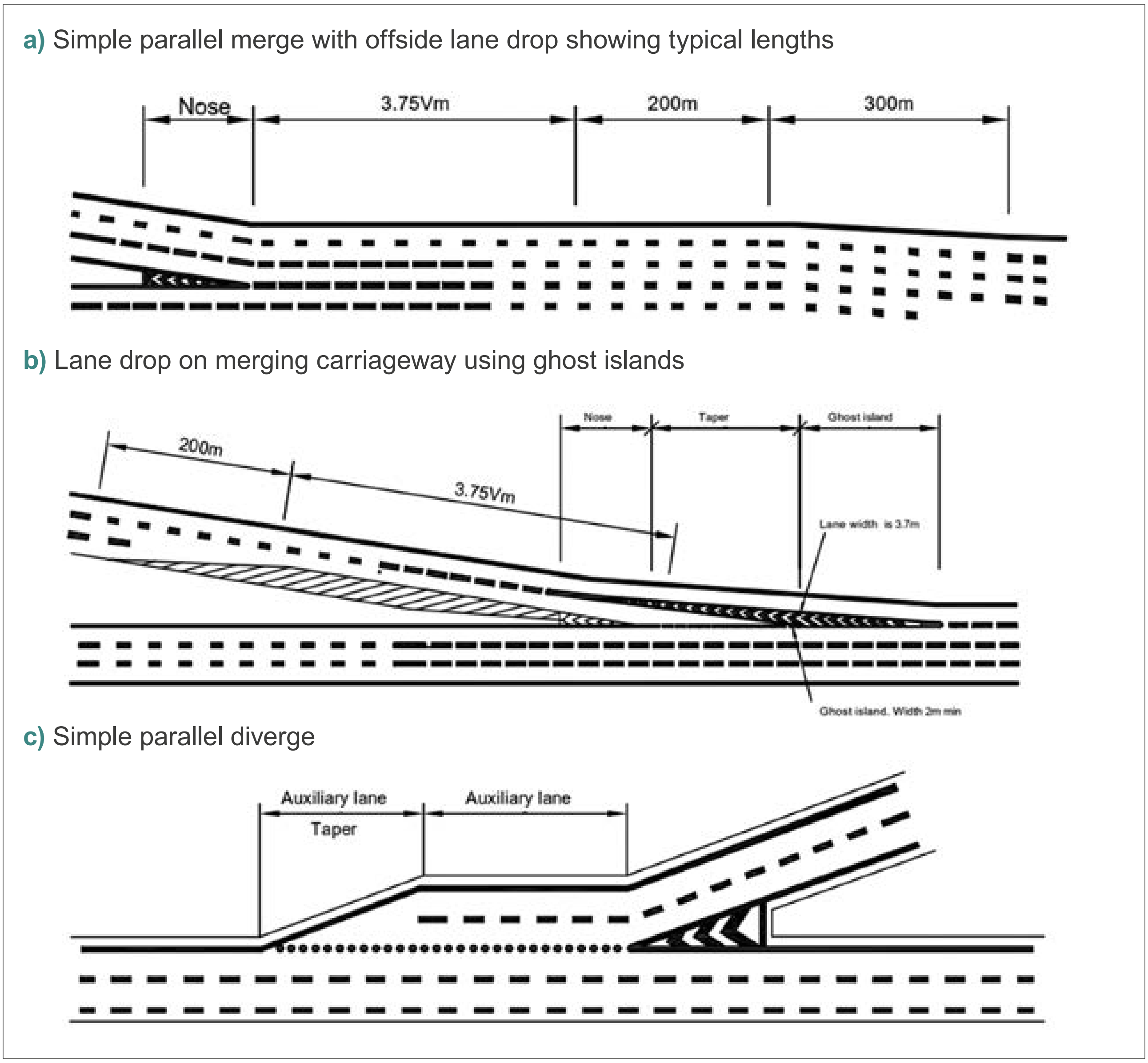
Figure 9.30 Layout for Right Turn Lane, Dual Carriageway



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 9.31 illustrates some typical designs.

Figure 9.31 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:

Case 1: One-lane one-way with no provision for passing stalled vehicles.

Case 2: One-lane one-way with provision for passing stalled vehicles.

Case 3: Two-lane one-way operation.

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Three traffic conditions must also be considered:

Condition A Insufficient trucks in the traffic stream to influence design.

Condition B Sufficient trucks to influence design.

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 9.18. 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.

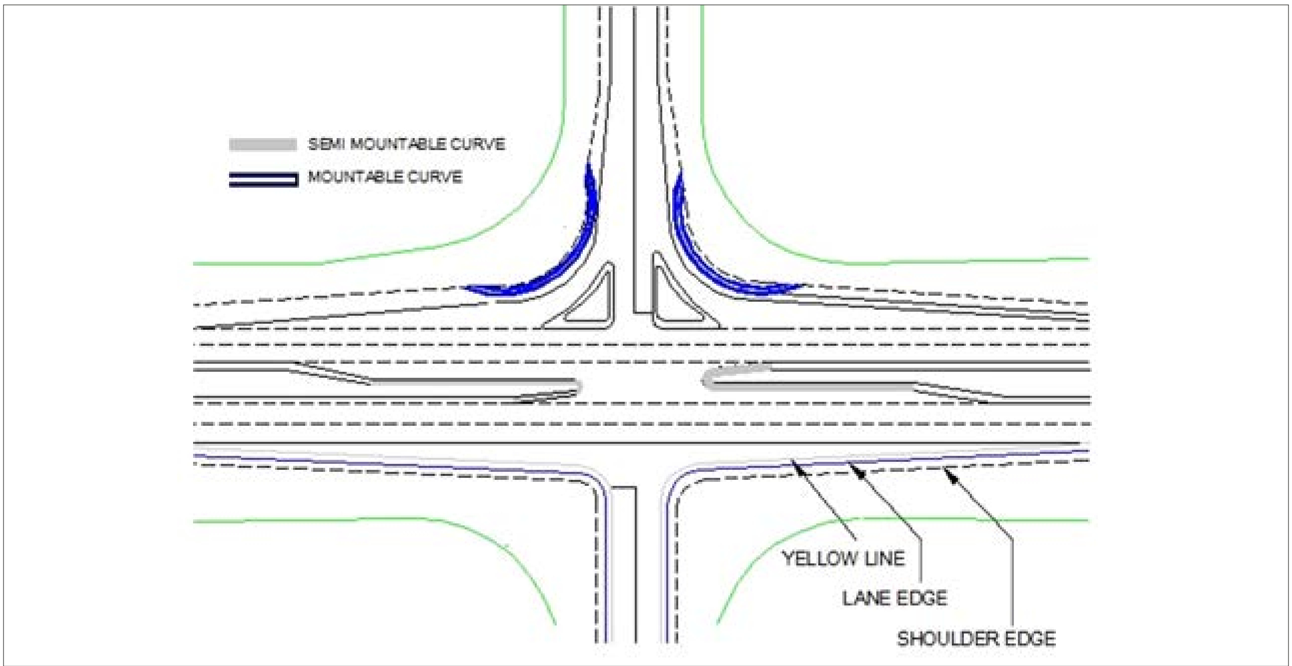
Table 9.18 Turning Roadway Widths

Inner Radius (m)	Case 1			Case 2			Case 3		
	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	7.0	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 9.32 shows the design of a typical crossroad intersection illustrating turning sections, channelisation islands, deceleration lanes, tapers, medians, and mountable kerbs.

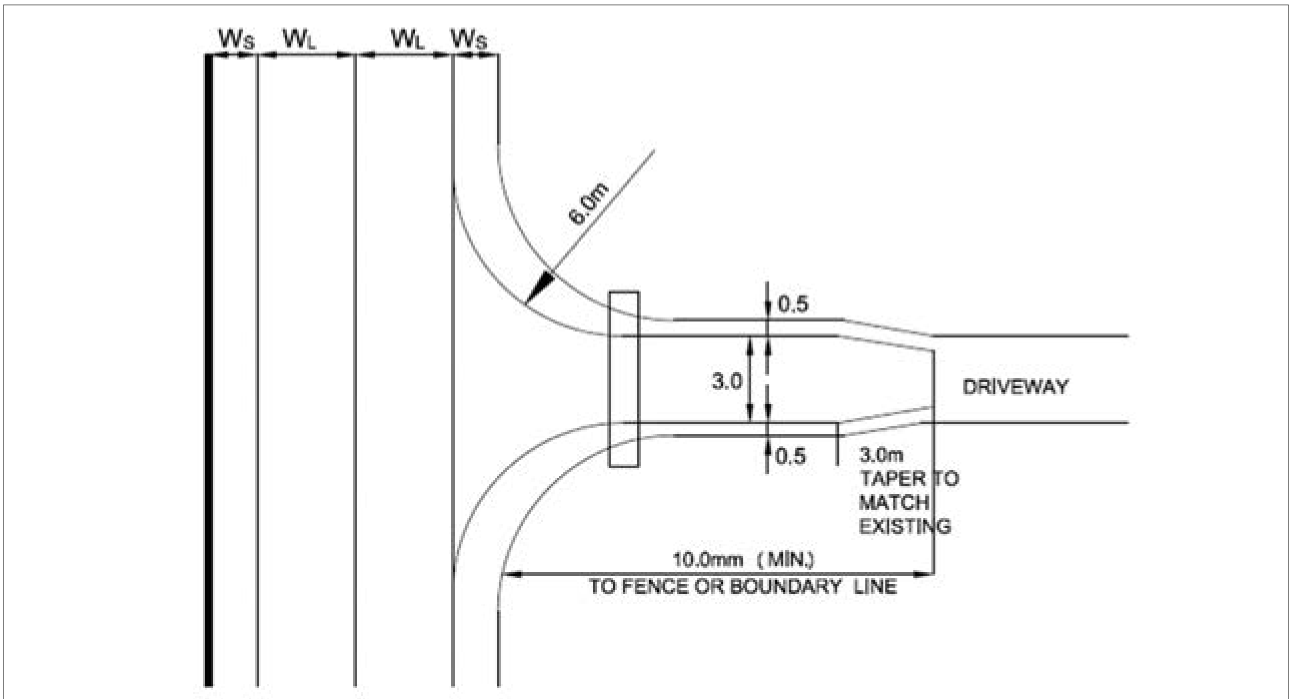
Figure 9.32 Typical Crossroad Intersection Showing all Elements



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 9.33. 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 9.33 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.

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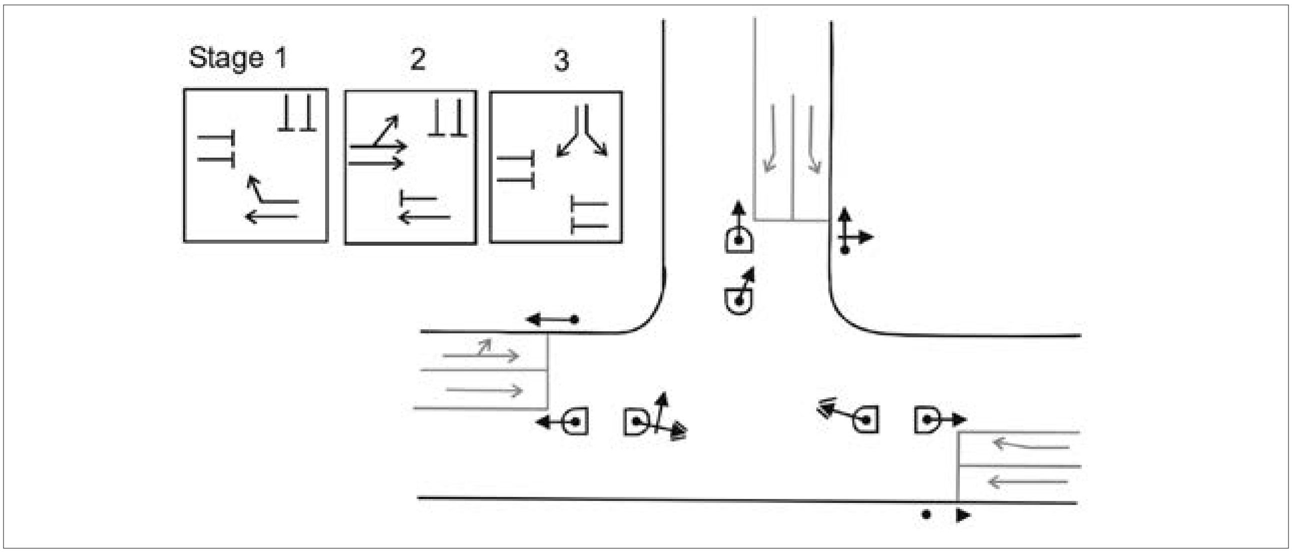
At-Grade Intersections

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 9.34.

Figure 9.34 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/hr and never where the speed exceeds 70 km/hr. Where signals are needed on roads with speeds higher than 50 km/hr 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 9.19 shows the criteria to be met for installing pedestrian crossing signals.

Table 9.19 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

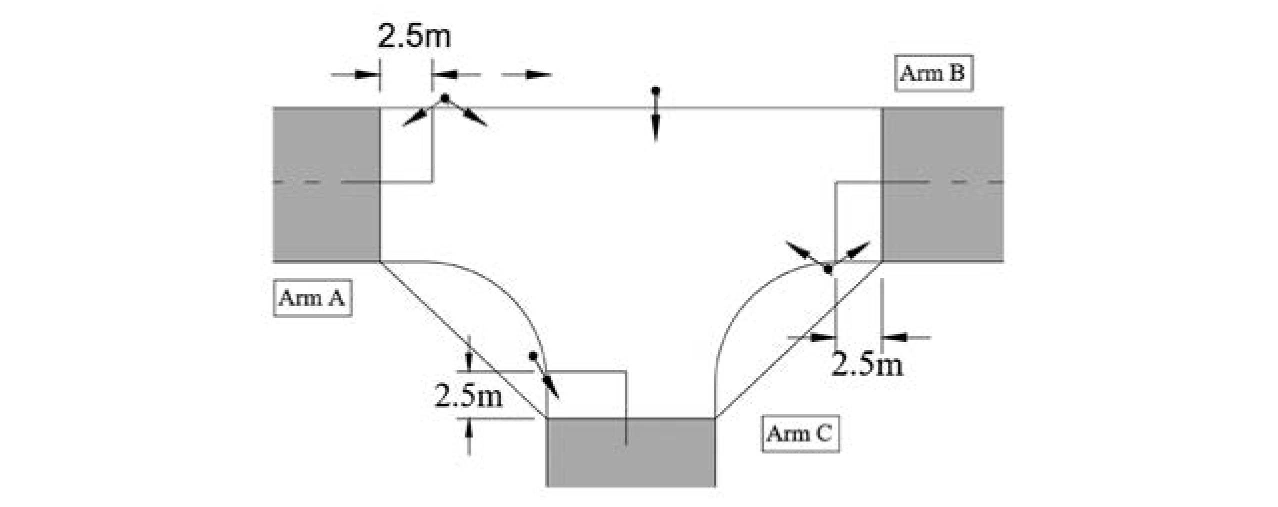
Each traffic lane must have clear vision of at least one primary signal head associated with its movement from the desirable stopping sight distance (70 m at 50 km/hr and 110 m at 70 km/hr). It is also important that the desirable stopping sight distance is available to all traffic entering the queue 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 9.35.

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 should be provided for the minor-road approaches to ensure a minimum level of safety when the signals are out of order.

Figure 9.35 Inter-visibility Zone without Pedestrian Crossing



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At-Grade Intersections

9.12.3 Lane Design

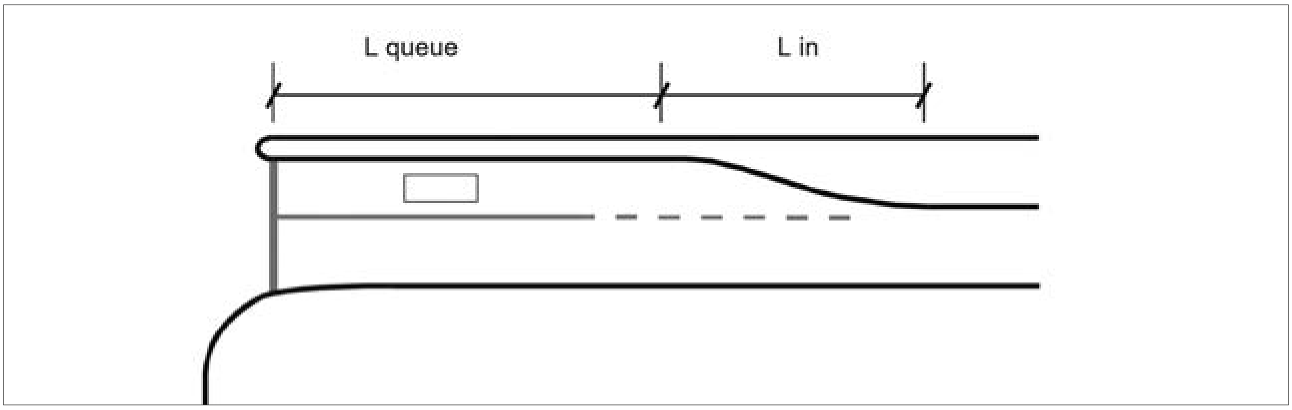
Traffic lanes should normally be 3.0 to 3.5 m wide. Nearside (kerb) lanes that are well-used by cyclists should be widened to 4 m if possible. The lane width may be narrowed to 2.75 m if space is very limited but only if there are few trucks or buses.

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.

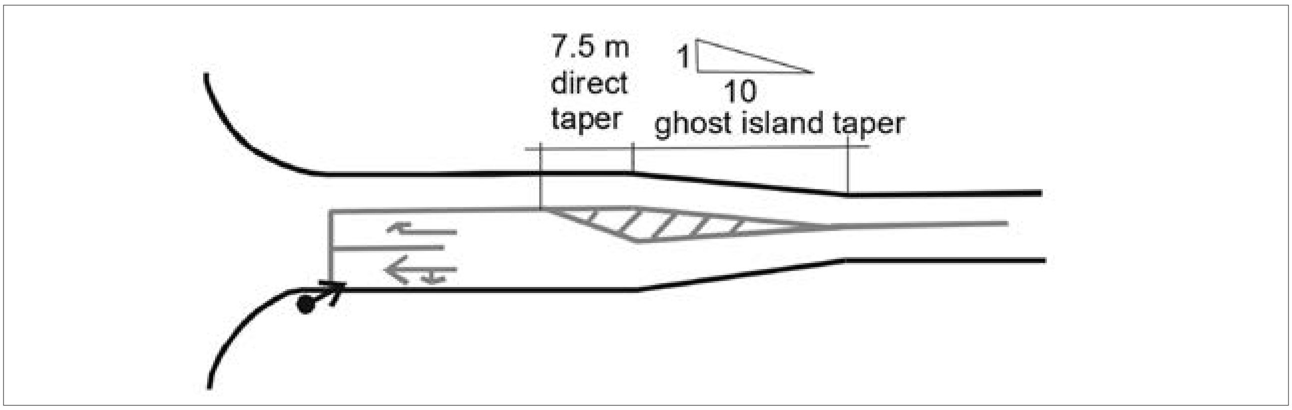
The entry taper, L_{in}, (Figure 9.36) of a kerbed entry lane should be a minimum of 30 m (taper 1:10) to allow a semi-trailer to cope with it. Tapers can be narrowed to 1:5 to allow more queuing space within the same total length.

Figure 9.36 Right Turn Lane Design



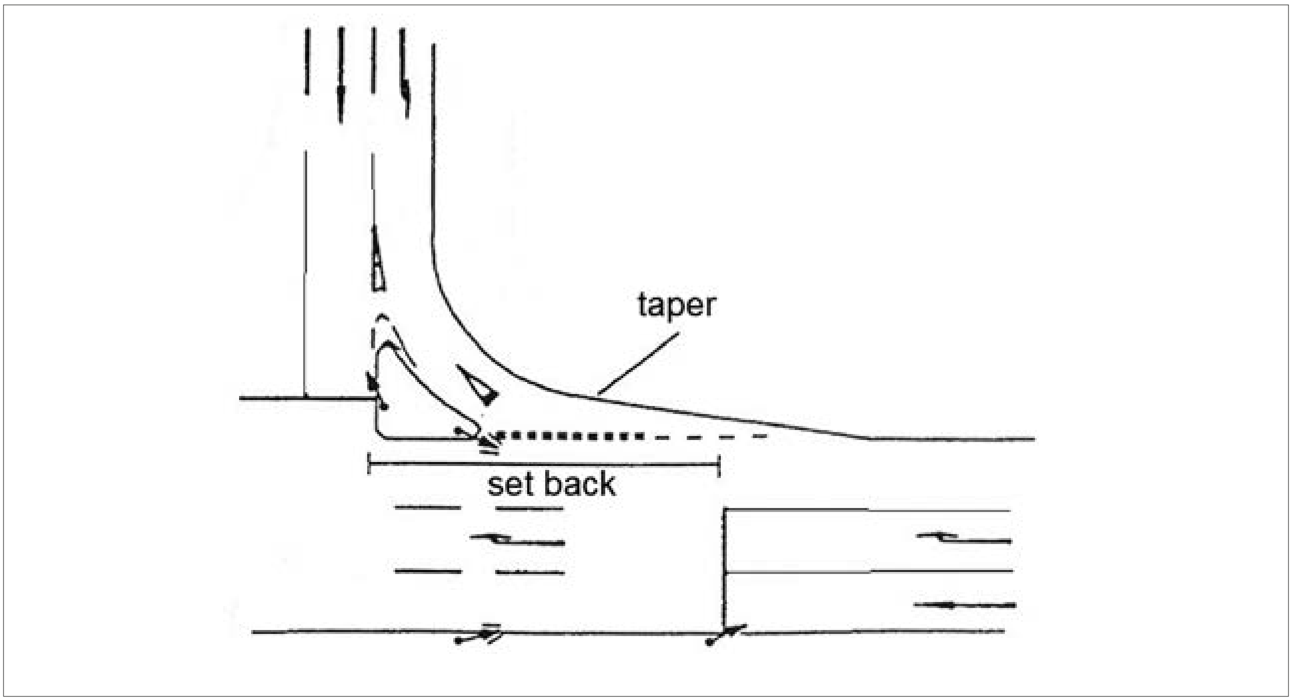
Minimum design measurements for a right turn with a ghost island are shown in Figure 9.37.

Figure 9.37 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 9.38). 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.

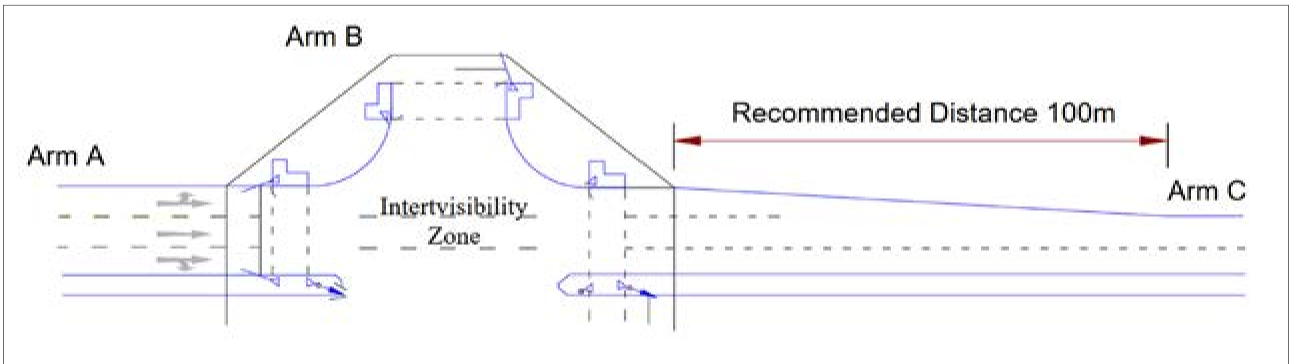
Figure 9.38 Left Turn Filter Lane with Taper to Facilitate Large Vehicles



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 9.39). 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.

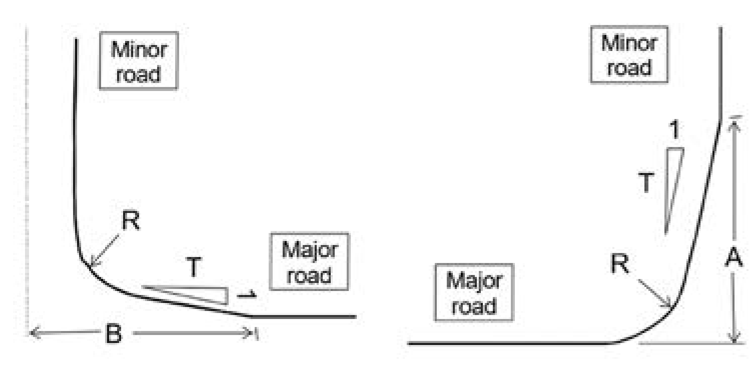
Figure 9.39 Lane Drop Design Principles



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 9.40.

Figure 9.40 Combinations of Tapers and Corner Radii

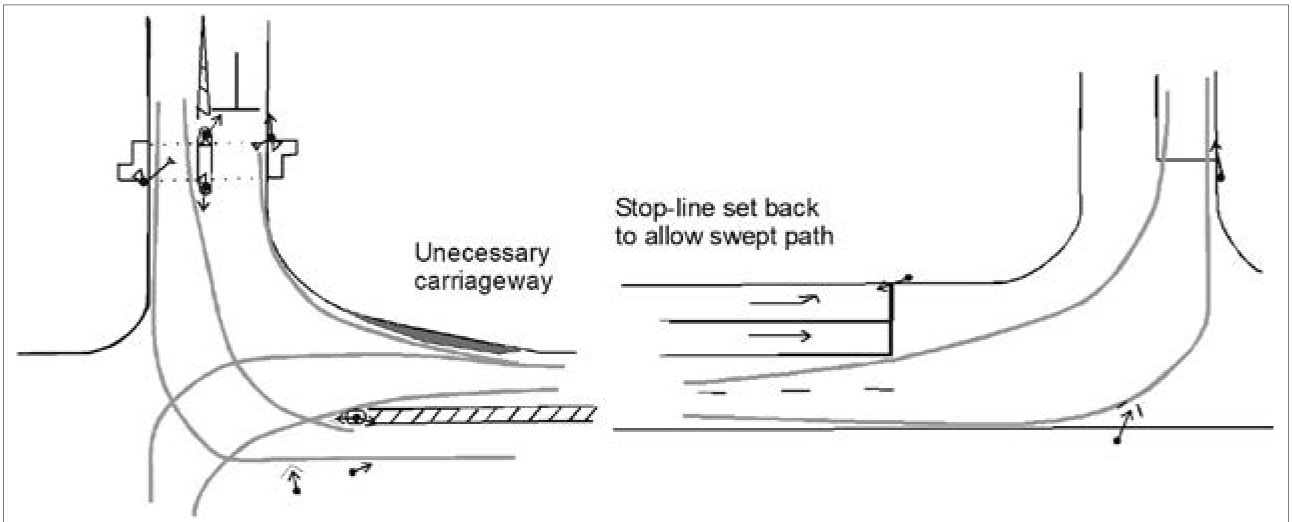


	R	T	A	B
Urban	10	5	30	30
Rural*	15	10	25	25

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 9.41. 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 9.41 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 9.35) (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 9.36.

Figure 9.42 Signal Location

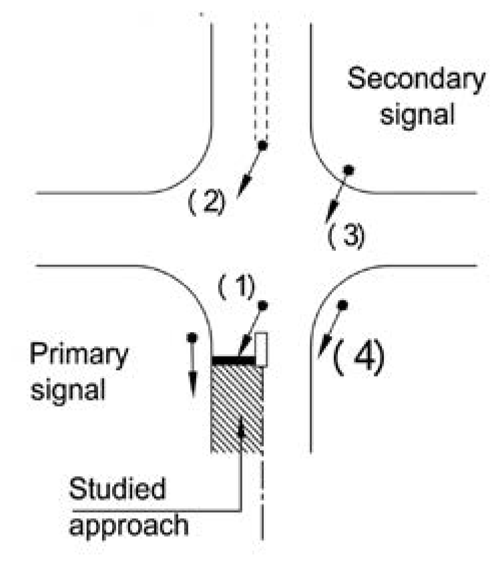
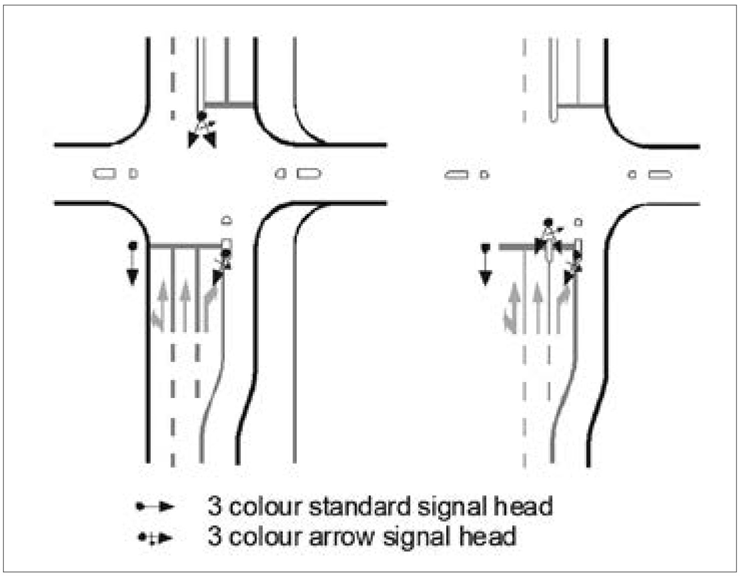


Figure 9.43 Alternative Signal Locations for Right Turn Lanes



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.

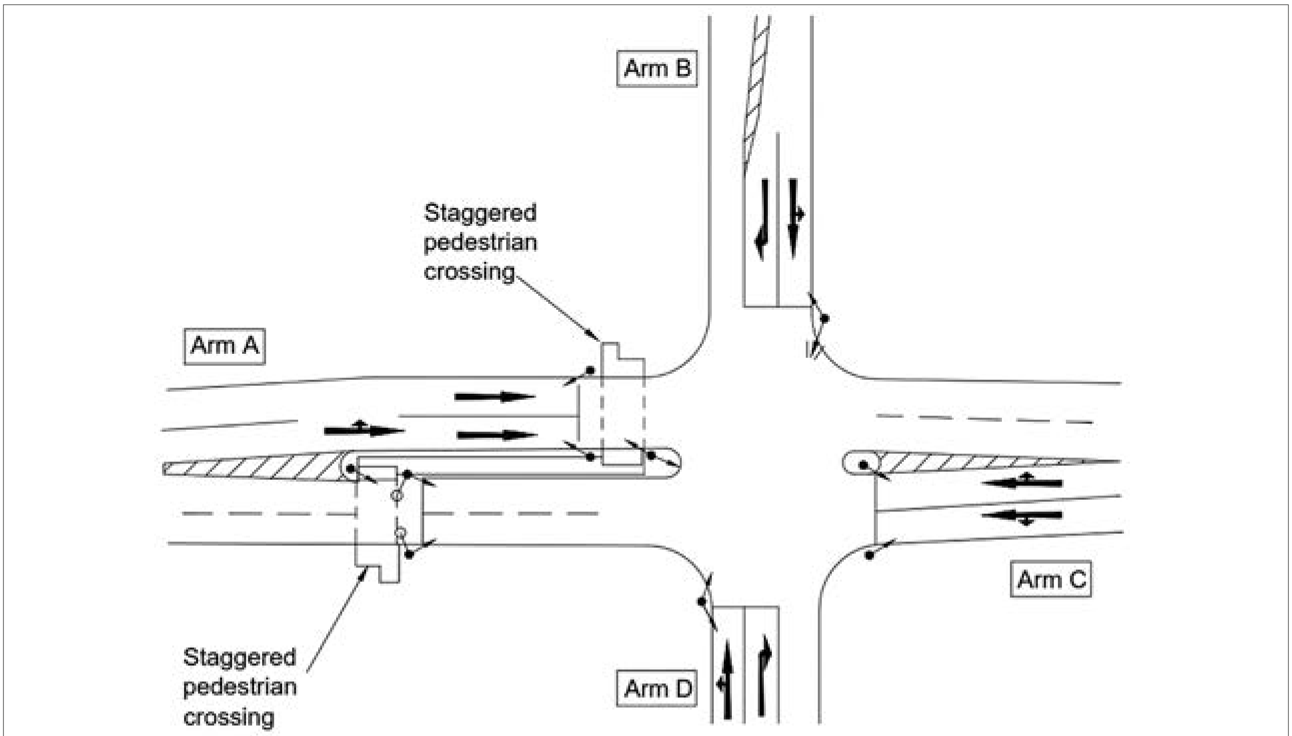
9.12.6 Pedestrian and Cyclist Facilities

Pedestrian crossings should be perpendicular to the edge of the carriageway to assist inter-visibility and to benefit visually impaired people. The Walkway 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 9.44.

Figure 9.44 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 walkways, refuges, and crossings, been provided for pedestrians?
16. Does 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 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, approximately by 75%, and should crashes occur, they are not as severe as crashes on roads with traffic travelling at normal speeds. The key elements of a roundabout as shown in Figure 10.1 are:

i. Entries and exits.

ii. Splitter islands

iii. The circulatory roadway.

iv. The central island.

v. Sight distances.

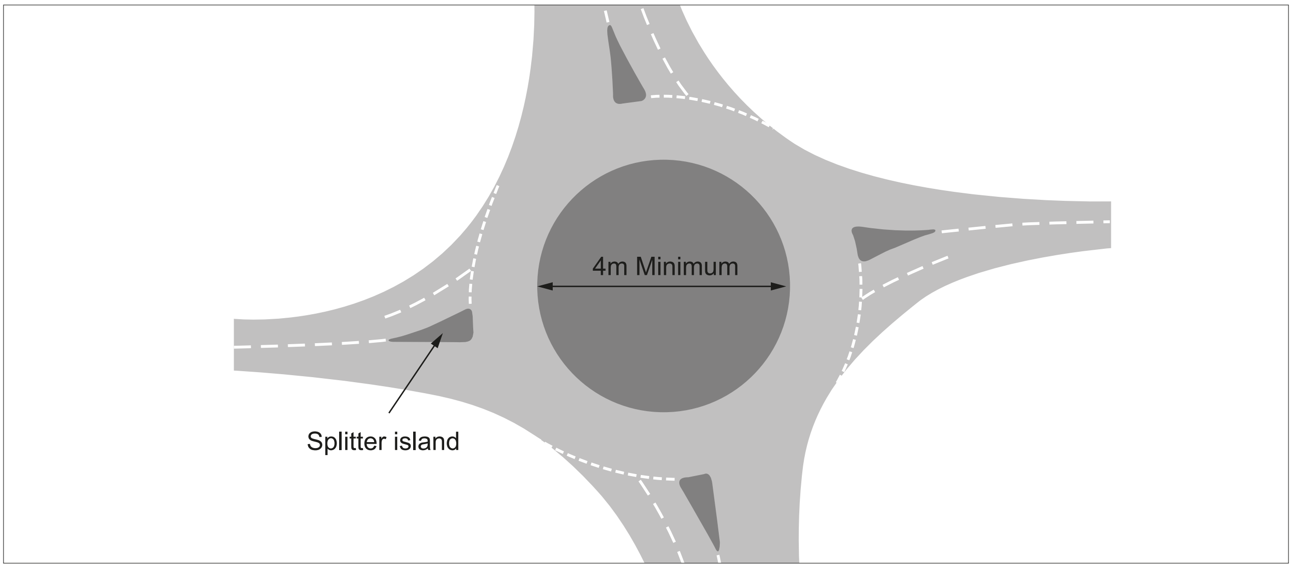
The main types of roundabouts are normal (in size) roundabouts, multi-lane, compact mini, signalised, and grade separated.

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.

Figure 10.1 Basic Roundabout Showing Key Features



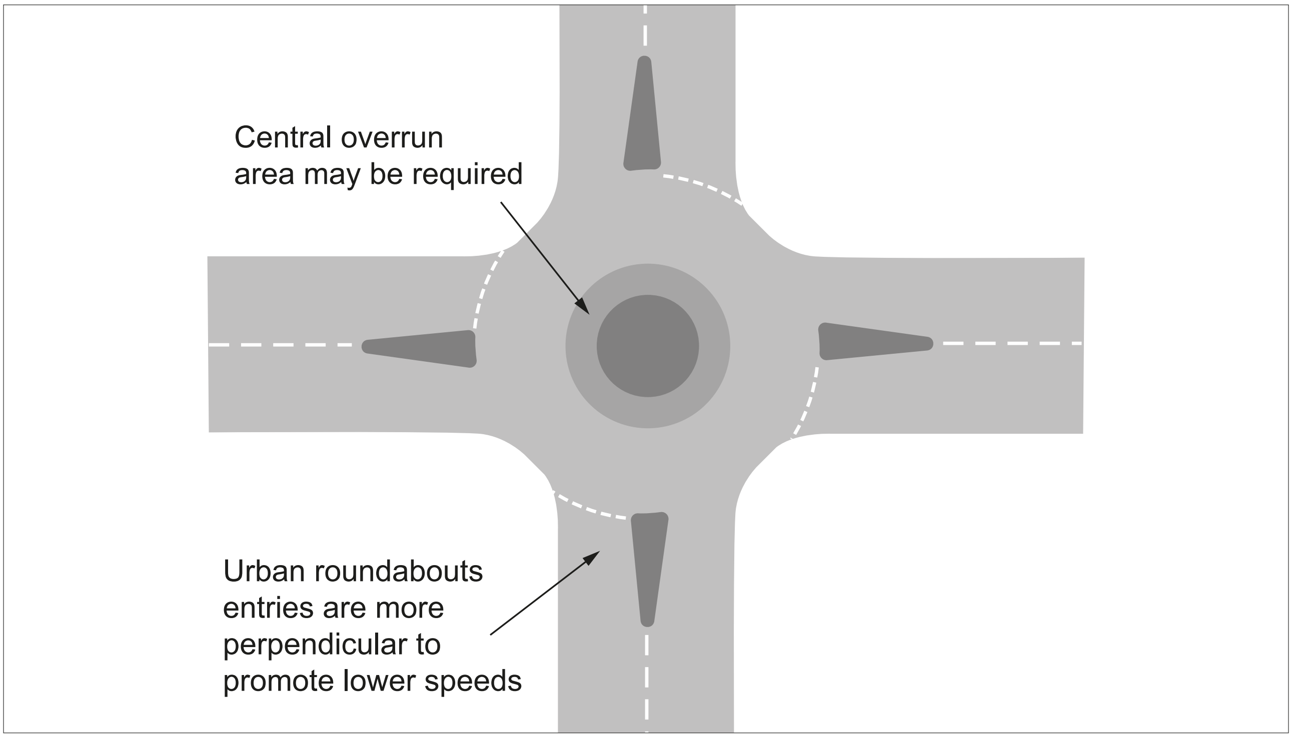
10.1.3 Multi-lane Roundabouts

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 10.2 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 10.19.

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 NMT adequate facilities.

10.3 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.

10.4 Design Requirements

10.4.1 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. All roundabouts must be designed with some entry deflection.

Entry width and sharpness of flare are the most important determinants of capacity.

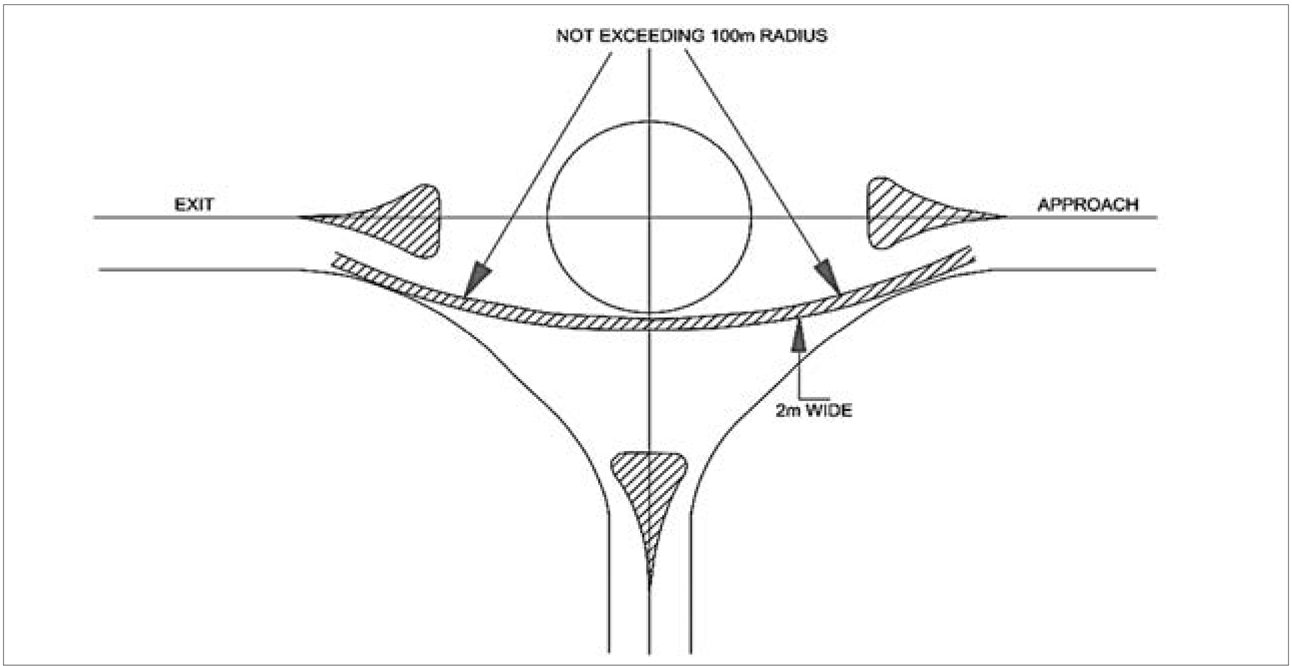
Splitter islands should also be used to help achieve adequate speed reductions and to:

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 10.12).

Figure 10.3 Vehicle Path through Roundabout



1 2 3 4 5 6 7 8 9 10 Roundabouts

10.4.2 Key Safety 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.

10.4.3 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.

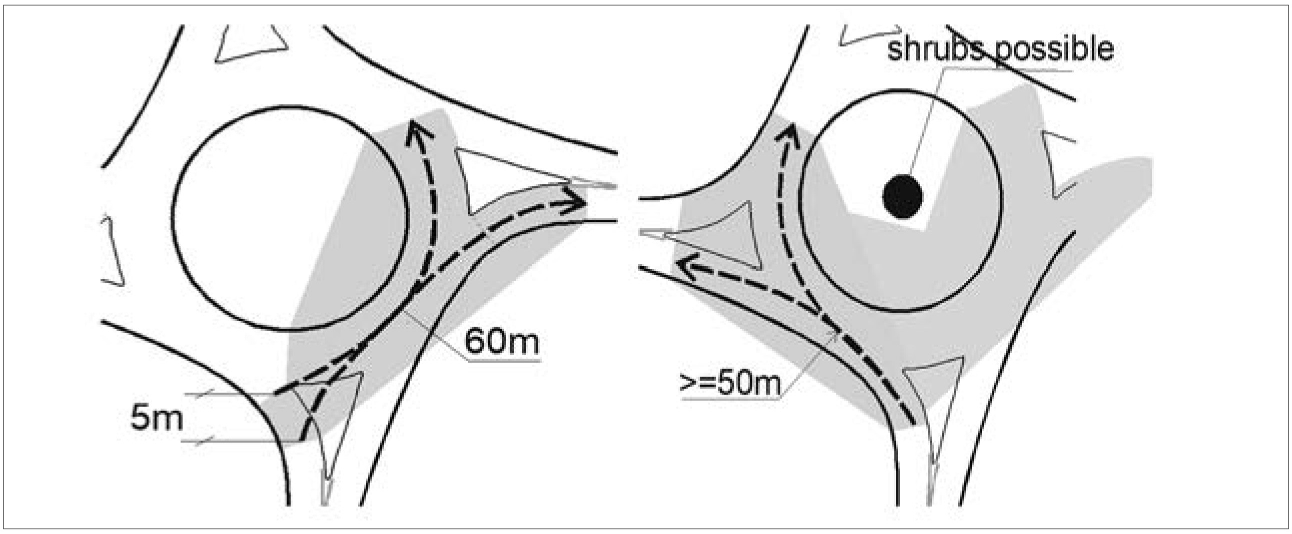
In five-arm intersections, the space for the extra connection can be created by making the central island elliptical or by increasing the radius of the central island to at least 20 m. However, elliptical central islands can be confusing.

10.4.4 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 sharp 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 15 m 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 Towards 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

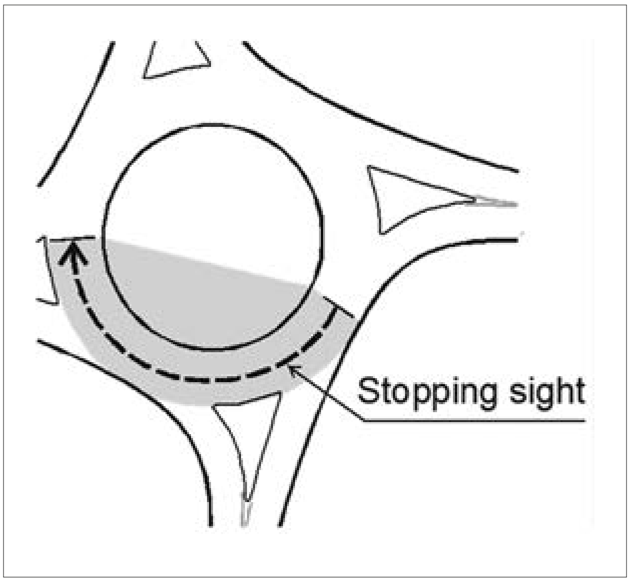
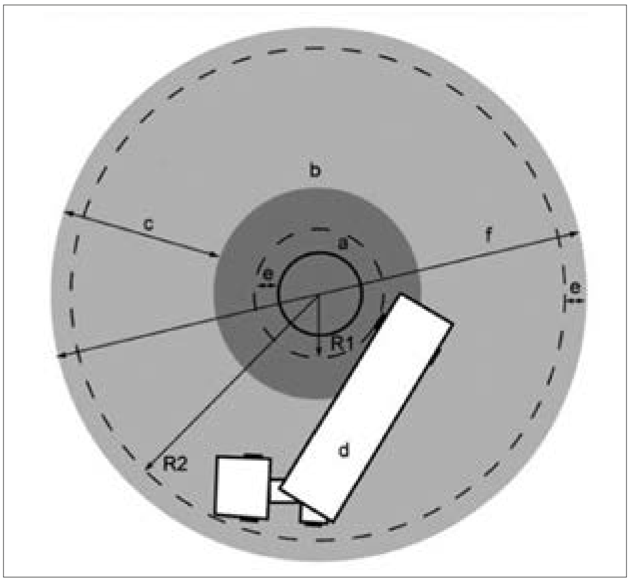


Figure 10.6 Nomenclature for Dimensions of Roundabouts Showing Design Vehicle.



The labels are defined for Figure 10.6 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.

10.5 Dimensions of Roundabouts

The dimensions of roundabouts are defined by the radii and widths shown in Figure 10.6 and Table 10.1.

Table 10.1 Roundabouts Dimensions

Central Island Diameter (m)	R_1	R_2	**Minimum ICD** (m)
4	3	13.0	28.0
6	4	13.4	28.8
8	5	13.9	29.8
10	6	14.4	30.8
12	7	15.0	32.0
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 100 m. 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 28 m and 36 m, 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, 12 m long coach, 15 m bus, 17.9 m, 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.

10.5.1 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 10 m and 25 m. 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.

10.5.2 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.

Figure 10.7 Minimum Width of Circulating Carriageway – One Lane

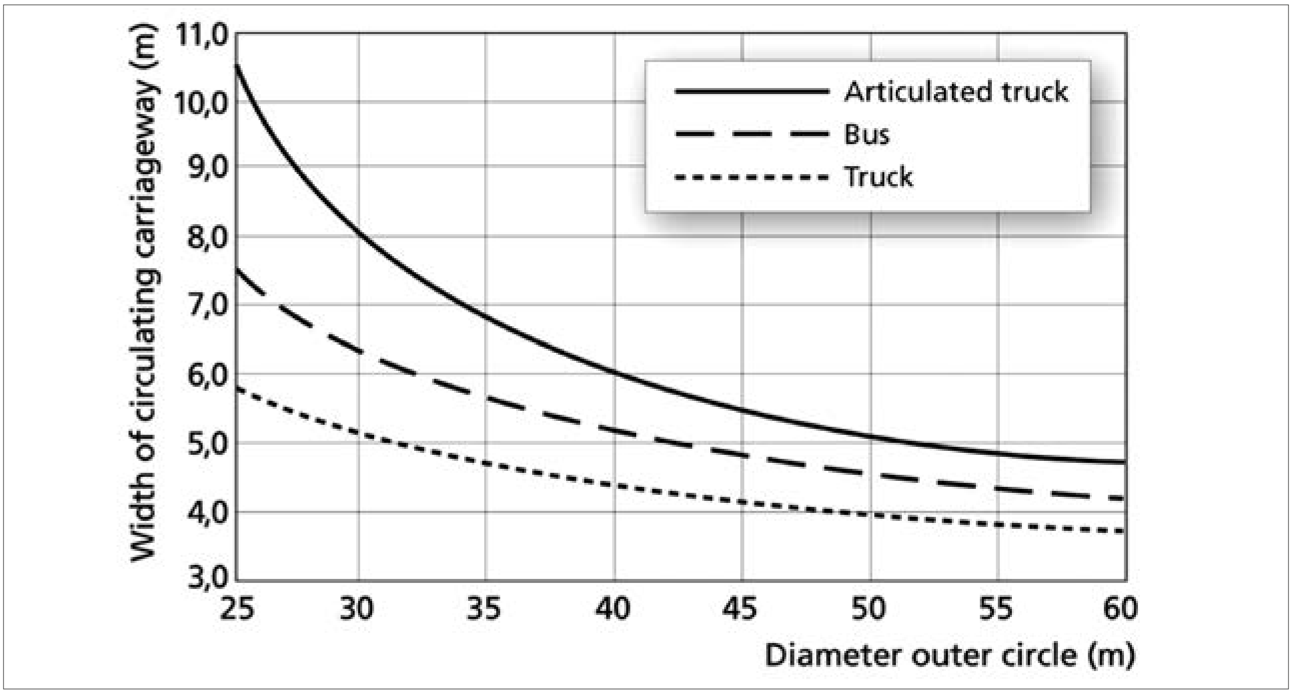
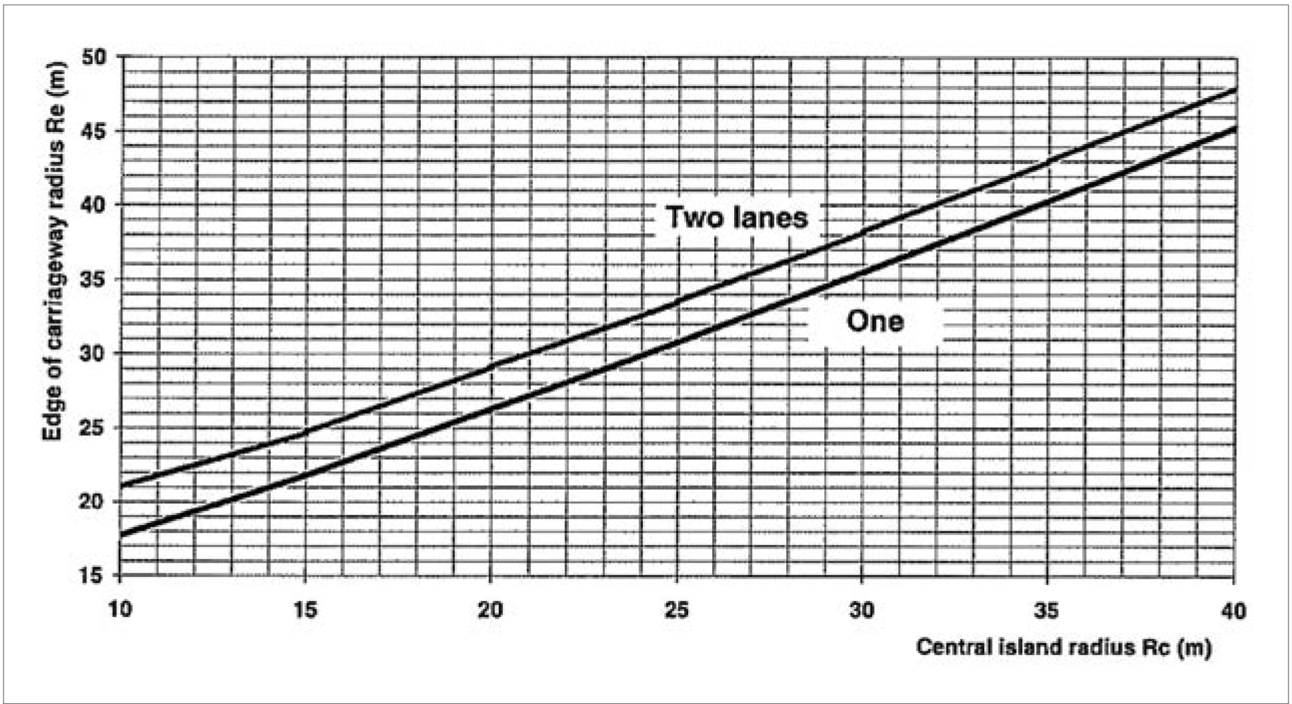


Figure 10.8 Radius of Central Island and Circulating Carriageway-Normal Roundabouts



1 2 3 4 5 6 7 8 9 10 Roundabouts

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.

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.

10.5.2.1 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.

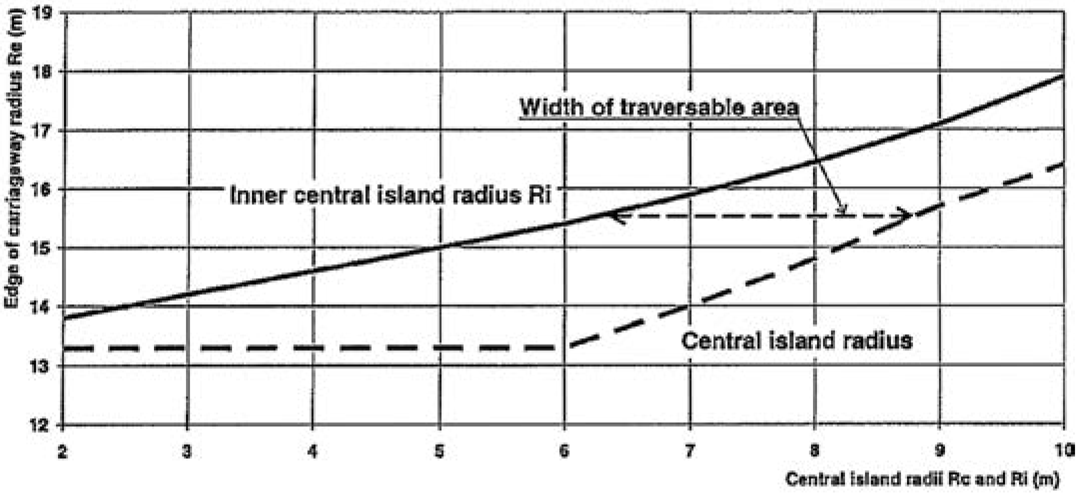
The inscribed circle diameter, the width of the circulatory carriageway and the central island diameter are interdependent. Once any two of these are established, the remaining measurement is determined automatically.

10.5.3 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 40 mm 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

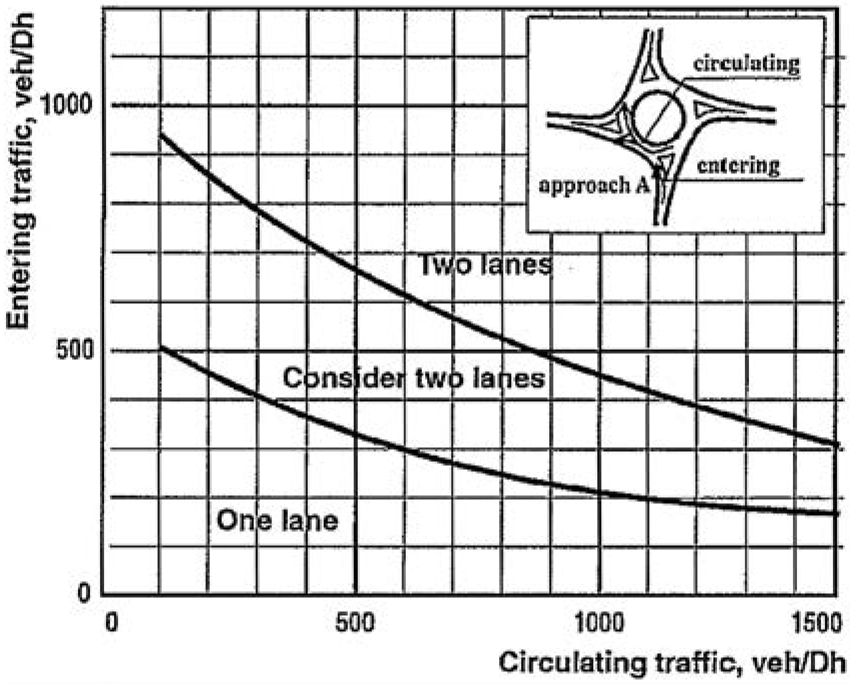


10.6 Entries

10.6.1 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.

10.6.2 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.

10.6.3 Entry Design

Several variables need to be considered in selecting an entry design which is safe and has adequate capacity. These variables are:

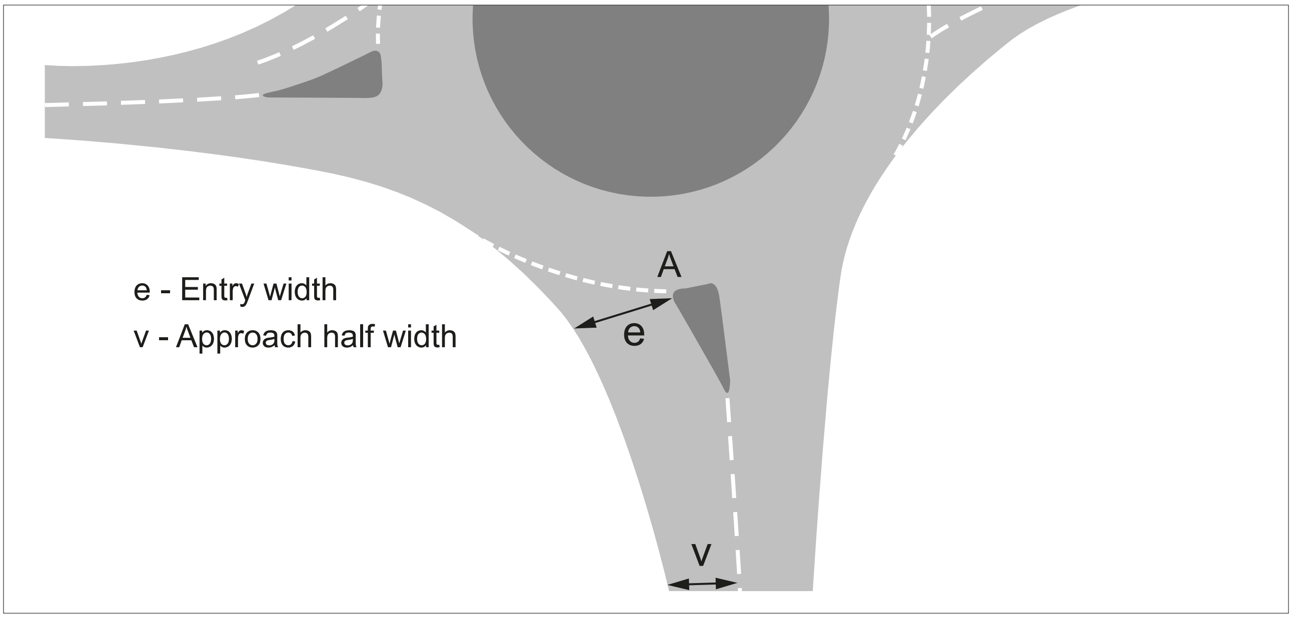
1. Approach half width;
2. Entry width;
3. Entry flaring;
4. Entry angle.

On multi-lane 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.

10.6.3.1 Approach Half Width

The approach half width, ‘v’ in Figure 10.11 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 10.11 Approach Half Width and Entry Width



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10.6.3.2 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 10.11). 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 3 m 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 multi-lane 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 2.5 m.

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.

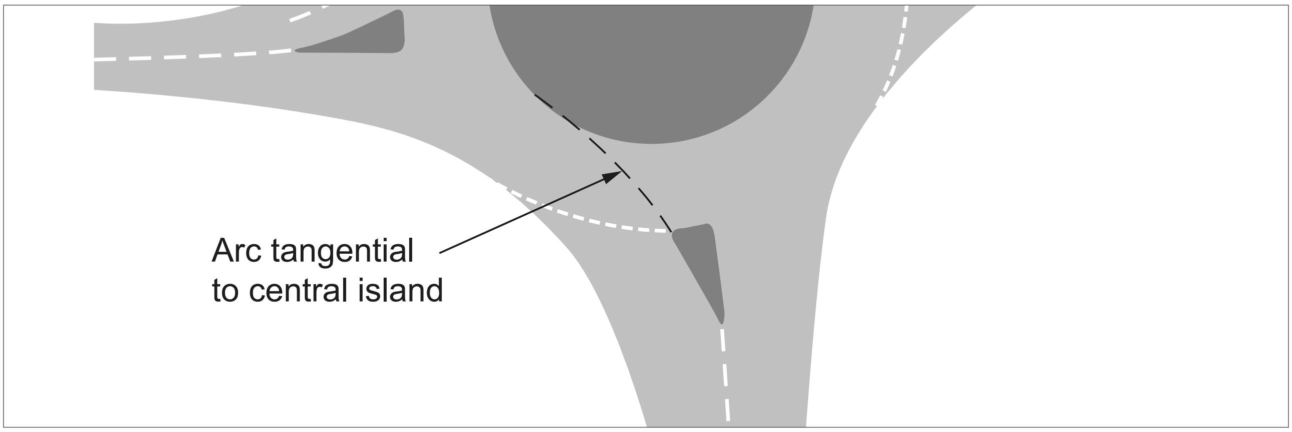
Table 10.2 Entry Widths

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

10.6.3.3 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 10.12 to reduce the likelihood of vehicle paths overlapping.

Figure 10.12 Example Showing an Arc Projected Forwards from the Splitter Island and Tangential to the Central Island

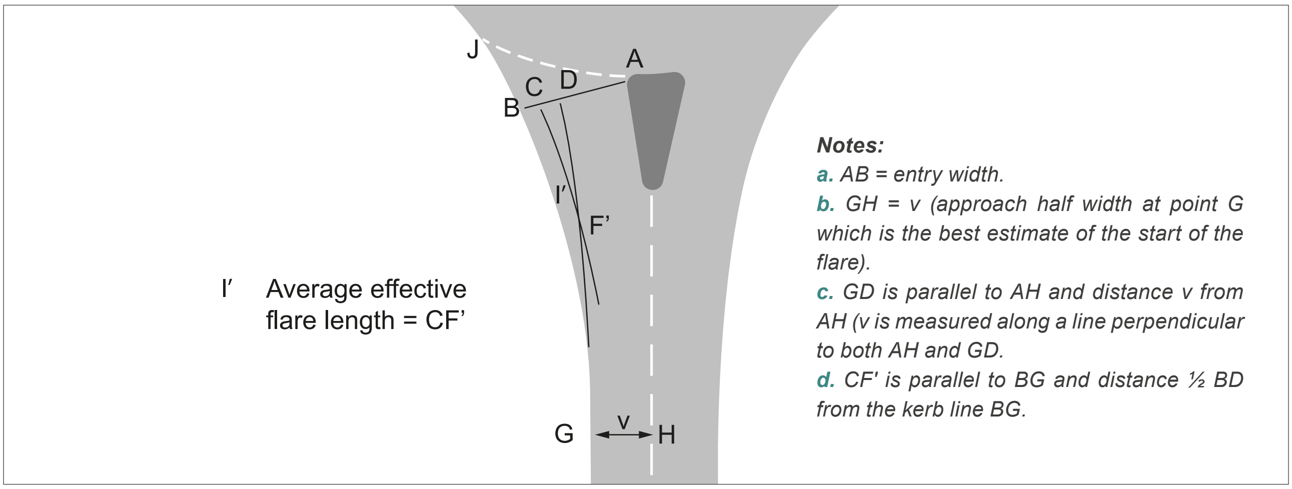


10.6.3.4 Entry Flaring

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 10.13.

Figure 10.13 Average Effective Flare Length



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 5 m in urban areas and 25 m in rural areas is desirable.

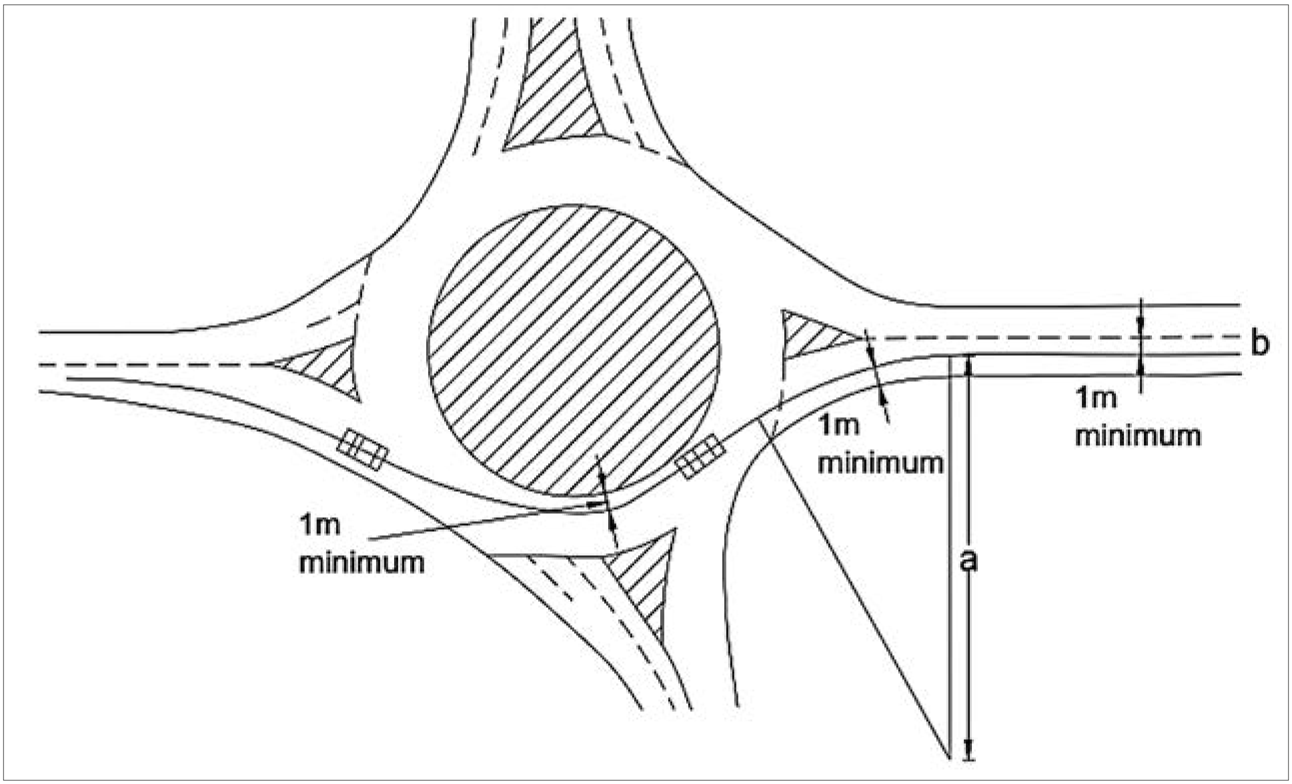
The entry width and the flare length are related. The capacity of a wide entry combined with a short flare can be similar to that of a narrow entry combined with a long flare.

10.6.3.5 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 10.14). 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 km/hr. At all other roundabout types, the entry path radius must not exceed 100 m.

Figure 10.14 Determination of Entry Path Radius for Ahead Movement at a 4-arm Roundabout



Notes

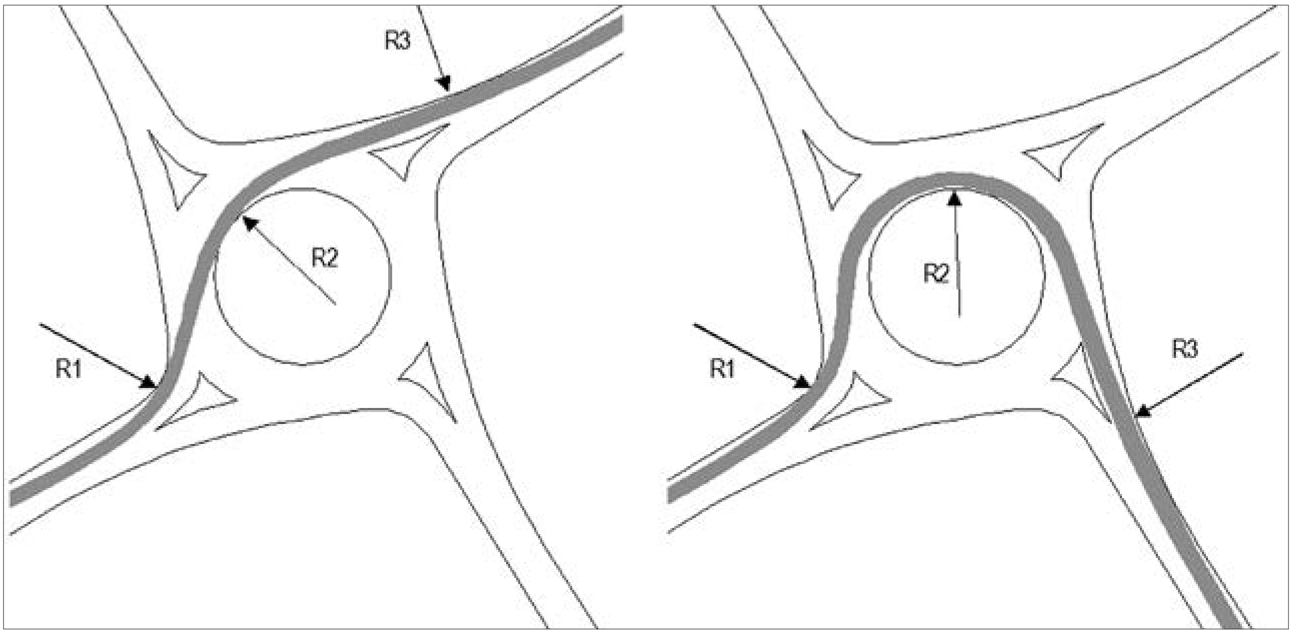
a. Entry path radius should be measured over the smallest best fit circular curve over a distance of 25 m occurring along the approach entry path in the vicinity of the give way line, but not more than 50 m in advance of it.

b. Commencement point 50 m from the give way line and at least 1m from the nearside kerb or centre line (or edge of central reserve).

10.7 Driving Paths

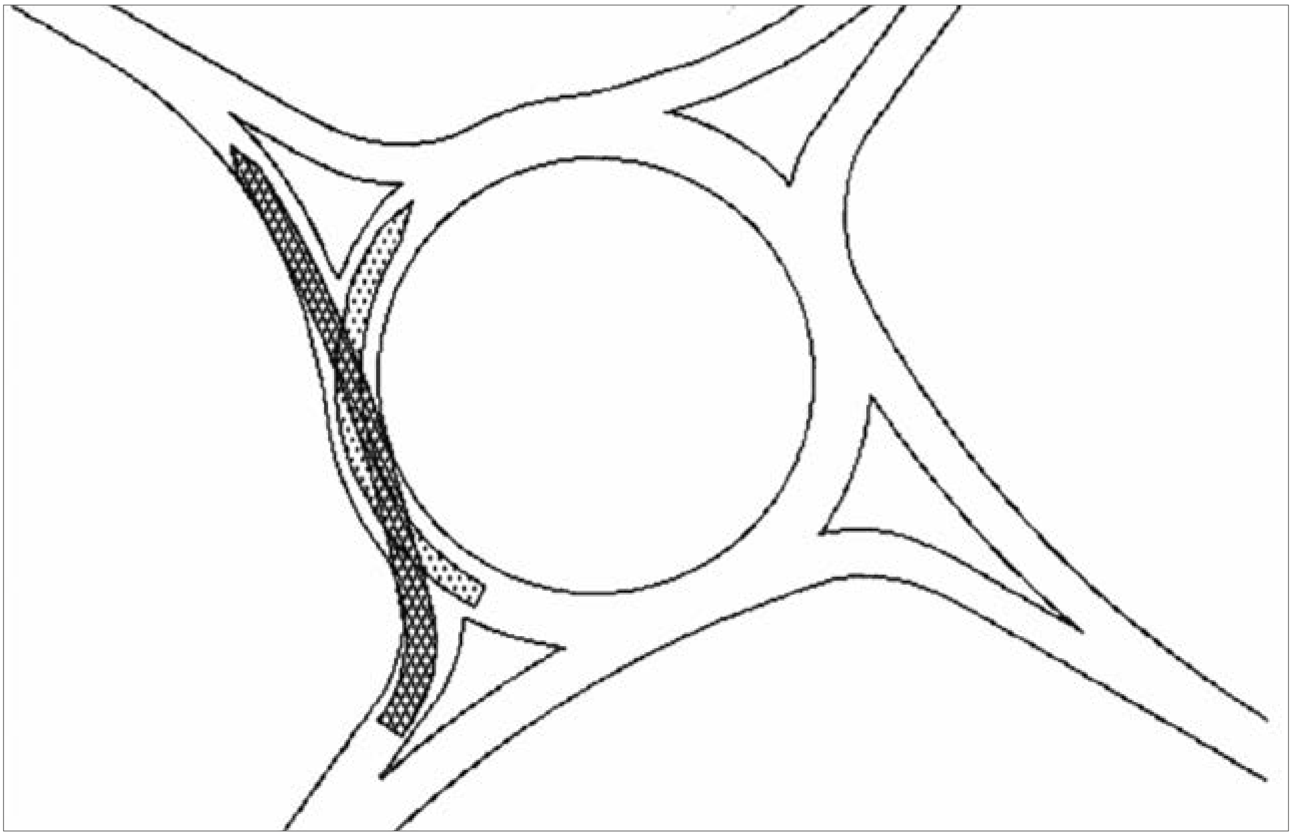
The alignment of the connecting roads can make it necessary to adjust the exit and entry curve radii. If larger radii than normal are used, the designer must check that all possible 2 m wide driving paths for passenger cars fulfil the requirement R1 ≤ R2 ≤ R3 ≤ 100 m to achieve speed control.

Figure 10.15 Driving Paths for Passenger Cars



It is preferable to avoid reverse curvature between the entry and the following exit (Figure 10.16). 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 10.16 Alignment Between Entry and Exit



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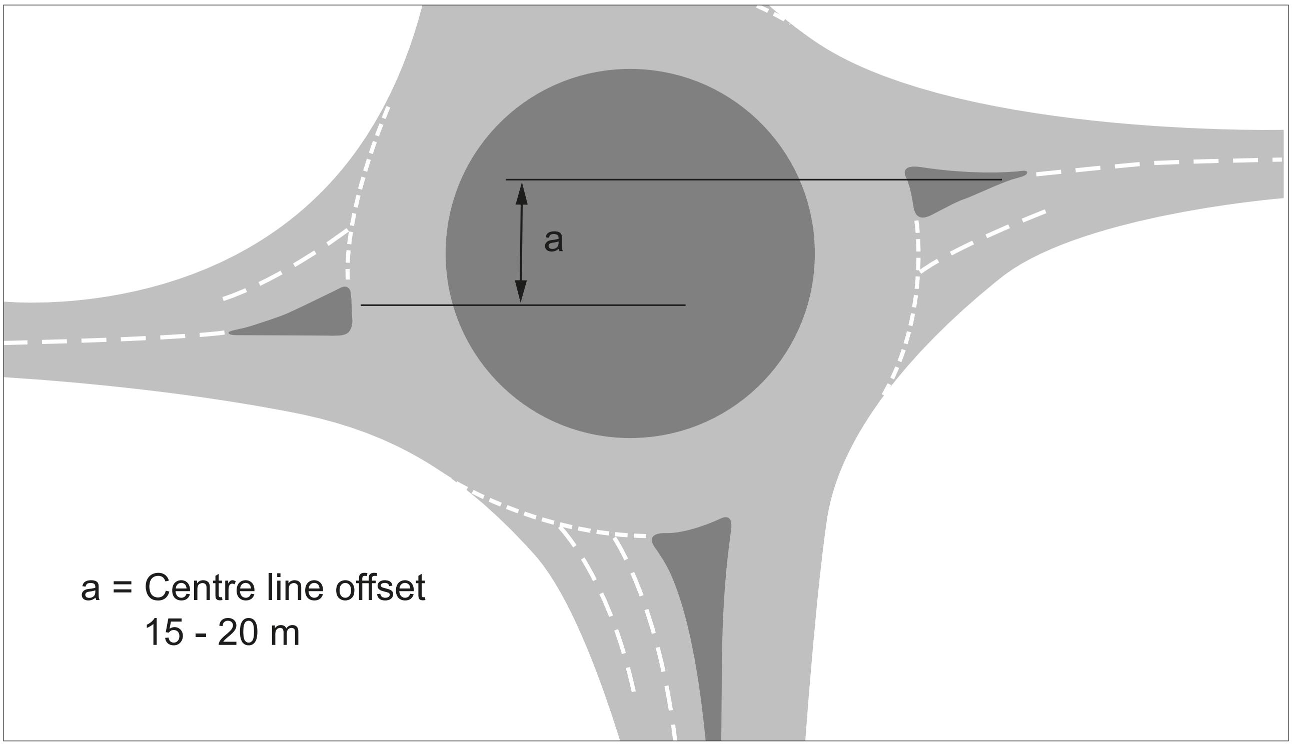
A method for creating entry deflection at a Normal Roundabout is to stagger the arms as shown in Figure 10.17. This will:

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 10.17 Staggering of East-West Arms to Increase Deflection



10.8 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.

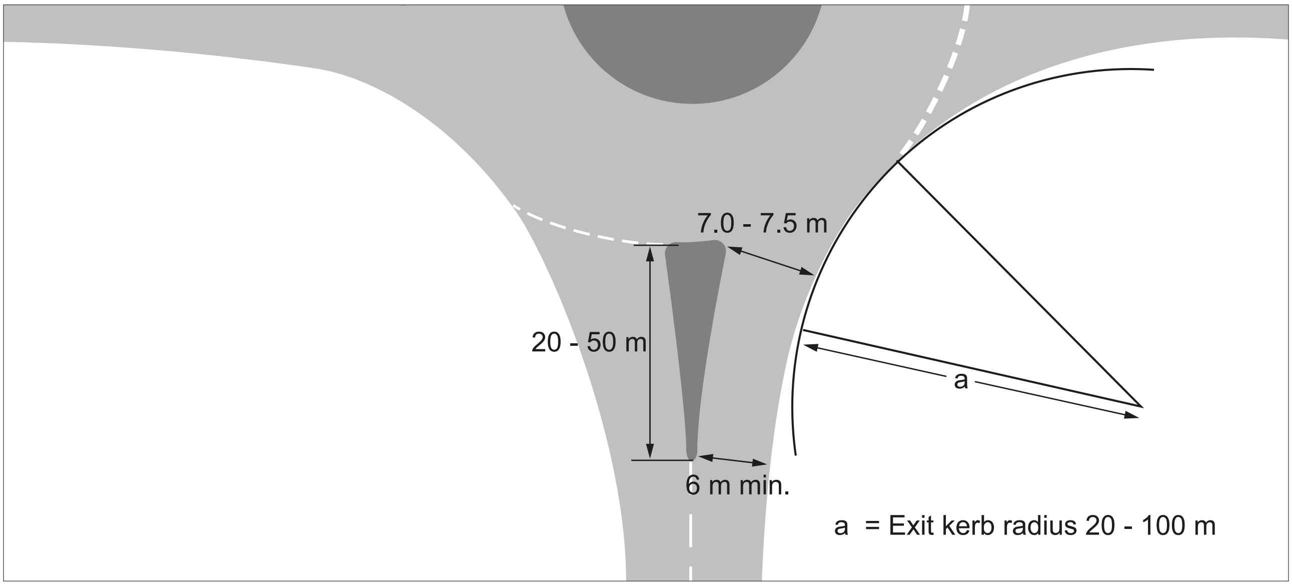
For example, at a normal roundabout, if the downstream link is a single carriageway road with a long splitter island, the exit width should be between 7 m and 7.5 m and the exit should taper down to a minimum of 6 m (Figure 10.18), allowing traffic to pass a broken down vehicle. If the link is an all-purpose 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.

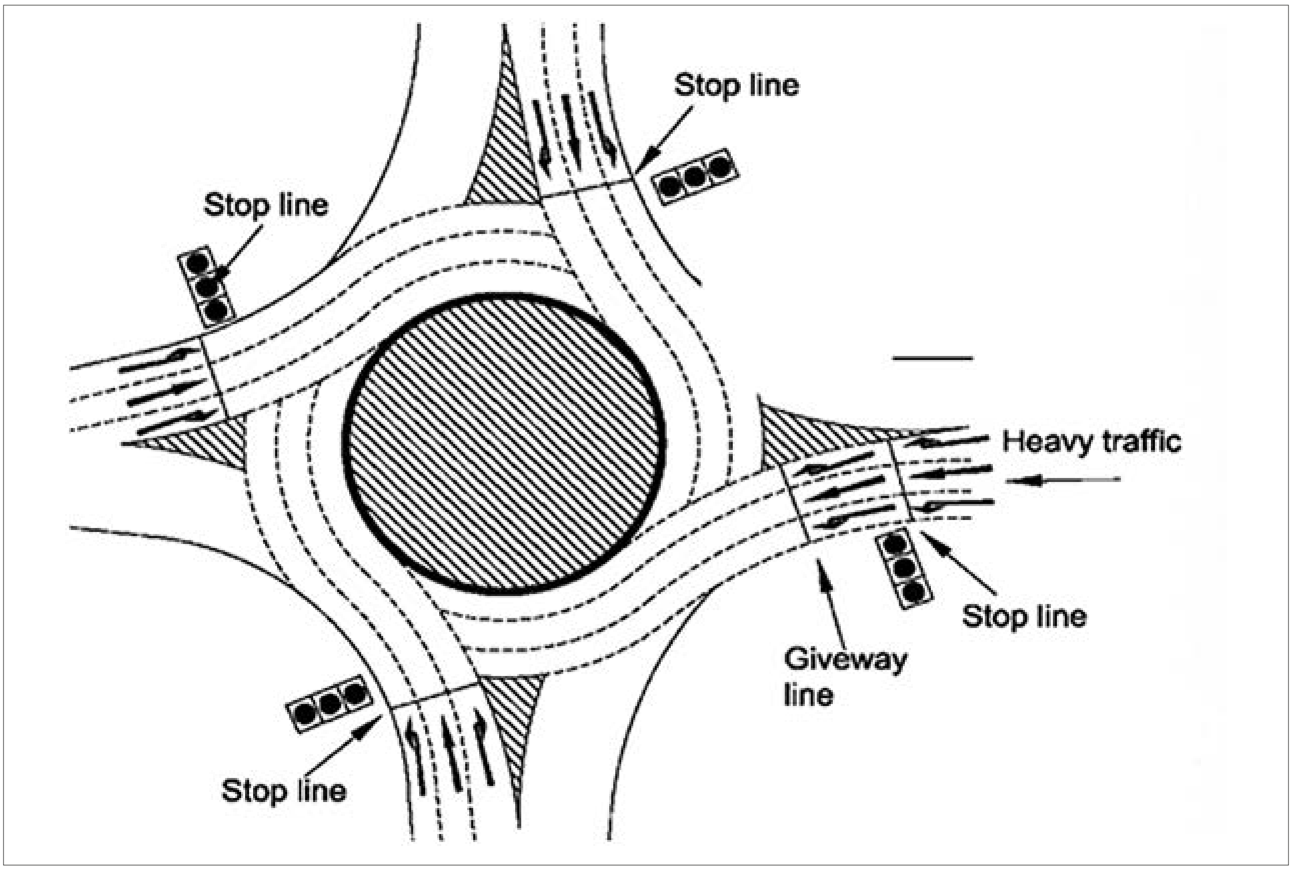
Figure 10.18 Single Carriageway Exit with a Long Splitter Island



10.9 Signalised Roundabouts

For large flows and therefore large roundabouts, traffic control will be required. Figure 10.19 is an example of such a roundabout.

Figure 10.19 Example of a Signalised Roundabout



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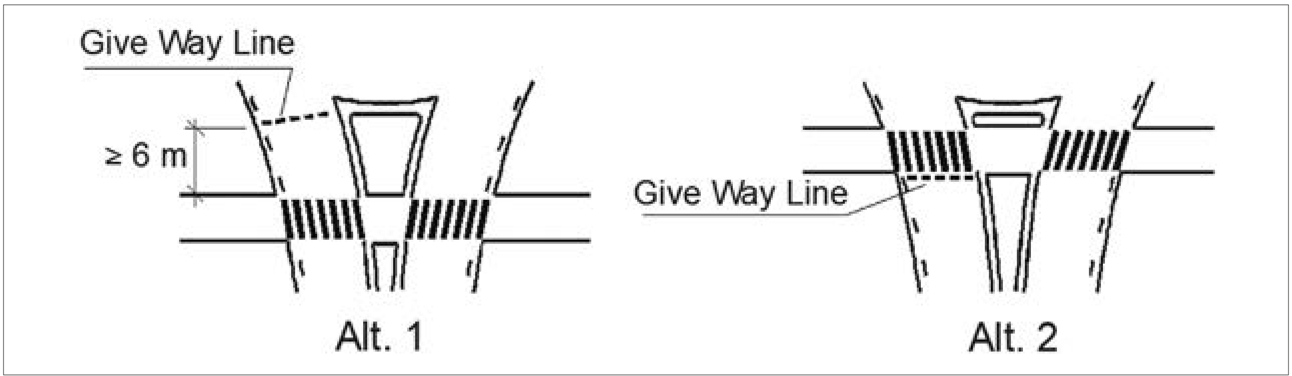
10.10 Pedestrian and Cycle Crossings

Pedestrian/cycle crossings are normally placed according to one of the two alternatives shown in Figure 10.20. They should never be located within the roundabout.

Figure 10.20 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 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 an expressway and any other road is that it is subject to rigid control of access. Entrances and exits to and from an expressway may take place only at specified points, typically remote from each other, and then only at very flat angles of merging and diverging. An expressway 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 expressways 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 UK's 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 expressway 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 signages.

The circumstances in which the use of a grade-separated interchange is warranted are usually as follows:

1. Where roads cross or connect to expressways.
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.

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Grade-separated Interchanges

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 expressway should then take place clear of the expressway itself. Thus, drivers are required to make two separate decisions. First, to leave the expressway 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 expressway are a disturbance 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 expressway 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 expressway or to depart from it.
3. Placing the expressway 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).

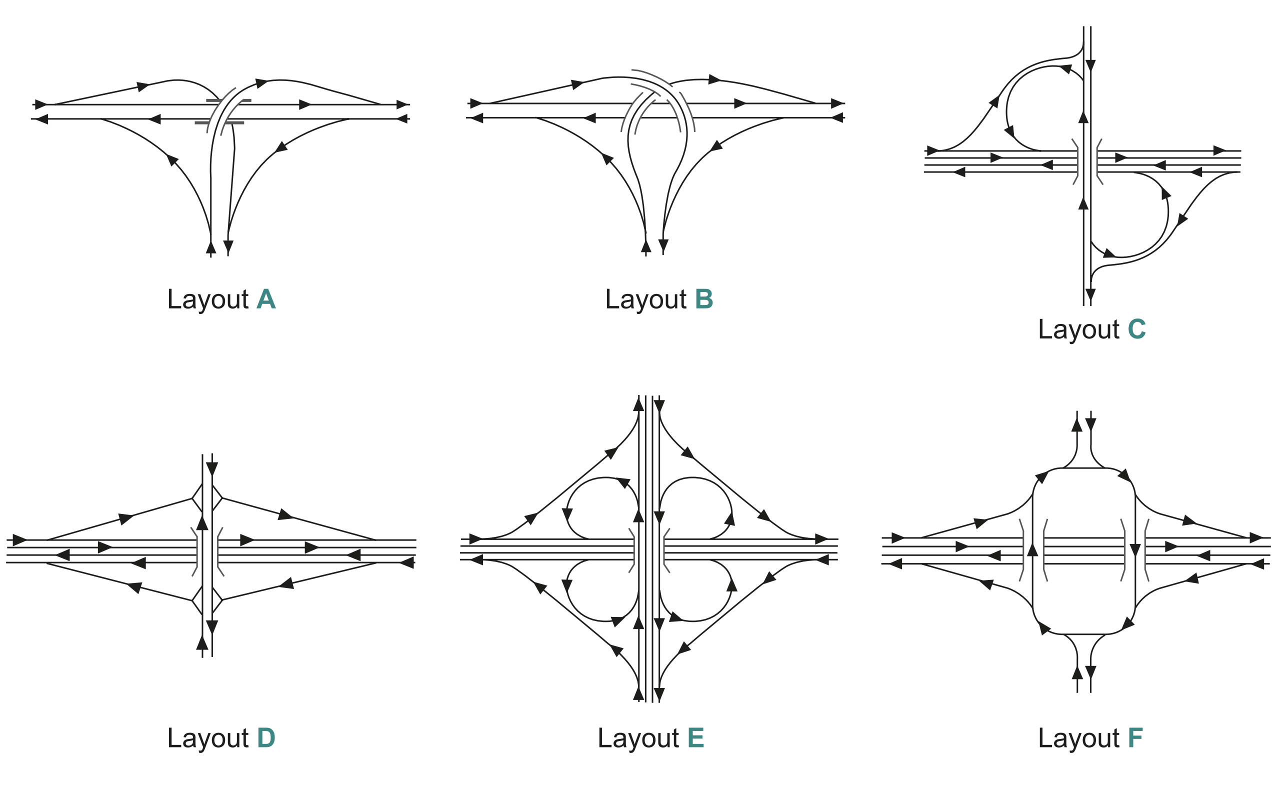
Grade-separated interchanges generally fall into four categories depending upon the number of roads involved and their relative importance. These categories are as follows:

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 Inter-change	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



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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 expressway terminates at its interchange with another expressway. 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 90º).

The Y-interchange may also be a part of a phased construction, for example, if the expressway which currently ends at the interchange, is planned to continue at some time in the future. In such a case the alignment of the expressway 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 expressway 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.

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 expressways 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 expressway 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.

Figure 11.2 Dumb-bell Layout (One Bridge, Two Roundabouts)

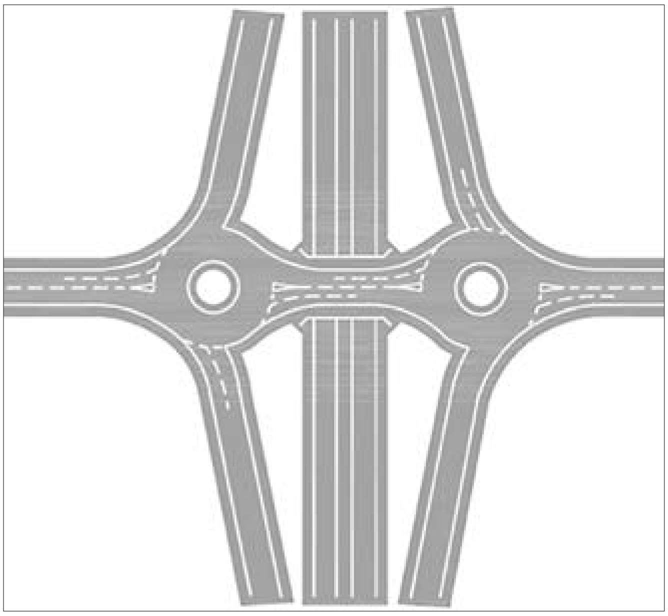
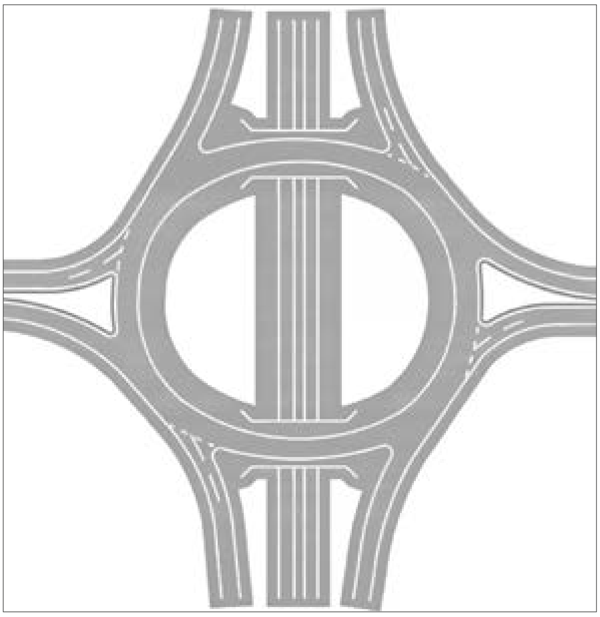


Figure 11.3 Use of a Large Roundabout



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.

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.

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Grade-separated Interchanges

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.

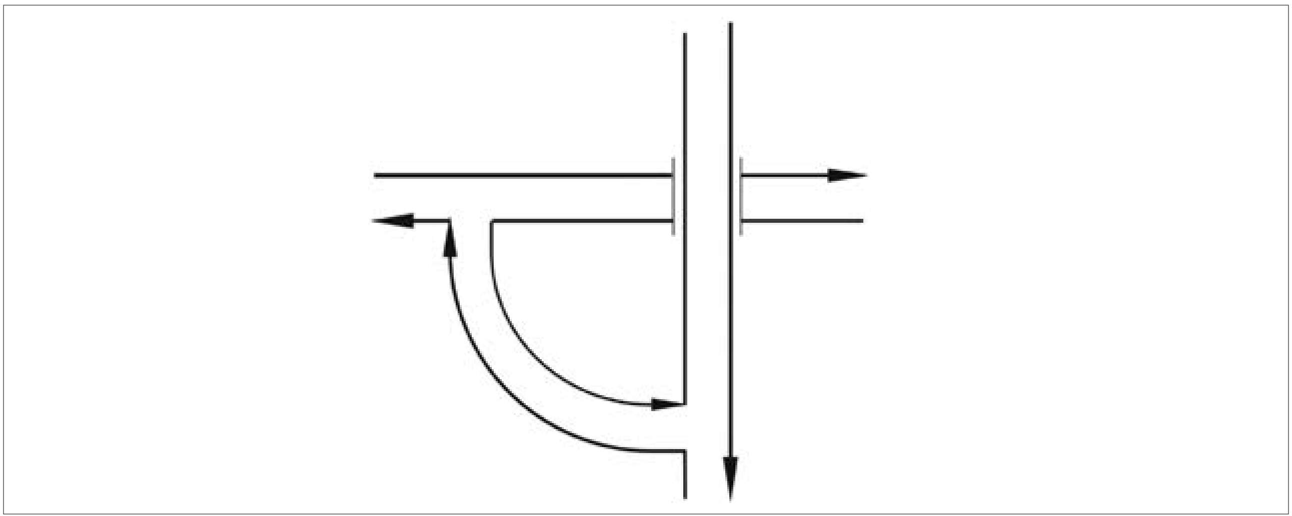
The exact layout and configuration of the ramps of the interchange depends on the interchange angles between the expressway and the intersecting road, the position of the interchange in the network and the layout of the total network. All movements may not necessarily be provided in all interchanges, 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-expressway Roads

Major routes that warrant interchanges are usually expressways. A major route that is not a expressway 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 expressway to reduce speed too quickly hence either compound curves are required suitable for an entry speed of 65% of the design speed of the expressway or a deceleration lane must be provided on the expressway.

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 expressway.

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Grade-separated Interchanges

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 expressway creating a hazardous situation and will, therefore, also affect the quality of traffic flow on the expressway. 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 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.

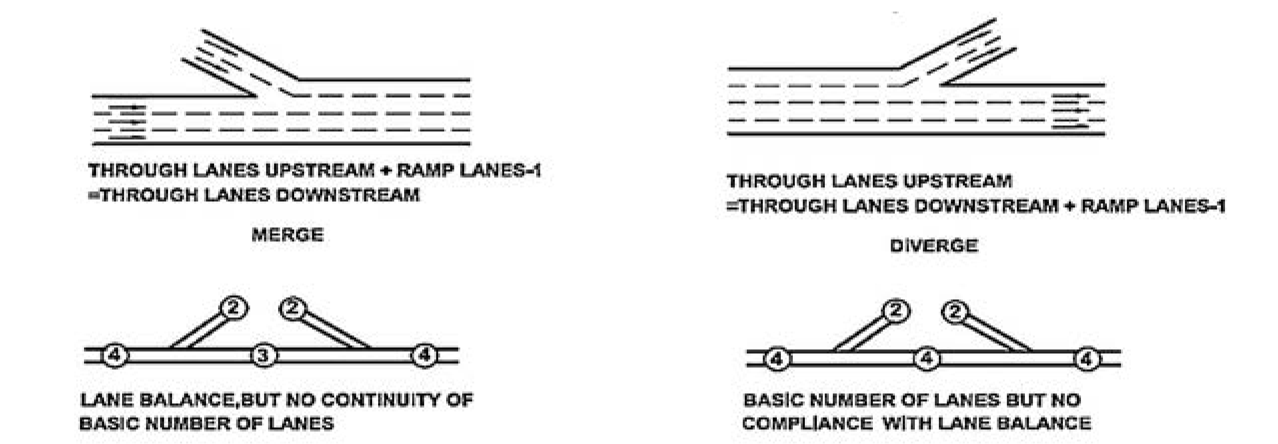
11.8.8 Basic Lanes and Lane Balance

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 expressway, the above rule indicates that the number of expressway 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.

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Grade-separated Interchanges

11.9 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.

11.10 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 an expressway 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 existing road system, may have extended up to the existing road boundary. Constraints may include the following:

A. Environmental Constraints:

1. Land take effect on property
2. Landscape
3. Ecology
4. Rights of way
5. Heritage
6. Noise and air quality
7. Visual impact

B. 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 expressway 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.

Step 8 Develop Traffic Flows

Derive low and high growth design year traffic flows for each section of expressway and arterial road.

Step 9 Lane Requirements

Determine expressway road lane requirements for each option.

Step 10 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 11 Road Signage

Ensure that an effective road signage scheme has been designed incorporating both advance direction signing and subsequent route confirmation.

Step 12 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 expressway 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 expressway 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 expressway boundaries.

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 4, 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

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.

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.

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In urban areas it is necessary to make provision for:

1. Passengers boarding and alighting on and off from public transport.
2. Disabled persons; and
3. Other non-vehicular users of the facility in addition to accommodating pedestrians and cyclists.

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.

Pedestrian safety is enhanced by the provision of median refuge islands of sufficient width at wide intersections (Section 4.13), and lighting at complex locations.

In urban areas, the presence of large numbers of pedestrians will require adequate sidewalk widths .

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 minimise 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. 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.

Improvements such as the following should me made:

1. Provision of wider paved shoulders.
2. Installation of 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.

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 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 12.1.

Table 12.1 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.

12.3 Traffic Calming

12.3.1 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.

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Road Safety Systems

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 built-up 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.

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**[2].

12.3.2 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.

12.3.3 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.

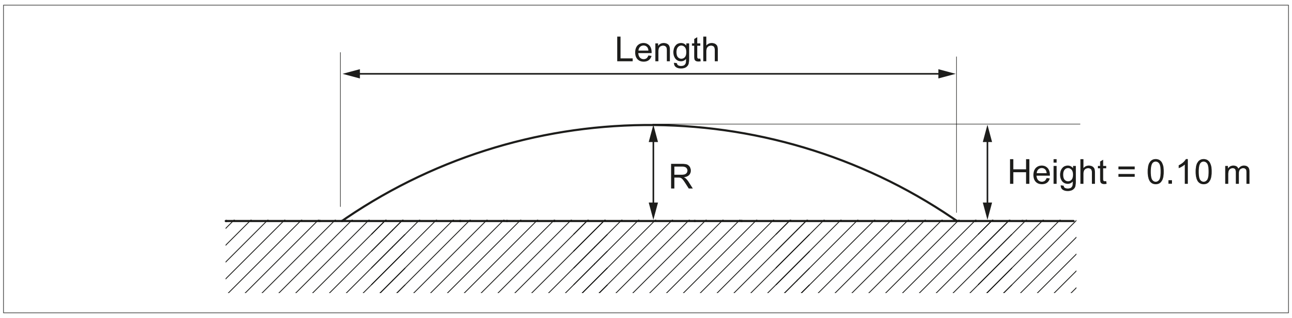
12.3.4 Speed Reduction Humps and Cushions

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 100 mm high with standard lengths of 3.7 m and 9.5 m for speed limits of 30 and 50 km/h respectively as shown in Figure 12.1.

[2] *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 12.2. 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 12.2 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
60	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 walkways. 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

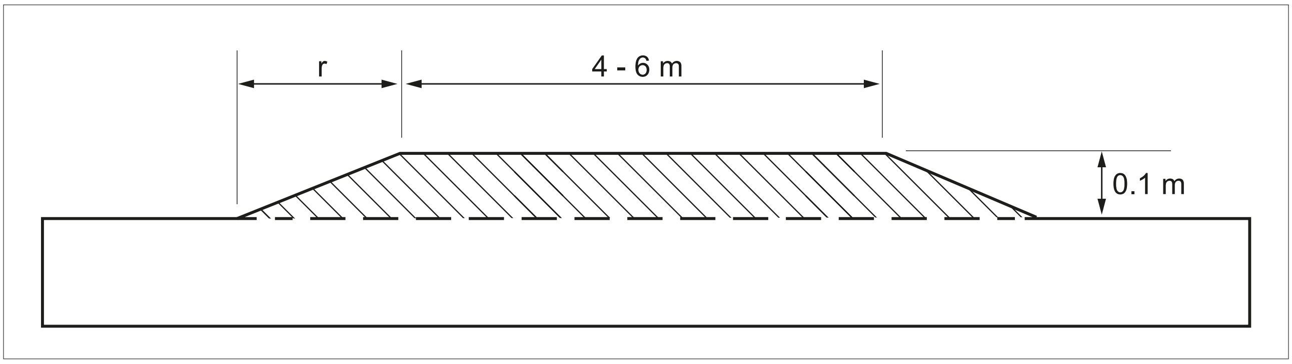


Table 12.3 Design of Flat-topped Road Humps

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
60	40	3.3	3.0

12.4 Treatment of Trading Centres

12.4.1 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.

12.4.2 The Approach Zone

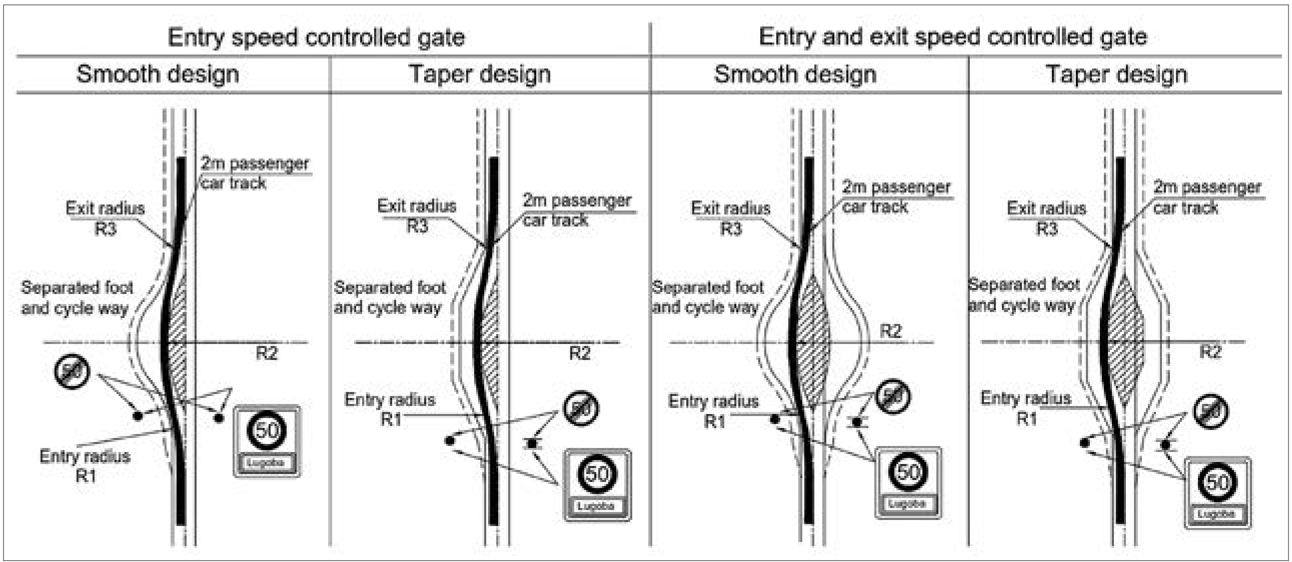
The Approach Zone is the section of road prior to entry into the centre where the driver needs to be made aware that the open road speed is no longer appropriate. This is the section of road where speed should be reduced typically from above 60/70 km/h down to 50 km/h before entering the built-up areas.

The entry should be marked by an obvious Gateway that marks the beginning of the Built-up areas. Drivers should be clearly informed that they are entering a section, where they are required to drive more slowly and carefully. 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



12.4.3 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.

12.4.4 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.

12.5 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 12.7).

12.6 Safety Barriers

12.6.1 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.

12.6.2 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 12.4 summarises the requirements for barriers for a range of roadside obstacles.

Table 12.4 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

12.6.3 Types of Longitudinal Barriers

12.6.3.1 General

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.

Longitudinal barriers are used to protect motorists from natural or man-made obstacles located along either side of the travelled way. They also sometimes protect pedestrians and cyclists.

Depending on their characteristics of deflection upon impact, roadside longitudinal barriers can be classified as flexible, semi rigid or rigid.

12.6.3.2 Flexible System

Flexible systems result in large lateral barrier deflections, but the lowest vehicle deceleration rates. Such systems have application in places where a substantial area behind the barrier is free of obstructions and/or other hazards within the zone of anticipated lateral deflection. These barriers usually consist of a weak post-and-beam system, and their design deflections are typically in the range of 3.2 m to 3.7 m but can be as low as 1.7 m.

12.6.3.3 Semi-rigid System

Semi-rigid systems, providing reduced lateral barrier deflections, but higher vehicle deceleration rates. These barrier systems have application in areas where lateral restrictions exist and where anticipated deflections have to be limited. They usually consist of a strong post-and-beam system and have design deflections ranging from 0.5 to 1.7 m.

12.6.3.4 Rigid System

Rigid systems, usually take the form of a continuous concrete barrier. These technologies result in no lateral deflection but impose the highest vehicle deceleration rates. They are usually applied in areas where there is very little room for deflection or where the penalty for penetrating the barrier is very high. Numerous shapes are available, including a high version for use where there is a high percentage of trucks.

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.

12.6.4 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 12.5 indicates the requirements of the containment levels.

Table 12.5 Containment Levels

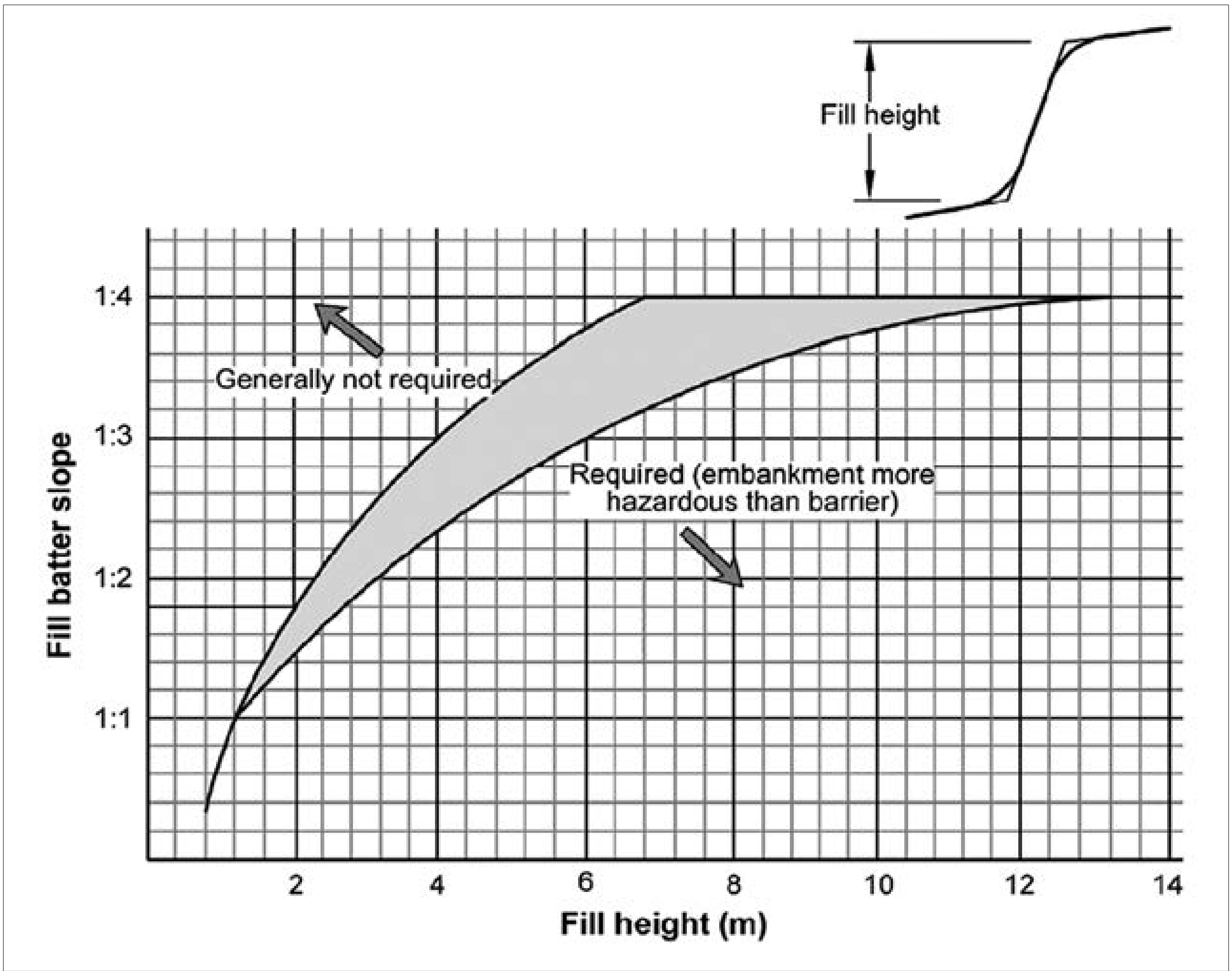
Level	Road Type and Condition
1	• Roads with speed limits ≤ 60 km/h and AADT ≤12,000. • Roads with speed limits ≥ 70 km/ and AADT ≤1,500.
2	• Roads with speed limits ≥ 60 km h and AADT > 12,000. • Roads with speed limits ≤ 70 km/ h and AADT > 1,500. • Expressways carrying high traffic volumes.
3	• Bridges 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 (<2 m) on expressways 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.

12.6.5 Requirement for Using Safety Barriers

The selection of the roadside barriers should be made on the basis of the system that will provide the required degree of shielding at the lowest cost. The lowest cost should also be based on a life cycle cost analysis, considering initial and maintenance costs and project life.

Roadside hazards that warrant shielding by barriers include embankments and roadside obstacles as provided in Table 12.4. Warrants for the use of barriers on embankments generally use embankment height and side slope as the parameters in the analysis and essentially compare the collision severity of hitting a barrier with the severity of going down the embankment. Figure 12.4 provides a chart for guidance for the installation of such barriers on embankments.

Figure 12.4 Warrants For Use of Roadside Barriers



The significance of this figure is that it provides a range of values of fill slope for which, at certain heights of fill, a barrier may be more or less hazardous than the embankment. For example, at a fill height of 6 m, a fill slope steeper than 1:3 would warrant the use of a barrier while a fill slope flatter than 1:4 would not require protection. On the intervening slopes, the designer should use his or her discretion in determining the need for a barrier.

In some situations, a measure of physical protection may be required for pedestrians or bicyclists using, or in close proximity to, a major road. Examples of such cases could include:

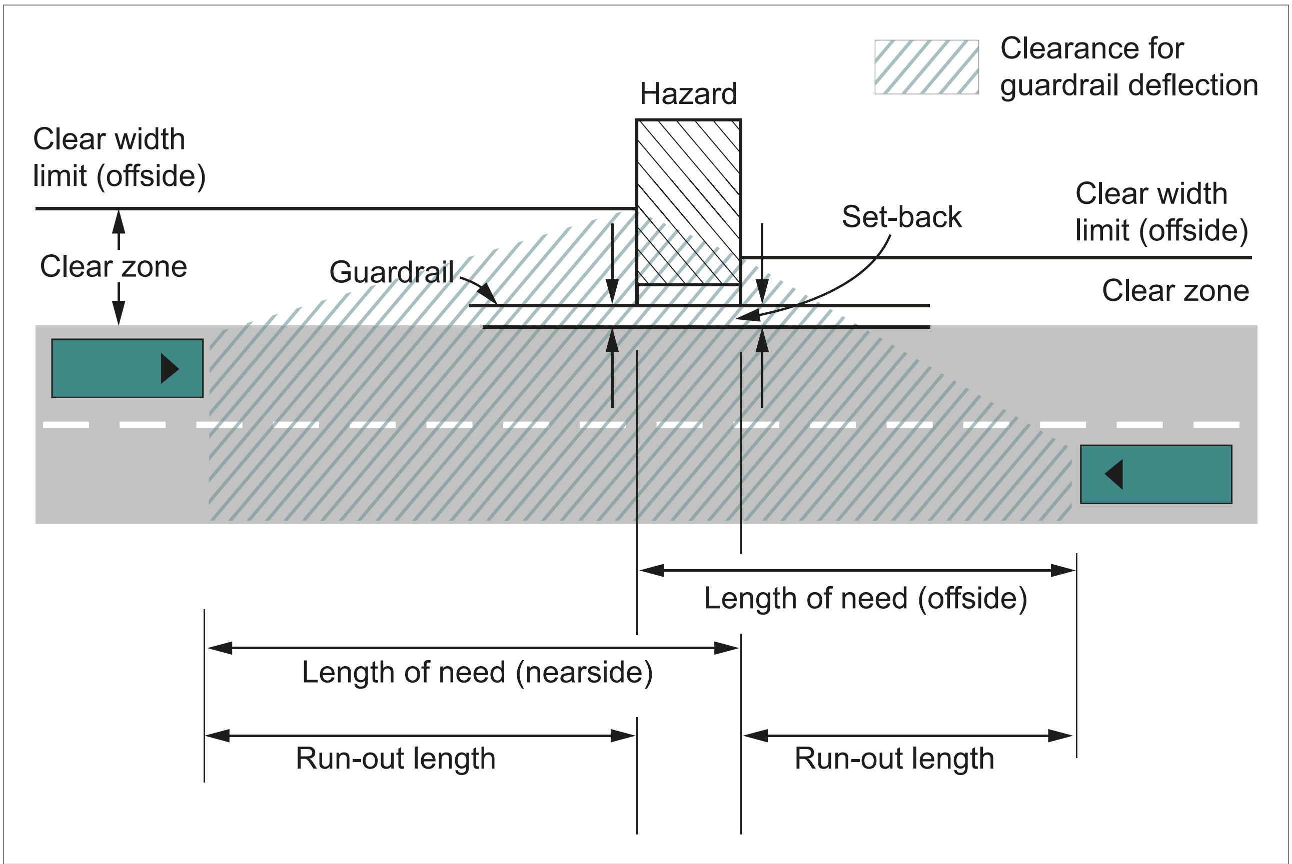
1. A barrier adjacent to a school boundary or property to minimise potential vehicle contact;
2. Shielding businesses or residences near the right of way in locations where there is a history of run-off-the road crashes; or
3. Separating pedestrians and/or cyclists from vehicle flows in circumstances where high-speed vehicle intrusions onto walkways or cycle track areas might occur.

In such cases the designer should study the needs and circumstances of the individual situation in order to take appropriate action.

12.6.6 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 12.5 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 12.5 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 80 km/h, the set-back should be 75 cm.

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

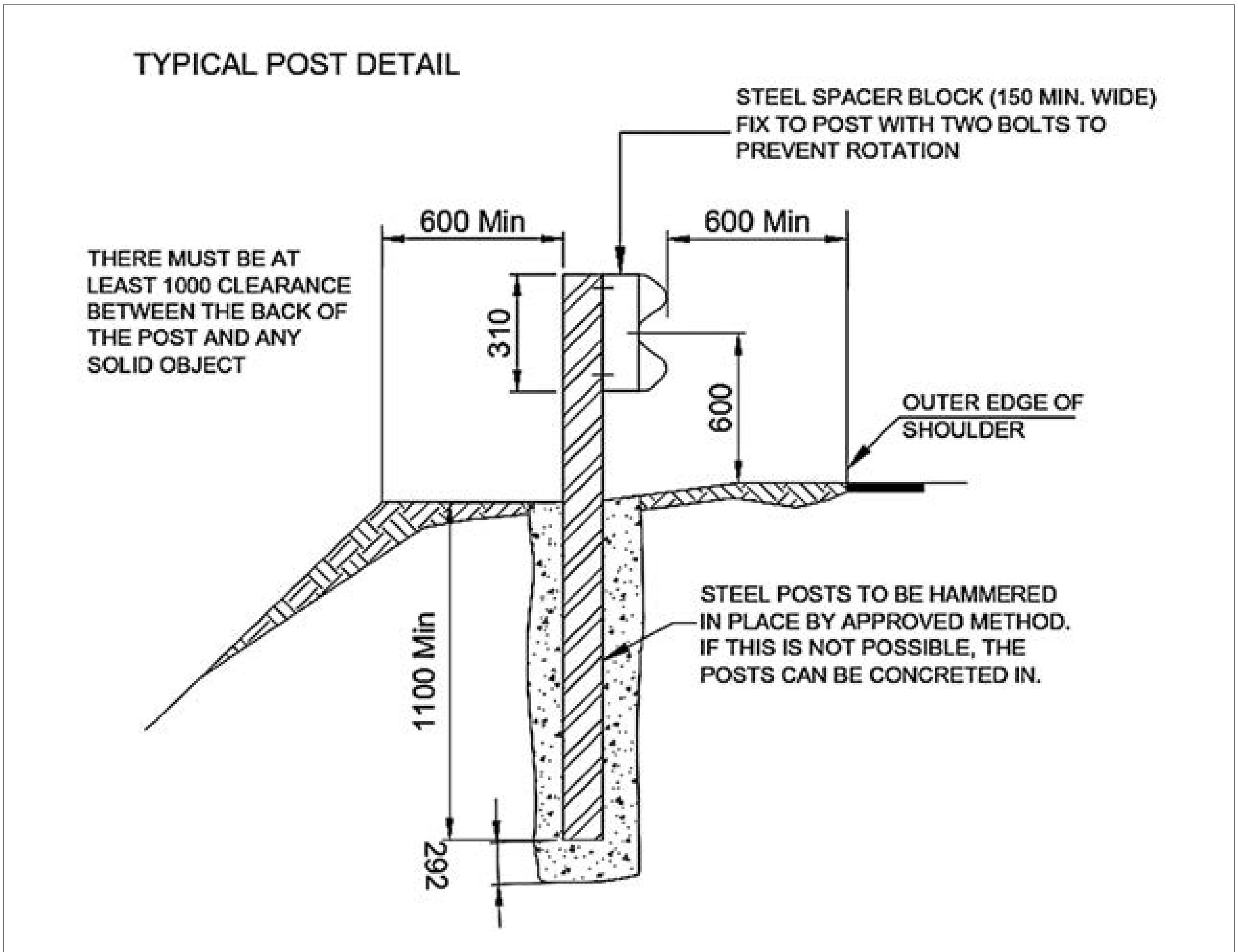
12.6.7 Steel Beam/Strong Post Guardrail

Steel beam, strong post guardrails are the most common type of safety barrier (Figure 12.6). 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 with one mounted above the other.

Figure 12.6 Typical Post Details, Steel Beam Safety Guardrail



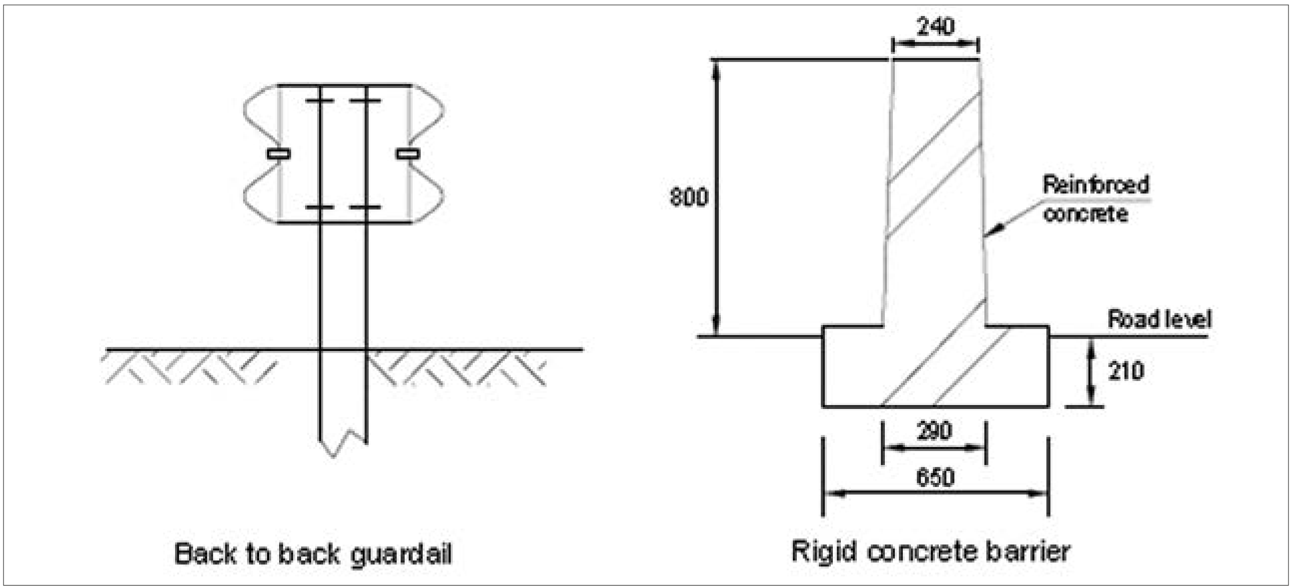
12.6.8 Steel Beam / Strong Post Guardrail Design Details

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.

12.6.9 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 12.7). Concrete barriers generally require very little routine maintenance except after a very severe impact.

Figure 12.7 Various Barriers



12.6.10 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.

12.6.11 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.

12.6.12 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.

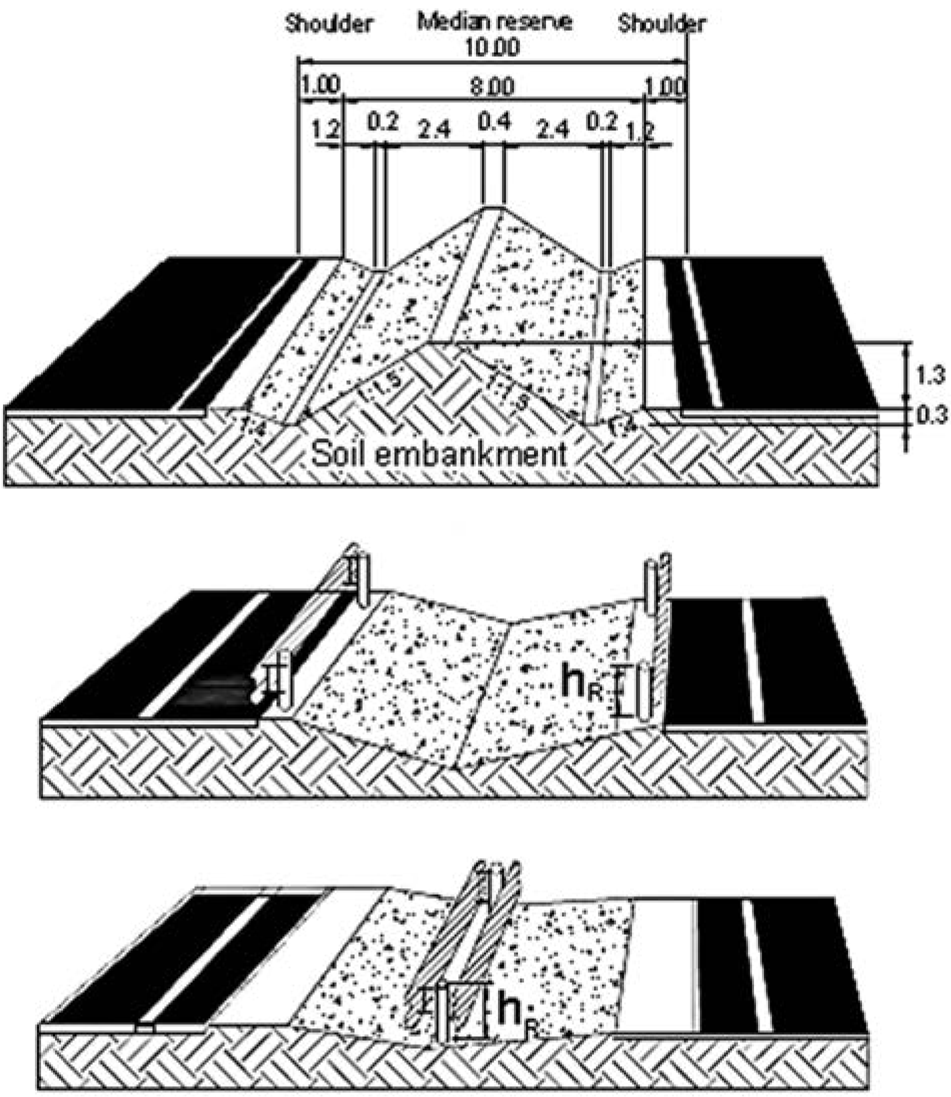
12.6.13 Median Barriers

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 12.8).

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 12.8 Different Types of Median Barrier



Height of rail (750-800mm depending on barrier type)

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.

12.6.14 Terminals of Barriers and Guardrails

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.

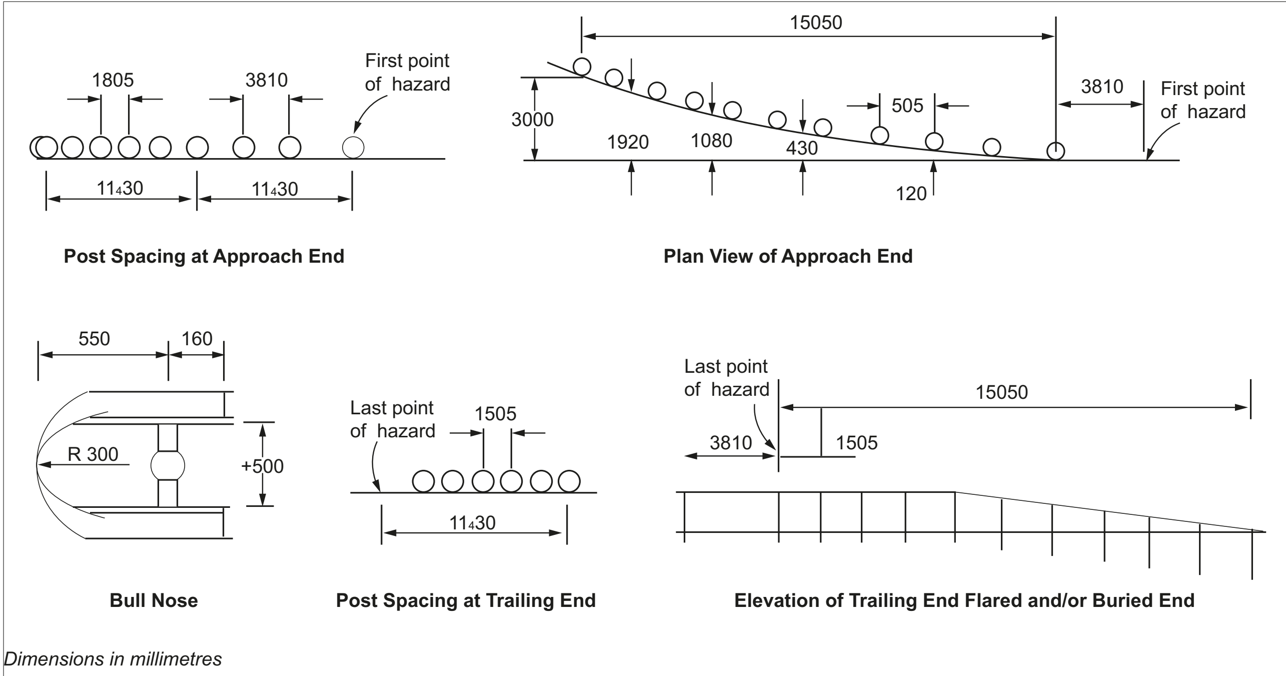
There is no completely safe way of terminating barriers and guardrails, but the general advice is:

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 12.9.

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.

Figure 12.9 Guardrail End Treatment

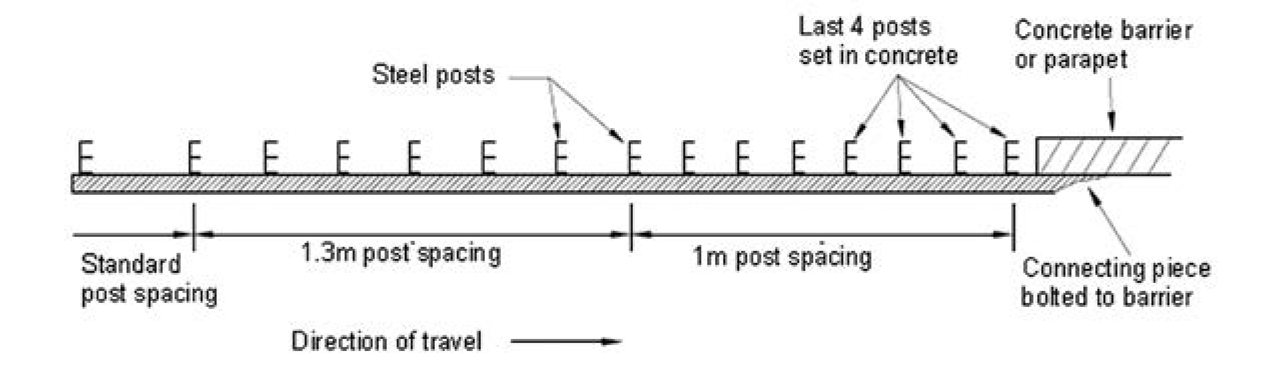


Source: *SATCC*

12.6.15 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 12.10). 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 Typical Transition (W-beam Guard-rail to Rigid Object)



12.6.16 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 pedestrian 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 Walkways are more likely to be effective.

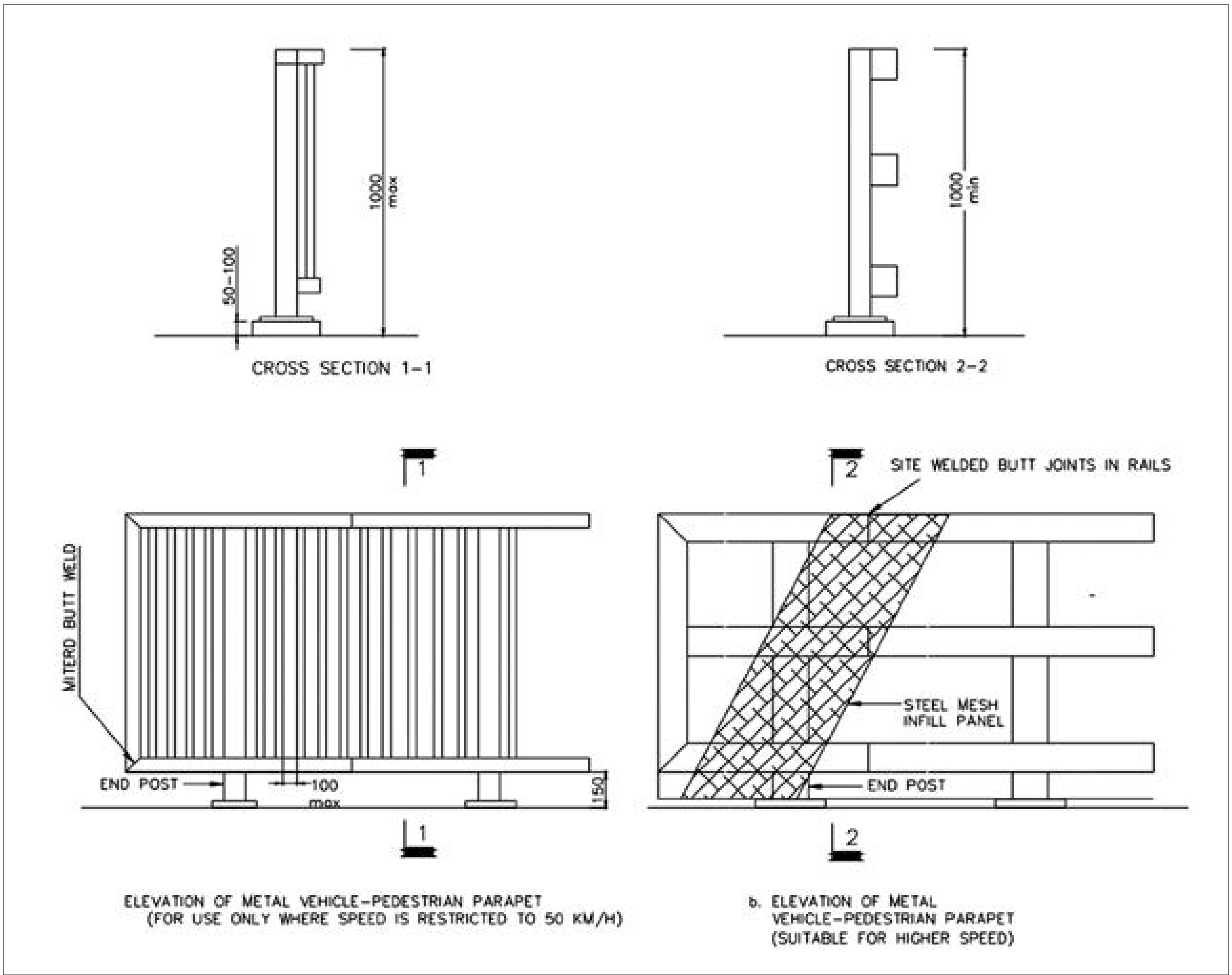
Pedestrian barriers should:

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. Be of neat aesthetical finish.

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 Walkway. Figure 12.11 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 12.11 Details of a Vehicle/Pedestrian Parapet



12.6.17 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.

12.7 Emergency Escape Ramps

12.7.1 Introduction

Where long, descending gradients exist, in addition to brake check areas (Section 12.5) 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.

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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.

12.7.2 Types of Emergency Escape Ramps

There are four types of emergency escape ramps.

1. Sand pile
2. Arrester beds- descending grade
3. Horizontal grade
4. Ascending grade

They are illustrated in Figure 12.12. All function by application of the decelerating effect of loose material.

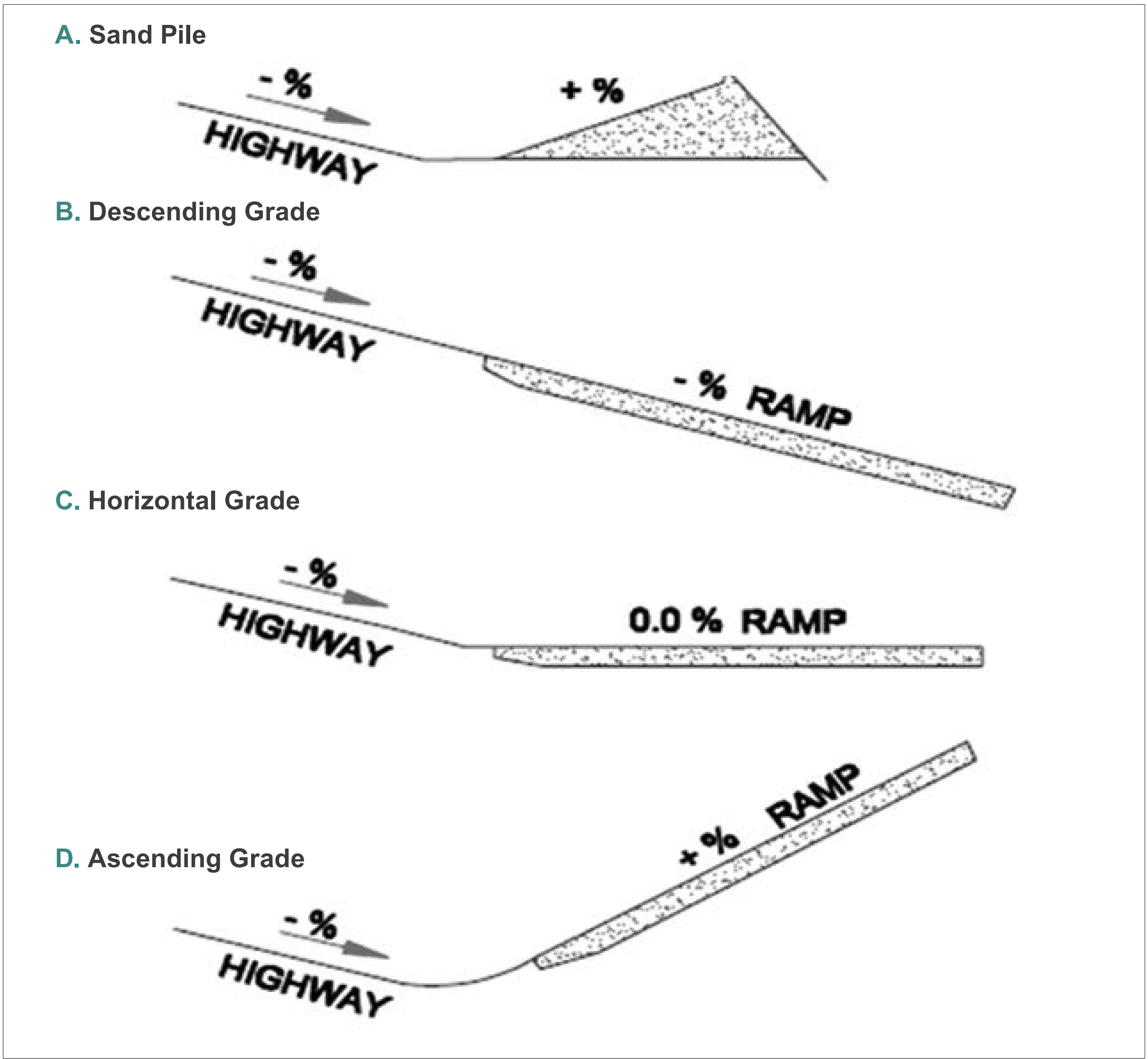
Sand piles, composed of loose, dry sand dumped at the ramp site are usually no more than 120 m in length. The influence of gravity is dependent on the slope of the surface. The increase in rolling resistance is supplied by the loose sand. Deceleration characteristics of sand piles are usually severe, 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 12.12 Basic Types of Emergency Escape Ramps



Note: *The profile is along the base line of the ramp*

12.7.3 Design Considerations

The design and construction of effective escape 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 maximise 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.

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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 12.1

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 12.1, 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.

12.7.4 Maintenance

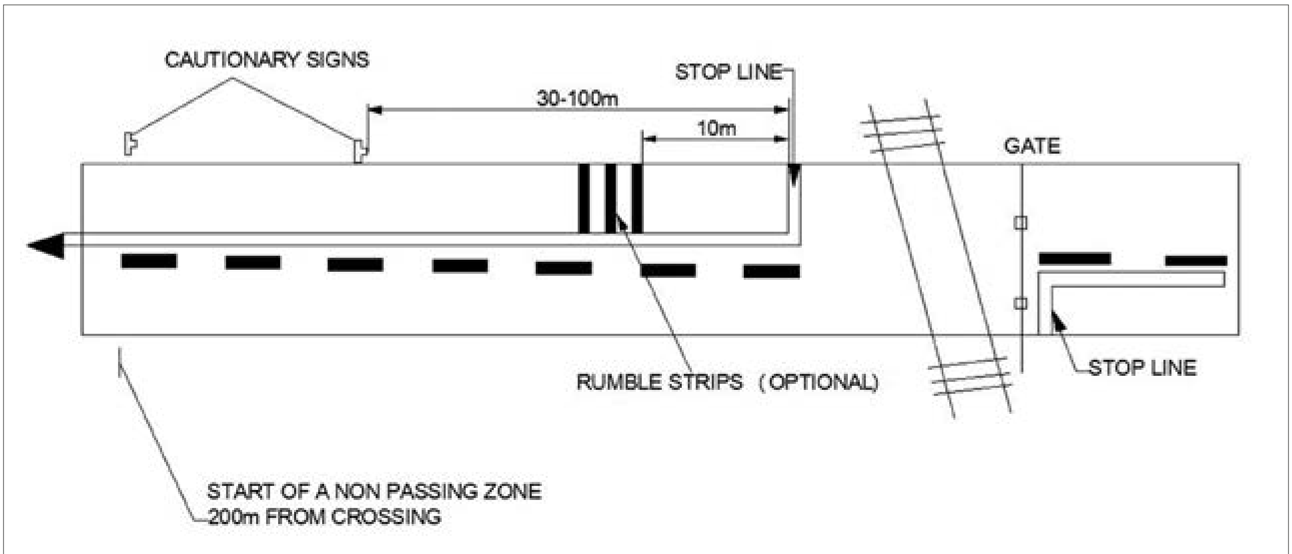
After each incident the in-fill materials should be reinstated. The arrester beds should be inspected periodically, and the in-fill materials replaced as necessary.

12.8 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 12.13). 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 45°. 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 12.13 Railway Crossing Details with Rumble Strips

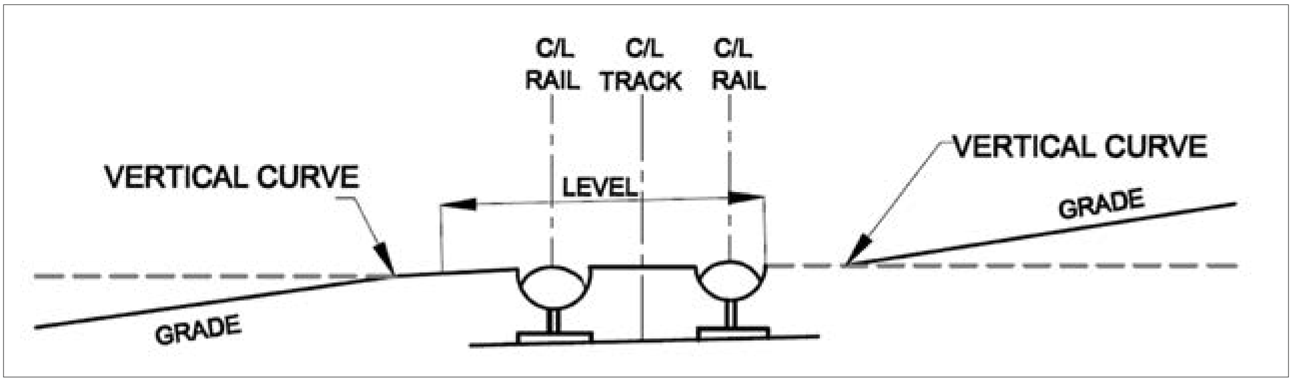


Where highways that are parallel with main tracks intersect highways that cross the tracks there should be sufficient distance between the tracks and the highway intersections to enable highway traffic in all directions to move expeditiously and safely.

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 12.14. 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 signages 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 12.14 Railway Crossings, Details on Vertical Curve



12.9 Kerbs

12.9.1 General

It is envisaged that all streets will be kerbed, whether surfaced or not. 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.

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.

Kerbing hewn from natural rock should approach semi-mountable kerbing in shape and could be used to replace mountable kerbing.

12.9.2 Function

Kerbs have several useful functions:

1. They define the edge of traffic lanes, traffic islands and walkways– during both day and night (they reflect vehicle headlights) and the ensure a neat amiable environment.
2. They support pavements and island structures so that edge break-up is avoided.
3. They protect adjacent areas from encroachment by vehicles.
4. They assist in drainage of the carriageway.

12.9.3 Types of Kerbs and Their Application

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 12.15.

12.9.3.1 Barrier Kerbs (non-mountable)

This kerb is used to provide protection to walkways traffic islands, pedestrian guardrail, traffic signs, etc. Kerbs on walkways should have a height of no more than 125 – 150 mm above road level. If they are higher than this, pedestrians may prefer to walk in the road. Barrier kerbs should not normally be used on roads with vehicle speeds in excess of 70 km/hr.

12.9.3.2 Semi-mountable Kerbs

These kerbs can be used in rural situations where high speeds would make the use of barrier kerbs risky. They are useful in defining and protecting the edges of the carriageway and traffic islands at intersections.

12.9.3.3 Mountable Kerbs

These kerbs are used to define traffic islands and road edges in urban and rural situations where there is a high risk of the kerbs being hit by vehicles. 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.

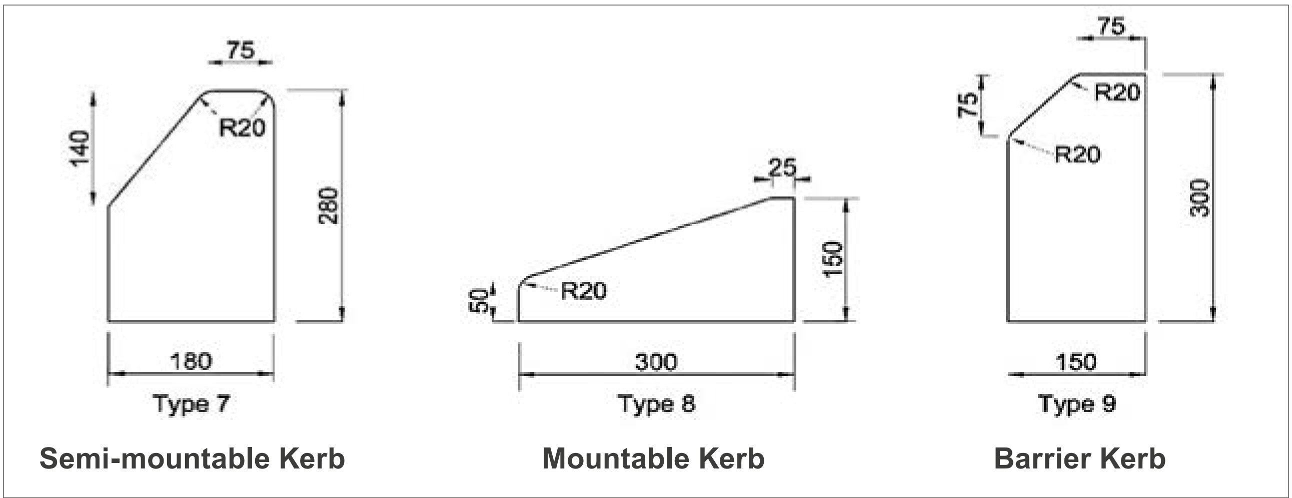
12.9.3.1 Flush Kerbs

These kerbs are used to protect and define an edge which can be crossed by vehicles.

12.9.3.2 Kerbs with Integral Drain

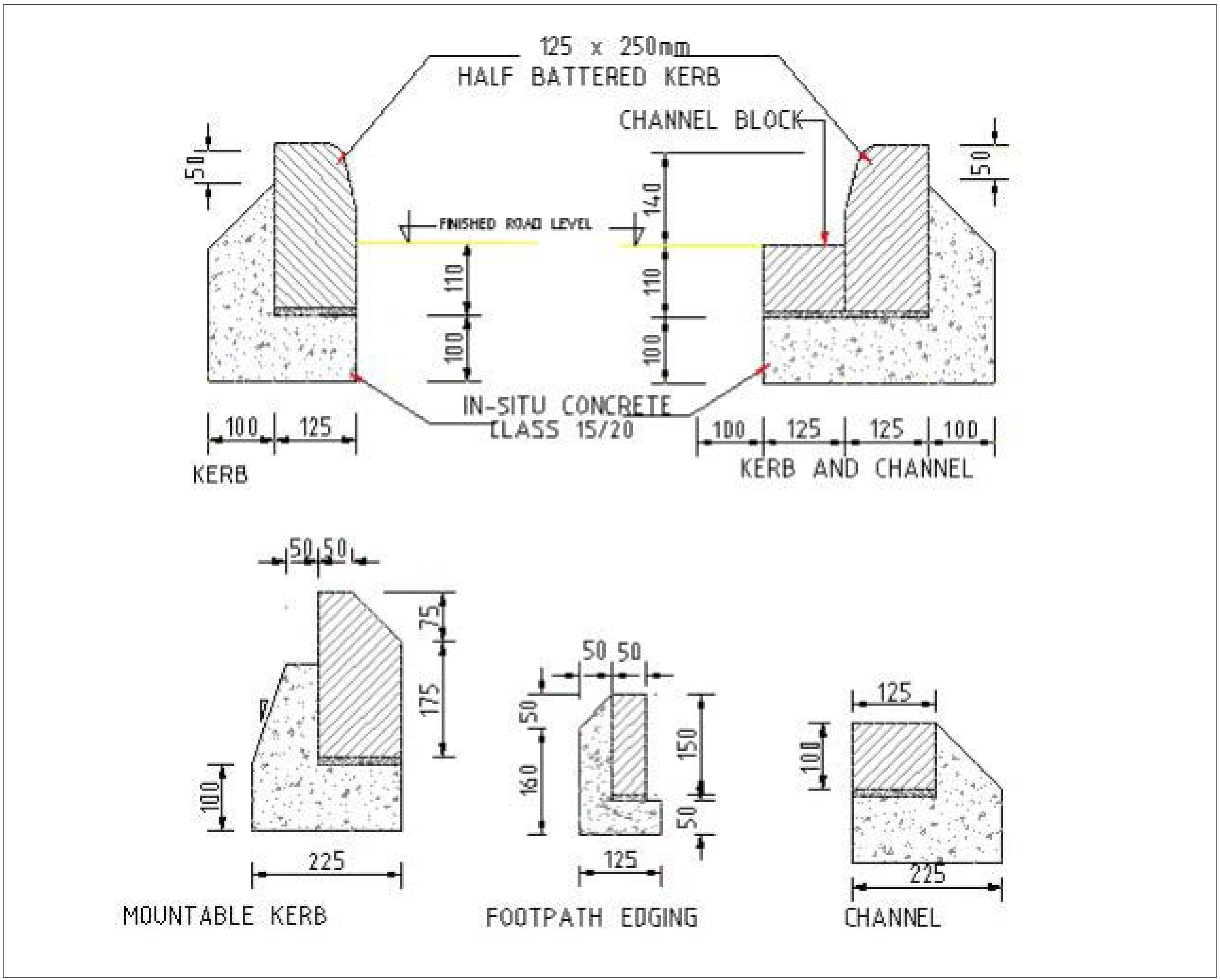
This is a neat and effective way of providing drainage in urban areas, and it reduces the risk of water penetration into the edge of the pavement. Other types of kerbs can be designed with integral drains.

Figure 12.15 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. Typical fixing of kerbs is shown in Figure 12.16.

Figure 12.16 Typical Fixing Details For Kerbing



13 Roadside Amenities and Facilities

13.1 Introduction

There are several 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 roadside amenities for travellers.

Provision of roadside amenities is one of the requirements to be incorporated in road design projects. It is necessary to establish amenities to meet the needs of long distance travellers and this is one element of road safety management, that includes:

1. Creation of rest areas at strategic location in identified zones where the onset of fatigue may appear in drivers.
2. Provision of rest areas for motorists.
3. Provision of heavy vehicle rest areas with exit and entry ramps, establish commercial and civic facilities
4. Provide stopping places for heavy and other vehicles, stopping places at public attraction areas or points of interest.
5. All the above shall be provided with proper signage in promoting and marketing the use of such roadside amenities.

Roadside amenities may be divided into the following broad categories:

1. Rest areas with amenities to enable drivers of light and heavy vehicles to rest and recuperate.
2. Commercial service centres.
3. Stopping places where stops will be short such as at points of interest (e.g. lookouts) and pull off areas.
4. Interception (inspection) sites for weighing and inspecting heavy vehicles, random breath testing etc.
5. Roadside vending sites.
6. Bus stops.
7. Emergency aides e.g. help phones.

Consistency is required in location, frequency, facilities, and signage. To achieve consistency, guidelines need to be developed that consider distances, traffic densities and driver needs. To achieve this goal, the following key principles shall be implemented:

1. Roadside amenities including driver rest areas and stopping places should be provided in appropriate locations such as in or as near as possible to townships for security reasons
2. Provision of roadside amenities is based on the value of fatigue management for long distance drivers and the reduction incidences of fatigue related accidents
3. Roadside amenities' facilities, access and signing should be consistent along routes and across provincial boundaries to assist in meeting the expectations of road users.

The following are considered as required components of roadside amenities and will be discussed in the subsequent sections:

1. Rest areas.
2. Service centres.
3. Stopping places.
4. Vehicle interception sites.
5. Roadside vending sites.

Bus Bays

Bus bays and parking bays are provided to prevent vehicles from stopping and standing on the carriageway. The siting of bus bays (which will also be used by matatus) and parking bays will depend greatly upon local conditions. The long established habits of public service and other vehicle drivers and their passengers shall not be disregarded. Bus and parking bays shall not be sited where visibility is restricted. Typical layouts for these facilities are shown in Figures 4.3.2, 4.3.3 and 4.3.4.

Figure 13.1 Simple Bus Bay (Length Dependent on the Number of Staged Vehicles)

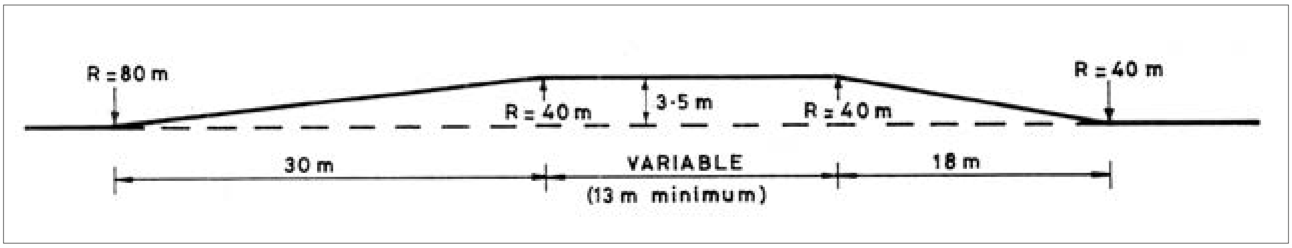


Figure 13.2 Heavily Uses Bus And Matatus Bay (Length Dependent on the Number of Staged Vehicles)

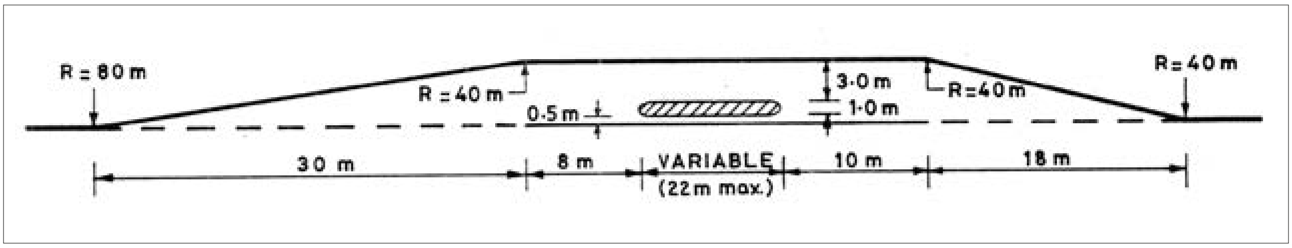
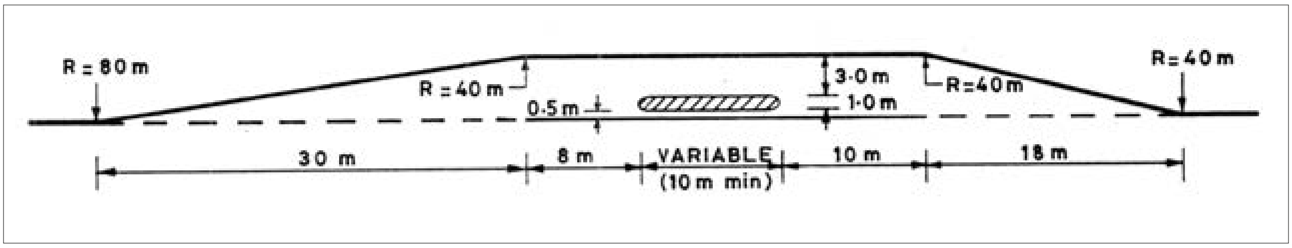


Figure 13.3 Parking Bay (Length Dependent on Number of Cars [N] or Trucks (2N) Likely to Use The Bay at the Same Time)



13.2 Rest Areas

13.2.1 General

Heavy vehicle operators drive for long distances, often through periods when they should naturally be feeling tired. It is believed that utilisation of rest time for the driver is of great importance for road safety. Roadside rest areas for heavy and commercial vehicle should therefore provide for the needs of the drivers.

13.2.2 Rest Area Categories

The following categories can be recommended based upon the location, length of travel and purpose:

13.2.2.1 Major Rest Area

These areas are designed for long rest breaks, offering a range of facilities and separate parking areas for heavy and light vehicles. These are designed to allow drivers to take rest and sleep breaks required under current driving hours regulations.

13.2.2.2 Minor Rest Area

These areas are designed for shorter rest breaks, and at a minimum should provide sufficient parking space for both heavy and light vehicles. While it is not anticipated that these stops will be used for long rest breaks/sleep opportunities, separate parking areas for heavy and light vehicles may be required at some locations.

13.2.2.3 Truck Parking Bays

These areas will primarily be designed to allow drivers of heavy vehicles to perform short checkups on their trucks such as load checks, tyre checks and other mechanical checks as required and for any other operational related needs.

On a given major arterial road design or upgrading projects where the route is usually used by freight transport, combinations of the above three types of rest area should be provided.

13.2.3 Siting of Rest Areas

13.2.3.1 Spacing

Intervals between rest areas depend on the category of rest area selected, the volume and mix of traffic and the demand for parking and rest opportunities.

However, the following can be applied as a general rule:

1. Major Rest Areas should be located at maximum intervals of 100 km.
2. Minor Rest Areas should be located at maximum intervals of 50 km.
3. Truck Parking Bays should be located at maximum intervals of 30 km.

The application of these recommended spacing intervals must be considered separately for each individual highway or road network, taking into account the mix and volume of traffic and the demand for rest opportunities. Differences in traffic volume and rest area demand will impact on the spacing of rest areas, and the number and size of parking spaces provided. Spacing may need to be reduced on highly trafficked routes where demand is evident.

13.2.3.2 Proximity to Villages or Towns

Rest areas should be located to promote the use of town facilities (including toilet and shower amenities and the purchase of food and fuel), where they are provided and accessible on a given route.

Where the traffic volume and demand justifies, consideration should be given to providing a Truck Parking Bay within or near a township to allow drivers the opportunity to take a rest break, check vehicle loads and for any repair needs.

13.2.3.3 Location

In planning the location of new rest areas and the upgrading of existing rest areas, the status and physical characteristics of the environment of a potential or existing site must be examined. Issues associated with topography, landmarks or scenic viewpoints and environment should be considered.

13.2.4 Design

13.2.4.1 Layout

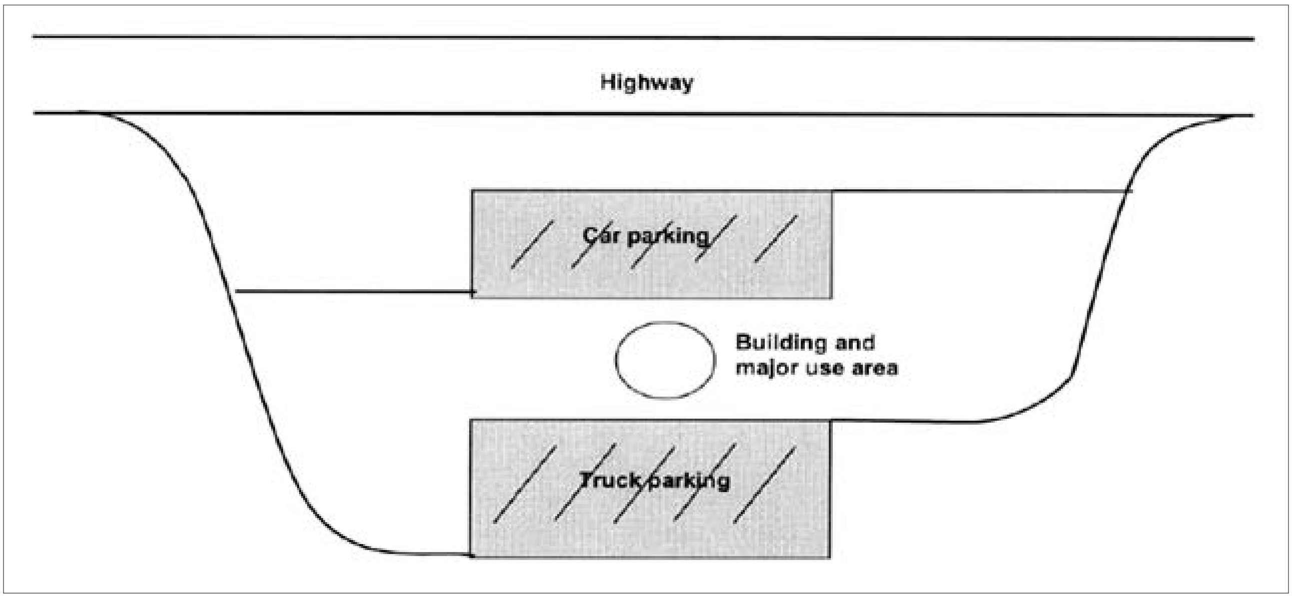
The primary goal of rest area layout design is to provide suitable facilities in an environment that promotes effective and safe rest and/or sleep opportunities, and to ensure that there is adequate provision for vehicles and pedestrians to move safely within the site.

The number of spaces provided at a given rest area site should be based on traffic volume and expected demand as described below. However the minimum numbers should be:

1. Major Rest Areas should provide sufficient parking space for at least 20 vehicles.
2. Minor Rest Areas should provide parking for up to 10 vehicles.
3. Truck Parking Bays should provide sufficient area to accommodate four to five heavy vehicles at any one time.

The number of parking bays provided should be carefully considered as insufficient parking space may discourage rest area use. In addition, the layout of the rest area should be designed, depending on category and size, to minimise conflict between pedestrians and vehicles. An example of layout is shown in Figure 13.4.

Figure 13.4 Example of Major Rest Area Layout



To determine the spacing requirements and design of rest area facilities, it is recommended that the following parking formula be used (AASHTO, 2001).

The parking formula requires knowledge of annual daily traffic (ADT), traffic volume on an hourly basis (DHV–Design Hourly Volume) and spacing intervals (BSL–Base Sector Length). It assumes the proportion of vehicles using a facility is 75% for cars and 25% for trucks, with the length of the stay between 15 minutes and 20 minutes respectively. The peak factor for the case in Kenya can be derived based on the above assumption.

The parking area analysis formula is given below:

$$N_c = \frac{ADT * P * DH * D_c * PF * VHS}{60} = \text{Car parking spaces required}$$

$$N = \frac{ADT * P * DH * D_t * PF * VHS_c}{60} = \text{Truck parking spaces required}$$

Where,

N_c = Number of car parking spaces required Nt = number of truck-parking spaces required.

P = Proportion of mainline stopping multiplied by the DSL/BSL (the adjusted proportion of mainline stopping in the overall corridor), established on a case-by case basis by usage surveys

DH = Design-hourly factor (DHV/ADT).

D_c = Proportion of cars using the facility normally assumed to be 0.75.

D_t = Proportion of trucks and oversised vehicles normally assumed to be 0.25.

PF = Peak factor, the ratio of the average day usage during the five summer months compared with the average day usage over the entire year, 1.8.

VHS = Average length of stay for cars and trucks determined on an hourly basis, normally assumed to be 15 minutes for cars and 20 minutes for trucks.

Applying constant usage design factors, as specified above:

N_c = ADT*P*0.051 (for cars).

N_t = ADT*P*0.023 (for trucks).

13.2.4.2 Security and Safety

It is recommended that the personal security of rest area users be considered in the siting and design of rest areas.

Large facilities, Major Rest Areas and Service Centres, will generally attract a level of use that will provide reasonable personal security for both car and heavy vehicle drivers. Smaller rest areas, particularly those likely to be used at night, should be located close to and within view of the road that they serve so that passing traffic can provide a base level of security. Where practical, the landscaping should maintain clear sight lines between the road and rest area. Rest areas that are used at night should be provided with adequate lighting.

13.2.4.3 Pedestrian Access and Visibility

It is recommended that rest areas and service centres be designed to ensure that potential conflict between vehicles and pedestrians is minimised, and that any necessary interaction occurs at a very low speed.

13.2.4.4 Speed

The recommended internal design speed for the assembly area is to be 10km/h. The design speed to this range is selected to minimise the speed with respect to conflicts between pedestrian and vehicle movements.

13.2.4.5 Access

When designing rest area layouts, consideration should be given to the provision of safe and effective access to the facility required for different standards of roads. Features including acceleration and deceleration lanes, entrance and exit ramps and slip lanes need to be designed in accordance with the provisions of Chapter 6, At- grade Intersection. Deceleration lanes should be provided at all facilities, to accommodate the needs of heavy vehicles entering rest areas from high-speed roads.

13.2.5 Signage for Rest Area

As a minimum requirement, signage for Rest Areas and Truck Parking Bays should be provided in accordance with the **RDM Volume 6, Part 2 Traffic Signs**.

Roadside amenities should be signed with the following:

1. **Advance Signs:** The legend on the signs should indicate of where the service is located adjacent to the road e.g. '250 m on left' or 'right' or where the service is located at the side of the road e.g. 'Turn left 250 m' or right.
2. **Position Signs:** At or directly opposite the point of entry to a service location adjacent to the road; or (b) At the turn-off to services or facilities along a side road, in conjunction with other intersection signs

13.2.6 Services and Facilities at Rest Areas

The minimum facilities that should be provided at Major and Minor Rest Areas are:

1. All weather pavements, with sealed pavements for access and egress roads/ramps.
2. Shade.
3. Rubbish bins.
4. Separate parking for heavy and light vehicles.
5. Sheltered areas.
6. Tables and/or benches, and
7. Provision for facilities for people with disabilities
8. Toilet, shower and drinking water facilities

Truck Parking Bays should include, at a minimum, rubbish bins, shade, toilet facilities and all-weather pavements. Where possible, sheltered areas, tables and benches should be provided.

13.3 Service Centres

13.3.1 General

'Service Centre' means a roadside development providing essential services for the safety, comfort and convenience of all road users, adjacent to or in close proximity to, and with direct or indirect access to the limited access road. Service centres are privately operated facilities. It is important for safety reasons to provide facilities that encourage drivers to break their journey to avoid driver fatigue and to minimise the risk of vehicles running out of fuel.

'Means of access' means a physical means of entry or exit between land and a road.

'Direct access' refers to a direct connection of a service centre's means of access to the through carriageway of the access-limited road. 'Indirect access' is where a service centre's means of access is from interchange ramps or from other roads adjacent to or connecting with the through carriageway of the access-limited road.

13.3.2 General Requirements for Service Centres

Service centres are expected to:

1. Comply with the Government's requirements for environmental protection, efficient traffic operations and safety;
2. Provide appropriate areas for rest unless deemed inappropriate in a particular circumstance by the relevant service centre strategy; and
3. Be designed to be attractive to all road users, with convenient and easy access from the through traffic carriageway.
4. Provide distinctly separate areas for trucks and other users. Facilities for truck drivers, including fuelling facilities, dining and ablutions must be separate from those for other motorists.

Service Centres should not:

1. Contain facilities which would have the effect of generating significant additional traffic and which are not essential for meeting the needs of motorists for service, safety, comfort or convenience;
2. Sell or supply alcoholic beverages; and
3. Have drive-through food service outlets, because of the intent to encourage rest and provide fatigue relief.

13.3.3 Specific Facility Requirements

The following facilities are commonly required, although may not be required at all sites:

1. Service station facilities;
2. Restaurant/fast food for motorists and heavy vehicle drivers;
3. Picnic area with tables, chairs and shelters;
4. Toilets and showers capable of servicing motorists, coach and heavy vehicle drivers and passengers;
5. Information services;
6. Enhanced landscaping;
7. Segmented and clearly defined parking zones including spaces for cars, buses, truck/semi-trailers, car/trailers and car/caravans; and
8. Space provision for Government services and emergency services.

13.3.4 Access

Accesses must be located and designed for safe operation, and not compromise the efficiency of existing and future traffic operations on the limited-access road. Access to service centre sites should be designed and constructed to operate efficiently under traffic volumes predicted to occur not less than in the design year after opening.

It is preferred that service centres are located approximately in pairs, to reduce the risk of drivers or passengers making risky manoeuvres to gain access to other road side service centre facilities. It is desirable that service centres should be located on opposite sides of the road such that the driver sees the near side service centre first.

13.4 Stopping Places

13.4.1 General

Stopping places are areas made available to enable drivers to undertake short stops for a variety of reasons, such as checking loads, attending to a vehicle breakdown or enjoying a scenic view. They have a purpose of providing a relatively safe location for immediate stopping needs at a safe distance from through traffic. Specific stopping places are required on sections of road which do not provide adequate shoulder width to allow vehicles to stop clear of the carriageway with sufficient frequency to meet unexpected stopping needs. There should be sufficient clearance from the through traffic pavement to allow drivers to inspect their vehicles safely.

Stopping places are designed to meet the specific needs of both motorists and heavy vehicle operators and can also located with a tourists' point of interest

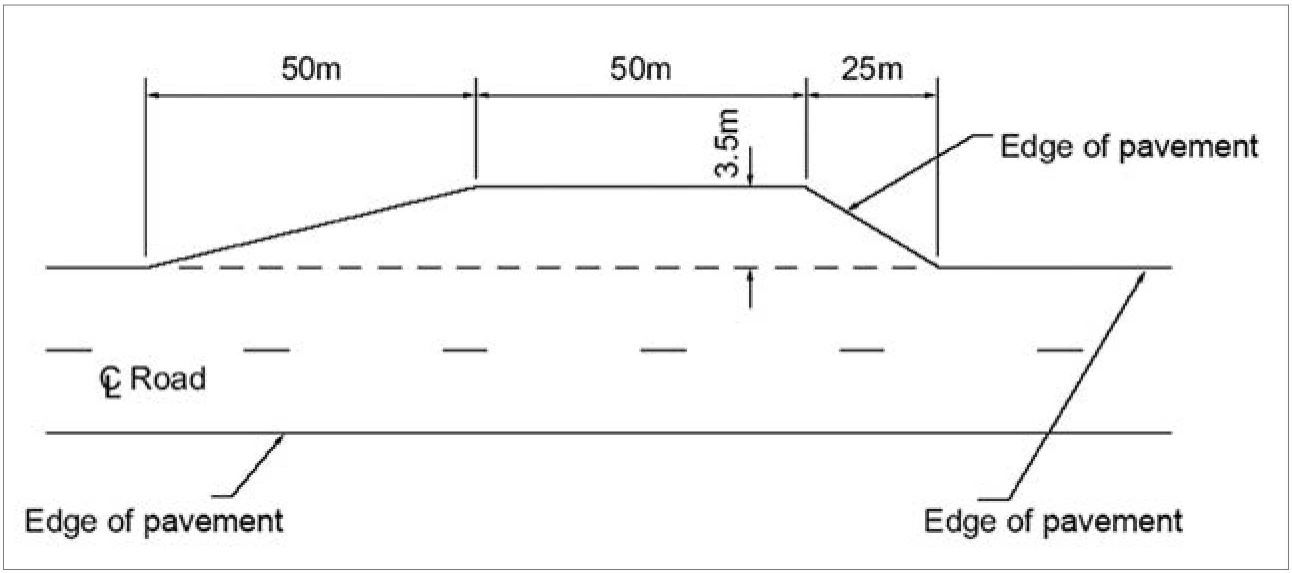
Stopping places for each direction should be located at approximately the same place, but separated to avoid having stopped vehicles on both sides of the road at the same time. A recommended spacing is 50 to 100m between adjacent tapers.

13.4.2 Motorist Stopping Places

Motorist stopping places are sealed or paved areas clear of the through pavement, allowing adequate space for a car towing a caravan or trailer, with safe access and exit, and sufficient width to allow safe inspection of the vehicle. A typical layout of such stopping places is illustrated in Figure 13.5.

In all cases, safe intersection sight distance (SISD) must be provided to the start of the approach taper of the stopping place (refer Chapter 6).

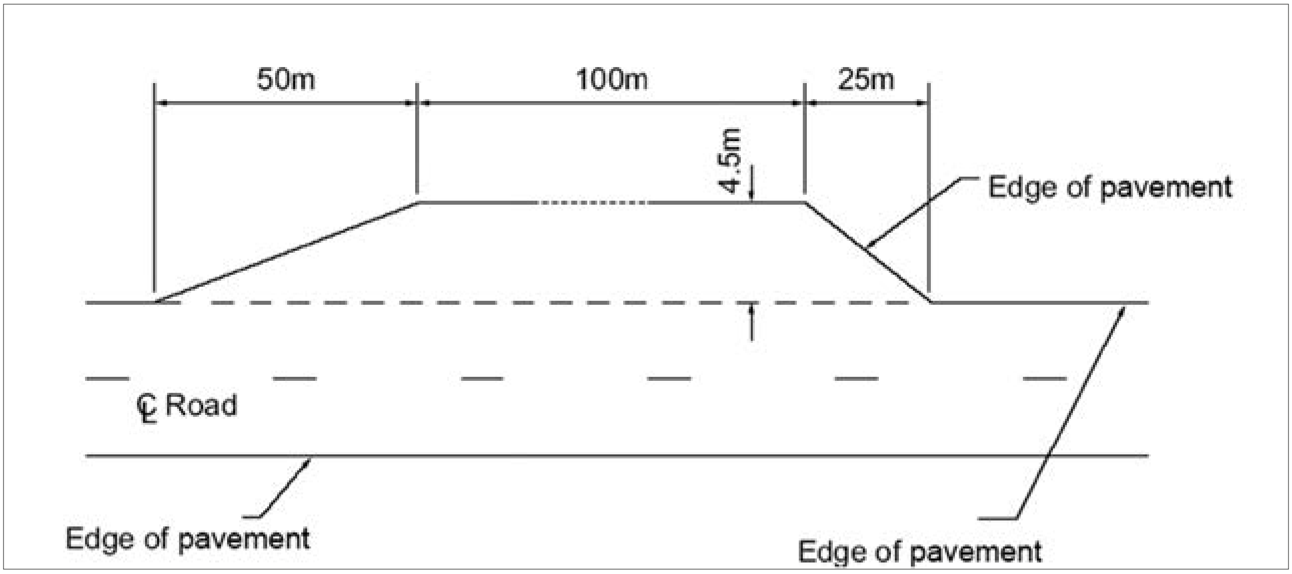
Figure 13.5 Typical Layout Of Stopping Places For Passenger Vehicles



13.4.3 Heavy Vehicle Stopping Places

Heavy vehicle stopping places are sealed or paved areas with safe entrances and exits for heavy vehicles and with sufficient clearance from the through pavement to allow loads to be inspected and adjusted safely. Figure 13.6 illustrates typical details for these stopping places. Safe Intersection Sight Distance (SISD) must be provided to the start of the approach taper of the stopping place (refer Chapter 9).

Figure 13.6 Typical Layout Of Stopping Places For Passenger Vehicles



13.4.4 Facilities

The minimum facility required in a stopping place is a rubbish bin, as stops at these places are expected to be of much shorter duration than at rest areas. Additional facilities should be considered, particularly at points of interest. Points of interest are stopping places provided to allow travellers to view and enjoy scenic areas and be informed about interesting local features.

13.4.5 Distance Between Stopping Places and Siting

The standard distance between motorist stopping places is 15 kilometres. As the standard distance between heavy vehicle stopping places is 45 kilometres, consideration should be given to making every third stopping place for motorists suitable for both heavy vehicles and motorists. If there are towns nearer than the above distances, however, the stopping places should be within or as near as possible to the townships for security reasons.

The following three points must be considered for siting for stopping places. In brief, they are:

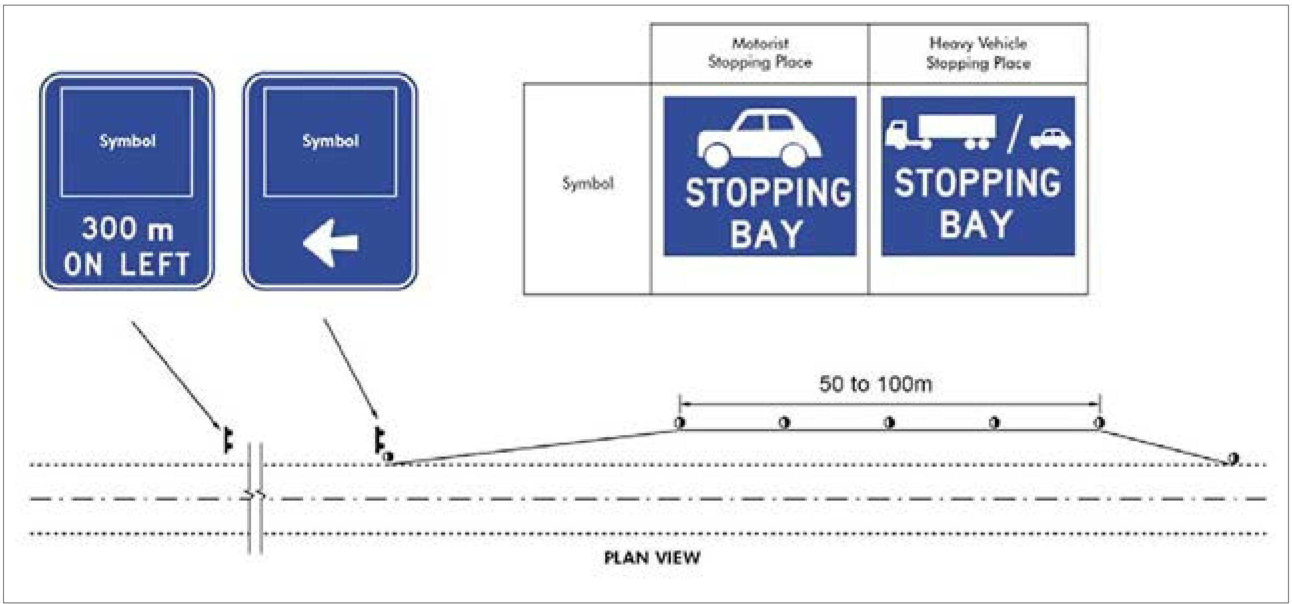
1. Sight distance for safe access and exist;
2. Placing heavy vehicle sites at or near a crest;
3. Locating at points of interest and minimising the impact on the environment.

When assessing a length of road for appropriate location of stopping bays, suitable sites for heavy vehicles (especially on crests) should be determined first. The other locations can then be determined to provide the required spacing.

13.4.6 Signage of Stopping Places

A similar approach as required for rest areas should be followed in providing the minimum signage for stopping places. An example of such signage is provided in Figure 13.7. The type of signs should be in accordance with **Road Design Manual: Volume 6 - Traffic Control Facilities and Communication Systems Design, Part 6.2 Traffic Signs**.

Figure 13.7 Signing Arrangement For Stopping Places



13.5 Traffic Interception and Inspection Sites

13.5.1 General

Interception sites are safe areas outside the road carriageway provided for:

1. Weighing and inspecting heavy and commercial vehicles;
2. Inspecting other vehicles; and
3. Other enforcement (e.g. random driving behaviour checking) by appropriate officials.

Motorists may use them for short stops to inspect their own vehicles provided the site is not being used for official purposes. Emergency vehicles may also use them.

These sites also allow over-weight vehicles to be detected and be directed to an interception site for more accurate weighing.

13.5.2 Siting Interception Sites

The location of interception sites is subject to the design requirements of the access and weighing areas and the needs of the transport inspection authorities. The routes taken by heavy vehicles (including alternative by-pass routes) are also considered.

Space required for interception for the following reasons in addition to the required sight distance and other safety needs:

1. Deceleration and acceleration tapers and access roads;
2. Storage for a number of vehicles on the approach side of the weigh site for vehicles waiting to be weighed;
3. The actual process of weighing;
4. A holding bay on the departure side of the weighing device for vehicles detected overloaded (Ideally, this area should allow for any necessary offloading to be undertaken, in addition to the area needed for safe access to both sides of the vehicle by inspectors to complete their records.); and
5. Sufficient width to permit safe passing of stationary vehicles and Inspecting Officers where a vehicle is allowed to proceed.

13.5.3 Geometry

General recommended geometric requirement are:

1. Maximum longitudinal grade over the weighing area is 2.0 % and maximum cross- fall is 3.0 %. Desirable limits are level grade and 2.0 % cross-fall. In addition the grade should be uniform over the site.
2. Visibility to the start of the exit taper and the end of the entrance taper should be not less than the heavy vehicle stopping distance and should be carefully considered in the context of the ruling traffic volume and speed.
3. Where weighing sites are required for both sides of the road to cover each travel direction, they need not be located directly opposite each other, although this is desirable due to the logistics of providing personnel for both directions.

The entry taper is longer than those for rest areas and stopping places as in those cases the drivers are expecting to stop and would be slowing down earlier. The overall dimensions of the site depend on the expected number of commercial vehicles and the extent of the holding areas required. Concrete pads with nominal dimensions of 40 m long x 5.0m wide are required for effective weighing of multi-combination vehicles.

A typical layout of vehicle interception is shown in Figure 13.8.

13.6 Roadside Vending Places

Roadside vending involves the selling of articles either directly or from a stall or standing vehicle on a road. The selling of goods and services in this way is potentially dangerous, and should be discouraged as vehicles may suddenly swerve or stop, creating unsafe situations with moving traffic.

13.7 Tolling Facilities

13.7.1 Planning and Positioning of Toll Plazas

A network analysis of travel volumes, origins and destination and the relative costs of using the tolled road is a prerequisite to locating and sizing of toll plazas.

Once the financial viability is established i.e. the income stream is in balance with the overall project costs, it is then necessary to position the plaza so as to maximise the defined catchment.

This would normally require the evaluation of a number of alternatives as they impact on:

1. Road user safety.
2. Land required.
3. Cost.
4. Operational efficiency.
5. Security.

13.7.2 Design Criteria

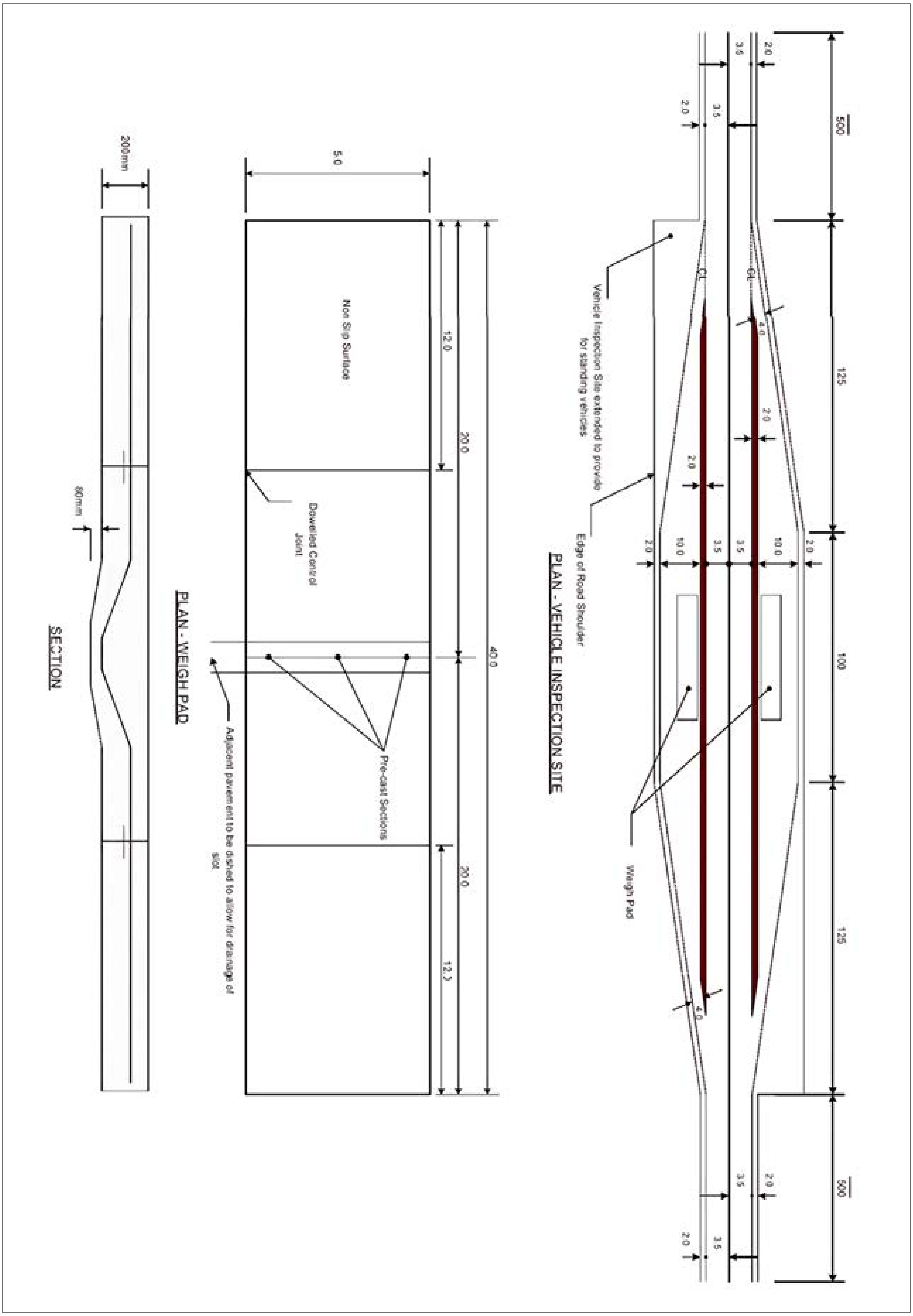
The toll plaza layout will ultimately be governed by the number of lanes required.

The standard toll booth module is 5.0 m wide consisting of a lane width of 3.0 metres and a toll island 2.0 metres wide.

An extra 3.0 m shoulder is recommended at the toll lanes on the periphery to accommodate abnormal vehicles.

For planning purposes, the average queuing and processing time per vehicle can be taken as 30 seconds and the maximum queue length should not exceed 5 vehicles.

Figure 13.8 Typical Layout of Vehicle Inspection Site



13.8 Mass Rapid Transit Facilities (MRT)

Mass transit , also called mass transportation, or public transportation, the movement of people within urban areas using group travel technologies such as buses and trains. The essential feature of mass transportation is that many people are carried in the same vehicle (e.g., buses) or collection of attached vehicles (trains). This makes it possible to move people in the same travel corridor with greater efficiency, which can lead to lower costs to carry each person or—because the costs are shared by many people—the opportunity to spend more money to provide better service, or both.

Mass transit systems may be owned by private, profit-making companies or by governments or quasi-government agencies that may not operate for profit. Whether public or private, many mass transportation services are subsidised because they cannot cover all their costs from fares charged to their riders. Such subsidies assure the availability of mass transit, which contributes to making cities efficient and desirable places in which to live. The importance of mass transportation in supporting urban life differs among cities, depending largely on the role played by its chief competitor, the private automobile.

People travel to meet their needs for subsistence (to go to work, to acquire food and essential services), for personal development (to go to school and cultural facilities), and for entertainment (to participate in or watch sporting events, to visit friends). The need for travel is a derived need, because people rarely travel for the sake of travel itself; they travel to meet the primary needs of daily life. Mobility is an essential feature of urban life, for it defines the ability to participate in modern society.

In Kenya, the policy for the provision of MRT facilities was adopted in 2009. NAMATA has since produced guidelines on planning, designing and operating BRT facilities, which are a sub-set of MRT facilities.

13.9 Public Utilities

13.9.1 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 13.1 indicates the utilities that are likely to be involved.

Table 13.1 Public Utilities

Surface Utilities	Underground Utilities
Sanitary sewers	Buried telecommunication 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.

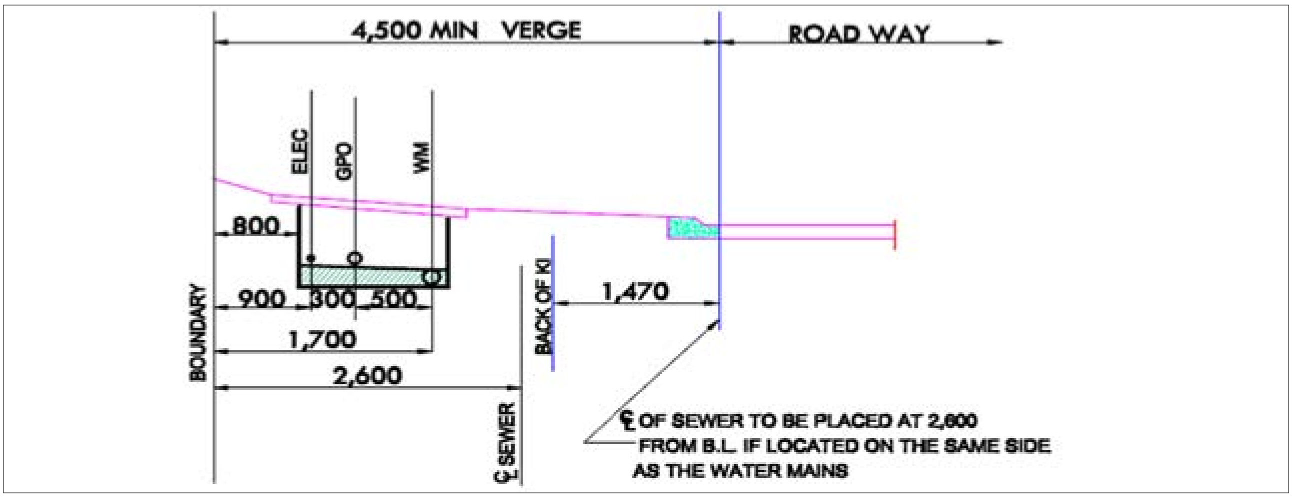
1
2
3
4
5
6
7
8
9
10
11
12
13
Roadside Amenities and Facilities

13.9.2 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 authorised. Electric and Telephone Cables and water main placing in one trench should be done according to Figure 13.10 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 Figure 13.9 as an example.

Figure 13.9 Utilities Placement Detail



Note:

1 After laying water main, backfill to 850 depth before installing electricity cables and telephone ducts.

2 Install cable ducts for telephone cables when installing other services.

All dimensions are given in mm.

14 Design of Interurban Roads

14.1 General

This Chapter attends specifically to the design of interurban roads in Kenya.

Chapters 1 to 13 of provide for all aspects of geometric design that are cross-cutting amongst all functional road classes. The designer shall refer to these Chapters for more information as necessary.

The interurban roads system in Kenya 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 multi-lane improvements.

Interurban roads are stratified into three classes: functional Class A, B, and C. The design standards applicable for such corridors prioritise mobility with separation of low and high-speed vehicles in highly trafficked areas.

With the low volumes of NMT expected on these roads, there is usually no separate provisions for such traffic. Shoulders may be used as walkways to accommodate pedestrians and cyclists. If need be, such as when such corridors pass through small urban centres, NMT facilities should be provided.

14.2 Design Standards for Interurban Roads

The applicable typical design standards for design of interurban roads are as shown in Table 14.1 below.

Derivation and assignment of these typical design standards to classified roads in Kenya is outlined in Chapter 2.

Table 14.1 Applicable Design Standards for Interurban Roads

Functional Classification	Applicable Design Standards
A	DR1, DR2
B	DR2, DR3, DR4
C	DR3, DR4, DR5

14.3 Design Controls and Criteria

14.3.1 General

Reference shall be made to Chapter 3 for details and guidance on all aspects of design controls and criteria for roads in Kenya. Below is a treatise of controls and criteria applying specifically to interurban roads.

14.3.2 Design Vehicle

For interurban roads in Kenya, the DV6 design vehicle, which is a truck and trailer combination as detailed in Table 3.3, shall be used for all conditions except for very severe escarpment terrain. The DV6 vehicle has a minimum turning radii of 12 m.

For mountainous area and steep escarpments the DV4 design vehicle shall be employed.

14.3.3 Design Speed

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 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.

The design speeds for the interurban roads and for the typical standards that apply to these classes of road are shown in Table 14.2, by terrain type and functional class.

Table 14.2 Design Speeds for Interurban Roads

<table>
<tr><th rowspan="2">Design Standards</th><th colspan="4">Maximum Design Speed (Km/H)</th><th rowspan="2" colspan="3">Application to Functional Classes</th></tr>
<tr><th>Flat</th><th>Rolling</th><th>Mountainous</th><th>Escarpment</th></tr>
<tr><td>DR1</td><td>120</td><td>100</td><td>80</td><td>-</td><td rowspan="2">A</td><td></td><td></td></tr>
<tr><td>DR2</td><td>110</td><td>90</td><td>70</td><td>50</td><td rowspan="3">B</td><td></td></tr>
<tr><td>DR3</td><td>100</td><td>90</td><td>70</td><td>50</td><td></td><td rowspan="3">C</td></tr>
<tr><td>DR4</td><td>90</td><td>85</td><td>60</td><td>50</td><td></td></tr>
<tr><td>DR5</td><td>80</td><td>65</td><td>50</td><td>40</td><td></td><td></td></tr>
</table>

14.3.4 Road Capacity

The design of any roadway and its features should explicitly consider traffic volumes, operational performance, and user characteristics for all transportation modes.

The designer shall determine the capacity of the road under consideration, and this will be the maximum hourly rate at which 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. To achieve this, traffic data shall be obtained from field studies in accordance with the **Road Design Manual Volume 1 Part 2: Traffic Surveys**.

Knowing the anticipated capacity of the highway the designer shall provide for features of the road to fit the requirements of the estimated traffic, selecting the best road type, dimensions of lanes, shoulders, weaving sections, geometry, etc.

Generally interurban roads in Kenya have an AADT of more than 1000 vehicles and shall be designed to accommodate the 30th to 50th highest hourly volume in year ten after opening (DHV = Design Hourly Volume), depending on economic considerations.

Design capacities for two-way interurban roads are shown in Table 14.3.

Table 14.3 Practical Design Capacities of 2-Lane Single Carriageway Interurban 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%	400
			80%	360
			60%	300
2000	80	90-100	100%	800
			80%	700
			60%	600
			40%	480
2000	65	80	100%	1120
			80%	1060
			60%	940
			40%	760
			20%	560
2000	55	70	100%	1340
			80%	1240
			60%	1140
			40%	1040
			20%	880

14.3.5 Level of Service

The quality of service provided by a specific road section under specific conditions is described as the Level of Service (LoS).

As detailed in Chapter 3, 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.

Interurban road in Kenya shall be designed for Level of Service E.

14.4 Design Catalogues for Interurban Roads

The following design catalogues summarise the design controls, cross-sectional elements, horizontal alignment characteristics and other elements for interurban roads:

Table 14.4: Design Standard DR1 – Interurban Multi-lane

Table 14.5: Design Standard DR2 – Rural Single Carriageway Class A, B

Table 14.6: Design Standard DR3 – Rural Single Carriageway Class B, C

Table 14.7: Design Standard DR4 – Rural Single Carriageway Class B,

Table 14.8: Design Standard DR5 – Rural Single Carriageway Class C

The designer shall use the catalogues as guides in preparing the design for each particular situation. It shall be the designer's responsibility to check and refine the outcome of the application of the guides against the design principles given in this manual, as well as good practice in general.

Table 14.4 Design Standard DR1 – Interurban Multi-lane

	Design Element			Ref. Section	Requirement/Specification					
	Applicable Functional Class				**Class A Multi-lane**					
Design Controls	Design Life				20 years					
Design Controls	Design Traffic (DHV)				> 8000					
Design Controls	Design Speed (Km/Hr)				Flat		Rolling		Mountainous	
Design Controls					120 – 110		100 - 90		80 - 70	
Design Controls	Control of Access				Full					
Design Controls	Level of Service Threshold				C					
Cross-Section Elements	Typical Cross-section				R1, R2, R3, R4					
Cross-Section Elements	Lane Width				3.65 m					
Cross-Section Elements	No. of Lanes each Direction				2 or more					
Cross-Section Elements	Outside Shoulder Width	Paved			R1, R2, R3 - 2.7 m; R4 – 2.5 m					
Cross-Section Elements		Un-paved			N/A					
Cross-Section Elements	Median Shoulder Width	Paved			R1, R2 – 1.5 m; R3 – 1.0 m; R4 – N/A					
Cross-Section Elements		Un-paved			N/A					
Cross-Section Elements	Crossfall Slope	Travel Lane			2.5 %					
Cross-Section Elements		Shoulder			4.0 %					
Cross-Section Elements	Auxiliary Lanes	Lane Width			3.0 m					
Cross-Section Elements		Shoulder Width			Paved: 1.0 m Unpaved – N/A					
Cross-Section Elements	Median Width	Depressed			2 m Minimum					
Cross-Section Elements		Concrete Median Barrier			0.75m					
Cross-Section Elements	Road Reserve				Desirable: 110 m; Reduced Desirable: 80 m					
Cross-Section Elements	Roadside Clear Zone	Low Traffic (Fill/Cut)			5 m; 3 m					
Cross-Section Elements		Medium Traffic (Fill/Cut)			8.5 m; 5.5 m					
Cross-Section Elements	Slopes	Cut	Cut Slope		1:4					
Cross-Section Elements			Depth of Ditch		0.75 m					
Cross-Section Elements			Backslope		1:3					
Cross-Section Elements		Fill	Safety Slope (within clear zone)		1:4					
Cross-Section Elements			Fill Slope (outside clear zone)		1:3					
Alignment Elements	**Design Speed (km/hr)**				**120**	**110**	**100**	**90**	**80**	**70**
Alignment Elements	Stopping Sight Distance (m)	g: 0%			285	245	205	170	140	110
Alignment Elements		g: 5%			330	285	235	195	155	120
Alignment Elements		g: 10%			400	330	280	230	180	140
Alignment Elements	Passing Sight Distance (m)	Desirable			780	703	670	615	550	485
Alignment Elements		Minimum			395	350	310	275	240	210
Alignment Elements	% Passing Opportunity				50		33		25	
Alignment Elements	Maximum Super-elevation Rate %				8	8	8	8	8	8
Alignment Elements	Minimum Horizontal Curve Radius (m)	SE: 4%			N/A	N/A	525	400	300	205
Alignment Elements		SE: 6%			755	595	465	355	265	185
Alignment Elements		SE: 8%			665	530	415	320	240	170
Alignment Elements	Maximum Grades	Flat			Desirable: 3 %; Absolute: 5 %					
Alignment Elements		Rolling			Desirable: 4 %; Absolute: 6 %					
Alignment Elements		Mountainous			Desirable: 6 %; Absolute: 8 %					
Alignment Elements	Minimum Grades				0.5 %					
Alignment Elements	Vertical Curve (K-values)	Crest			185	140	100	67	45	30
Alignment Elements		Sag (Comfort)			36	30	25	20	16	12
Alignment Elements		Sag (Headlights)			70	60	50	40	32	25

Table 14.5 Design Standard DR2 – Rural Single Carriageway Class A, B

	Design Element			Ref. Section	Requirement/Specification							
	Applicable Functional Class				**Class A, Class B**							
Design Controls	Design Life				20 years							
	Design Traffic (AADT or DHV)				AADT>4000 or 500<DHV<8000							
	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	Typical Cross-section				R4, R5,							
	Lane Width				R4 - 3.65 m, R5 – 3.5 m							
	No. of Lanes each Direction				2							
	Outside Shoulder Width	Paved			R4 – 2.5 m, R5 – 2.0 m							
		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			3.0 m							
		Shoulder Width			Paved: 1.0 m Unpaved – N/A							
	Median Width	Depressed			2 m 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 14.6 Design Standard DR3 – Rural Single Carriageway Class B, C

	Design Element			Ref. Section	Requirement/Specification
	Applicable Functional Class				**Class B, Class C**
Design Controls	Design Life				20 years
Design Controls	Design Traffic (AADT or DHV)				2000 < AADT < 4000 or 250 < DHV < 500
Design Controls	Design Speed (Km/Hr)				see below
Design Controls	Control of Access				Full
Design Controls	Level of Service Threshold				C
Cross-Section Elements	Typical Cross-section				R4, R5,
Cross-Section Elements	Lane Width				R4 - 3.65 m, R5 – 3.5 m
Cross-Section Elements	No. of Lanes each Direction				1
Cross-Section Elements	Outside Shoulder Width	Paved			R4 – 2.5 m, R5 – 2.0 m
Cross-Section Elements	Outside Shoulder Width	Un-paved			N/A
Cross-Section Elements	Median Shoulder Width	Paved			N/A
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			4.0 %
Cross-Section Elements	Auxiliary Lanes	Lane Width			3.0 m
Cross-Section Elements	Auxiliary Lanes	Shoulder Width			Paved: 1.0 m Unpaved – N/A
Cross-Section Elements	Median Width	Depressed			2 m Minimum
Cross-Section Elements	Median Width	Concrete Median Barrier			N/A
Cross-Section Elements	Road Reserve				Class B: Desirable: 60 m; Reduced: 40 m
Cross-Section Elements	Road Reserve				Class C: Desirable: 40 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	Backslope		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

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

Alignment Elements		Ref. Section	110	100	90	80	70	60	50
Design Speed (km/hr)			**110**	**100**	**90**	**80**	**70**	**60**	**50**
Stopping Sight Distance (m)	g: 0%		245	205	170	140	110	85	65
Stopping Sight Distance (m)	g: 5%		285	235	195	155	120	95	70
Stopping Sight Distance (m)	g: 10%		330	280	230	180	140	105	75
Passing Sight Distance (m)	Desirable		703	670	615	550	485	420	345
Passing Sight Distance (m)	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
Minimum Horizontal Curve Radius (m)	SE: 6%		595	465	355	265	185	130	85
Minimum Horizontal Curve Radius (m)	SE: 8%		530	415	320	240	170	120	80
Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %						
Maximum Grades	Rolling		Desirable: 4 %; Absolute: 6 %						
Maximum Grades	Mountainous		Desirable: 6 %; Absolute: 8 %						
Minimum Grades			0.5 %						
Vertical Curve (K-values)	Crest		140	100	67	45	30	17	10
Vertical Curve (K-values)	Sag (Comfort)		30	25	20	16	12	9	6.5
Vertical Curve (K-values)	Sag (Headlights)		60	50	40	32	25	19	14

Table 14.7 Design Standard DR4 – Rural Single Carriageway Class B, C

	Design Element			Ref. Section	Requirement/Specification						
	Applicable Functional Class				**Class B, Class C**						
Design Controls	Design Life				20 years						
	Design Traffic (AADT or DHV)				500 < AADT < 2000						
	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				Partial						
	Level of Service Threshold				A						
Cross-Section Elements	Typical Cross-section				R5, R6,						
	Lane Width				R5 - 3.5 m, R6 – 3.25 m						
	No. of Lanes each Direction				1						
	Outside Shoulder Width	Paved			R5 – 2.0 m, R6 – 1.5 m						
		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: 60 m; Reduced: 40 m						
					Class : 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 14.8 Design Standard DR5 – Rural Single Carriageway Class C

	Design Element			Ref. Section	Requirement/Specification			
	Applicable Functional Class				Class C			
Design Controls	Design Life				20 years			
Design Controls	Design Traffic (AADT)				500 < AADT < 2000			
Design Controls	Design Speed (Km/Hr)				Flat	Rolling	Mountainous	Escarpment
Design Controls		Class C			100 - 80	90 - 65	70 - 50	50
Design Controls	Control of Access				Partial			
Design Controls	Level of Service Threshold				A			
Cross-Section Elements	Typical Cross-section				R5, R6,			
Cross-Section Elements	Lane Width				R5 - 3.5 m, R6 – 3.25 m			
Cross-Section Elements	No. of Lanes each Direction				1			
Cross-Section Elements	Outside Shoulder Width	Paved			R5 – 2.0 m, R6 – 1.5 m			
Cross-Section Elements		Un-paved			N/A			
Cross-Section Elements	Median Shoulder Width	Paved			N/A			
Cross-Section Elements		Un-paved			N/A			
Cross-Section Elements	Crossfall Slope	Travel Lane			2.5 %			
Cross-Section Elements		Shoulder			N/A			
Cross-Section Elements	Auxiliary Lanes	Lane Width			N/A			
Cross-Section Elements		Shoulder Width			N/A			
Cross-Section Elements	Median Width	Depressed			N/A			
Cross-Section Elements		Concrete Median Barrier			N/A			
Cross-Section Elements	Road Reserve				Desirable: 40 m; Reduced: 40 m			
Cross-Section Elements	Roadside Clear Zone	Low Traffic (Fill/Cut)			5 m; 3 m			
Cross-Section Elements		Medium Traffic (Fill/Cut)			8.5 m; 5.5 m			
Cross-Section Elements	Slopes	Cut	Cut Slope		1:4			
Cross-Section Elements			Depth of Ditch		0.75 m			
Cross-Section Elements			Backslope		1:3			
Cross-Section Elements		Fill	Safety Slope (within clear zone)		1:4			
Cross-Section Elements			Fill Slope (outside clear zone)		1:3			

	Design Element		Ref. Section						
Alignment Elements	**Design Speed (km/hr)**			**100**	**90**	**80**	**70**	**60**	**50**
Alignment Elements	Stopping Sight Distance (m)	g: 0%		205	170	140	110	85	65
Alignment Elements		g: 5%		235	195	155	120	95	70
Alignment Elements		g: 10%		280	230	180	140	105	75
Alignment Elements	Passing Sight Distance (m)	Desirable		670	615	550	485	420	345
Alignment Elements		Minimum		310	275	240	210	180	155
Alignment Elements	% Passing Opportunity			33		25			
Alignment Elements	Maximum Super-elevation Rate %			8	8	8	8	8	8
Alignment Elements	Minimum Horizontal Curve Radius (m)	SE: 4%		525	400	300	205	140	95
Alignment Elements		SE: 6%		465	355	265	185	130	85
Alignment Elements		SE: 8%		415	320	240	170	120	80
Alignment Elements	Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %					
Alignment Elements		Rolling		Desirable: 4 %; Absolute: 6 %					
Alignment Elements		Mountainous		Desirable: 6 %; Absolute: 8 %					
Alignment Elements	Minimum Grades			0.5 %					
Alignment Elements	Vertical Curve (K-values)	Crest		100	67	45	30	17	10
Alignment Elements		Sag (Comfort)		25	20	16	12	9	6.5
Alignment Elements		Sag (Headlights)		50	40	32	25	19	14

14.5 Typical Cross-sections for Interurban Roads

Typical cross-sections for interurban roads in Kenya are as shown in Figure 14.1 to Figure 14.3. The designer shall use these as a guide and refine the same for specific design requirements of individual projects.

Figure 14.1 Interurban Multi-lane Road Type R1

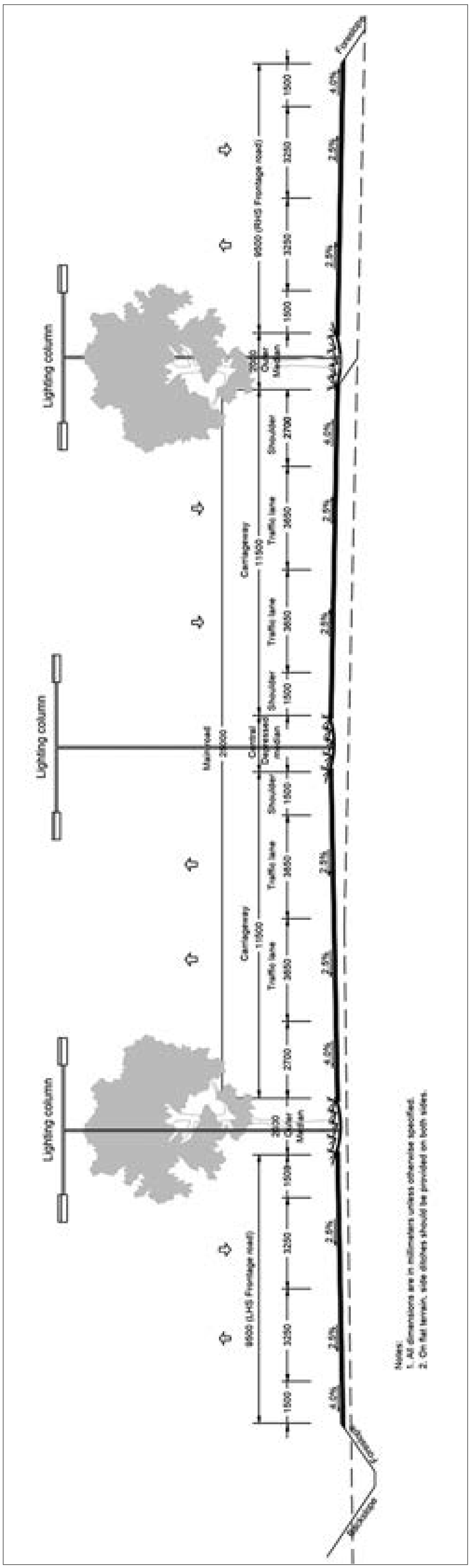


Figure 14.2 Dual Carriageway Type R3

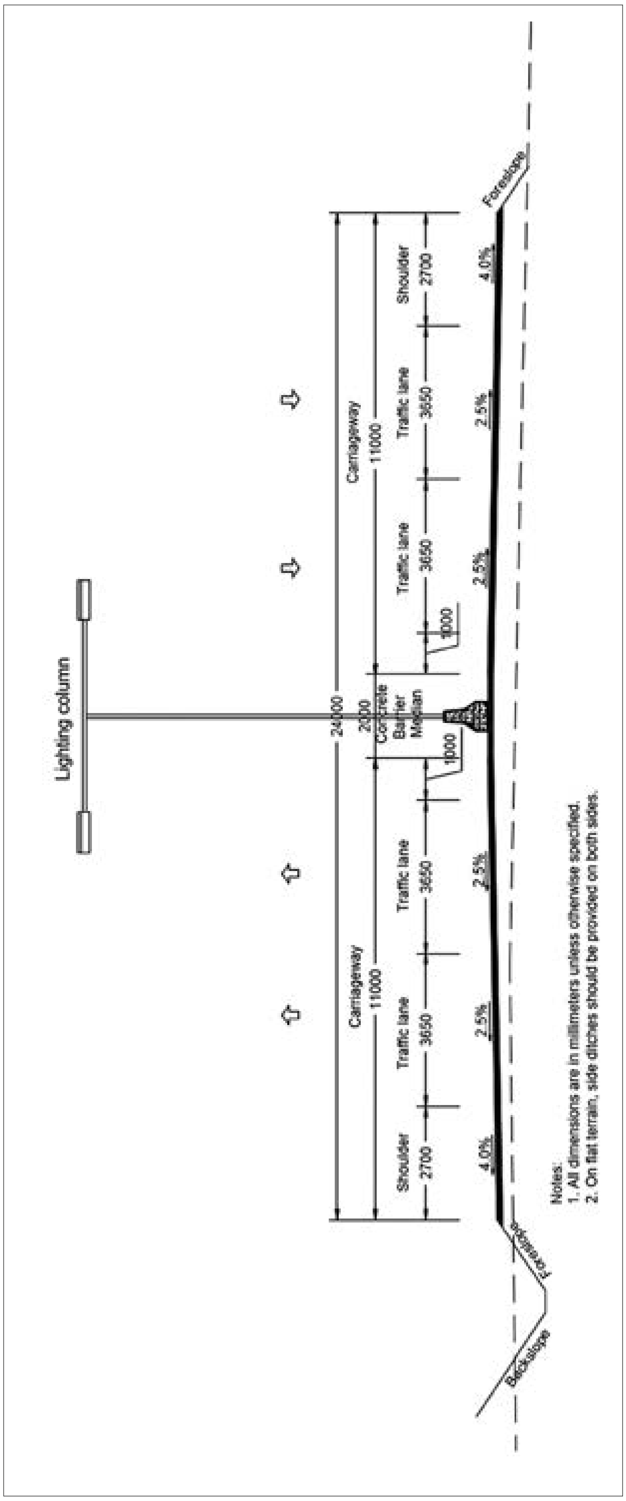
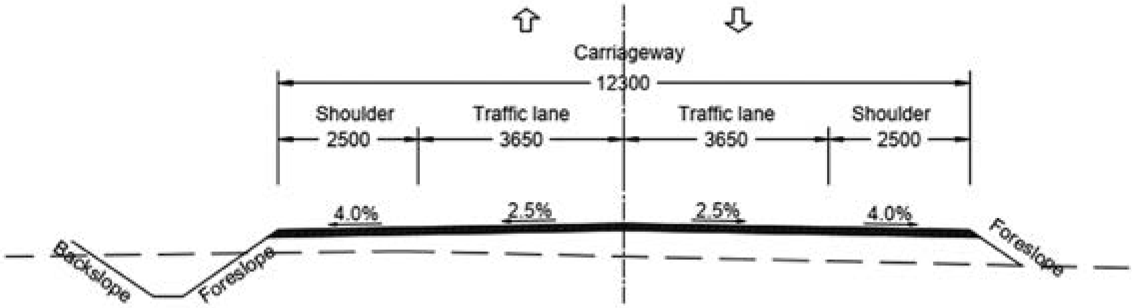


Figure 14.3 Single Carriageway Type R4



Notes:
1. All dimensions are in millimeters unless otherwise specified.
2. On flat terrain, side ditches should be provided on both sides.

15 Design of Rural Roads

15.1 General

This chapter provides guidance to the designer for specifics that are particular to rural roads in Kenya.

Chapters 1 to 13 of this manual provide all aspects of geometric design that are cross-cutting amongst all functional road classes in Kenya. The designer shall refer to these Chapters for more information.

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 walkways where pedestrian traffic volumes are low, and walkway provided within sections of high pedestrian activity.

The applicable design standards for design of rural roads are as shown in Table 15.1. Derivation and assignment of these typical design standards to classified roads in Kenya is outlined in Chapter 3.

Table 15.1 Applicable Design Standards for Rural Roads

Functional Classification	Applicable Design Standards
D	DR4, DR5, DR6
E	DR6, DR7
F, K, G,K,L,P,R,S, W (Minor Roads)	DR7

15.2 Design Controls and Criteria

15.2.1 Design Vehicle

For rural roads in Kenya, the DV6 design vehicle, which is a truck and trailer combination as detailed in Table 3.3, shall be used for all conditions except for very severe escarpment terrain.

15.2.2 Design Speed

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 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.

The design speeds for rural roads for the recommended typical standards are shown in Table 15.2.

Table 15.2 Design Speeds for Rural Roads

<table>
<tr><th rowspan="2">Design Standards</th><th colspan="4">Maximum Design Speed (Km/H)</th><th colspan="3" rowspan="2">Application to Functional Classes</th></tr>
<tr><th>Flat</th><th>Rolling</th><th>Mountainous</th><th>Escarpment</th></tr>
<tr><td>DR4</td><td>90</td><td>85</td><td>60</td><td>50</td><td rowspan="3">D</td><td></td><td></td></tr>
<tr><td>DR5</td><td>80</td><td>65</td><td>50</td><td>40</td><td></td><td></td></tr>
<tr><td>DR6</td><td>70</td><td>65</td><td>50</td><td>40</td><td rowspan="2">E</td><td></td></tr>
<tr><td>DR7</td><td>50</td><td>40</td><td>30</td><td>30</td><td></td><td>E to W</td></tr>
</table>

15.2.3 Passenger Car Units

In determining traffic volumes the appropriate passenger car unit values for different types of vehicles for rural roads shall be as shown in Table 3.7.

15.2.4 Road Capacity

Design capacities for two-way rural roads are shown in Table 15.3.

Table 15.3 Practical Design Capacities of 2-Lane Single Carriageway 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%	400
			80%	360
			60%	300
2000	80	90 - 100	100%	800
			80%	700
			60%	600
			40%	480
2000	65	80	100%	1120
			80%	1060
			60%	940
			40%	760
			20%	560
2000	55	70	100%	1340
			80%	1240
			60%	1140
			40%	1040
			20%	880

15.2.5 Level of Service

The quality of service provided by a specific road section under specific conditions is described as the Level of Service (LoS).

As detailed in Chapter 3, 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.

Rural roads in Kenya shall be designed for Level of Service E.

15.3 Design Catalogues for Rural Roads

The following deign catalogues summarise the design controls, cross-sectional elements, horizontal alignment characteristics and other elements for rural roads:

Table 15.4: Design Standard DR4 – Rural Single Carriageway Class D

Table 15.5: Design Standard DR5 – Rural Single Carriageway Class D, E

Table 15.6: Design Standard DR6 – Rural Single Carriageway Class D, E

Table 15.7: Design Standard DR7 – Rural Minor Roads Class E, F, G, K, L, P, R, S, T, W

The designer shall use the catalogues as guides in preparing the design for each particular situation. It shall be the designer's responsibility to check and refine the outcome of the application of the guides against the design principles given in this manual, as well as good practice in general.

Table 15.4 Design Standard DR4 – Rural Single Carriageway Class D

	Design Element			Ref. Section	Requirement/Specification					
	Applicable Functional Class				**Class D**					
Design Controls	Design Life				20 years					
	Design Traffic (DHV)				150 < AADT < 2000					
	Design Speed (Km/Hr)				Flat	Rolling	Mountainous	Escarp-ment		
					90 – 70	85 – 65	60 – 50	50 - 40		
	Control of Access				Partial					
	Level of Service Threshold				A					
Cross-Section Elements	Typical Cross-section				R6, R7					
	Lane Width				3.0 m					
	No. of Lanes each Direction				N/A					
	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				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					
Alignment Elements	**Design Speed (km/hr)**				**90**	**80**	**70**	**60**	**50**	**40**
	Stopping Sight Distance (m)	g: 0%			170	140	110	85	65	45
		g: 5%			195	155	120	95	70	47
		g: 10%			230	180	140	105	75	50
	Passing Sight Distance (m)	Desirable			615	550	485	420	345	275
		Minimum			275	240	210	180	155	135
	% Passing Opportunity				25					
	Maximum Super-elevation Rate %				8	8	8	8	8	8
	Minimum Horizontal Curve Radius (m)	SE: 4%			400	300	205	140	95	55
		SE: 6%			355	265	185	130	85	50
		SE: 8%			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			67	45	30	17	10	5
		Sag (Comfort)			20	16	12	9	6.5	4
		Sag (Headlights)			40	32	25	19	14	9

Table 15.5 Design Standard DR5 – Rural Single Carriageway Class D, E

	Design Element			Ref. Section	Requirement/Specification			
	Applicable Functional Class				**Class D, Class E**			
Design Controls	Design Life				20 years			
Design Controls	Design Traffic (AADT)				500 < AADT < 2000			
Design Controls	Design Speed (Km/Hr)				Flat	Rolling	Mountainous	Escarpment
Design Controls	Design Speed (Km/Hr)	Class D			90 – 70	85 – 65	60 – 50	50 - 40
Design Controls	Design Speed (Km/Hr)	Class E			70 - 50	65 - 40	50 – 30	30
Design Controls	Control of Access				Partial			
Design Controls	Level of Service Threshold				A			
Cross-Section Elements	Typical Cross-section				R7, R8			
Cross-Section Elements	Lane Width				Classes D, E: 3.25 m			
Cross-Section Elements	No. of Lanes each Direction				1			
Cross-Section Elements	Outside Shoulder Width	Paved			N/A			
Cross-Section Elements	Outside Shoulder Width	Un-paved			N/A			
Cross-Section Elements	Median Shoulder Width	Paved			N/A			
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			N/A			
Cross-Section Elements	Auxiliary Lanes	Lane Width			N/A			
Cross-Section Elements	Auxiliary Lanes	Shoulder Width			N/A			
Cross-Section Elements	Median Width	Depressed			N/A			
Cross-Section Elements	Median Width	Concrete Median Barrier			N/A			
Cross-Section Elements	Road Reserve				Class D: Desirable: 25 m; Reduced: 25 m			
Cross-Section Elements	Road Reserve				Class E: Desirable: 20 m; Reduced: 20 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	Backslope		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			

	Design Element		Ref. Section	Requirement/Specification						
Alignment Elements	**Design Speed (km/hr)**			**90**	**80**	**70**	**60**	**50**	**40**	**30**
Alignment Elements	Stopping Sight Distance (m)	g: 0%		170	140	110	85	65	45	30
Alignment Elements	Stopping Sight Distance (m)	g: 5%		195	155	120	95	70	47	31
Alignment Elements	Stopping Sight Distance (m)	g: 10%		230	180	140	105	75	50	33
Alignment Elements	Passing Sight Distance (m)	Desirable		615	550	485	420	345	275	195
Alignment Elements	Passing Sight Distance (m)	Minimum		275	240	210	180	155	135	115
Alignment Elements	% Passing Opportunity				25					
Alignment Elements	Maximum Super-elevation Rate %			8	8	8	8	8	8	8
Alignment Elements	Minimum Horizontal Curve Radius (m)	SE: 4%		400	300	205	140	95	55	30
Alignment Elements	Minimum Horizontal Curve Radius (m)	SE: 6%		355	265	185	130	85	50	26
Alignment Elements	Minimum Horizontal Curve Radius (m)	SE: 8%		320	240	170	120	80	45	24
Alignment Elements	Maximum Grades	Flat		Desirable: 3 %; Absolute: 5 %						
Alignment Elements	Maximum Grades	Rolling		Desirable: 4 %; Absolute: 6 %						
Alignment Elements	Maximum Grades	Mountainous		Desirable: 6 %; Absolute: 8 %						
Alignment Elements	Minimum Grades			0.5 %						
Alignment Elements	Vertical Curve (K-values)	Crest		67	45	30	17	10	5	2
Alignment Elements	Vertical Curve (K-values)	Sag (Comfort)		20	16	12	9	6.5	4	2.5
Alignment Elements	Vertical Curve (K-values)	Sag (Headlights)		40	32	25	19	14	9	5

Table 15.6 Design Standard DR6 – Rural Single Carriageway Class D, E

	Design Element			Ref. Section	Requirement/Specification							
	Applicable Functional Class				**Class D, Class E**							
Design Controls	Design Life				20 years							
	Design Traffic (AADT)				500 < AADT < 2000							
	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	
	Control of Access				None							
	Level of Service Threshold				A							
Cross-Section Elements	Typical Cross-section				R6, R7							
	Lane Width				Class D: 3.5 m; Class E: 3.25 m – 3.5 m							
	No. of Lanes each Direction				1							
	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							
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 15.7 Design Standard DR7 – Rural Minor Roads Class E, F, G, K, L, P, R, S, T, W

	Design Element			Ref. Section	Requirement/Specification							
	Applicable Functional Class				**Classes E, F, G, K, L, P, R, S, T, W**							
Design Controls	Design Life				20 years							
	Design Traffic (AADT)				< 50							
	Design Speed (Km/Hr)				Flat	Rolling	Mountainous	Escarpment				
		Class E to W			70 - 50	65 - 40	50 – 30	30				
	Control of Access				None							
	Level of Service Threshold				A							
Cross-Section Elements	Typical Cross-section				R8, R9							
	Lane Width				3.0 m							
	No. of Lanes each Direction				1							
	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							
Alignment Elements	**Design Speed (km/hr)**					**70**	**60**	**50**	**40**	**30**	**40**	**30**
	Stopping Sight Distance (m)	g: 0%				110	85	65	45	30	45	30
		g: 5%				120	95	70	47	31	47	31
		g: 10%				140	105	75	50	33	50	33
	Passing Sight Distance (m)	Desirable				485	420	345	275	195	275	195
		Minimum				210	180	155	135	115	135	115
	% Passing Opportunity											
	Maximum Super-elevation Rate %					8	8	8	8	8	8	8
	Minimum Horizontal Curve Radius (m)	SE: 4%				205	140	95	55	30	55	30
		SE: 6%				185	130	85	50	26	50	26
		SE: 8%				170	120	80	45	24	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				30	17	10	5	2	5	2
		Sag (Comfort)				12	9	6.5	4	2.5	4	2.5
		Sag (Headlights)				25	19	14	9	5	9	5

15.4 Typical Cross-sections for Rural Roads

Typical cross-sections for rural roads in Kenya are as shown in Figure 15.1 to Figure 15.4. The designer shall use these as a guide and refine the same for specific design requirements of individual projects.

Figure 15.1 Single Carriageway Type R6

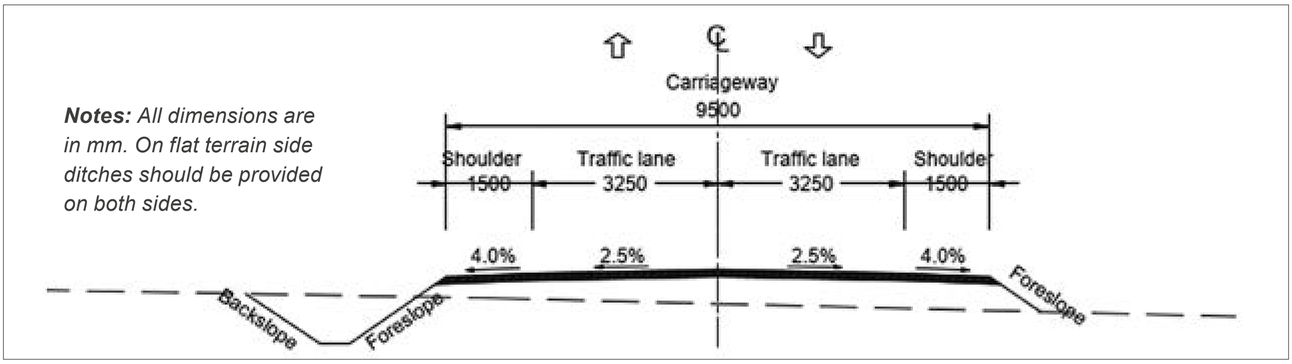


Figure 15.2 Single Carriageway Type R6 with Super-elevation

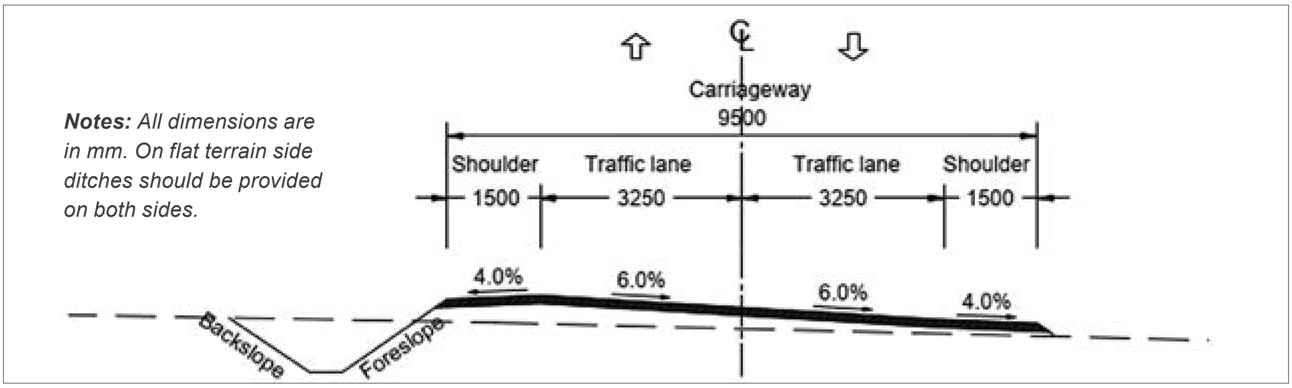


Figure 15.3 Single Carriageway Type R7

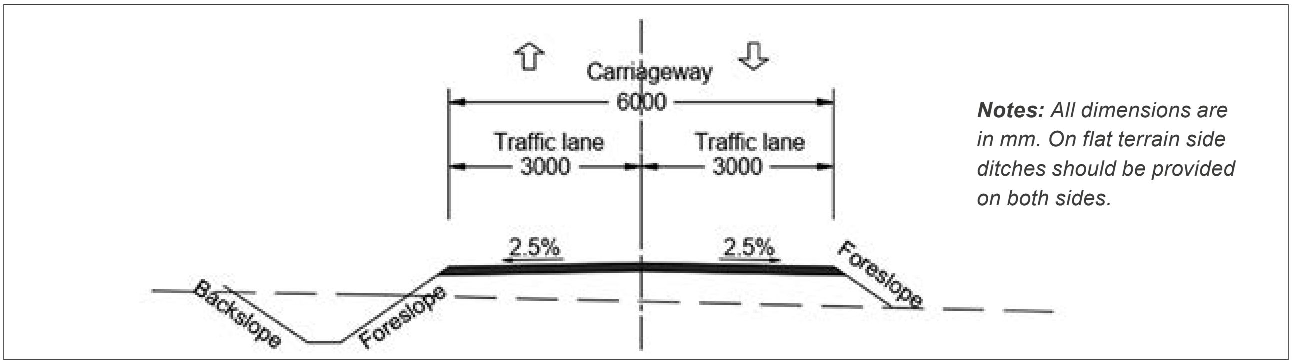
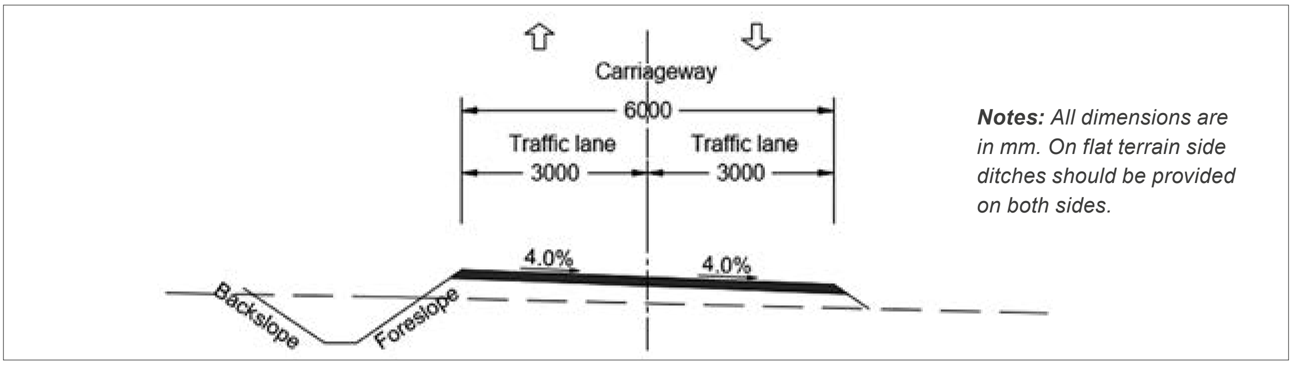


Figure 15.4 Single Carriageway Type R7 with Super-elevation



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Design of Urban Arterial and Collector Roads

16 Design of Urban Arterial and Collector Roads

16.1 Introduction

16.1.1 General

This chapter provides guidance to the designer for specifics that are particular to urban arterial and collector roads in Kenya.

Chapters 1 to 13 of this manual provide all aspects of geometric design that are cross-cutting amongst all functional road classes in Kenya.

The designer shall refer to these Chapters for more information and guidance as necessary.

16.1.2 Functions of Urban Arterials and Collectors

On the one hand, the prime function of urban arterial roads is the movement of traffic, and these roads should primarily cater for long distance movements in the urban system and through traffic. The arterial roads carry most of the trips entering or leaving the conurbation. In the urban areas, these roads provide continuity for major rural arterials that intercept the urban boundary.

Access to abutting land is subordinate to the free movement of traffic.

The design standards recommended for the urban arterial roads may be of too high for the smaller towns and those recommended for the urban collector roads may be suitable.

On the other hand, the function of the collector roads is to carry traffic from the local roads and streets to the arterial network. They will function both to serve the movement of traffic and provide access to local roads and adjacent properties. Although they may link adjoining districts, they are not intended for long cross-town journeys for which the arterial network should provide a more attractive alternative. Their prime function is to facilitate the safe and unhindered movement of traffic in the districts they serve. These roads place more emphasis on land access than arterial roads and offer lower mobility.

The main functions served by urban arterials and collectors in Kenya are as given in Table 16.1 below and design standards have been developed to ensure efficient fulfilment of those functions.

Table 16.1 Functions of Urban Arterial, Collector and Local Roads

Road Class	Main road user	Access	Transit traffic	Use by public transport	Operating speeds (Km/hr)
Urban arterial (UA)	Motorised traffic	No access to properties, no link to local access roads,	Serve transit traffic	Allowed, carries the main bus routes	100
Major collector (UC)	Mixed: Motorised traffic, NMT	Limited access to properties, serve as link between arterials	No transit traffic allowed	Carry main bus routes	70
Minor collector (UC)	Mixed: Motorised traffic, NMT, pedestrians	Access to properties	No transit traffic	Buses not allowed	50
Local Roads/ Streets	Mixed: Motorised traffic, NMT, pedestrians	Access to properties	No transit traffic	Buses, HGVs not allowed	15-30
Service Roads	Mixed: Motorised traffic, NMT, pedestrians	Access to properties	No transit traffic	Buses not allowed	10

16.1.3 Design Standards

The applicable design standards for design urban arterial and collector roads are as indicated in Table 16.2.

Development of the standards is explained in Chapter 3. These typical standards are intended to be used as a guide by the designer.

Table 16.2 Applicable Design Standards for Urban Arterial and Collector Roads

Functional Classification	Applicable Design Standards
UA	DU1
UC	DU2, DU3, DU4

16.2 Design Controls and Criteria

16.2.1 General

Reference shall be made to Chapter 3 for details and guidance on all aspects of design controls and criteria for all roads in Kenya.

Below is a treatise of controls and criteria applying to urban arterial and collector roads, in particular.

16.2.2 Road Reserve Width

The road reserve widths given in Table 16.3 shall be adopted for arterial and collector road corridors in urban areas. These align with the normal road widths that are available in Kenyan urban road contexts.

Table 16.3 Road Reserve Widths for Urban Arterial and Collector Roads

Functional Class	Desirable	Reduced
UA	30-40	30
UC	18-25	18

16.2.3 Design Vehicles

In an urban setting the design vehicle shall be defined for the various design elements inclusive of NMT and PT facilities.

For urban arterial and collector roads the passenger car should be used for speed-related standards and the medium heavy goods vehicle of configuration DV6 as given in Chapter 3, Table 3.3 shall be adopted for standards relating to manoeuvrability, typically at intersections.

For areas designated for use by NMT's only the heaviest vehicle in that class, which is the cart, with an approximate weight of 1 tonne, shall be adopted.

Table 16.4 gives the dimensional details of NMT's design vehicles which shall be adopted for urban collector and arterial roads as well.

Table 16.4 Pedestrian and NMT Dimensional Details

Vehicle/road User	Width (m)	Length (m)	Sideways Clearance (m)	Turning Circle Radii (m)
Pedestrian	0.7	1.0	0.3	1.5
Bicycle	0.75	2.0	0.5	6.0
Bicycle with goods	1.0	3.0	0.8	6.0
Cart	1.60	3.0	0.8	4.0

16.2.4 Design Speed

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 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**.

Kenyan cities continue to experience significant road safety challenges. One of the key causes is the design of urban roads that prioritises high operational speeds, which can create unwalkable environments and pose risks to pedestrians and cyclists.

The design speeds of up to 100 km/h dictated by regional and international practice are excessive in the context of urban roads and have warranted lowering.

The design speeds for open road travel for urban arterial and collector roads, for the typical standards, are therefore as shown in Table 16.5 and are stratified by functional class.

Table 16.5 Design Speeds for Urban Arterial And Collector Roads

<table>
<tr><th>Design Standards</th><th>Desirable design speed (km/h)</th><th>Reduced design speed (km/h)</th><th colspan="2">Application to Functional Classes</th></tr>
<tr><td>DU1</td><td>100</td><td>50</td><td rowspan="2">UA</td><td></td></tr>
<tr><td>DU2</td><td>80</td><td>40</td><td rowspan="3">UC</td></tr>
<tr><td>DU3</td><td>70</td><td>30</td><td></td></tr>
<tr><td>DU4</td><td>50</td><td>30</td><td></td></tr>
</table>

For the design of turning spaces and intersection corners design speeds of 10-15 km/hr shall be adopted.

16.2.5 Passenger Car Units

In the process of determining design volumes for arterial and collector roads, the appropriate passenger car unit values for different types of vehicles under different conditions shown in Table 16.6 shall be adopted.

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Design of Urban Arterial and Collector Roads

Table 16.6 Passenger Car Units Applicable To Urban Roads

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

16.2.6 Determining Motorised Traffic Volumes

Traffic data for use in determining traffic volumes shall be obtained from field surveys undertaken in accordance with the **Road Design Manual Volume 1: Geometric Design of Highways, Rural and Urban Roads, Part 2 - Traffic Surveys**.

The design of main traffic routes in urban areas shall 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.

Therefore, in urban area design, the 30th highest hourly volume can be a reasonable representation of daily peak hours during the year.

The designer is referred to Chapter 1 for detailed explanation of the derivation of the Design Hourly Volume (DHV) for urban roads in general.

16.2.7 Road Capacity and Level of Service

The design of any roadway and its features should explicitly consider traffic volumes, operational performance, and user characteristics for all transportation modes.

The designer shall determine the capacity of the road under consideration and this will be the maximum hourly rate at which 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.

Knowing the anticipated capacity of the highway the designer shall provide for features of the road to fit the requirements of the estimated traffic, selecting the best road type, dimensions of lanes, shoulders, weaving sections, geometry, etc.

Design capacities for two-way and one-way urban roads are shown in Table 16.7 and Table 16.8.

Table 16.7 Practical Capacities of Two-way Urban Roads

Effective Width of Carriageway	2 Lane		3 Lane	4 Lane	6 Lane
	6 - 7.0 m	7.3 m	10 – 11 m	14 – 14.5 m	20 m
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 16.8 Practical Capacities of One-way Urban Roads

Effective Width of Carriageway excluding refuges, or central; reservation	2 Lane		3 Lane	4 Lane	Comments
	5.5 m	7.0-7.3 m	10-11 m	14-14.5 m	
Description	Capacity in PCU's per hour				
Urban expressways 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.

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16.2.8 Pedestrian Walkway Capacity

The recommended walkway capacities in each direction for walkways are given in Table 16.9.

Table 16.9 Walkways Capacity in One Direction

Type of Flow	Walking Speed (km/hr)	Capacity (persons/m/hr)	Level of Service
Uninterrupted flow	5.0	1000	**A** - Good
Slightly interrupted flow	4 – 5	2000	**B** - Acceptable
Disturbed flow	3 – 4	2500	**C** - Poor
Highly disturbed flow	2 – 3	3000	**D** - Unacceptable

Source: *Productive and Liveable Cities, SSATP, 2001*

16.2.9 Carts

Cart paths shall only be provided in locations of close proximity to markets and their movement shall be separated from pedestrian movement.

Where cart use in prevalent and there is a tendency for mixing with pedestrians, walkways shall be constructed to a width of 3 m as a minimum.

16.3 Cross Section

16.3.1 General

The designer shall refer to Chapter 4 for guidance and details on the selection of cross-sectional elements of all roads in Kenya, in general.

16.3.2 Traffic Lane Widths

No other features of the highway design have a greater influence on the safety and comfort of driving than the lane width and the condition of the surface. It is crucial to prioritise the safety of all road users in urban environments and offer flexibility in cases where the right-of-way is limited.

Lane widths of 2.5 m to 3.6 m are generally used.

For the sake of safety, efficiency and ease of operation, the width of the traffic lanes should be chosen with particular regard to the type, volume and speed of the traffic using the road. Where the volume of cyclists is high and no separate cycle track is provided, consideration should be given to increasing the width of the outside lanes by about one metre.

The trafficked lane widths for urban arterial and collector roads shall be as given in Table 16.10.

Table 16.10 Traffic Lane Widths for Urban Roads

Road category	Most common functional class and type	Surface type	Lane width range
Urban Roads	UA (urban arterial roads)	Paved	**Multi-lane or Dual carriageway:** (3.0 m – 3.75 m lane width)
	UC (urban collector roads)	Paved	**Single carriageway:** (3.0 m – 3.65 m lane width)
	UL (local urban streets)	Paved	**Single carriageway:** (3.0 m – 3.65 m lane width)

16.3.3 Shoulders

On rural roads a shoulder is necessary to provide lateral structural support to the pavement, to accommodate broken down vehicles and for emergency use. However, in the urban environment, the shoulder is often omitted as kerbs and channels usually provide structural support.

On arterial and collector roads where traffic volumes are high, shoulders may be provided for emergency or refuge purposes. The shoulders should be surfaced, preferably with a material that contrasts with the main carriageway in colour and texture, in order to protect the pavement under the carriageway from the ingress of water and erosion. Consideration should be given to constructing rumble strips on the shoulder in order to discourage drivers from using the shoulders as an additional traffic lane.

Some of the more important advantages of shoulders are as follows:

1. As a refuge for broken down vehicles .
2. As a place to rest, consult maps, etc.
3. As a space for road maintenance equipment and personnel.
4. Stormwater is collected further away from the road pavement, thus reducing the ingress of moisture under the trafficked lane.
5. As an additional area to avoid potential accidents.
6. Sight distance is improved.
7. Road capacity is increased; uniform speed in encouraged.
8. Lateral clearance is provided for signs and guard-rails.
9. Space is provided for cyclists and pedestrians.
10. The driver feels less constrained thus reducing stress.

The minimum shoulder width should be 0.6 m for the lightly trafficked roads, but the desirable width should be 1.5 to 2.5 m.

It may be impractical to provide shoulders because of land constraints. In such cases consideration should be given to providing emergency stopping bays at suitable intervals (500 m).

Shoulders should drain away from the road and should be constructed with a camber similar to or slightly greater than that of the adjacent road.

16.3.4 Kerbs

Kerbs should normally be light-coloured and should be clearly distinguishable from other parts of the road and walkway, by day or night and in wet or dry weather.

Kerbs are intended to achieve one or more of the following:

1. Indicate to the driver the edge of the running surface.
2. Protect the edge of the pavement construction from damage.
3. Control surface water run-off.

The following types of kerb are in general use in Kenya:

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
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1. **Half Battered/non-mountable Kerbs**

   This kerb has a high upstand that is intended to prevent vehicles from mounting it. It is used to separate the vehicle running surface from pedestrian or no parking areas and should be provided where walkways or cycle tracks are adjacent to the trafficked lane.

2. **Battered/semi-mountable**

   The semi-mountable kerb has a splay face that permits a vehicle moving with substantial lateral force to mount it without undue vehicle damage. This type of kerb would be suitable where vehicles are not encouraged to leave the carriageway, but where it would be safer for a wayward vehicle to leave the carriageway rather than deflect across the carriageway into oncoming traffic.

3. **Mountable**

   The low profile mountable kerb is intended to allow a vehicle to mount it with ease without damage to the vehicle or the kerb. This type of kerb is suitable for use at intersections where turning radii are relatively small and it is acceptable that vehicles will occasionally cross the line of the kerb.

4. **Flush/block**

   These consist of plane channel blocks approximately 100 x 125 mm in size.

   This type of kerb is intended to demarcate the edge of the running surface or shoulder and protect the edge of the pavement construction without preventing vehicles or NMT from crossing it. It would be used where vehicles commonly leave the carriageway or in conjunction with an upstand kerb as part of a drainage channel. The use of flush concrete blocks to demarcate the edge of cycle tracks or walkways is usually expensive and their economic justification needs to be considered carefully.

When incorporating kerbs in a road design, it should be remembered that upstand type kerbs reduce the effective width of the running surface since drivers tend to maintain a safe distance between the vehicle and the kerb.

16.4 At-Grade Intersections

Reference shall be made to Chapter 9 for guidance on the design of at-grade intersections for all road in general.

Intersection geometry in urban areas must accommodate the needs of a diverse range of vehicles and NMT users commonly found in such environments. Intersection design should facilitate visibility and predictability for all users, creating an environment in which complex movements feel safe, easy, and intuitive.

In particular:

1. Pedestrian crossings shall be provided on all legs of all signalised intersections.
2. Pedestrian crossings must be aligned with the general crossing desire lines for the envisaged users.
3. It is desirable to provide 5m wide pedestrian crossings, the minimum allowable shall be 3m.
4. All cycle track crossings shall have a minimum width of 2 m.
5. Raised crosswalks (tabletop crossings) shall be set out at a level of +150 mm above finished road level at un-signalised zebra crossings.
6. Refuge islands shall be provided where there are more than three lanes in total to be crossed.
7. Provide protected crossings for bicycles through intersections.
8. 'Median tips' shall be provided where there is a zebra crossing and a median at an intersection.
9. Stop lines perpendicular to the travel lane and set back at least 3 m from the zebra crossings shall be marked clearly.

16.5 Roadside Facilities and Amenities

16.5.1 General

Reference shall be made to Chapter 13 for guidance and details on roadside facilities and amenities for urban arterial and collector roads.

16.5.2 Bus Stops

The following considerations shall be taken into account in providing public service vehicle stopping places on urban roads:

1. PSV stops should be provided at intervals of 200-400 m, depending on the level of public transport demand.
2. The minimum width of the bay shall me 2.5 m.
3. On a street with two or more carriageway lanes per direction, the PSV stop should be placed adjacent to the vehicle's line of travel so that the PSV does not need to pull over.
4. On a street with one carriageway lane per direction or at terminal locations, the stop may incorporate a bus bay provided that there is sufficient clear space for walking behind the shelter.
5. Shelter with adequate lighting, protection from sun and rain, and customer information.
6. Cycle tracks should be routed behind bus shelters.

16.5.3 Passenger Service Vehicle Terminals

The following standards shall be adopted for provision of terminals for public passenger vehicles:

1. PSV terminals shall be designed to ensure that the entire facility offers universal access.
2. Minimal walking distances and vertical displacement between platforms is necessary to avoid user discomfort.
3. Bottleneck shall be avoided, wider spaces where different pedestrian streams intersect must be provided.
4. User of the terminals must be protected from the elements such as the sun, rain and wind. Roofs should extend above buses in loading bays.
5. PSV terminals must contain ample seating for waiting passengers and other users.
6. Complete customer information, including real-time departure information should be easily available.
7. Full provision must be made for adequate lighting to enhance safety and security for passengers.
8. Public toilets must be easily accessible for all classes of users and must include baby changing facilities.
9. Organised vending kiosks are a requirement, priority being given to those that provide food and drink.
10. Special provision must be made for cycle parking facilities.

16.5.4 Parking

16.5.4.1 General

Parking availability and characteristics can strongly influence a driver's choice of destination. The success of shopping centres is often attributed to the provision of large areas for free car parking near the shops. Adequate parking may be an important factor in securing the future economic viability of a city centre, provided that the availability of parking does not attract more traffic into the centre than the road network can accommodate. Drivers will also consider security when choosing where to park. Because of the role it can play, policy on parking is normally an integral part of the local town planning scheme and it is essential that this policy is known to the designer early in the planning process.

16.5.4.2 Parking Demand

Parking demand is established on the basis of land use, floor area and employment. The following criteria may be used as a rough guide:

1. On the basis of land use, the designer should provide:
   a. One (1) parking space per 50 sq. m for offices.
   b. One (1) parking space per 30 sq., m for shopping centres
   c. One (1) Parking space per 100 sq. m for commercial areas.
   d. 0.15 spaces per seat for places of worship.
   e. One (1) parking space per 200 sq. m for others.

OR

2. On the basis of employment, the designer should provide:
   a. For short term parking one (1) parking space per 12 employees.
   b. For long term parking one (1) parking space per 5 employees.

16.5.4.3 Longitudinal Arrangement

This arrangement allows parking of vehicles behind one another, parallel to the kerb. Although this method uses less width of the roadway, it allows parking space for fewer vehicles in the longitudinal direction. This arrangement is recommended for narrow roads.

16.5.4.4 Inclined Arrangement (Angle Parking)

This arrangement allows a larger number of vehicles to be parked and it is more convenient for drivers.

This arrangement is recommended for roads with sufficient width.

16.6 NMT Facilities

16.6.1 Walkways

The desirable widths of walkways shall be as indicated in Table 16.11. This width represents the clear width, excluding furniture and frontage zones.

Table 16.11 Desirable Walkway Widths for Urban Arterial and Collector Roads

Situation	Desirable Clear Width (m)	Comments
General low volume	2.0	• General minimum width for most roads and streets shall be 1.5m • Clear width required for one wheelchair with 0.15 m allowance both sides.
High pedestrian volumes	3.0 (or higher based on volume)	• Generally desirable for commercial and shopping areas.

16.6.2 Pedestrian Crossings

Pedestrian crossings shall be designed well to ensure maximum road user safety for all road users. At-grade crossings for urban roads shall cater for pedestrians of all ages and PWDs, with traffic calming measures or signalisation to manage vehicle movements.

The following principles shall apply:

1. Crossings should be signalised or raised to the level of the walkway to provide universal access and traffic calming. A ramp slope of 1:8 is preferred.
2. Pedestrian crossings should match pedestrian desired travel lines.
3. If a speed hump is used, the hump should be placed 5 m before the crossing.
4. The pedestrian crossing should have a width of 5 m or equivalent width to the adjacent walkway, whichever is larger.
5. Bulb-outs should be added in parking lanes to reduce the crossing distance.
6. Footbridges should be avoided except when crossing limited-access Class A highways since they are inconvenient, inaccessible, and unsafe.

16.6.3 Footbridges

Pedestrian bridges will be required at busy junctions and other points where pedestrians need to cross the road in large numbers. As pedestrian bridges are fairly short and are intended solely for movement, they can reasonably be assumed to have higher acceptable capacity than normal walkways.

Capacity of a pedestrian bridge can be assessed on the basis of 70 persons per minute per metre width on the level and 50 persons per metre width on steps or ramps.

Where possible, bridges should have ramped approaches as well as steps to accommodate the disabled. Continuous ramps should, preferably be 1 in 16, if this is not possible than they should not be steeper than 1 in 10. Bridges should have a clear headroom of 5.5 m above the carriageway and a deck width of at least 2 m. If a central support is required, it should be well protected against vehicle damage.

16.6.4 Cycle Tracks

In general, cycle tracks should be provided on urban roads with an operational speed exceeding 30 km/h.

Table 16.12 gives guidance on widths for cycle tracks.

Table 16.12 Cycle Track Widths Guidance

Situation	Minimum Width (m)	Desirable Width (m)
General low volume	1.2	2.0
High cyclist volumes	2.0 (or higher based on volume)	3.0 (or higher based on volume)

Cycle tracks shall be designed to the following standards:

1. Preferably, cycle tracks shall be physically separated from the carriageway—as distinguished from painted cycle lanes, which offer little protection to cyclists.
2. A clear width of at least 2.0 m for one-way movement shall be provided and 3.0 m of clear width for two-way movement.
3. A smooth surface material—asphalt or concrete shall be adopted to offer acceptable levels of comfort to road users; paving blocks should be avoided.
4. Cycle tracks shall be elevated 150 mm above the carriageway.
5. In terms of positioning between the walkway and carriageway a buffer of at least 0.5 m between the cycle track and carriageway is necessary. The buffer should be paved if it is adjacent to a parking lane. The width should be increased to 0.75 m next to buildings, walls, etc.
6. Bollards to prevent encroachments by cars shall be placed in the middle of the cycle track, still allowing for cyclists to pass on either side. Bollard spacing width of 1.2 m shall be adopted.

16.7 Bus Rapid Transit Facilities (BRT)

16.7.1 General

BRT corridor design requires careful planning covering cross section designs, busway placement, intersection treatments, and station positions. BRT corridors function best if they are designed to meet the needs of all users, including public transport users, pedestrians, cyclists, and personal motor vehicle users.

The designer is referred to **NAMATA Guidelines for BRT, 2023** for information on the planning process for BRT in Kenya.

16.7.2 Design of Street Elements for BRT

BRT facilities shall be provided on high-volume collector roads only.

Various elements of a BRT corridor and their suggested dimensions are given below:

1. A BRT lane for one-way movement should have a width of 3.5 m. The width of a passing lane, where required, should be 3.5 m.
2. A divider minimum 0.5 m wide should separate BRT lanes from mixed traffic. These should be expanded to at least 1 m at street crossing points by marginally reducing carriageway and BRT lane width or utilising the space created when the cross section at a station tapers to the standard cross section.
3. Median stations that serve both directions of BRT services should have a minimum internal clear width of 3.5 m. The outer width would be at least 4 m, with wider stations provided where demand is higher.
4. The width of BRT elements, at stations without passing lanes, would be 11 m. In case of systems that require a passing lane, the total width of BRT elements at station expands to 19 m (or 15.5 m in the case of a staggered station). Since passing lanes are not required at non-station locations, the width of BRT elements drops to 8 m in both cases.

5. Walkways are essential for safe pedestrian access to BRT stations. Walkways with a minimum clear width of 2 m should be provided on either side of the carriageway. A tree line next to the walkway – with a minimum width of 1 m – should be included where adequate right-of-way is available. Existing trees should be preserved wherever possible during corridor construction.
6. Cycle tracks may be provided along the corridor for the safety and convenience of cyclists where adequate right of way is available.
7. In case of BRT, since a majority of large vehicles (buses) do not use the carriageway, the carriageway width may be reduced from 7 m to 6.0 – 6.5 m for two lanes.
8. At non-station locations along the BRT corridor, parallel parking may be provided at the edge of a carriageway or a service lane, depending on the section.

BRT requires wider cross sections at stations. Elsewhere, a multi-utility zone that provides space for on-street parking and bus stops can occupy the extra 4 m of ROW that is available between stations. Walking and cycling provide last-mile connectivity to BRT stations, and space for these modes should not be compromised in station areas. BRT lanes require physical separation to prevent entry by mixed traffic. Physical delineators should be paired with adequate signage and road markings to alert personal motor vehicle users that they may not enter the lanes.

Table 16.13 and Table 16.14 summarise the dimensional details of various BRT corridor elements. Further details may be obtained from **NAMATA Guidelines for BRT Facilities, 2023**.

Table 16.13 BRT Corridor Elements Dimensions: Widths

Street Element	Specifications	Minimum Width (m)	Maximum Width (m)
BRT	One-way lane	3.5	4.0
	Two-way lane	7.0	7.5
	Median station	4.0	*
	Passing lane at station	3.5	4.0
Buffer		0.5	*
Pedestrian refuge		1.0	*
Carriageway	Mixed traffic lane (per lane for carriageways with two or more lanes per direction)	3.0	3.5
Loading bays	Parallel orientation	2.0	2.2
Bus & BRT direct services	One-way	2.0	*
Cycle track	Two-way	3.0	*
	One-way on each side	2.0	*
Walkway	Total width including furniture zone and frontage zone	3.5	*
	Clear space	2.0	*
Tree line	Next to the walkway or in the parking lane	1.0	*

Source: *NAMATA BRT Guidelines, 2023*

Table 16.14 BRT Corridor Elements Dimensions: Heights

Street Element	Specifications	Minimum Height (mm)	Maximum Height (mm)
BRT	BRT lane between stations	0	0
	BRT lane at station	0	150
	Station	At the same height as the bus floor	
	Median between BRT lanes and carriageway	300	400
Carriageway	Raised zebra crossings	100	150
Walkway		100	150
Cycle track		100	100
Bus stop	Kerb-side bus shelter	150	150

Source: *NAMATA BRT Guidelines, 2023*

16.8 Design Catalogues for Urban Arterial and Collector Roads

The following design catalogues summarise the design controls, cross-sectional elements, horizontal alignment characteristics and other elements for urban arterial and collector roads:

Table 16.15: Design Standard DU1 – Urban Arterial Multi-lane

Table 16.16: Design Standards DU2 – Urban Arterial and Collector

Table 16.17: Design Standard DU3 – Urban Collector

Table 16.18: Design Standards DU4 – Urban Collector Road

The designer shall use the catalogues as guides in preparing the design for each situation. It shall be the designer's responsibility to check and refine the outcome of the application of the guides against the design principles given in this manual, as well as good practice in general.

Table 16.15 Design Standard DU1 – Urban Arterial Multi-lane, Class A

	Design Element			Ref. Section	Requirement/Specification	
	Applicable Functional Class				**Class A**	
Design Controls	Design Life				20 years	
Design Controls	Design Traffic (DHV)				> 18 000 per lane	
Design Controls	Design Speed (km/hr)				80 - 100	
Design Controls	Control of Access				Full	
Design Controls	Level of Service Threshold				C	
Cross-Section Elements	Typical Cross-section				DU1, DU2	
Cross-Section Elements	Lane Width				2 or 3	
Cross-Section Elements	No. of Lanes each Direction				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	
Cross-Section Elements	On Street Parking - Bay Width	Bus			3.5	
Cross-Section Elements	On Street Parking - Bay Width	Car			2.4	
Cross-Section Elements	Bus Stop – Bay Width				5 m	
Cross-Section Elements	Cycle Track Width				2 m	
Cross-Section Elements	Walkway Width				1.5 – 3.0 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	Backslope		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
Alignment Elements	Decision Sight Distance (m)				310	310
Alignment Elements	Passing Sight Distance (m)	Desirable			550	550
Alignment Elements	Passing Sight Distance (m)	Minimum			240	240
Alignment Elements	% Passing Opportunity				50	50
Alignment Elements	Maximum Super-elevation Rate %				8	8
Alignment Elements	Minimum Horizontal Curve Radius (m)	SE: 4%			300	525
Alignment Elements	Minimum Horizontal Curve Radius (m)	SE: 6%			265	465
Alignment Elements	Minimum Horizontal Curve Radius (m)	SE: 8%			240	415
Alignment Elements	Maximum Grades	Flat			Desirable: 3 %; Absolute: 5 %	
Alignment Elements	Maximum Grades	Rolling			Desirable: 4 %; Absolute: 6 %	
Alignment Elements	Maximum Grades	Mountainous			Desirable: 6 %; Absolute: 8 %	
Alignment Elements	Minimum Grades				0.5 %	
Alignment Elements	Vertical Curve (K-values)	Crest			45	100
Alignment Elements	Vertical Curve (K-values)	Sag (Comfort)			16	25
Alignment Elements	Vertical Curve (K-values)	Sag (Headlights)			32	50

Table 16.16 Design Standards DU2 – Urban Arterial and Collector

	Design Element			Ref. Section	Requirement/Specification			
	Applicable Functional Class				**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	Typical Cross-section				DU2, DU3			
	Lane Width				2			
	No. of Lanes each Direction				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 Park-ing - 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	
	Walkway 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			
			Backslope		1:3			
		Fill	Safety Slope (within clear zone)		1:4			
			Fill Slope (outside clear zone)		1:3			
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 (m)	Desirable			550	550	345	485
		Minimum			240	240	155	210
	% 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 16.17 Design Standard DU3 – Urban Collector

	Design Element			Ref. Section	Requirement/Specification	
	Applicable Functional Class				**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	Typical Cross-section				DU3, DU4	
	Lane Width				2	
	No. of Lanes each Direction				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			2 m Minimum	
		Raised Concrete			2 m 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	
	Walkway 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	
			Backslope		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 (m)	Desirable			345	485
		Minimum			155	210
	% Passing Opportunity				33	
	Maximum Super-elevation Rate %				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 16.18 Design Standard DU4 – Urban Collector Road

	Design Element			Ref. Section	Requirement/Specification			
	Applicable Functional Class				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	Typical Cross-section				DU4, DU5			
	Lane Width				1 or 2		1	
	No. of Lanes each Direction				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	Painted			0.75 m Minimum			
		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	
	Walkway Width				1.5 m		2.4 m	
	Service Road Width				3 m		N/A	
	Road Reserve				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			
			Backslope		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 (m)	Desirable			345	485	195	345
		Minimum			155	210	115	155
	% Passing Opportunity				50			
	Maximum Super-elevation Rate %				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

16.9 Typical Cross-sections

Typical cross-sections for urban arterial and collector roads in Kenya are as shown in Figure 16.1 to Figure 16.5.

The designer shall use these as a guide and refine the same for specific design requirements of individual projects.

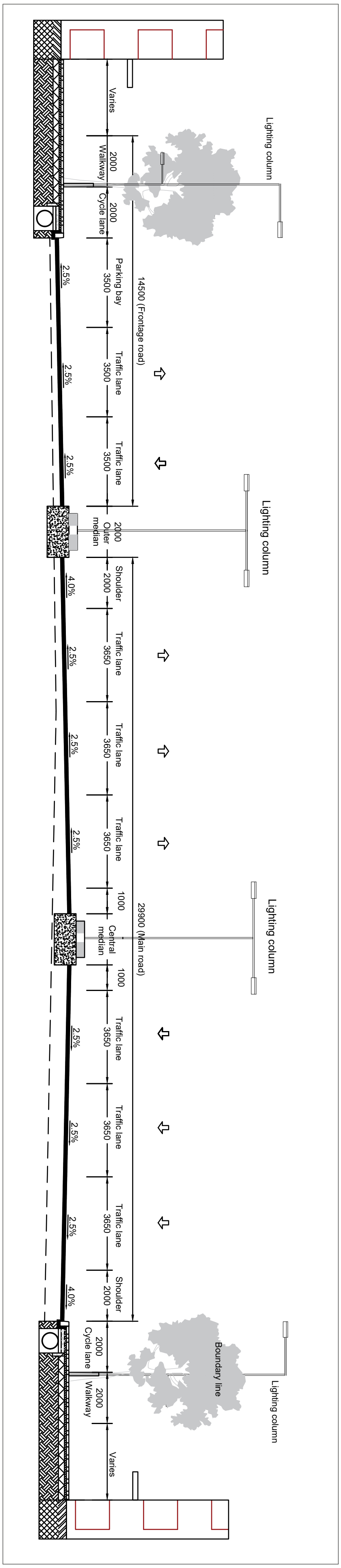

Figure 16.1 Urban Arterial Road – Type U1

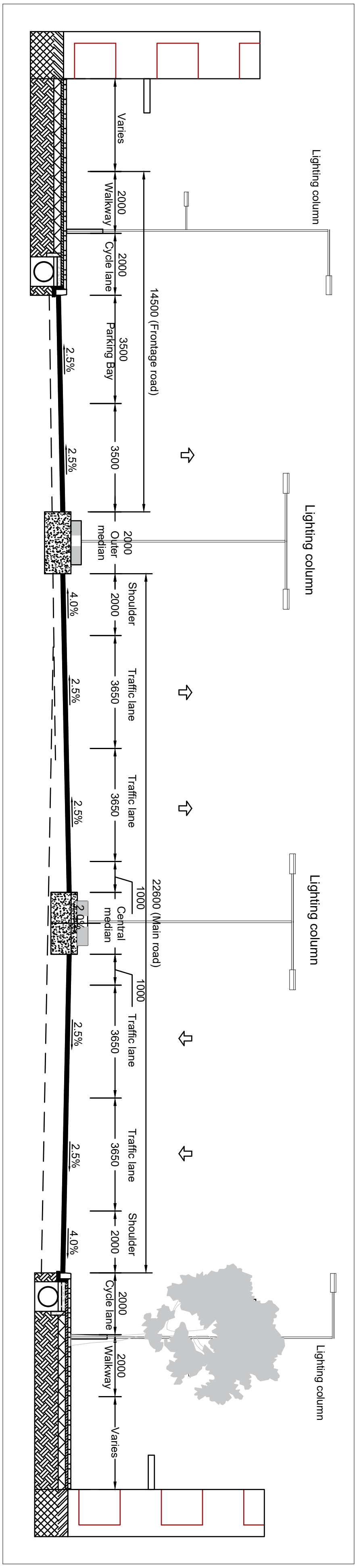

Figure 16.2 Urban Arterial Road – Type U2

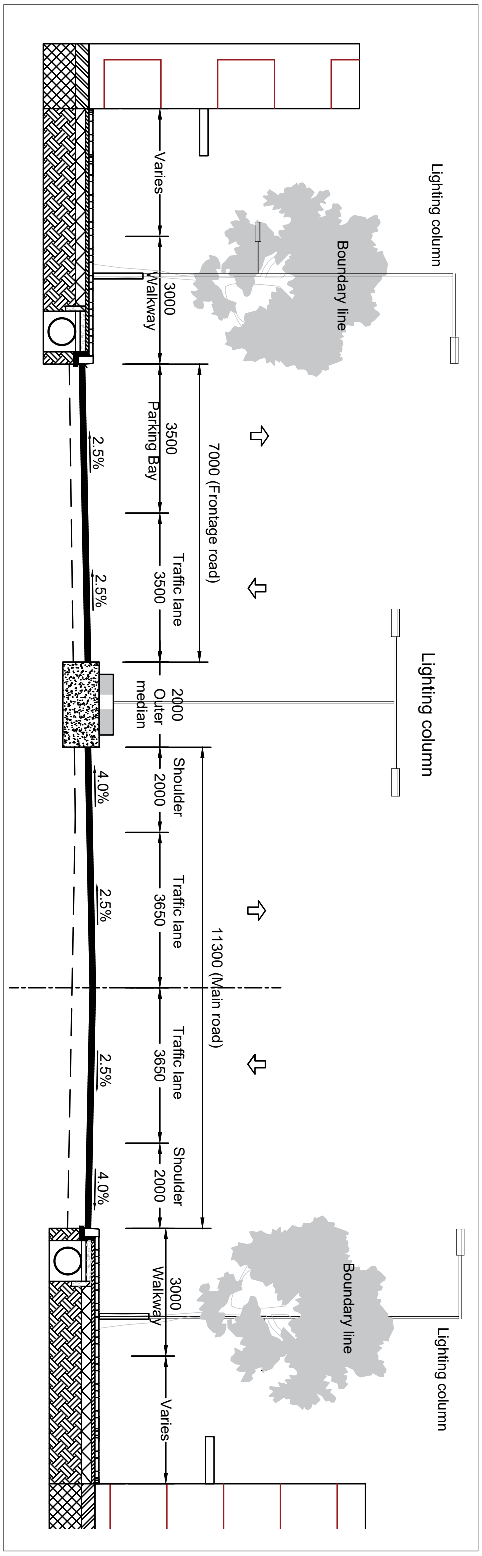

Figure 16.3 Urban Arterial Road – Type U3

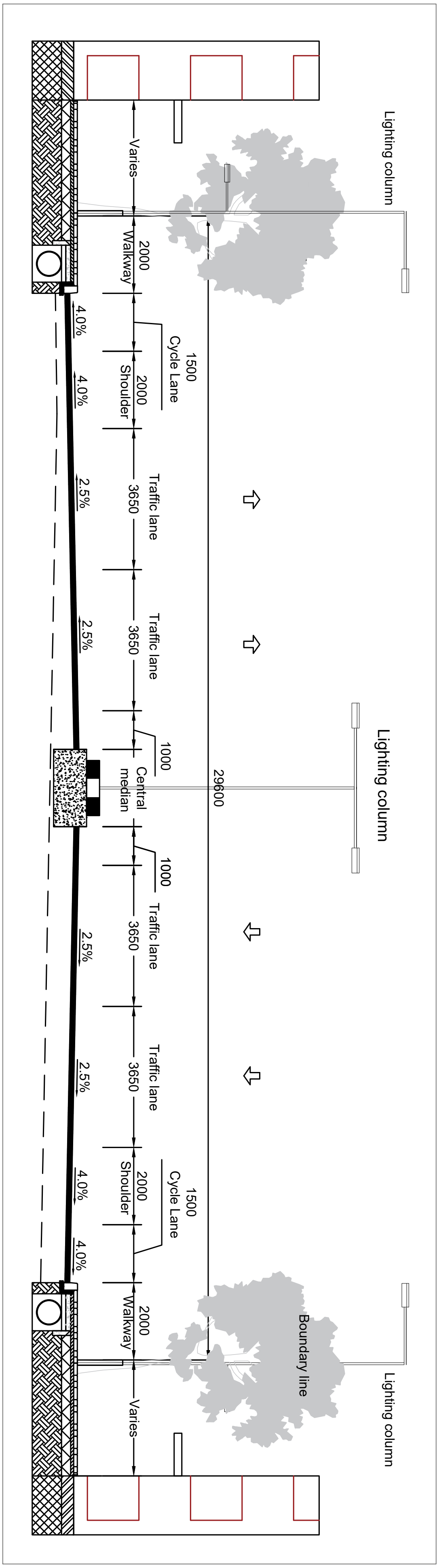

Figure 16.4 Urban Collector Road – Type U4

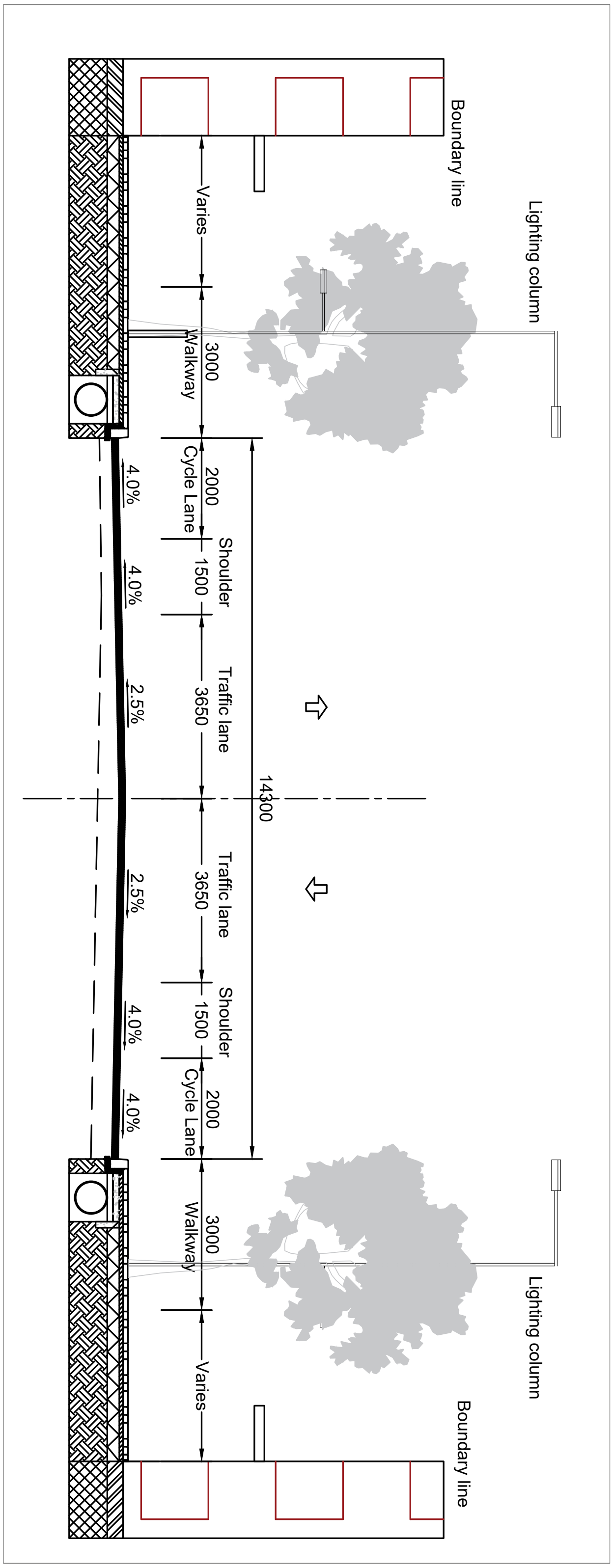

Figure 16.5 Urban Collector Road – Type U5

17 Design of Urban Streets/Local Roads

17.1 Introduction

17.1.1 General

This chapter provides guidance to the designer for specifics that are particular to urban streets (local roads) in Kenya.

Chapters 1 to 13 of this manual provide all aspects of geometric design that are cross-cutting amongst all functional road classes in Kenya.

The designer shall refer to these Chapters for more information and guidance as necessary.

17.1.2 Functions of Urban Streets

The function of urban streets is primarily to provide access to properties and feed into collector roads. Through traffic is discouraged. Residential streets serve residential stands including cul-de-sacs, where there is no through traffic. Commercial and industrial stand streets serve commercial and industrial stands, where there is little or no through traffic. Shopping and central business streets prioritise access by pedestrians and limit access by motorised vehicles.

The main functions served by urban streets in Kenya are summarised in Table 17.1 below and design standards have been developed to ensure efficient fulfilment of those functions.

Table 17.1 Functions of Urban Streets

Road Class	Main road user	Access	Transit traffic	Use by public transport	Operating speeds (Km/hr)
Local Roads/ Streets	Mixed: Motorised traffic, NMT, pedestrians	Access to properties	No transit traffic	Buses, HGVs not allowed	15-30
Service Roads	Mixed: Motorised traffic, NMT, pedestrians	Access to properties	No transit traffic	Buses not allowed	10

17.1.3 Design Standards

The applicable design standards for design of local roads/urban streets are as indicated in Table 17.2.

Development of the standards is explained in Chapter 2. These typical standards are intended to be used as a guide by the designer.

Table 17.2 Applicable Design Standards for Urban Streets

Functional Classification	Applicable Design Standards
UL	DU4, DU5, DU6

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Design of Urban Roads

17.2 Design Controls and Criteria

17.2.1 General

Reference shall be made to Chapter 1 for details and guidance on all aspects of design controls and criteria for all roads in Kenya.

This section attends to controls and criteria applying to urban streets, in particular.

17.2.2 Road Reserve

The road reserve widths given in Table 17.3 shall be adopted for urban streets/local roads. These align with the constrained road corridor widths that are available in the Kenyan urban road contexts.

Table 17.3 Road Reserve Widths for Urban Streets/Local Roads

Functional Class	Desirable (m)	Reduced (m)
UA	12	6

Note: *that the 6M road reserve shall be used exclusively as a NMT-Only corridor to provide for Walkways, cycle tracks, drainage (covered) and passage of utilities.*

17.2.3 Design Vehicles and NMT Characteristics

In an urban setting the design vehicle shall be defined for the various design elements, inclusive of NMT and PT facilities.

For urban streets the passenger car design vehicle DV1 should be used for speed-related standards and the medium heavy goods vehicle of configuration DV5 as given in Table 3.3 shall be adopted for standards relating to manoeuvrability, typically at intersections.

For areas designated for use by NMT's only the heaviest vehicle in that class, which is the cart, with an approximate weight of 1 tonne, shall be adopted.

Table 17.4 gives the dimensional details of the DV1 design vehicle and NMT's which shall be adopted for urban streets.

Table 17.4 Design Vehicles, Pedestrians and NMT Dimensional Details

Vehicle/road User	Width (m)	Length (m)	Sideways Clearance (m)	Turning Circle Radii (m)
Pedestrian	0.7	1.0	0.3	1.5
Bicycle	0.75	2.0	0.5	6.0
Bicycle with goods	1.0	3.0	0.8	6.0
Cart	1.60	3.0	0.8	6.0
Car (DV1)	1.3	2.1	1.0	7.3

17.2.4 Design Speed

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 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.**

The design speeds for urban streets, for the typical standards as indicated above, are therefore as shown in Table 17.5 and are stratified by functional class.

Table 17.5 Design Speeds for Urban Local Streets

Design Standards	Desirable design speed (km/h)	Reduced design speed (km/h)	Application to Functional Classes
DU4	50	30	**UL**
DU5	30	10 – 15	
DU6	20	10	

For the design of turning spaces and intersection corners design speeds of 10 – 15 km/hr shall be adopted.

17.2.5 Passenger Car Units

In the process of determining design volumes for urban street/local roads, the appropriate passenger car unit values for different types of vehicles under different conditions shown in Table 3.7 shall be adopted.

17.2.6 Determining Motorised Traffic Volumes

Traffic data for use in determining traffic volumes shall be obtained from field surveys undertaken in accordance with the **Road Design Manual Volume 1: Geometric Design of Highways, Rural and Urban Roads, Part 2 - Traffic Surveys**.

The design of main traffic routes in urban areas shall 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.

Therefore, in urban area design, the 30th highest hourly volume can be a reasonable representation of daily peak hours during the year.

The designer is referred to Chapter 3 for detailed explanation of the derivation of the Design Hourly Volume (DHV) for urban roads in general.

17.2.7 Road Capacity and Level of Service

The design of any roadway and its features should explicitly consider traffic volumes, operational performance, and user characteristics for all transportation modes.

The designer shall determine the capacity of the road under consideration and this will be the maximum hourly rate at which 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.

Knowing the anticipated capacity of the highway, the designer shall provide for features of the road to fit the requirements of the estimated traffic, selecting the best road type, dimensions of lanes, shoulders, weaving sections, geometry, etc.

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17.2.8 Pedestrian Walkway Capacity

Separate walkways shall be provided on all arterial and collector roads. The required widths depend on the peak volume of pedestrians anticipated per hour. On average a width of 1m is required for each 1000 pedestrians per hour.

Recommended walkway capacities in each direction are given in Table 17.6.

Table 17.6 Walkways Capacity in One Direction

Type of flow	Walking Speed	Capacity (persons/m/hr)	Level of Service
Uninterrupted flow	5.0	1000	**A** - Good
Slightly interrupted flow	4-5	2000	**B** - Acceptable
Disturbed flow	3-4	2500	**C** - Poor
Highly disturbed flow	2-3	3000	**D** - Unacceptable

Source: *Productive and Liveable Cities, SSATP, 2001*

17.2.9 Bicycle Lane Capacity

For design purposes a peak capacity of single lane (1.0 m wide) bicycle lane separated from other traffic shall be 1000 cyclists/hr. For a 2.0 m wide track, one-way flow the capacity shall be taken as 2000 cyclists/hr.

17.2.10 Carts

Cart paths shall only be provided in locations of close proximity to markets and their movement shall be combined with pedestrian movement.

Hence, where cart use in prevalent, walkways shall be constructed to a width of 3 m as a minimum.

17.3 Cross Section

17.3.1 General

The designer shall refer to Chapter 4 for guidance and details on the selection of cross-sectional elements in general.

17.3.2 Traffic Lane Widths

It is crucial to prioritise the safety of all road users in urban environments and offer flexibility in cases where the right-of-way is limited.

The trafficked lane widths for urban local roads shall be as given in Table 17.7.

Table 17.7 Traffic Lane Widths For Urban Streets

Type of flow	Walking Speed	Capacity (persons/m/hr)	Level of Service
Urban Roads	**UL** (local urban streets)	Paved	**Single carriageway** (lanes of 3.0-3.65m width)
	UL (local urban streets)	Unpaved	**Single carriageway** (lanes of 2.75m - 3.5m width)

17.4 At-Grade Intersections

Reference shall be made to Chapter 9 for guidance on the design of at-grade intersections for all roads in general.

Intersection geometry in urban areas must accommodate the needs of a diverse range of vehicles and NMT users commonly found in such environments. Intersection design should facilitate visibility and predictability for all users, creating an environment in which complex movements feel safe, easy, and intuitive.

In particular:

1. Zebra crossings shall be provided on all legs of all signalised intersections.
2. Pedestrian crossings must be aligned with the general crossing desire lines for the envisaged users.
3. It is desirable to provide 5 m wide pedestrian crossings, the minimum allowable shall be 3 m.
4. All cycle track crossings shall have a minimum width of 2 m.
5. Raised crosswalks (tabletop crossings) shall be set out at a level of +150 mm above finished road level at un-signalised zebra crossings.
6. Refuge islands shall be provided where there are more than three lanes in total to be crossed.
7. Provide protected crossings for bicycles through intersections.
8. 'Median tips' where there is a zebra crossing and a median at an intersection.
9. Stop lines perpendicular to the travel lane and set back at least 3 m from the zebra crossings.

17.5 Roadside Facilities and Amenities

17.5.1 General

Reference shall be made to Chapter 13 for guidance and details on roadside facilities and amenities for urban arterial and collector roads.

17.5.2 Bus Stops

The following considerations shall be taken into account in providing public service vehicle stopping places on urban roads:

1. PSV stops should be provided at intervals of 200-400 m, depending on the level of public transport demand.
2. The minimum width of the bay shall me 2.5 m.
3. On a street with two or more carriageway lanes per direction, the PSV stop should be placed adjacent to the vehicle's line of travel so that the PSV does not need to pull over.
4. On a street with one carriageway lane per direction or at terminal locations, the stop may incorporate a bus bay provided that there is sufficient clear space for walking behind the shelter.
5. Shelter with adequate lighting, protection from sun and rain, and customer information.
6. Cycle tracks should be routed behind bus shelters.

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17.5.3 Passenger Service Vehicle Terminals

The following standards shall be adopted for provision of terminals for public passenger vehicles:

1. PSV terminals shall be designed to ensure that the entire facility offers universal access.
2. Minimal walking distances and vertical displacement between platforms is necessary to avoid user discomfort.
3. Bottleneck shall be avoided, wider spaces where different pedestrian streams intersect must be provided.
4. User of the terminals must be protected from the elements such as the sun, rain and wind. Roofs should extend above buses in loading bays.
5. PSV terminals must contain ample seating for waiting passengers and other users.
6. Complete customer information, including real-time departure information should be easily available.
7. Full provision must be made for adequate lighting to enhance safety and security for passengers.
8. Public toilets must be easily accessible for all classes of users and must include baby changing facilities.
9. Organised vending kiosks are a requirement, priority being given to those that provide food and drink.
10. Special provision must be made for cycle parking facilities.

17.6 NMT Facilities

17.6.1 Walkways

The minimum clear width of walkways shall be as indicated in Table 17.8. This width represents the clear width, excluding furniture and frontage zones.

Table 17.8 Walkway Widths For Local Roads

Situation	Minimum Clear Width (m)	Comments
General low volume	2.0	• General minimum widths for most roads and streets. • Not adequate for commercial or shopping environments. • Clear width required for one wheelchair with 0.15 m allowance both sides.
High pedestrian volumes	3.0 (or higher based on volume)	• Generally commercial and shopping areas.

17.6.2 Pedestrian Crossings

In the urban local road context, at-grade crossings shall be provided to cater for pedestrians of all ages and PWDs, with traffic calming measures or signalisation to manage vehicle movements.

The following principles shall apply:

1. Crossings should be signalised or raised to the level of the walkway to provide universal access and traffic calming. A ramp slope of 1:8 is preferred.
2. Pedestrian crossings should match pedestrian desire lines.

3. If a speed hump is used, the hump should be placed 5 m before the crossing.
4. The pedestrian crossing should have a width of 5 m or equivalent width to the adjacent walkway, whichever is larger.
5. Bulb-outs should be added in parking lanes to reduce the crossing distance.
6. Footbridges should be avoided except when crossing limited-access highways since they are inconvenient, inaccessible, and unsafe.

17.6.3 Cycle Tracks

In general, cycle tracks should be provided on urban roads with an operational speed exceeding 30 km/h.

Table 17.9 Cycle Track Widths Guidance

Situation	Minimum Clear Width (m)	Improved Minimum Width (m)
General low volume	1.2	2.0
High cyclist volumes	2.0 (or higher based on volume)	3.0 (or higher based on volume)

Cycle tracks shall be designed to the following standards:

1. Preferably, cycle tracks shall be physically separated from the carriageway—as distinguished from painted cycle lanes, which offer little protection to cyclists.
2. A clear width of at least 2.0 m for one-way movement shall be provided and 3.0 m of clear width for two-way movement.
3. A smooth surface material—asphalt or concrete shall be adopted to offer acceptable levels of comfort to road users; paver blocks should be avoided.
4. Cycle tracks shall be elevated 150 mm above the carriageway.
5. In terms of positioning between the walkway and carriageway a buffer of at least 0.5 m between the cycle track and carriageway is necessary. The buffer should be paved if it is adjacent to a parking lane. The width should be increased to 0.75 m next to buildings, walls, etc.
6. Bollards to prevent encroachments by cars shall be placed in the middle of the cycle track, still allowing for cyclists to pass on either side. Bollard spacing width of 1.2 m shall be adopted.

17.7 Design Catalogues for Urban Local Roads/Streets

The following deign catalogues summarise the design controls, cross-sectional elements, horizontal alignment characteristics and other elements for urban arterial and collector roads:

Table 17.10: Design Standards DU4 – Urban Collector and Local

Table 17.11: Design Standard DU5 – Urban Local (CBD and Residential)

Table 17.12: Design Standard DU6 – Urban Local (Residential)

The designer shall use the catalogues as guides in preparing the design for each situation. It shall be the designer's responsibility to check and refine the outcome of the application of the guides against the design principles given in this manual, as well as good practice in general.

Table 17.10 Design Standards DU4 – Urban Collector and Local

	Design Element	Ref. Section	Requirement/Specification			
	Applicable Functional Class		**Urban Collector**		**Urban Local**	
Design Controls	Design Life		20 years			
Design Controls	Design Traffic (DHV)		500 - 2000			
Design Controls	Design Speed (km/hr)		50 – 70		30 - 50	
Design Controls	Control of Access		Full		Partial	
Design Controls	Level of Service Threshold		D		B	
Cross-Section Elements	Typical Cross-section		DU4, DU5			
Cross-Section Elements	Lane Width		1 or 2		1	
Cross-Section Elements	No. of Lanes each Direction		3.65 m		3.65 m	
Cross-Section Elements	Outside Shoulder Width – Paved		1.5 m		0 - 1 m	
Cross-Section Elements	Outside Shoulder Width – Un-paved		N/A			
Cross-Section Elements	Median Shoulder Width – Paved		N/A			
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		3.0 m			
Cross-Section Elements	Auxiliary Lanes – Shoulder Width		Paved: 1.0 m Unpaved – N/A			
Cross-Section Elements	Median Width – Depressed		N/A			
Cross-Section Elements	Median Width – Concrete Median Barrier		N/A			
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	Walkway Width		1.5 m		2.4 m	
Cross-Section Elements	Service Road Width		3 m		N/A	
Cross-Section Elements	Road Reserve		Desirable: 60 m; Reduced: 40 m			
Cross-Section Elements	Roadside Clear Zone – Low Traffic (Fill/Cut)		4 m; 2.5 m		2.5 m; 2.5 m	
Cross-Section Elements	Roadside Clear Zone – Medium Traffic (Fill/Cut)		5.5 m; 4 m		3 m; 3 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 – Backslope		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)**		**50**	**70**	**30**	**50**
Alignment Elements	Stopping Sight Distance (m) – g: 0%		65	110	30	65
Alignment Elements	Stopping Sight Distance (m) – g: 5%		70	120	31	70
Alignment Elements	Stopping Sight Distance (m) – g: 10%		75	140	33	75
Alignment Elements	Decision Sight Distance (m)		155			
Alignment Elements	Passing Sight Distance (m) – Desirable		345	485	195	345
Alignment Elements	Passing Sight Distance (m) – Minimum		155	210	115	155
Alignment Elements	% Passing Opportunity		50			
Alignment Elements	Maximum Super-elevation Rate %		8			
Alignment Elements	Minimum Horizontal Curve Radius (m) – SE: 4%		95	205	30	95
Alignment Elements	Minimum Horizontal Curve Radius (m) – SE: 6%		85	185	26	85
Alignment Elements	Minimum Horizontal Curve Radius (m) – SE: 8%		80	170	24	80
Alignment Elements	Maximum Grades		Desirable: 3 %; Absolute: 5 %			
Alignment Elements	Minimum Grades		0.5 %			
Alignment Elements	Vertical Curve (K-values) – Crest		10	30	2	10
Alignment Elements	Vertical Curve (K-values) – Sag (Comfort)		6.5	12	2.5	6.5
Alignment Elements	Vertical Curve (K-values) – Sag (Headlights)		14	25	5	14

Table 17.11 Design Standard DU5 – Urban Local (CBD and Residential)

	Design Element			Ref. Section	Requirement/Specification	
	Applicable Functional Class				CBD/Industrial	Residential
Design Controls	Design Life				20 years	
	Design Traffic (DHV)				500 - 1000	
	Design Speed (km/hr)				10 – 30	
	Control of Access				Partial	
	Level of Service Threshold				B	
Cross-Section Elements	Typical Cross-section				DU5, DU6	
	Lane Width				1	
	No. of Lanes each Direction				3.65 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	
	Walkway 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	
			Backslope		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	
	Passing Sight Distance (m)	Desirable			195	345
		Minimum			115	155
	% Passing Opportunity				50	
	Maximum Super-elevation Rate %				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

Table 17.12 Design Standard DU6 – Urban Local (Residential)

	Design Element				Ref. Section	Requirement/Specification	
	Applicable Functional Class					CBD/Industrial	Residential
Design Controls	Design Life					20 years	
	Design Traffic (DHV)					< 500	
	Design Speed (km/hr)					10 – 20	
	Control of Access					Partial	
	Level of Service Threshold					B	
Cross-Section Elements	Typical Cross-section					DU6, DU7, DU8, DU9	
	Lane Width					1	
	No. of Lanes each Direction					3.65 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	
	Walkway 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	
				Backslope		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	
	Passing Sight Distance (m)			Desirable		195	345
				Minimum		115	155
	% Passing Opportunity					50	
	Maximum Super-elevation Rate %					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

17.8 Typical Cross-sections

Typical cross-sections for urban streets in Kenya are as shown in Figure 17.1 to Figure 17.3. The designer shall use these as a guide and refine the same for specific design requirements of individual projects.

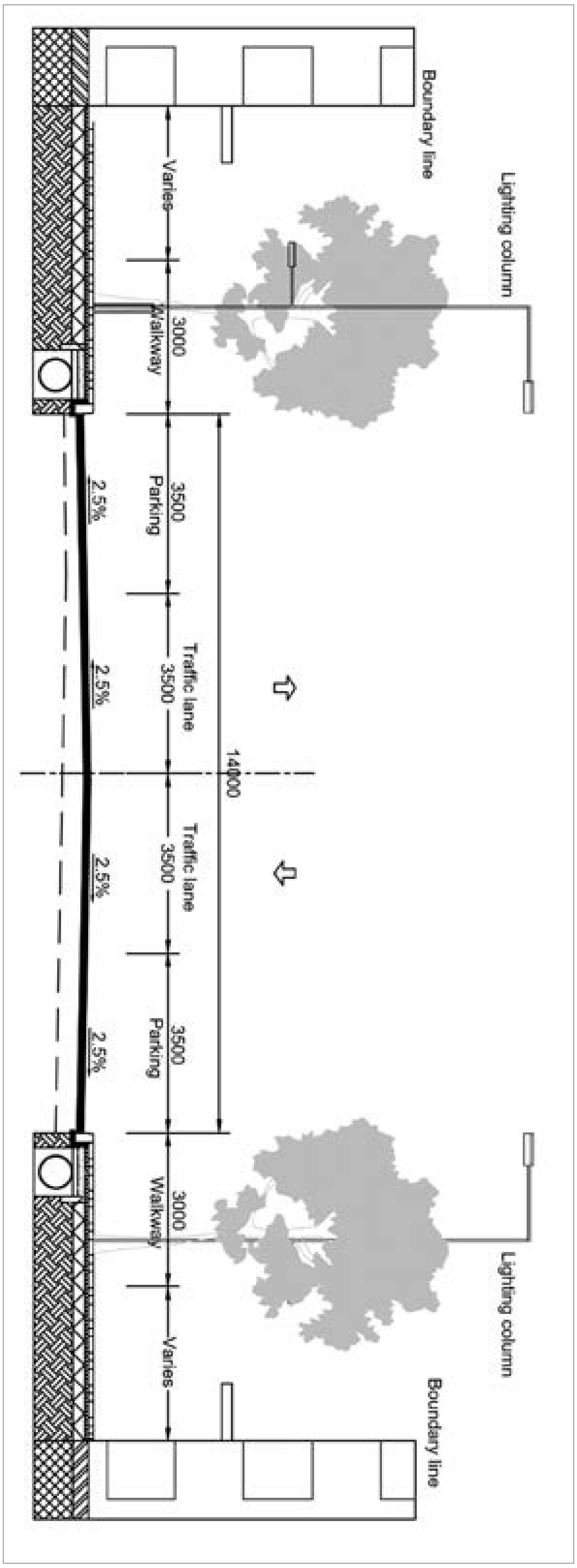


Figure 17.2 Urban Local Street (On-street Parking, Pedestrian and Cyclist Lanes)

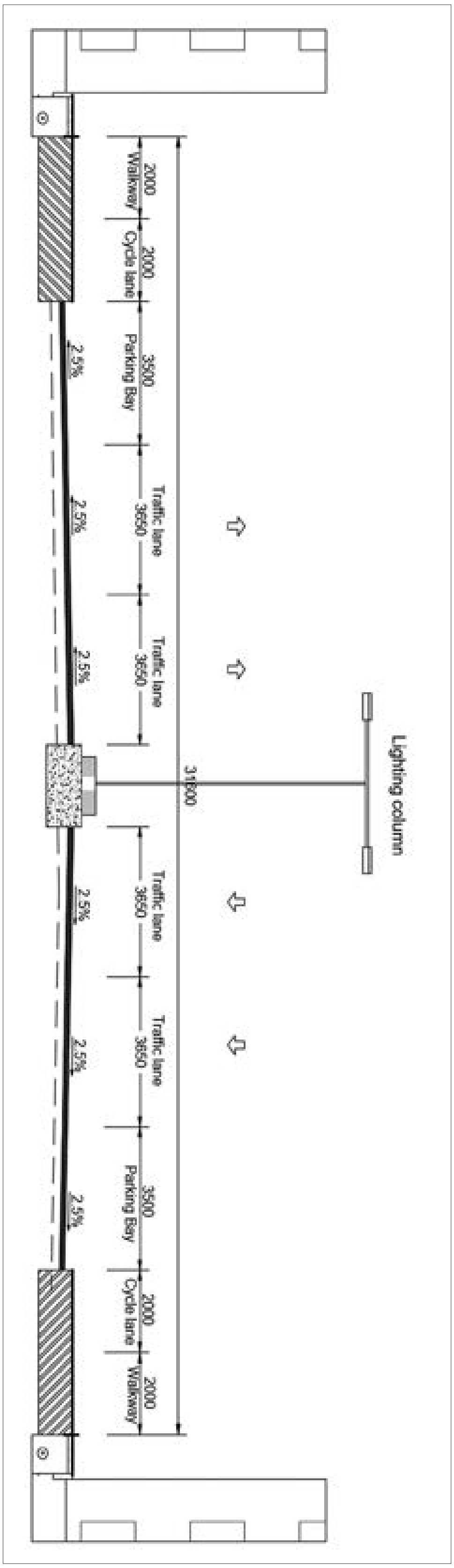
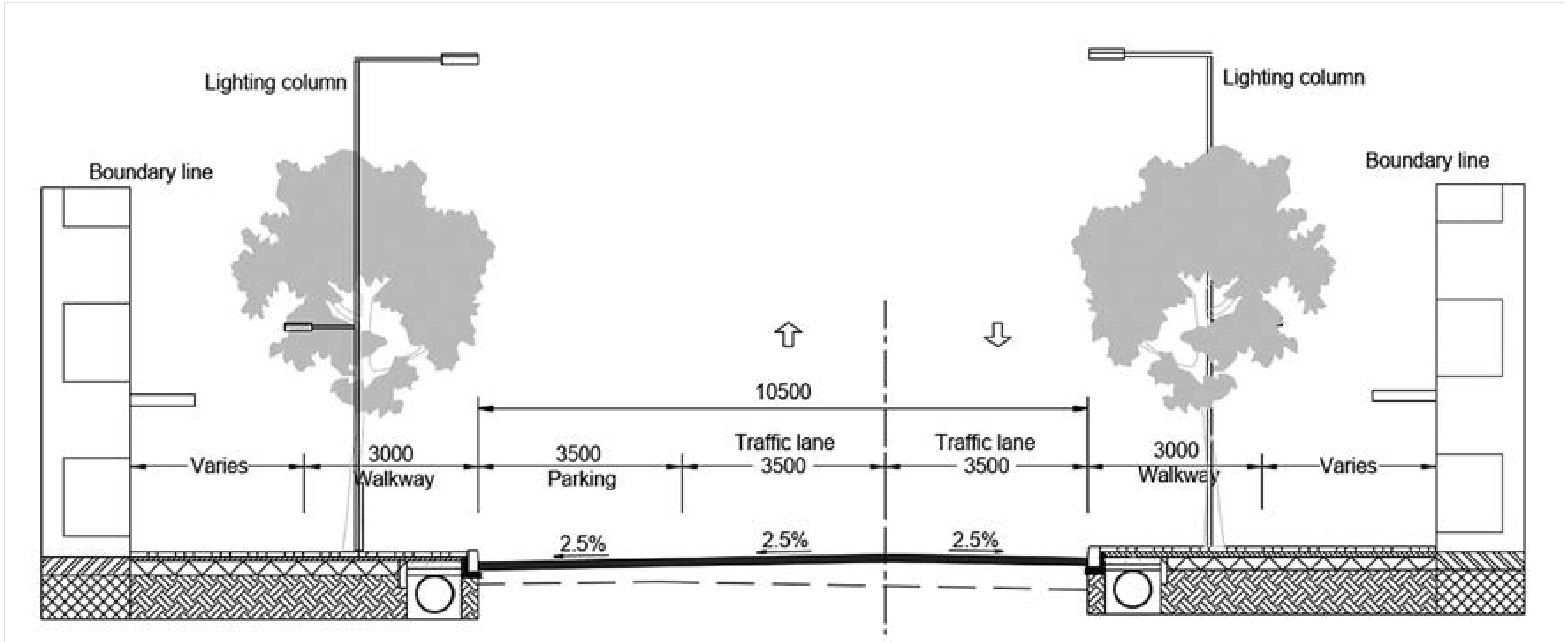


Figure 17.1 Urban Local Street Road (On-street Parking)

Figure 17.3 Urban Local Street (On-street Parking and Pedestrian Lane)



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