FOREWORD

The road network in Ethiopia provides the dominant mode of freight and passenger transport and thus plays a vital role in the economy of the country. The network comprises a huge national asset that requires adherence to appropriate standards for design, construction and maintenance in order to provide a high level of service. As the length of the road network is increasing, appropriate choice of methods to preserve this investment becomes increasingly important.

In 2002, the Ethiopian Roads Authority (ERA) first brought out road design manuals to provide a standardized approach for the design, construction and maintenance of roads in the country. Due to technological development and change, these manuals require periodic updating. This current version of the manual has particular reference to the prevailing conditions in Ethiopia and reflects the experience gained through activities within the road sector during the last 10 years. Completion of the review and updating of the manuals was undertaken in close consultation with the federal and regional roads authorities and the stakeholders in the road sector including contracting and consulting industry.

Most importantly, in supporting the preparation of the documents, a series of thematic peer review panels were established that comprised local experts from the public and private sector who provided guidance and review for the project team.

This Manual supersedes the Geometric Design Manual part of the ERA 2002 series of Manuals. The standards set out shall be adhered to unless otherwise directed by ERA. However, I should emphasize that careful consideration to sound engineering practice shall be observed in the use of the manual, and under no circumstances shall the manual waive professional judgment in applied engineering. For simplification in reference this manual may be cited as ERA's *Geometric Design Manual - 2013.*

On behalf of the Ethiopian Roads Authority I would like to thank DFID, Crown Agents and the AFCAP team for their cooperation, contribution and support in the development of the manual and supporting documents for Ethiopia. I would also like to extend my gratitude and appreciation to all of the industry stakeholders and participants who contributed their time, knowledge and effort during the development of the documents. Special thanks are extended to the members of the various Peer Review Panels, whose active support and involvement guided the authors of the manual and the process.

It is my sincere hope that this manual will provide all users with a standard reference and a ready source of good practice for the geometric design of roads, and will assist in a cost effective operation, and environmentally sustainable development of our road network.

I look forward to the practices contained in this manual being quickly adopted into our operations, thereby making a sustainable contribution to the improved infrastructure of our country.

Comments and suggestions on all aspects from any concerned body, group or individual as feedback during its implementation is expected and will be highly appreciated.

Addis Ababa, 2013

Zaid Wolde Gebriel

Director General, Ethiopian Roads Authority

PREFACE

The Ethiopian Roads Authority is the custodian of the series of technical manuals, standard specifications and bidding documents that are written for the practicing engineer in Ethiopia. The series describes current and recommended practice and sets out the national standards for roads and bridges. The documents are based on national experience and international practice and are approved by the Director General of the Ethiopian Roads Authority.

The *Geometric Design Manual -2013* forms part of the Ethiopian Roads Authority series of Road and Bridge Design documents. The complete series of documents, covering all roads and bridges in Ethiopia, is as follows:

1. Geometric Design Manual
2. Site Investigation Manual
3. Geotechnical Design Manual
4. Route Selection Manual
5. Pavement Design Manual Volume I Flexible Pavements
6. Pavement Design Manual Volume II Rigid Pavements
7. Pavement Rehabilitation and Asphalt Overlay Design Manual
8. Drainage Design Manual
9. Bridge Design Manual
10. Low Volume Roads Design Manual
11. Standard Environmental Procedures Manual
12. Standard Technical Specifications
13. Standard Detailed Drawings.
14. Best Practice Manual for Thin Bituminous Surfacings
15. Standard Bidding Documents for Road Work Contracts – A series of Bidding Documents covering the full range of projects from large scale works unlimited in value to minor works with an upper threshold of $300,000. The higher level documents have both Local Competitive Bidding and International Competitive Bidding versions.

These documents are available to registered users through the ERA website: www.era.gov.et

Manual Updates

Significant changes to criteria, procedures or any other relevant issues related to new policies or revised laws of the land or that are mandated by the relevant Federal Government Ministry or Agency should be incorporated into the manual from their date of effectiveness.

Other minor changes that will not significantly affect the whole nature of the manual may be accumulated and made periodically. When changes are made and approved, new page(s) incorporating the revision, together with the revision date, will be issued and inserted into the relevant chapter.

All suggestions to improve the manual should be made in accordance with the following procedures:

1. Users of the manual must register on the ERA website: www.era.gov.et
2. Proposed changes should be outlined on the Manual Change Form and forwarded with a covering letter of its need and purpose to the Director General of the Ethiopian Roads Authority.
3. After completion of the draft review period, proposed modifications will be assessed by the requisite authorities in ERA.
4. Agreed changes will be approved by the Director General of the Ethiopian Roads Authority on recommendation from the Deputy Director General (Engineering Operations).
5. The release date will be notified to all registered users and authorities.

Addis Ababa, 2013

Zaid Wolde Gebriel

Director General, Ethiopian Roads Authority

ETHIOPIAN ROADS AUTHORITY
CHANGE CONTROL DESIGN MANUAL

MANUAL CHANGE		This area to be completed by the ERA Director of Quality Assurance
Manual Title:______________________________ ____		CHANGE NO.______________ (SECTION NO. CHANGE NO. ____________________________
Section Table Figure Page	**Explanation**	**Suggested Modification**

Submitted by:

Name:_____________________________________Designation:_______________________________

Company/Organisation Address

____

____email:__________________________Date:________

Manual Change Action

Authority	**Date**	**Signature**	**Recommended Action**	**Approval**
Registration				
Director Quality Assurance				
Deputy Director General Eng.Ops				

Approval / Provisional Approval / Rejection of Change:

Director General ERA**:**___________________________________ **Date:** ___________________

ACKNOWLEDGEMENTS

The Ethiopian Roads Authority (ERA) wishes to thank the UK Government's Department for International Development (DFID) through the Africa Community Access Programme (AFCAP) for their support in developing this *Geometric Design Manual – 2013*. The manual will be used by all authorities and organisations responsible for the provision of roads in Ethiopia.

This *Geometric Design Manual-2013* is based on a review of the design standards of several countries including the USA (AASHTO, FHWA), the United Kingdom (TRL, HA), RSA (CSIR, SANRAL), and Ethiopia (TCDE). It is based largely on ERA's *Geometric Design Manual – 2002* but includes improvements resulting from recent research and extensions to deal with topics that were not included in the earlier manual. This manual is consistent with the relevant sections of ERA's *Low Volume Roads Design Manual*.

From the outset, the approach to the development of the manual was to include all sectors and stakeholders in Ethiopia. The input from the international team of experts was supplemented by our own extensive local experience and expertise. Local knowledge and experience was shared through review workshops to discuss and debate the contents of the draft manual. ERA wishes to thank all the individuals who gave their time to attend the workshops and provide valuable inputs to the compilation of the manual.

In addition to the workshops, Peer Review Groups comprising specialists drawn from within the local industry were established to provide advice and comments in their respective areas of expertise. The contribution of the Peer Review Group participants is gratefully acknowledged.

The final review and acceptance of the document was undertaken by an Executive Review Group. Special thanks are given to this group for their assistance in reviewing the final draft of the document.

Finally, ERA would like to thank Crown Agents for their overall management of the project.

As with the other manuals of this series, the intent was, where possible, and in the interests of uniformity, to use those tests and specifications included in the AASHTO and/or ASTM Materials references. Where no such reference exists for tests and specifications mentioned in this document, other references are used.

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Addis Ababa, 2013

Zaid Wolde Gebriel

Director General, Ethiopian Roads Authority

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GLOSSARY OF TERMS

Acceleration/ deceleration lane	An auxiliary lane used by a vehicle about to enter (acceleration lane) or leave (deceleration lane) the travelled way to adjust its speed as required for safety.
Access	Way whereby the owner or occupier of any land has access to a public road, whether directly or across land lying between his land and such public road.
Access Control	The condition whereby the road agency either partially or fully controls the right of direct access to and from a public highway or road.
Arterial	Highway designed to move relatively large volumes of traffic at high speeds over long distances. Typically, arterials offer little or no access to abutting properties.
At-grade Junction	Junction where all roadways join or cross at the same level.
Auxiliary Lane	Part of the roadway adjoining the carriageway for parking, speed change, turning, storage for turning, weaving, truck climbing, and for other purposes supplementary to through traffic movement.
Aggregate	Hard mineral elements of construction material mixtures, for example: sand, gravel (crushed or uncrushed) or crushed rock.
Asphalt	In American literature asphalt is another term for bitumen. The term is also commonly used in this way in Ethiopia. In other countries, asphalt is commonly used as a shorthand for asphaltic concrete or, indeed, any design of high quality bitumen/aggregate mixture.
Asphalt Concrete	A mixture to predetermined proportions of aggregate, filler and bituminous binder material plant mixed and usually placed by means of a paving machine.
Asphalt Surfacing	The layer or layers of asphalt concrete constructed on top of the roadbase, and, in some cases, the shoulders.
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.
Axis of Rotation	The line about which the pavement is rotated to super-elevate the roadway. This line normally maintains the highway profile.
Back Slope	Area between the ditch bottom and the limit of the earthworks.

Base Course	This is the main component of the pavement contributing to the spreading of the traffic loads. In many cases, it will consist of crushed stone or gravel, or of good quality gravelly soils or decomposed rock. Bituminous base courses may also be used (for higher classes of traffic). Materials stabilised with cement or lime may also be contemplated.
Binder Course	The lower course of an asphalt surfacing laid in more than one course.
Bitumen	The most common form of bitumen is the residue from the refining of crude oil after the more volatile material has been distilled off. It is essentially a very viscous liquid comprising many long-chain organic molecules. For use in roads it is practically solid at ambient temperatures but can be heated sufficiently to be poured and sprayed. Some natural bitumens can be found worldwide that are not distilled from crude oil but the amounts are very small.
Borrow Area	An area within designated boundaries approved for the purpose of obtaining borrow material. A borrow pit is the excavated pit in a borrow area.
Borrow Material	Any gravel, sand, soil, rock or ash obtained from borrow areas, dumps or sources other than cut within the road prism and which is used in the construction of the specified works. Not including crushed stone or sand obtained from commercial sources.
Boulder	A rock fragment, usually rounded by weathering or abrasion, with an average dimension of 0.30 m or more.
Bound Pavement Materials	Pavement materials held together by an adhesive bond between the materials and another binding material such as bitumen.
Broken back curve	Two curves in the same direction with a tangent shorter than 500 metres connecting them.
Bus Lay-byes	Lay-by reserved for public service vehicles.
Camber	The convexity given to the curved cross-section of a roadway.
Capacity	Maximum practicable traffic flow in given circumstances.
Capping Layer	(Selected or improved subgrade). The top of an embankment or bottom of an excavation prior to construction of the pavement structure. Where very weak soils and/or expansive soils (such as black cotton soils) are encountered, a capping layer is sometimes necessary. This consists of better quality subgrade material imported from elsewhere or subgrade material improved by stabilisation (usually mechanical), and may also be considered as a lower quality sub-base.
Carriageway	That portion of the roadway including the various traffic lanes and auxiliary lanes but excluding shoulders.

Centre Line	Axis along the middle of the road
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.
Coefficient of Friction	Ratio of the frictional force on the vehicle and the component of the weight of the vehicle perpendicular to the frictional force
Collector Roads	Secondary roads linking locally important centres to each other, to more important centres, or to higher class roads
Compound Curve	Curve consisting of two or more arcs of different radii curving in the same direction and having a common tangent or transition curve where they meet.
Contraction Joint	A joint normally placed at recurrent intervals in a rigid concrete slab to control transverse cracking.
Control of Access	Conditions where the right of owners or occupants of adjoining land or other persons to access, light, air or view in connection with a road is fully or partially controlled by public authority.
Crest	Peak formed by the junction of two gradients.
Crest Curve	Convex vertical curve with the intersection point of the tangents above the road level.
Critical Slope	Side slope on which a vehicle is likely to overturn.
Cross Roads	Four-leg junction formed by the intersection of two roads at approximately right angles.
Cross-Section	A vertical section showing the elevation of the existing ground, ground data and recommended works, usually at right angles to the centreline.
Crossfall	The difference in level measured transversely across the surface of the roadway.
Crown	Highest portion of the cross-section of a cambered roadway.
Crown Runoff	Also referred to as Tangent Runout. The rotation of the outer lane of a two lane road from zero cross fall to normal camber (NC).
Critical Length of Grade	The maximum length of a specific upgrade on which a loaded truck can operate without an unreasonable reduction in speed. Very often, a speed reduction of 15 km/h or more is considered “unreasonable”.
Culvert	A structure, other than a bridge, which provides an opening under the

	carriageway or median for drainage or other purposes.
Curb	Border of stone, concrete or other rigid material formed at the edge of the roadway or footway.
Cutting	Cutting shall mean all excavations from the road prism including side drains, and excavations for intersecting roads including, where classified as cut, excavations for open drains.
Chippings	Stones used for surface dressing (treatment).
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.
Deformed Bar	A reinforcing bar for rigid slabs conforming to “Requirements for Deformations” in AASHTO Designations M 31M.
Depressed Median	A median lower in elevation than the travelled way and so designed to carry a portion of the storm water falling on the road.
Design Capacity	Maximum number of vehicles that can pass over a lane or a roadway during a given time period without operating conditions falling below a pre-selected design level.
Design Period	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 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. In practice, most roads will only be constrained to minimum parameter values over short sections or on specific geometric elements.
Design Traffic Volume	Number of vehicles or persons that pass over a given section of a lane or roadway during a given time period.
Design Vehicle	Vehicle whose physical characteristics and proportions are used in setting geometric design.
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	Movement of a vehicle out of a traffic stream.
Diverted Traffic	Traffic that changes from another route (or mode of transport) to the project road because of the improved pavement, but still travels

	between the same origin and destination.
Divided Road	Road in which there are two physically separated roadways reserved for travelling in opposite directions.
Dowel	A load transfer device in a rigid concrete slab, usually consisting of a plain round steel bar. Unlike a tie bar, a dowel may permit horizontal movement.
Economical Limit of Haul	Distance through which it is more economical to haul excavated material than to waste and borrow.
Embankment	That portion of the road prism composed of approved fill material, which lies above the original ground and is bounded by the side slopes, extending downwards and outwards from the outer shoulder breakpoints and on which the pavement is constructed.
Equivalent Standard Axles (ESAS)	A measure of the potential damage to a pavement caused by a vehicle axle load expressed as the number of 8.16 metric tonnes 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.
Equivalent Single Axle Load (ESA)	Summation of equivalent 8.16 ton single axle loads used to combine mixed traffic to design traffic for the design period.
Escarpment	Geological features that are very steep and extend laterally for considerable distances, making it difficult or impossible to construct a road to avoid them. They are characterised by more than 50 five-metre contours per km and the transverse ground slopes perpendicular to the ground contours are generally greater than 50%.
Expansion Joint	A joint located to provide for expansion of a rigid concrete slab without damage to itself, adjacent slabs, or structures.
Eye Height	Assumed height of a driver's eyes above the surface of the roadway used for the purpose of determining sight distances.
Feeder Road	Lowest level of road in the network hierarchy with the function of linking traffic to and from rural areas, either directly to adjacent urban centres, or to the Collector road network.
Fill	Material of which a man-made raised structure or deposit such as an embankment is composed, including soil, soil-aggregate or rock. Material imported to replace unsuitable roadbed material is also classified as fill.
Flexible Pavements	Includes primarily those pavements that have a bituminous (surface dressing or asphalt concrete) surface. The terms "flexible and rigid" are somewhat arbitrary and were primarily established to

	differentiate between asphalt and Portland cement concrete pavements.
Footpath	Portion of a road reserved exclusively for pedestrians.
Formation Level	Level at top of subgrade.
Free Haul	Maximum distance through which excavated material may be transported without added cost above the unit bid price.
Freeway	Highest level of arterial characterised by full control of access and high design speeds.
Gap	The elapsed time between the back of one vehicle passing a point on the road or highway and the nose of the following vehicle passing the same point.
Generated Traffic	Additional traffic which occurs in response to the provision of improvement of the road.
Grade Separated Junction	Junction where two roads cross at different levels and are connected by ramps.
Gradient	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), typically expressed in percentage form as the vertical rise or fall in metres/100 metres. In the direction of increasing stake value, upgrades are taken as positive and downgrades as negative.
Grading Modulus (GM)	Related to the cumulative percentages by mass of material in a representative sample of aggregate, gravel or soil retained on the 2.36 mm, 0.425 mm and 0.075 mm sieves; $GM = 3 - \left(\frac{P_{2.36} + P_{0.425} + P_{0.075}}{100} \right)$ where: $P_{2.36}$ = percentage passing 2.36 mm sieve; $P_{0.425}$ = percentage passing 0.425 mm sieve; $P_{0.075}$ = percentage passing 0.075 mm sieve
Guard Rail	Continuous barrier erected alongside a road to prevent traffic from accidentally leaving the roadway or from crossing the median.
Heavy Vehicles	Those having an unloaded weight of 3000 kg or more.
Horizontal Alignment	Direction and course of the road centreline in plan.
Horizontal Clearance	Lateral clearance between the edge of shoulder and obstructions.
Horizontal Curve	Curve in plan-view.

Hot mix asphalt (HMA)	Generic name for all high quality mixtures of aggregates and bitumen that use the grades of bitumen that must be heated in order to flow sufficiently to coat the aggregates. Includes Asphaltic Concrete, Dense Bitumen Macadam and Hot Rolled Asphalt.
Interchange	Network of roads at the approaches to a junction at different levels that permits traffic movement from one to the other or to more roadways or roads.
Junction	Common zone of two or more roads allowing vehicles to pass from one to the other. Meeting of one road with another.
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.
K Value	Ratio of the minimum length of vertical crest curve in meters to the algebraic difference in percentage gradients adjoining the curve.
Lane	Strip of roadway intended to accommodate a single line of moving vehicles.
Lay-by	Part of the road set aside for vehicles to draw out of the traffic lanes for short periods.
Left Hand Lane	On a dual roadway, the traffic lane nearest to the central reserve.
Left Turn Lane	An auxiliary lane to accommodate deceleration and storage of left-turning vehicles at junctions.
Level of Service	Qualitative rating of the effectiveness of a road in serving traffic, measured in terms of operating conditions.
Limited Access Road	Road with right of access only at a limited number of places.
Link Road	National Road linking nationally important centres.
Local Road	Road (or street) primarily for access to adjoining property. It may or may not be a classified road.
Longitudinal Joint	A joint normally placed between traffic lanes in rigid pavements to control longitudinal cracking.
Longitudinal Profile	Outline of a vertical section of the ground, ground data and recommended works along the centreline.
Maintenance	Routine work performed to keep a pavement as nearly as possible in its as-constructed condition under normal conditions of traffic and forces of nature.
Markers	Post, generally fitted with reflective material or small reflecting studs, but not usually lighted, erected off the roadway to give

	warning or guidance to traffic.
Meeting Sight Distance	Distance required to enable the drivers of two vehicles 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.
Median	Area between the two carriageways of a dual carriageway road. Excludes the inside shoulders.
Merging	Movement of a vehicle or vehicles into a traffic stream.
Mountainous (Terrain)	Terrain that is rugged and very hilly with substantial restrictions in both (terrain) horizontal and vertical alignment. Defined as having 26-50 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally above 25%.
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.
Non-recoverable Slope	Transverse side slope where the motorist is generally unable to stop or return to the roadway
Object Height	Assumed height of a notional object on the surface of the roadway used for the purpose of determining sight distance.
Operating Speed	Highest overall speed at which a driver can travel on a given road under 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.
Overlay	One or more courses of asphalt construction on an existing pavement. The overlay often includes a levelling course, to correct the contour of the old pavement, followed by a uniform course or courses to provide needed thickness.
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.
Passenger Car Equivalents (PCE)	A measure of the impedance offered by a vehicle to the passenger cars in the traffic stream. Usually quoted as the number of passenger cars required to offer a similar level of impedance to the other cars in the stream.
Passing Bay	Widened section of an otherwise single lane road where a vehicle may move over to enable another vehicle to pass.

Passing Sight Distance	Minimum sight distance on two-way single roadway roads that must be available to enable the driver of one vehicle to pass another vehicle safely and comfortably without interfering with the speed of an oncoming vehicle travelling at the design speed, should it come into view after the overtaking manoeuvre is started.
Pavement Layers	The layers of different materials which comprise the pavement structure.
Pedestrian Crossing	Transverse strip of roadway intended for the use of pedestrians crossing the road. The crossing may be uncontrolled or controlled.
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.
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.
Point of Vertical Curvature (PVC)	The point at which a grade ends and the vertical curve begins, often also referred to as BVC.
Point of Vertical Intersection (PVI)	The point where the extension of two grades intersect. The initials are sometimes reversed to VPI.
Point of Vertical Tangency (PVT)	The point at which the vertical curve ends and the grade begins. Also referred to as EVC.
Project Specifications	The specifications relating to a specific project, which form part of the contract documents for such project, and which contain supplementary and/or amending specifications to the Standard Specifications.
Pumping	The ejection of foundation material, either wet or dry, through joints or cracks, or along edges of rigid slabs resulting from vertical movements of the slab under traffic.
Quarry	An area within designated boundaries, approved for the purpose of obtaining rock by sawing or blasting.
Ramp	a) Inclined section of roadway over which traffic passes for the primary purpose of ascending or descending so as to make connections with other roadways. b) Interconnecting length of road of a traffic interchange or any connection between roads of different levels, on which vehicles

	may enter or leave a designated road.
Reconstruction	The process by which a new pavement is constructed, utilizing mostly new materials, to replace an existing pavement.
Recoverable Slope	Side slope of limited grade such that a motorist can generally return to the roadway.
Recycling	The reuse, usually after some processing, of a material that has already served its first-intended purpose.
Rehabilitation	Work undertaken to significantly extend the service life of an existing pavement. This may include overlays and pre overlay repairs, and may include complete removal and reconstruction of the existing pavement, or recycling of part of the existing materials.
Reinforcement	Steel embedded in a rigid slab to resist tensile stresses and detrimental opening of cracks.
Reverse Camber (RC)	A super-elevated section of roadway sloped across the entire travelled way at a rate equal to the normal camber.
Reverse Curve	Composite curve consisting of two arcs or transitions curving in opposite directions.
Right Hand Lane	On a dual roadway, the traffic lane nearest to the shoulder.
Right Turn Lane	Auxiliary lane to accommodate deceleration and storage of right-turning vehicles at junctions.
Right of Way	Strip of land legally awarded to the Roads Authority, 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. The width of the road reserve is measured at right angles to the centreline.
Rigid Pavement	A pavement structure which distributes loads to the subgrade having, as the main load bearing course, a Portland cement concrete slab of relatively high-bending resistance.
Roadbase	A layer of material of defined thickness and width constructed on top of the sub-base, or in the absence thereof, the subgrade. A roadbase may extend to outside the carriageway.
Road Bed	The natural in situ material on which the fill, or in the absence of fill, any pavement layers, are to be constructed.
Road Bed Material	The material below the subgrade extending to such depth as affects the support of the pavement structure.
Road Prism	That portion of the road construction included between the original ground level and the outer lines of the slopes of cuts, fills, side fills and side drains. It does not include sub-base, roadbase, surfacing, shoulders, or existing original ground.

Roadway	The area normally travelled by vehicles and consisting of one or a number of contiguous traffic lanes, including auxiliary lanes and shoulders.
Rolling (Terrain)	Terrain with low hills introducing moderate levels of rise and fall with some restrictions on vertical alignment. Defined as terrain with 11-25 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally between 3% and 25%.
Roundabout	Road junction designed for movement of traffic in one direction around a central island.
Safety Rest Area	Roadside area with parking facilities for the motorist to stop and rest.
Sag Curve	Concave vertical curve with the intersection point of the tangents below the road level.
Shoulder	Part of the road outside the carriageway, but at substantially the same level, for accommodation of stopped vehicles for emergency use, and for lateral support of the carriageway.
Shoulder Breakpoint	The point on a cross section at which the extended flat planes of the surface of the shoulder and the outside slope of the fill and pavement intersect.
Side Fill	That portion of the imported material within the road prism which lies outside the fills, shoulders, roadbase and sub-base and is contained within such surface slopes as shown on the Drawings or as directed by the Engineer. A distinction between fills and side fill is only to be made if specified.
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.
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.
Speed Hump	Device for controlling the speed of vehicles, consisting of a raised

(Bump)	area or recess on the roadway.
Spiral Curve	Transition curves from straight (tangent) sections of road and a circular curve.
Stabilisation	The treatment of materials used in the construction of the road bed, fill or pavement layers by the addition of a cementitious binder such as lime or Portland Cement or the mechanical modification of the material through the addition of a soil binder or a bituminous binder. Concrete and asphalt are not considered as materials that have been stabilised.
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.
Sub-base	The layer of material of specified dimensions on top of the subgrade and below the roadbase. It is the secondary load-spreading layer underlying the base course. It will usually consist of a material of lower quality than that used in the base course and particularly of lower bearing strength. Materials may be unprocessed natural gravel, gravel-sand, or gravel-sand-clay, with controlled gradation and plasticity characteristics. The sub-base also serves as a separating layer preventing contamination of the base course by the subgrade material and may play a role in the internal drainage of the pavement.
Subgrade	The surface upon which the pavement structure and shoulders are constructed. The top portion of the natural soil, either undisturbed (but re-compacted) local material in cut sections, or soil excavated in cut or borrow areas, and placed as compacted embankment.
Subsurface Drain	Covered drain constructed to intercept and remove subsoil water, including any pipes and permeable material in the drains.
Super-elevation	Inward tilt or transverse inclination given to the cross section of a roadway throughout the length of a horizontal curve to reduce the effects of centrifugal force on a moving vehicle; expressed as a percentage.
Super-elevation Run-off	Length of road over which super-elevation is reduced from its maximum value to zero.
Surface Treatment	The sealing or resealing of the carriageway or shoulders by means of one or more successive applications of bituminous binder and an aggregate such as crushed stone chippings.
Surfacing	This comprises the top layers(s) of the flexible pavement and consists of a bituminous surface dressing or one or two layers of premixed bituminous material (generally asphalt concrete). Where premixed materials are laid in two layers, these are known as the wearing course and the binder course.

Switchbacks	Sequence of sharp curves at or near minimum radius employed to traverse a mountainous or escarpment terrain section.
Tangent	Portion of a horizontal alignment of straight geometrics.
Taper	Transition length between a passing place, auxiliary lane or climbing lane and the standard roadway.
Tie Bar	A deformed steel bar or connector embedded across a joint in a rigid concrete slab to prevent separation of abutting slabs.
Traffic	Vehicles, pedestrians and animals travelling along a route.
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.
Traffic Volume	Volume of traffic usually expressed in terms of average annual daily traffic (AADT).
Transition Curve	Curve whose radius changes continuously along its length, used for the purpose of connecting a tangent with a circular arc or two circular areas of different radii.
Transition Length	Length of the transition curve.
Trunk Road	International Trunk Road linking centres of international importance and crossing international boundaries or terminating at international ports.
Turning Lanes	Lanes which separate turning vehicles from the through traffic lanes.
Turning Roadway	Channelized turning lane at an at-grade intersection.
Typical Cross-Section	A cross-section of a road showing standard dimensional details and features of construction.
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.
Unbound Pavement Materials	Naturally occurring or processed granular material which is not held together by the addition of a binder such as cement, lime or bitumen.
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. For example, a warrant for signalisation of an intersection would include the traffic volumes that should be exceeded before signalisation is considered as a traffic control option. Note that, 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.
Waste	Material excavated from roadway cuts but not required for making the embankments. It must be pointed out that this material is not necessarily wasted as the word implies, but can be used in widening embankments, flattening slopes, or filling ditches or depressions for erosion control.
Wearing Course	The top course of an asphalt surfacing or, for gravel roads, the uppermost layer of construction of the roadway made of specified materials.
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.
Welded Wire Fabric	Welded steel wire fabric for concrete reinforcement.

ABBREVIATIONS

AADT	Average Annual Daily Traffic
AASHO	American Association of State Highway Officials (previous designation)
AASHTO	American Association of State Highway and Transportation Officials
ADT	Average Daily Traffic
AC	Asphalt Concrete
ACV	Aggregate Crushing Value – a measure of aggregate strength
a_1, a_2, a_3	Strength coefficients. The empirical strength coefficients used for weighting the contribution of each layer of the pavement to the overall structural number (SN). They are modified by the drainage coefficients, m (see below).
ASTM	American Society for Testing Materials
BS	British Standard
BVC	Beginning of Vertical Curve
CADD	Computer Aided Design and Drafting
CBR	California Bearing Ratio (as described in AASHTO T 193)
CMP	Corrugated Metal Pipe
CRCP	Continuously Reinforced Concrete Pavement
DCP	Dynamic Cone Penetrometer
DHV	Daily High Volume
DS	Design Standard
DTM	Digital Terrain Model
DV	Design Vehicle
EELPA	Ethiopian Electric Light and Power Authority
ELH	Economic Limit of Haul
EMA	Ethiopian Mapping Authority
ERA	Ethiopian Road Authority
ESA	Equivalent standard axles. A measure of the damaging effect of vehicle axles (*see ERA Pavement Design Manual Volume I*).

ETB	Ethiopian Birr
ETC	Ethiopian Telecommunications Corporation
EVC	End of Vertical Curve
FH	Free Haul
FWD	Falling Weight Deflectometer
GM	Grading Modulus
GPS	Global Positioning System
h_1, h_2, h_3	Thicknesses of pavement surface, base and sub-base layers (existing or required)
HAL	Horizontal Alignment Listing
HMA	Hot Mixed Asphalt
IDA	International Development Agency
ICL	Initial Consumption of Lime test
IRI	International Roughness Index
JPCP	Jointed Plain Concrete Pavement
JRCP	Jointed Reinforced Concrete Pavement
LAA	Los Angeles Abrasion Value – a measure of aggregate strength
LoS	Loss of Service
m_2, m_3	Drainage coefficients. Factors used to modify *layer coefficients* in flexible pavements to take account of climate, the effectiveness of internal pavement drainage and moisture sensitivity.
MDD	Maximum Dry Density
MUTCD	Manual on Uniform Traffic Control Devices
NC	Normal Crown section (or Normal Camber)
NDT	Non-destructive test
PC	Point of Curvature
PCC	Portland Cement Concrete
PI	Point of Intersection
PMS	Pavement Management System
PSD	Passing Sight Distance

PT	Point of Tangency
PVI	Point of Vertical Intersection
RCP	Reinforced Concrete Pipe
RoW	Right of Way
RPSD	Reduced Passing Sight Distance
RRD	Representative Rebound Deflection
RTA	Road Transport Authority
SC	Spiral to circular curve transition point
SSD	Stopping Sight Distance
S1 to S6	Subgrade strength classes used to characterize the subgrade in pavement design (see *ERA Pavement Design Manual Volume I Flexible Pavements*).
SN and MSN	Structural Number and Modified Structural Number. An index of overall pavement strength based on the thicknesses and strengths of each pavement layer.
SN_{eff} and MSN_{eff}	Effective Structural Number of an existing pavement
T1 to T8	Traffic classes used to characterize the anticipated traffic in terms of ESA for flexible pavement design purposes
TBM	Temporary Benchmark
ToR	Terms of Reference
TRL	Transport Research Laboratory, UK (formerly TRRL)
TRRL	Transport and Road Research Laboratory, UK
TS	Tangent to Spiral transition point
VOC	Vehicle Operating Costs
VFB	Voids Filled with Bitumen
VIM	Voids in the Mix
VMA	Voids in the Mineral Aggregate
VPI	Vertical Point of Intersection

1 INTRODUCTION

1.1 General

The procedures for the geometric design of roads presented in this manual are applicable to most trunk roads, link roads, main access roads, collector roads, feeder roads and unclassified roads as defined by ERA in this Geometric Design Manual.

The manual does not deal in detail with the design of major grade separated junctions between restricted access freeways (motorways) although the topic is introduced sufficiently for some of the simpler designs to be carried out; nor does the manual deal with the detailed design of drainage features. For this the reader should refer to ERA's *Drainage Design Manual.*

The use of the procedures described in this manual will help in achieving reasonable uniformity in geometric design for a given set of conditions.

1.2 Organization of the Manual

The organization of this manual and the design process are outlined in this Section.

After this introduction, a summary of the standards developed within the manual together with departures from standards and the method of dealing with departures from standards, are given in Chapter 2.

Chapter 3 deals with preliminary design considerations. Specifically, it lists procedures for identifying potential alignments in a 'route corridor' selection process.

Chapter 4 discusses survey requirements.

Chapter 5 discusses those external controls and criteria affecting the selection of the geometric design values. These include a discussion of the road hierarchy and functional classification; terrain considerations; the design vehicle; the importance of population density and character of adjoining land use; design traffic volume; and design speed.

Cross sectional elements of the road include lane widths, shoulders, cross-fall, side slopes and back slopes, roadside ditches, clear zones, and right-of-way. These issues are discussed in Chapter 6.

A significant element in the geometric design of roads is sight distance. Chapter 7 develops the formulae and application of both stopping and passing sight distances.

Chapter 8 deals with horizontal alignment and provides information on tangent sections, transition curves, curve elements including circular curves, reverse, broken-back, compound and isolated curves. Lane widening requirements for curves are explained, and the special considerations in switchback, or hairpin, curves are discussed. Super-elevation standards are then developed, including rates, run-off and shoulder super-elevation.

Chapter 9 is devoted to the issue of vertical alignment. Sub-sections deal with the topics of crest and sag curves, maximum and minimum gradients, climbing lanes, and vertical clearances.

Chapter 10 discusses the need for phasing between horizontal and vertical design, problems associated with mis-phasing and possible corrective actions.

Chapter 11 discusses at-grade junctions, including design requirements, selection of junction type, T-junctions, cross junctions and roundabouts; sight distances; and junction elements including turning lanes and traffic islands.

Grade-separated junctions are discussed in a similar manner in Chapter 12. Topics included are the choice of scheme, geometric standards, design principles and types of junctions.

Safety and miscellaneous items are dealt with in Chapter 13. These include the design of safety rest areas and scenic overlooks, bus lay-bys and parking bays, parking lanes, public utilities, railway grade crossings, safety barriers and emergency escape ramps.

Items defined as roadway furniture are discussed in Chapter 14. This includes traffic signs, road markings, marker posts, traffic signals and lighting.

Finally, the appendices provide information on numerous aspects deemed best placed in an appendix rather than in the main body of the text. This includes the classification of roads and the current extent of ERA's road network; details of friction factors for road surfaces; further explanation of the method of determining sight distances; requirements for the location of utilities; typical cross sections; supplementary information on procedures for design and on the preparation of plans and drafting; terms and definitions, and abbreviations used in the manual; and information concerning the use of mass haul diagrams for minimizing the extent of earthworks.

2 SUMMARY OF STANDARDS AND DEPARTURES FROM STANDARDS

2.1 Introduction

Geometric design is the process whereby the layout of the road through the terrain is designed to meet the needs of the road users. The principal geometric features are the road cross-section and the horizontal and vertical alignments. Appropriate standards depend upon the following factors:

1. Topography.
2. Traffic volume and traffic composition.
3. Design vehicle characteristics.
4. Road function.
5. Design speed and other speed controls.
6. Control of access.
7. Road safety considerations.
8. Land use and physical features.
9. Economic and financial considerations.
10. Environmental considerations.
11. Alternative construction technologies.

2.2 Design Standards

The design standards apply to divided highways, trunk and link roads, main access and collector roads, and feeder roads.

An overview of the design standards for each road class is shown in Table 2-1 relating road functional classification, traffic volumes and design speeds. Table 2.2 summarises shoulder widths and the widening of shoulders to provide the facilities required in urban and peri-urban areas.

It will be noted that the design standards do not include any three-lane, two-way roads. Such roads were intended to function as two-lane two-way roads but with a continuous central lane for overtaking manoeuvres to minimise congestion. Such roads were found to have a considerably higher capacity than two-lane, two-way roads but they were found to be dangerous because the practical effect of the three-lane cross-section was to concentrate the faster vehicles of the two opposing traffic streams in a common lane resulting in unsafe operations. Such roads have been universally abandoned.

Table 2.3 summarises the geometric adjustments to be made to the standards for roads that generally carry only relatively light traffic, when the number of large and heavy vehicles (3-axles and > 10 tonnes) comprises a significant proportion of the traffic stream.

Similarly Table 2.5 summarises the geometric adjustments to be made if the number of non 4- wheeled (and above) motorised traffic (i.e. motor cycles, etc.) and non-motorized traffic including pedestrians exceeds certain values and requires special provision for reasons of safety and smooth traffic flow.

Finally, Tables 2.6 to 2.17 show the design standards in more detail for each class of road.

The manual describes the derivation of the standards and provides information on how to use them.

2.3 Departures from Standards

It is anticipated that there may be situations where the designer will be compelled to deviate from these standards. An example of a Departure from Standard is the inclusion of a switchback or the use of a gradient greater than the desirable value. Where the designer departs from a standard, he must obtain written approval from ERA. The Designer shall submit the following information to ERA:

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

The Designer must submit all major and minor Departures from Standards to the respective regional directorate for evaluation. If the proposed Departures from Standards are acceptable, the Departures from Standards will be submitted to the Quality Assurance, Road Inspection and Safety Directorate for final approval.

Table 2-1: Road Classification, AADT, Carriageway Widths and Design Speeds

<table>
<tr><th colspan="5" rowspan="2">Road Functional Classification</th><th rowspan="2">Design Standard</th><th rowspan="2">Design Traffic Flow (AADT) (Mid-life)</th><th rowspan="2">Surface Type</th><th>Width (m)</th><th colspan="5">Design Speed (km/hr.)</th></tr>
<tr><th>Carriageway</th><th>Flat</th><th>Rolling</th><th>Mountainous</th><th>Escarpment</th><th>Urban/Peri-Urban</th></tr>
<tr><td rowspan="6"></td><td rowspan="4"></td><td rowspan="3"></td><td></td><td rowspan="6">TRUNK</td><td>DC8</td><td>10,000 –15,000</td><td>Paved</td><td>Dual 2 x 7.3</td><td>120</td><td>100</td><td>85</td><td>70</td><td>50</td></tr>
<tr><td rowspan="7">LINK</td><td>DC7</td><td>3,000 – 10,000</td><td>Paved</td><td>7.3</td><td>120</td><td>100</td><td>85</td><td>70</td><td>50</td></tr>
<tr><td>DC6</td><td>1,000 – 3,000</td><td>Paved</td><td>7.0</td><td>100</td><td>85</td><td>70</td><td>60</td><td>50</td></tr>
<tr><td rowspan="7">MAIN ACCESS</td><td>DC5</td><td>300 – 1,000</td><td>Paved</td><td>7.0</td><td>85</td><td>70</td><td>60</td><td>50</td><td>50</td></tr>
<tr><td rowspan="6">COLLECTOR</td><td rowspan="2">DC4 [2]</td><td rowspan="2">150 – 300</td><td>Paved</td><td>6.5 – 7.0 [1]</td><td rowspan="2">70</td><td rowspan="2">60</td><td rowspan="2">50
45[3]</td><td rowspan="2">30</td><td rowspan="2">50</td></tr>
<tr><td>Unpaved</td><td>7.0 – 7.5 [1]</td></tr>
<tr><td rowspan="6">FEEDER</td><td rowspan="6"></td><td rowspan="2">DC3 [2]</td><td rowspan="2">75 – 150</td><td>Paved</td><td>6.0</td><td rowspan="2">70</td><td rowspan="2">60</td><td rowspan="2">50
45[3]</td><td rowspan="2">30</td><td rowspan="2">50</td></tr>
<tr><td>Unpaved</td><td>7.0</td></tr>
<tr><td rowspan="4"></td><td rowspan="2">DC2 [2]</td><td rowspan="2">25 – 75</td><td>Paved</td><td>3.3</td><td rowspan="2">60</td><td rowspan="2">50</td><td rowspan="2">40
35[3]</td><td rowspan="2">25</td><td rowspan="2">50</td></tr>
<tr><td>Unpaved</td><td>6.0</td></tr>
<tr><td rowspan="2"></td><td rowspan="2"></td><td>DC1</td><td>1 – 25</td><td>Unpaved</td><td>4.5</td><td>50</td><td>40</td><td>30</td><td>20</td><td>40</td></tr>
<tr><td>Basic Access</td><td><10</td><td>Unpaved</td><td>3.5</td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

Notes

1. Choice of *carriageway and shoulder widths* also depends on numbers of Large Heavy Vehicles defined as vehicles with 3 or more axles and with GVW > 10 tonnes (see Table 2.3)
2. The choice of *design standard* depends on the numbers of Large Heavy Vehicles as well as AADT.
3. Design speed is adjusted slightly to provide the same minimum radius of horizontal curvature as for the paved road option in mountainous terrain.
4. Transition curves are required for all road standards except roads traversing escarpments and road classes DC1 and Basic Access.

Table 2-2: Shoulder Widths on Each Side of the Carriageway

Design Standard	Rural Terrain/Shoulder Width (m)				High numbers of PCUs[2]	Town Section Widths[2] (m)		
	Flat	Rolling	Mountainous	Escarpment		Parking Lane	Foot way	Median[!]
DC8	3.0	3.0	0.5 – 2.5	0.5 – 2.5		3.5	2.5 (min)	5.0 (min)
DC7	3.0	3.0	0.5 – 2.5	0.5 – 2.5		3.5	2.5	Barrier[!]
DC6	1.5 - 3.0++	1.5 - 3.0	0.5 – 1.5	0.5 – 1.5	3.0	3.5	2.5	n/a
DC5	1.5	1.5	0.5	0.5	2.75	3.5	2.5	n/a
DC4 paved	1.25	1.25	0.5	0.5	2.75	3.5	2.5	n/a
DC4 unpaved	Note 1	Note 1	Note 1	Note 1	+2.0	3.5	2.5	n/a
DC3 paved	1.0	1.0	0.5	0.5	2.5	3.5	2.5	n/a
DC3 unpaved	Note 1	Note 1	Note 1	Note 1	+1.5	3.5	2.5	n/a
DC2 paved	1.5	1.5	1.0	1.0	None	n/a	n/a	n/a
DC2 unpaved	Note 1	Note 1	Note 1	Note 1	+1.25	n/a	n/a	n/a
DC1	Note 1	Note 1	Note 1	Note 1	None	n/a	n/a	n/a
Basic Access	Note 1	Note 1	Note 1	Note 1	None	n/a	n/a	n/a

Notes 1 Shoulders not defined for unpaved roads.

2 Additional widths not applicable on escarpments and may not be possible in mountainous terrain.

Modifications to the standards summarised above are made for excessive numbers of large heavy vehicles in the traffic stream and for high volumes of non-motorized vehicles, motor cycles, pedestrians (and other forms of intermediate transport) on some of the lower road standards. Large heavy vehicles are defined as vehicles with three or more axles and gross vehicle weights of greater than 10 tonnes.

Table 2-3: Adjustments for excessive numbers of large heavy vehicles

<table>
<tr><th>AADT</th><th>Original design class</th><th>No. of large heavy vehicles</th><th>Revised design class or modification to standards</th></tr>
<tr><td rowspan="2">150 - 300</td><td rowspan="2">DC 4 paved</td><td>Heavy vehicles > 80</td><td>DC 5</td></tr>
<tr><td>40 < Heavy vehicles < 80</td><td>Increase running surface to 7.0m and decrease shoulders to 1.0m</td></tr>
<tr><td rowspan="2">150 - 300</td><td rowspan="2">DC 4 unpaved</td><td>Heavy vehicles > 80</td><td>DC 5</td></tr>
<tr><td>40 < Heavy vehicles < 80</td><td>Increase road width to 7.5m</td></tr>
<tr><td rowspan="2">75 - 150</td><td>DC 3 paved</td><td rowspan="2">Heavy vehicles > 30</td><td rowspan="2">DC 4</td></tr>
<tr><td>DC 3 unpaved</td></tr>
<tr><td rowspan="2">25 - 75</td><td rowspan="2">DC 2</td><td>Heavy vehicles > 20</td><td>DC 3</td></tr>
<tr><td>Heavy vehicles < 10</td><td>DC 1 may be used</td></tr>
</table>

Passenger Car Units (PCUs) are defined as shown in Table 2.4 and the modifications are summarised in Table 2.5. The modifications are not possible on escarpments. In mountainous terrain they are only possible along relatively flat sections. In these circumstances the PCU 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 2-4: PCU values

Vehicle	PCU value
Pedestrian	0.15
Bicycle	0.2
Motor cycle	0.25
Bicycle with trailer	0.35
Motor cycle taxi (bajaj)	0.4
Motor cycle with trailer	0.45
Small animal-drawn cart	0.7
Bullock cart	2.0
All based on a passenger car = 1.0	

Table 2-5: Adjustments for PCUs greater than 300 AADT.

Standard	AADT	Surface	Modification
DC 8	>10,000	Paved	None
DC 7	3,000 – 10,000	Paved	None
DC 6	1,000 – 3,000	Paved	Shoulders width increased to 3.0 m each side
DC 5	300 – 1,000	Paved	Shoulders increased to 2.75 m each side
DC 4	150 - 300	Paved	Shoulder width increased to 2.75 m each side
		Unpaved	Increase carriageway width by 4.0 m
DC 3	75 - 150	Paved	Shoulder width increased to 2.5 m each side
		Unpaved	Increase carriageway width by 3.0 m
DC 2	25 - 75	Paved	None (paved sections will be short)
		Unpaved	Increase carriageway width by 2.5 m

Table 2-6: Geometric Parameters for Design Class DC8 Paved (AADT > 10,000 Dual Carriageway)

Design Element		Unit	Flat	Rolling	Mountain	Escarp't	Urban Peri-Urban
Design Speed		km/hr	120	100	85	70(2)	50
Width of running surface		m	2x7.3				2x7.3+
Width of shoulders		m	Table 2.2				
Minimum. Stopping Sight Distance	g = 0%	m	285	210	155	110	65
	g = 5%	m	330	240	175	120	70
	g = 10%	m	400	285	205	140	75
Minimum Horizontal Curve Radius(2)	SE = 4%	m	780	515	350	215	95
	SE = 6%	m	685	455	310	195	85
	SE = 8%	m	610	410	280	175	80
Transition Curves Required			Yes	Yes	Yes	No	No
Max. Gradient (desirable)		%	3	4	6	6	6
Max. Gradient (absolute)		%	5	6	8	8	7
Minimum Gradient		%	0.5	0.5	0.5	0.5	0.5
Maximum Super-elevation		%	8	8	8	8	4
Min. Crest Vertical Curve (1)		K	185	100	55	30	10
Min. Sag Vertical Curve		K	36	25	18	12	7
Normal Cross-fall		%	2.5	2.5	2.5	2.5	2.5
Shoulder Cross-fall		%	4	4	4	4	4
Right of Way		m	50	50	50	50	50

Notes

1. These values are based on an object height of 0.2m. Use of a different sized object (see Chapter 9) requires ERA approval.
2. The design speed on escarpments may be dictated by the severity of the terrain and the curve radius (plus curve widening) that can be achieved on the hairpin bends.

Table 2-7: Geometric Parameters for Design Class DC7 Paved (AADT 3,000-10,000)

Design Element		Unit	Flat	Rolling	Mountain	Escarp't	Urban Peri-Urban
Design Speed		km/hr	120	100	85	70(3)	50
Width of Running Surface		m	7.3				7.3+
Width of Shoulders		m	Table 2.2				
Minimum. Stopping Sight Distance	g = 0%	m	285	210	155	110	65
	g = 5%	m	330	240	175	120	70
	g = 10%	m	400	285	205	140	75
Min. Passing Sight Distance		m	805	675	330(2)	270(2)	180(2)
% Passing Opportunity		%	50	50	25	0	20
Minimum Horizontal Curve Radius(3)	SE = 4%	m	780	515	350	215	95
	SE = 6%	m	685	455	310	195	85
	SE = 8%	m	610	410	280	175	80
Transition Curves Required			Yes	Yes	Yes	No	No
Max. Gradient (desirable)		%	3	5	7	7	6
Max. Gradient (absolute)		%	5	7	9	9	8
Minimum Gradient		%	0.5	0.5	0.5	0.5	0.5
Maximum Super-elevation		%	8	8	8	8	4
Min. Crest Vertical Curve (1)		K	185	100	55	30	10
Min. Sag Vertical Curve		K	36	25	18	12	7
Normal Cross-fall		%	2.5	2.5	2.5	2.5	2.5
Shoulder Cross-fall		%	4	4	4	4	4
Right of Way		m	50	50	50	50	50

Notes
1. These values are based on an object height of 0.2m. Use of a different sized object (see Chapter 9) requires ERA approval.
2. To abort passing manoeuvre.
3. The design speed on escarpments may be dictated by the severity of the terrain and the curve radius (plus curve widening) that can be achieved on the hairpin bends.

Table 2-8: Geometric Parameters for Design Class DC6 Paved (AADT 1,000-3,000)

Design Element		Unit	Flat	Rolling	Mountain	Escarp't	Urban Peri-Urban
Design Speed		km/hr	100	85	70	60[3]	50
Width of running surface		m	7.0				7.0+
Width of shoulders		m	Table 2.2 and Table 2.5				
Minimum. Stopping Sight Distance	g = 0%	m	210	155	110	85	65
	g = 5%	m	240	175	120	90	70
	g = 10%	m	285	205	140	105	75
Min. Passing Sight Distance[2]		m	375	330	270	230	180
% Passing Opportunity		%	50	33	25	0	20
Minimum Horizontal Curve Radius[4])	SE = 4%	m	515	350	215	145[3]	95
	SE = 6%	m	455	310	195	135[3]	85
	SE = 8%	m	410	280	175	120[3]	-
Transition Curves Required			Yes	Yes	Yes	No	No
Max. Gradient (desirable)		%	3	5	7	7	6
Max. Gradient (absolute)		%	5	7	9	9	8
Minimum Gradient		%	0.5	0.5	0.5	0.5	0.5
Maximum Super-elevation		%	8	8	8	8	4
Min. Crest Vertical Curve [1]		K	100	55	30	17	10
Min. Sag Vertical Curve		K	25	18	12	9	7
Normal Cross-fall		%	2.5	2.5	2.5	2.5	2.5
Shoulder Cross-fall		%	4	4	4	4	4
Right of Way		m	50	50	50	50	50

Notes

1. These values are based on an object height of 0.2m. Use of a different sized object (see Chapter 9) requires ERA approval.
2. To abort passing manoeuvre.
3. Target value. The design speed on escarpments may be dictated by the severity of the terrain and the curve radius (plus curve widening) that can be achieved on the hairpin bends.

Table 2-9: Geometric Parameters for Design Class DC5 Paved

(AADT 300-1,000)

Design Element		**Unit**	**Flat**	**Rolling**	**Mountain**	**Escarp’t**	**Urban Peri-Urban**
Design Speed		km/hr	85	70	60	50(3)	50
Width of Running Surface		m	7.0				7.0+
Width of Shoulders		m	Table 2.2 and Table 2.5				
Minimum. Stopping Sight Distance	g = 0%	m	155	110	85	65	65
	g = 5%	m	175	120	90	70	70
	g = 10%	m	205	140	105	75	75
Min. Passing Sight Distance(2)		m	330	270	230	180	180
% Passing Opportunity		%	25	25	15	0	20
Minimum Horizontal Curve Radius(4)	SE = 4%	m	350	215	145	95(3)	95
	SE = 6%	m	310	195	135	85(3)	85
	SE = 8%	m	280	175	120	80(3)	-
Transition Curves Required			Yes	Yes	No	No	No
Max. Gradient (desirable)		%	4	6	8	8	7
Max. Gradient (absolute)		%	6	8	10(4)	10(4)	9
Minimum Gradient		%	0.5(5)	0.5	0.5	0.5	0.5
Maximum Super-elevation		%	8	8	8	8	4
Min. Crest Vertical Curve (1)		K	55	30	17	10	10
Min. Sag Vertical Curve		K	18	12	9	7	7
Normal Cross-fall		%	2.5	2.5	2.5	2.5	2.5
Shoulder Cross-fall		%	4	4	4	4	4
Right of Way		m	50	50	50	50	50

Notes

1 These values are based on an object height of 0.2m. Use of a different sized object (see Chapter 9) requires ERA approval.

2 To abort passing manoeuvre.

3 The design speed on escarpments may be dictated by the severity of the terrain and the curve radius (plus curve widening) that can be achieved on the hairpin bends.

4 Length not to exceed 200m and relief gradients required (< 6% for minimum of 200m).

5 In some circumstances in very flat terrain this can be reduced to 0.3%

Table 2-10: Geometric Parameters for Design Class Paved DC4[1] (AADT 150-300)

Design Element		Unit	Flat	Rolling	Mountain	Escarp't	Urban Peri-Urban
Design Speed		km/hr	70	60	50	25	50
Width of Running Surface		m	6.5[2]	6.5[2]	6.5	6.5	6.5[2]
Width of Shoulders		m	Table 2.2, Table 2.3 and Table 2.5				
Minimum. Stopping Sight Distance	g = 0%	m	110	85	65	25	65
	g = 5%	m	120	90	70	25	70
	g = 10%	m	140	105	75	25	75
Min Passing Sight Distance		m	270	230	180	50	180
Minimum Horizontal Curve Radius	SE = 4%	m	215	145	95	20[3]	95
	SE = 6%	m	195	135	85	18[3]	85
	SE = 8%	m	175	120	80	17[3]	-
	SE = 10%	m	160	110	75	16[3]	-
Max. Gradient (desirable)		%	4	6	8	8	7
Max. Gradient (absolute)		%	6	8	10[4] [5]	10[4] [5]	9
Minimum Gradient		%	0.5[7]	0.5	0.5	0.5	0.5
Min. Crest Vertical Curve [6]		K	30	17	10	2	10
Min. Sag Vertical Curve		K	12	9	7	2	7
Normal Cross-fall		%	3	3	3	3	3
Shoulder Cross-fall		%	6	6	3	3	6

Notes

1. If there are more than 80 Large Heavy Vehicles per day then DC5 should be used (Table 2.9).
2. If the number of Large Heavy Vehicles is >40 per day then this should be increased to 7.0m. (Table 2.3).
3. On hairpin stacks the minimum radius may be reduced to a minimum of 15m.
4. Length not to exceed 200m and relief gradients required (< 6% for minimum of 200m).
5. If the number of Large Heavy Vehicles < 10 this can be increased to 12%.
6. These values are based on an object height of 0.2m. Use of a different sized object (see Chapter 9) requires ERA approval.
7. In some circumstances in very flat terrain this can be reduced to 0.3%

Table 2-11: Geometric Parameters for Unpaved DC4[(1)]
(AADT 150-300)

Design Element		Unit	Flat	Rolling	Mountain	Escarp't	Urban Peri-Urban
Design Speed		km/hr	70	60	45[(7)]	25	50
Road Width [(4)]		m	7.0[(2,3)]	7.0[(2,3)]	7.0	7.0	7.0[(2, 3, 4)]
Minimum. Stopping Sight Distance	g = 0%	m	125	95	60	25	70
	g = 5%	m	145	110	70	25	80
	g = 10%	m	175	130	75	30	90
Min. Passing Sight Distance		m	270	230	165	50	180
Min. Horizontal Radius		m	255	175	90	25[(5)]	115
Max. Gradient (desirable)		%	4	6	6	6	4
Max. Gradient (absolute)		%	6	9	9	9	6
Minimum Gradient		%	0.5[9]	0.5	0.5	0.5	0.5
Max. Super-elevation		%	6	6	6	6	6
Min Crest Vertical Curve [(8)]		K	35	20	9	1	11
Min Sag Vertical Curve		K	12	9	5	2	7
Normal Cross-fall[(6)]		%	6	6	6	6	6

Notes

1 If there are more than 80 Large Heavy Vehicles then DC5 should be used (Table 2.3).

2 If the number of Large Heavy Vehicles is >40 but <80 then the road width should be increased to 7.5m (Table 2.3).

3 If the number of PCUs is high, see Table 2.5.

4 In urban and peri-urban areas parking lanes and footpaths might be required and the roadway may need to be paved (Table 2.2).

5 On hairpin stacks the minimum radius may be reduced to a minimum of 15m.

6 Cross-fall can be reduced to 4% where warranted (e.g. poor gravel - for safety, low rainfall).

7 The design speed has been adjusted to provide the same minimum radii of curvature as for the paved DC4 standard.

8 These values are based on an object height of 0.2m. Use of a different sized object (see Chapter 9) requires ERA approval.

9 In some circumstances in very flat terrain this can be reduced to 0.3%.

Table 2-12: Geometric Parameters for Design Class Paved DC3[1] (AADT 75-150)

Design Element		Unit	Flat	Rolling	Mountain	Escarp't	Urban Peri-Urban
Design Speed		km/hr	70	60	50	25	50
Width of Running Surface		m	6.0	6.0	6.0	6.0	6.0
Width of Shoulders[2] [3]		m	Table 2.2 and Table 2.5				
Minimum. Stopping Sight Distance	g = 0%	m	110	85	65	25	65
	g = 5%	m	120	90	70	25	70
	g = 10%	m	140	105	75	25	75
Min. Passing Sight Distance		m	275	225	175	60	175
Minimum Horizontal Curve Radius	SE = 4%	m	215	145	95	20[3]	95
	SE = 6%	m	195	135	85	18[3]	85
	SE = 8%	m	175	120	80	17[3]	-
Max. Gradient (desirable)		%	6	7	10[4]	10[4]	7
Max. Gradient (absolute)		%	8	9	12[4)]	12[4]	9
Minimum Gradient		%	0.5[6]	0.5	0.5	0.5	0.5
Min. Crest Vertical Curve [5]		K	30	17	10	2	10
Min. Sag Vertical Curve		K	12	9	7	2	7
Normal Cross-fall		%	3	3	3	3	3
Shoulder Cross-fall		%	6	6	3	3	6

Notes

1 If there are more than 30 Large Heavy Vehicles, then DC4 should be used (Table 2.3).
2 On hairpin stacks the minimum radius may be reduced to a minimum of 15m.
3 In urban and peri-urban areas parking lanes and footpaths may be required (Table 2.2).
4 Length not to exceed 200m and relief gradients required (< 6% for minimum of 200m).
5 These values are based on an object height of 0.2m. Use of a different sized object (see Chapter 9) requires ERA approval.
6 In some circumstances in very flat terrain this can be reduced to 0.3%.

Table 2-13: Geometric Parameters for Design Class Unpaved DC3[1]

(AADT 75-150)

Design Element		Unit	Flat	Rolling	Mountain	Escarp't	Urban[2] Peri-Urban
Design Speed		km/hr	70	60	45[6]	25	50
Road Width		m	7.0[3]	7.0[3]	6.5	6.5	7.0[2] [3]
Minimum Stopping Sight Distance	g = 0%	m	125	95	60	25	70
	g = 5%	m	145	110	70	25	80
	g = 10%	m	175	130	75	30	90
Min Horizontal Radius		m	255	175	90	25[4]	115
Max. Gradient (desirable)		%	4	6	6	6	4
Max. Gradient (absolute)		%	6	9	9	9	6
Minimum Gradient		%	0.5[8]	0.5	0.5	0.5	0.5
Max. Super-elevation		%	6	6	6	6	6
Min. Crest Vertical Curve [7]		K	35	20	9	1	11
Min. Sag Vertical Curve		K	12	9	5	2	7
Normal Cross-fall[5]		%	6	6	6	6	6

Notes

1 If there are more than 30 Large Heavy Vehicles, then DC4 should be used (Table 2.3).

2 In urban and peri-urban areas, parking lanes and footpaths may be required and the roadway may need to be paved (Table 2.2)

3 If the number of PCUs is high, see Table 2.5.

4 On hairpin stacks the minimum radius may be reduced to a minimum of 15m.

5 Cross-fall can be reduced to 4% where warranted (e.g. poor gravel -for safety, low rainfall).

6 Design speed is adjusted to provide the same minimum radii of curvature as for the paved DC3 standard.

7 These values are based on an object height of 0.2m. Use of a different sized object (see Chapter 9) requires ERA approval.

8 In some circumstances in very flat terrain this can be reduced to 0.3%.

Table 2-14: Geometric Parameters for Design Class Paved DC2[(1)] (AADT 25-75)

Design Element		Unit	Flat	Rolling	Mountain	Escarp't	Urban Peri-Urban
Design Speed		km/hr	60	50	40	20	50
Width of Running Surface		m	3.3				
Width of Shoulders[(3)]		m	1.5	1.5	1.0	1.0	1.5[(2)]
Minimum. Stopping Sight Distance	g = 0%	m	85	65	45	20	65
	g = 5%	m	90	70	47	20	70
	g = 10%	m	105	75	50	20	75
Minimum Horizontal Curve Radius	SE = 4%	m	145	95	55	15[(7)]	95
	SE = 6%	m	135	85	50	15[(7)]	85
	SE = 8%	m	120	80	50	15[(7)]	-
Max. Gradient (desirable)		%	6	7	10[(4)]	10[(4)]	7
Max. Gradient (absolute)		%	8	9	12[(4)]	12[(4)]	9
Minimum Gradient		%	0.5[6]	0.5	0.5	0.5	0.5
Max. Super-elevation		%	6	6	6	6	6
Min. Crest Vertical Curve[(5)]		K	17	10	5	2	10
Min. Sag Vertical Curve		K	9	7	4	1	7
Normal Cross-fall		%	3	3	3	3	3
Shoulder Cross-fall		%	6	6	3	3	6

Notes

1. If there are more than 20 Large Heavy Vehicles, then DC3 should be used.
2. In urban and peri-urban areas, parking lanes and footpaths may be required and the roadway may need to be paved (Table 2.2).
3. If the number of PCUs is high, see Table 2.5.
4. Length not to exceed 200m and relief gradients required (< 6% for minimum of 200m).
5. These values are based on an object height of 0.2m. Use of a different sized object (see Chapter 9) requires ERA approval.
6. In some circumstances in very flat terrain this can be reduced to 0.3%.
7. On hairpin stacks the minimum radius may be reduced to a minimum of 13m.

Table 2-15: Geometric Parameters for Design Class Unpaved DC2 (1, 2)

(AADT 25-75)

Design Element		Unit	Flat	Rolling	Mountain	Escarp't	Urban Peri-Urban
Design Speed		km/hr	60	50	35(7)	20	50
Road Width(4,5)		m	6.0				6.0(3)
Minimum. Stopping Sight Distance	g = 0%	m	85	65	40	20	65
	g = 5%	m	90	70	45	20	70
	g = 10%	m	105	75	50	20	75
Min. Horizontal Radius		m	175	115	55	15(6)	115
Max. Gradient (desirable)		%	4	6	6	6	4
Max. Gradient (absolute)		%	6	9	9	9	6
Minimum Gradient		%	0.5(10)	0.5	0.5	0.5	0.5
Max. Super-elevation		%	6	6	6	6	6
Min. Crest Vertical Curve (9)		K	20	11	5	1	11
Min. Sag Vertical Curve		K	9	7	3	1	7
Normal Cross-fall (8)		%	6	6	6	6	6

Notes

1 If the number of Large Heavy Vehicles >20 then DC3 should be used.

2 If the number of Large Heavy Vehicles <10 then DC1 may be used.

3 In urban and peri-urban areas, parking lanes and footpaths may be required and the roadway may need to be paved (Table 2.2).

4 If the number of PCUs is high, see Table 2.5.

5 Road widths may be reduced at the discretion of the Engineer and approval of the Client to address specific local conditions, especially in mountainous areas.

6 On hairpin stacks the minimum radius may be reduced to a minimum of 13m.

7 Design speed is adjusted to provide the same minimum radii of curvature as for paved standard.

8 Cross-fall can be reduced to 4% where warranted (e.g. poor gravel-for safety, low rainfall).

9 These values are based on an object height of 0.2m. Use of a different sized object (see Chapter 9) requires ERA approval.

10 In some circumstances in very flat terrain this can be reduced to 0.3%.

Table 2-16: Geometric Parameters for Design Class DC1
(AADT 1-25)

Design Element		Unit	Flat	Rolling	Mountain	Escarp't (1) (2)	Urban Peri-Urban
Design Speed		km/hr	50	40	30	20	40
Road Width		m	4.5				
Minimum. Stopping Sight Distance	g = 0%	m	70	60	30	20	50
	g = 5%	m	80	70	35	20	55
	g = 10%	m	90	75	37	20	60
Min. Horizontal Radius		m	115	65	35	15(1)	65
Max. Gradient (desirable)		%	4	6	6	6	4
Max. Gradient (absolute)		%	10[2]	10[2]	12[2]	12(2)	9
Minimum Gradient		%	0.5[3]	0.5	0.5	0.5	0.5
Min. Crest Vertical Curve		K	11	6	2	1	6
Min. Sag Vertical Curve		K	7	4	3	1	4
Normal Cross-fall		%	6	6	6	6	6

Notes
1 On hairpin stacks the minimum radius may be reduced to 13m.
2 Length not to exceed 200m and relief gradient required (< 6% for minimum of 200m).
3 In some circumstances in very flat terrain this can be reduced to 0.3%.

For the lowest category of road it may sometimes be necessary to adopt a basic access only approach. For such roads it may be too expensive to provide a design speed but minimum absolute standards must be applied. These are summarised in Table 2.17.

Table 2-17: Minimum Standards for Basic Access

Characteristic	Minimum requirements	
Radius of horizontal curvature	12m absolute but up to 20m depending on expected vehicles	
Vertical curvature K value for crests K value for sags	 2.5 0.6	
Maximum gradients Open to all vehicles Open only to cars and pick-ups	 14% 16%	
Minimum stopping sight distance	Flat and Rolling terrain Mountainous Escarpments	50m 35m 20m

3 SITE INVESTIGATION FOR ROUTE SELECTION AND DESIGN

3.1 Introduction

Site investigation is a vital and integral part of the location, design and construction of a road. It provides essential information on the characteristics of the soils along the possible alignments, availability of construction materials, topography, land use, environmental issues and socio-political considerations related to the following:

- i) Selection of the route/alignment of the road;
- ii) Location of water crossings and drainage structures;
- iii) Design information for the road pavements, bridges and other structures;
- iv) Identifying areas of possible geotechnical problems requiring specialist investigation;
- v) Identifying areas of possible problem soils requiring additional investigation and treatment;
- vi) Location and assessment of suitable, locally available, borrow and construction material.

This list indicates that the main component of site investigations is focussed on what is generally described as ‘engineering’ or, more precisely, ‘geotechnical engineering’. However, various other types of survey are required. Hydrological surveys are required to determine the water flows that determine the drainage design of the road, including bridges; traffic surveys are required to estimate the numbers of vehicles, both motorised and non-motorised, that will use the road; surveys are required to evaluate environmental impacts and how to control them; surveys are required in which the local communities are consulted about the road project; and so on.

Information obtained during the site investigation is used by the design engineer to prepare and refine the detailed engineering design. This information is usually contained within a series of documents that are prepared by the design engineer, initially for consideration by the client and, ultimately, to develop the tender and draft contract documents. These documents normally include separate volumes dealing with the following design aspects:

- i) Alignment survey and geometric design;
- ii) Traffic volume and traffic loading;
- iii) Construction materials and subgrade properties;
- iv) Pavement structural design;
- v) Hydrology, drainage and water crossings;
- vi) Ground stability and geotechnical design;
- vii) Environmental considerations (EIA and outline EMP);
- viii) Social and complementary activities;
- ix) Engineer’s cost estimate.

Not all projects require the same detailed surveys. Road projects fall into the following categories:

1) A new road that follows the general alignment of an existing track or trail;
2) Upgrading a lower class of road to a higher class

3) A completely new road where nothing currently exists.

Some realignment, and therefore site investigation, will almost certainly be necessary when upgrading an existing road and considerably more will be required when converting a track into an all-weather route. Major site investigations are usually only needed when designing and building a completely new road. In all cases the extent and quality of any investigation has a strong influence on the selection of the most cost-effective route and road design.

Roads of all standards require sufficient investigation to provide enough data and information to enable the engineer to optimise the design. In this respect, it is the job of the design engineer to ensure that a well-designed and organised site investigation is undertaken. The design engineer must therefore specify a programme for the site investigation teams (survey, materials, geotechnical, socio-environmental) that will provide adequate information and data to examine the feasibility of all the options under consideration.

3.2 Site Investigation Techniques

Site investigation techniques encompass a large range of methods. The amount and type of investigation that is needed for a specific road depends on the nature of the proposed project and the environment in which it is to be built. For full details of individual site investigation techniques the reader is referred to the appropriate manuals as follows:

Route Selection Manual (ERA). This deals with the selection of the optimal route for a new road. It provides guidance on the appraisal of factors affecting route selection including topography, engineering geology, hydrology, social and environmental factors, and economic return on investment. It recommends a multi-criteria analysis for the comparison of competing options, and the identification of the most favourable solution.

Site Investigation Manual (ERA). This deals with site investigation procedures for materials for road construction and geotechnical factors affecting the performance of the road. It covers subgrade soil investigations, including problem soils, selection of materials for the construction of the pavement layers, foundation investigations for structures, and the assessment of slope stability.

Pavement Design Manual Volume I Flexible Pavements (ERA). This manual includes information on surveys for estimating traffic volumes and axle loads for design purposes, information on the use of subgrade strength data and information on the required properties of the materials for the pavement layers.

Drainage Design Manual (ERA). This includes information for assessing the size of culverts and bridge openings based on estimates of the water flows obtained from survey information including rainfall intensity, catchment area and catchment characteristics.

Design Manual for Low Volume Roads (ERA). This manual includes guidance on all required survey information for the design of low volume roads. For such roads a lower level of survey effort is acceptable in view of the lower cost of a typical LVR compared with the cost of higher road classes.

4 SURVEY REQUIREMENTS

4.1 Introduction

This chapter describes the survey requirements associated with the geometric design process. Survey data for design purposes consist of mapping in sufficient detail for the level of design being undertaken. In some instance a Digital Terrain Model (DTM) for use with computer design software may be required.

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

The following factors should be considered when determining the survey data required:

1. Size and scope of the project
2. Time requirements to move from data collection to the start of design
3. Estimated data collection cost
4. Level of accuracy and detail needed

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

4.2 Method of Data Collection: Photogrammetry and Field Survey

Topographical ground survey has the capability of achieving greater accuracy than photogrammetry. The effectiveness of aerial photography depends on location (urban or rural), ground cover, etc.

Photogrammetry is sufficiently accurate for most applications and can be more cost effective for all but small projects. For mapping and DTMs, photogrammetry is usually the preferred choice. However, if a project road is short, has dense foliage, or requires only mapping of limited features, a field survey is the logical choice. Some fieldwork will be required for most projects to compile property lines, right-of-way (ROW) information, and data for utilities, culverts, trees, buildings, bridges and road sign data unavailable through aerial photography.

Elevations of photogrammetric DTM points on hard surfaces are accurate to within ±60 millimetres. If more precise vertical accuracy is required for areas of a project, the data must be obtained through a field survey. If precise vertical accuracy is required, such as for highway pavement elevations, or if obstructed views occur, photogrammetric data can be supplemented with survey elevations. It is recommended that survey data be collected before the photogrammetric data to help assure the accuracy of the DTM. Table 4-1 provides guidelines for when photogrammetry, survey, or a combination of both should be used. It should be noted that this table is a guideline only, and that appropriate methods also depend on factors such as project location (rural or urban), and road length.

4.3 Survey Data Products

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

Mapping used for design development and right-of-way is generally provided at 1:2000 scale. In addition, 1:500 scale mapping can be provided for highly complex projects or bridge sites.

If vertical data is required, it is provided as DTMs that have replaced contours and cross sections. DTMs allow more flexibility for the designer and potential follow-up use in construction. Using the DTM approach, earthwork quantities for payment purposes can be calculated based on the final design centreline. With DTM data, cross sections for stakeout purposes can be generated at any desired interval and with any desired station numbering. DTMs should be requested if the project will be designed using CADD design software, if cross sections along multiple alignments are required, or if construction needs require centreline cross section data.

Photogrammetric mapping products consist of 2D graphic files and 3D DTM surfaces (i.e. *.ttn or *. dtm files) for use in the CADD system.

Field survey data must be obtained using Total Stations or GPS, the output from which should be compatible with the CADD system to be used.

Table 4-1: Survey Data Requirements

Project Type	Appropriate Terrain Data	Collection Method	Typical Bandwidth
New Construction or Upgrading	Mapping[1]& DTM[2]	Photogrammetry and field survey	100 m - 150 m
Resurfacing or Rehabilitation			
Over 0.5 km long	Mapping[1]& DTM[2]	Field survey	30-120m
Under 0.5 km long	No Mapping[2]	N/A	N/A
Resurfacing	Mapping[1]and DTM (may be required)	Field survey	30-100m
Intersection Reconstruction	Mapping[1] and DTM	Photogrammetry and field survey	100-150m
New Bridge and Bridge Replacement			
Over 6 ha	Mapping[1] and DTM	Survey	100-150m
Under 6 ha	No Mapping[2]	N/A	100-150m

1. Mapping scales will be 1:2000
2. Instead of new mapping, consider the use of record plans and photo enlargements supplemented by field survey checks.
3. For bridges, mapping scales must be enlarged. The bandwidth should be a distance up/downstream consistent with the ERA *Bridge Design Manual*.

4.4 Survey Data by Project Type

Table 4.1 shows the amount and type of survey data that is generally required and the method of data collection for each project type. Some projects may require a combination of products.

Bridge rehabilitation projects generally do not require any terrain data unless necessary for major rehabilitation. When terrain data is necessary, use the same terrain guidelines used for new bridges and bridge replacement projects.

Resurfacing and rehabilitation projects usually do not require project wide mapping. However, limited data may be required such as pavement elevations where super-elevation adjustments are anticipated. Other projects may require very limited data such as pavement and shoulder edges. If only limited data is needed then a field survey is preferred.

4.5 Field Surveys

Detailed ground surveys along the length of the proposed project roads should use the most up-to-date surveying equipment such as Total Stations or GPS to examine the road alignment and cross sections and any bridge sites and culvert sites that are considered necessary to complete the detailed design and the estimation of quantities.

A controlled traverse should be established using GPS, coordinated and tied into the national grid system. These points shall be referenced in the field in permanent concrete posts and shall be shown on the plan and profile drawings. Since projects are to be carried out using CADD it is essential to organize the topographic surveys as the first step of a coherent data collection - design chain. Therefore the whole topographic survey should be made using Total Stations which will directly record the alignment, profile, and cross section data on computer files which will be retrieved by the CADD system during the design stage.

The existing road centreline should be identified and staked every 20 metres. The coordinates are recorded automatically using the Total Station.

The start and end of horizontal curves, and roadway cross sections are also recorded.

The following methodology is used to establish the original setting out data for the reestablishment of the centreline:

i) The control traverse is established, monumented, and the coordinates in X, Y, Z accurately measured and tied in to the National Grid System. Concrete beacons are established at intervals of 150 - 300 metres. These beacons are located as close as possible to the limit of the road reserve and where one beacon is visible from the other along the road.
ii) Using the established polygon network of beacons, each of the centreline points is coordinated.
iii) Using the method of least squares, the best-fit horizontal alignment through the coordinated points is established.

Cross sections are levelled for each centreline point to a minimum of 25 metres distance from the centreline. Road edges, cuts, ditch edges, culverts, hilltops, water crossings and embankments are taken. Topographic survey information is collected for an adequate distance on each side of the centreline and cross sections at appropriate intervals, depending on the type of terrain.

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

New alignments are recommended where inadequate horizontal sight distances and sharp curves exist and wherever the existing route is not to the standards. Therefore, the vertical and horizontal alignments should be given due attention with respect to sight distance, maximum grade, maximum length of grade criteria, and safety. In introducing new alignments, major bridges and drainage structures should be retained as control points or as node points on the new centreline wherever they are in good condition. Should there be a need for realignment of the existing road, topographic surveys along the chosen realignment should be established. The centreline of the road is defined at every 20 metre interval. Topographical cross-sections, extending at least 25 metres either side of the centreline, are taken at each of the centreline reference points.

Recommended bridge and major culvert sites are surveyed and mapped at a scale of 1:500 with contours at 0.5 metres intervals or greater in the more severe sections. Each of the site surveys is tied to the elevation of the primary traverse.

Topographic data are processed using the project computer as work progress.

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

Each survey crew shall be equipped with an electronic Total Station, a three-prism line road, and an electronic field book. The Total Station will have unlimited on-board data storage by utilizing integrated circuit data storage cards. Each card should be capable of storing 500 points. The use of an electronic field book allows the Total Station operator to code in descriptions and other important information for each data point.

Survey teams can carry out the topographic field work requirements as follows:

i) One team for the location of the control points, whether GPS or National Grid;
ii) One team to survey the centre line and the longitudinal profile,
iii) One team to survey the cross sections, and
iv) One team for the land acquisition survey.

The output from the Total Station and data collection is a computer file which contains horizontal coordinate points, vertical elevations, and a description of all points needed to develop a full topographic map of the area. The computer file must be capable of being downloaded directly into a computerized design and drafting program. These programs should then be able to generate, if so desired, a three-dimensional Digital Terrain Model. The plot should be checked and verified by the surveyors shortly after the fieldwork. The step-by-step procedure to be used for data collection is as follows:

i) Base map information may be obtained from the Ethiopian Mapping Authority.
ii) The road is divided into survey subsections at a distance of 2 km. A coordinate system is established for the roadway sections.
iii) A Global Positioning System (GPS) Survey Control System is used to locate precisely the topographic control points required for the project. A series of receiver stations is used to generate these points with a high degree of accuracy by measuring signals generated from a group of three or more geosynchronous satellites. The establishment of the zero-zero coordinate point is determined after a review of the existing coordinate information.
iv) A bench circuit is run using accepted level procedures and degrees of accuracy. Each benchmark is monumented and assigned coordinate points. Once horizontal and vertical controls are established, the survey crews cross-section the roadway alignment. The width of each cross-section is dependent on the terrain of the roadway and the different natural and man-made features. The minimum distance from centreline for cross-sectioning is 25 metres. The number of points depends on the topography, road lane characteristics, project road features within the right-of-way, and as required for the purpose of design and computations of quantities for earthwork and profile course. Areas where existing roads cross the alignment and areas affected by cross drainage require additional topographic survey. Such supplementary topographic survey is carried out concurrently during the progress of survey along the alignment.

4.6 Topography

All points of detail are located by a right-angle offset wherever possible, with chainage and offset being recorded. Only when this method is not possible are other techniques such as bearing and distance, to be used. Structures (buildings, bridges, culverts, etc.) are checked by detailed face measurements. All physical features adjacent to the line, whether natural or artificial, are recorded within a range of 25 metres either side of the centreline in open country and in small villages (market centres) and towns.

4.7 Bench Marks

Standard benchmark levelling procedure is followed with the following limitations observed:

i) A benchmark is established every 300 - 500 metres along the line close to the right of way, and at all major structures (bridges and box culverts). Bench marks must be inter-visible (see Figure 4.1 Standard Bench Mark).
ii) Every benchmark is check levelled by a forward run and a subsequent backward run forming a closed "loop".
iii) The following standard of accuracy is to be maintained:

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

Where C = maximum permissible error of closure in centimetres,
K = distance between bench marks in kilometres

This gives Table 4.2 for comparison of accuracy

Table 4-2: Required Level of Accuracy for Surveys

K (km)	0.5	1.0	2.0	5.0	10.0
C (cm)	± 0.7	± 1.0	± 1.4	± 2.2	± 3.2

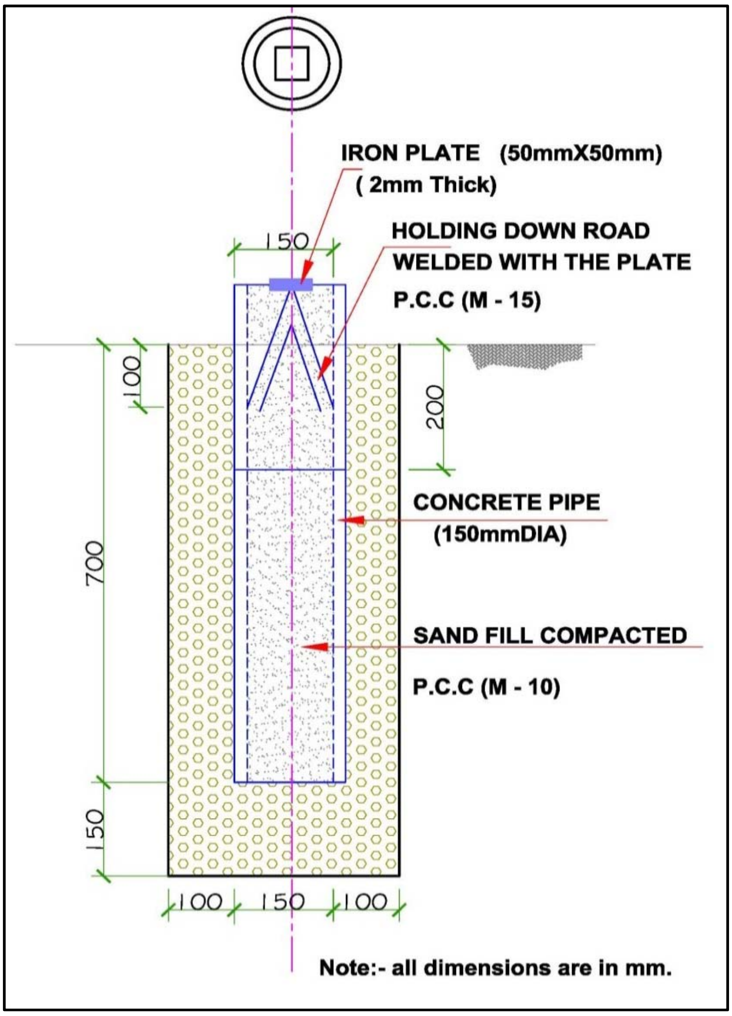


Figure 4-1: Standard Bench Mark (alternative designs are available)

4.8 Profile and Cross Sections

Profile and cross-section levelling can be run simultaneously. All profiling is done by direct levelling to two decimal places of a metre, and wherever practicable the cross section levels are to be obtained in the same manner. Where impracticable, direct levelling may be replaced or extended by the use of either a hand level or Rhodes arc for cross-section work.

Where it is not possible to close a day's work on a permanent benchmark e.g. because of failing light, a sudden storm, etc. a Temporary Benchmark (TBM) shall be established from which the work may be resumed.

Cross-sections shall be taken to a minimum distance of 25 metres each side of the centreline. Profile levelling is run between each pair of consecutive benchmarks, previously established, and the leveller must close on each successive benchmark as a turning point. For each succeeding length of profile any error from the preceding length shall be discarded; the elevation of the previously established intervening benchmark is accepted and used for the succeeding length of profile.

The disclosure on each previously established benchmark shall not exceed 3.2 centimetres. The surveyor should check that closure on each successive benchmark is within the prescribed tolerance. Where the difference is outside this limit the run must be repeated.

4.9 Photogrammetry

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

As an example, photogrammetry from aerial photographs of 1:25,000 scale can yield uncontrolled contour mapping at a maximum scale of 1:5,000, with contours at 5-metre intervals. It is advisable to correct the contour model by establishing two ground control points in each stereo pair, by tying points on the photographs either to the national or local grid, or by GPS. The main problems associated with the use of photogrammetry relate to the lack of ground definition in areas of shade, cloud or dense forest cover.

It may be worthwhile taking photography with an 80 percent photograph overlap in order to be able to select an appropriate air base for the amount of exaggeration required. For interpretation in areas of high relief the viewer should use every photograph in the run (80 percent overlap, giving a short air base and minimum relief exaggeration). For areas of low relief the viewer can select every other photograph, to double relief exaggeration. In extreme circumstances of very flat ground the viewer can select every third or even every fourth photograph, doubling the relief exaggeration each time.

The scale of photography is an important factor to consider in the reliability and ground resolution of the interpretation. Table 4.3 indicates the optimum scales of photography required to perform various desk study and design tasks.

Table 4-3: Air Photo Scales for Various Project Tasks

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

4.10 Detailed Survey and Alignment Design

With the route corridor confirmed, the alignment engineer, with a survey team, will flag the approximate centreline. An approximate alignment should first be drawn onto photogrammetrically plotted contour maps and enlarged prints of aerial photographs in the office prior to embarking on detailed fieldwork.

If slope stability is critical to the alignment, then geotechnical-mapping surveys should be undertaken at scales of between 1:1,000 and 1:5,000. It is easier for personnel to locate themselves with the required accuracy if an approximate centreline has been set out, but the engineer should be prepared to modify the location of the centreline in the light of the geotechnical survey. In very difficult ground, these surveys should ideally be carried out prior to the centreline flagging exercise using aerial photograph enlargements or compass traverse as a means of location positioning.

With the alignment confirmed, detailed design of all subsequent works can proceed. Design of the detailed vertical and horizontal alignments requires topographical mapping at a scale of 1:1,000 with contour intervals at a maximum of 2 metres, using ground survey, photogrammetry or a combination of the two. Ground survey may be preferable at this stage due to the greater survey accuracy required. The use of photogrammetry requires the establishment of a base line traverse and the commissioning of air photography at a scale of between 1:5,000 and 1:10,000. Plan and profile drawings and schedules of earthwork and retaining wall designs and quantities can then be produced for contract documentation.

5 DESIGN CONTROLS AND CRITERIA: THE PRINCIPAL FACTORS DETERMINING GEOMETRIC STANDARDS

5.1 Introduction

The geometric design is influenced by numerous factors and controlled by specific design criteria as follows,

i) traffic volume expected to use the road;
ii) composition of the traffic;
iii) the design vehicle;
iv) design speed;
v) terrain;
vi) roadside population and land use;
vii) pavement type;
viii) soil type and climate;
ix) construction technology;
x) functional classification of the road;
xi) safety;
xii) economic and environmental considerations.

For a long road these factors usually vary along the route and therefore the design does not have to be constant for the whole length of a road. On the contrary, changes in the design are usually required in order to obtain proper balance between the road layout and the above factors, whilst maintaining construction costs at realistic levels.

The design process is illustrated in Figure 5.1. The following text describes some of the factors affecting design controls and criteria.

5.2 Traffic

For geometric design it is the physical dimensions of a vehicle that are important. A truck requires more space than a car, for example, and requires a larger curve radius for turning and a wider road for safe passing.

The way that vehicle size influences the geometric design of low and high volume roads is fundamentally different. When the volume of traffic is high, the road space occupied by different types of vehicle is an important element in designing for *capacity*, namely the highest traffic flow per hour that the road can carry. Large trucks, buses, and non-motorized vehicles, for example, have a strong influence. As traffic increases, traffic interaction increases until the traffic level exceeds the capacity of the road. When this occurs the traffic speeds and traffic flow decrease rapidly, and the level of service is severely compromised.

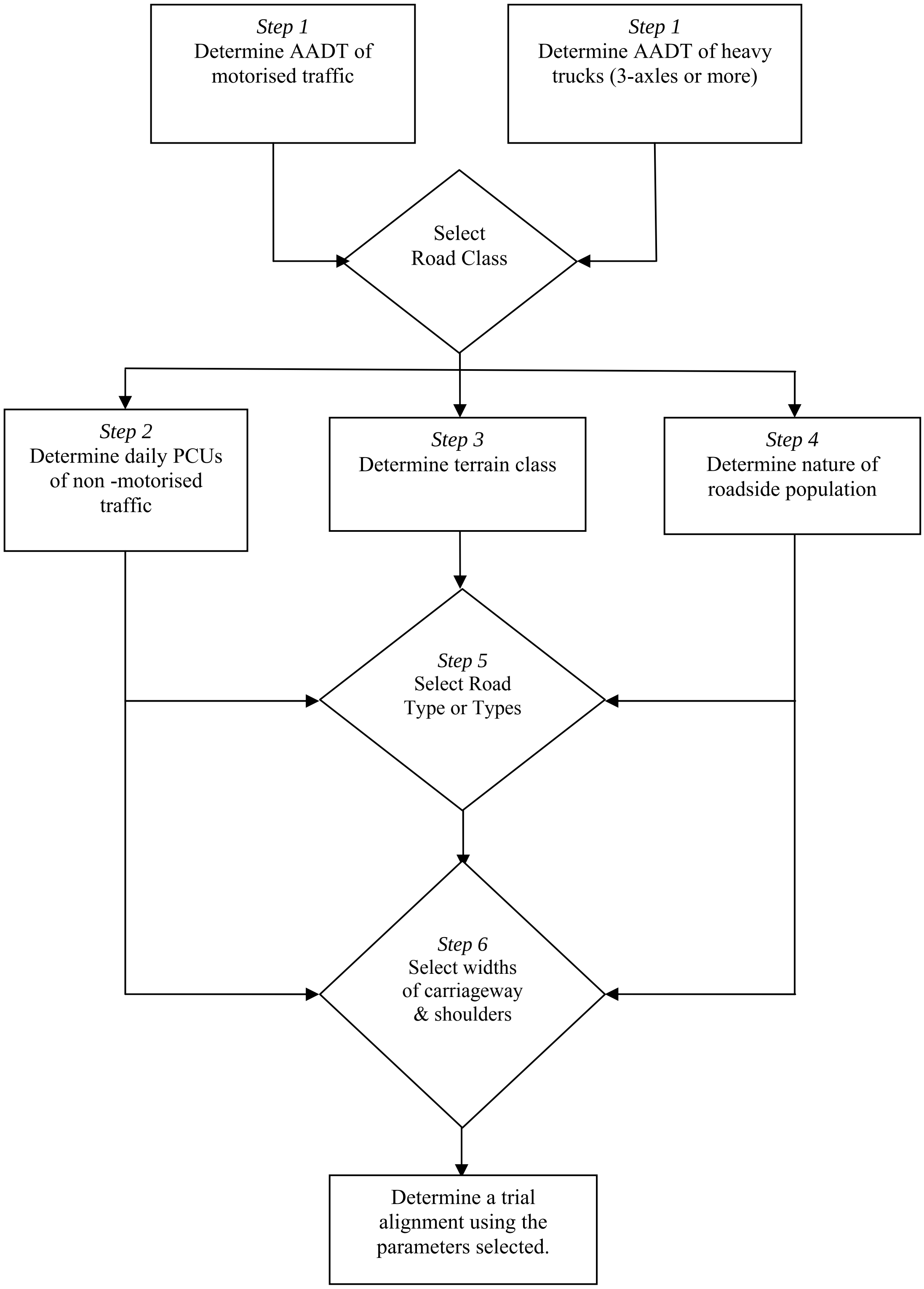


Figure 5-1: Selection Procedure for Appropriate Geometric Standards

Capacity is usually only an issue during the busiest hours of the day and for the heaviest trafficked roads, hence the number of vehicles carried during the peak hour defines the criterion for whether a higher capacity road is required. Normally, when a high volume road is being designed, traffic data should be available to determine the likely peak hour flow and traffic composition. For a two-lane, two-way road in flat terrain, capacity is reached when the traffic level (sum of both directions) approaches 3,200 equivalent passenger cars per hour (Harwood et al.; 1999). This peak traffic (per hour) is usually between 12 and 18 per cent of the ADT and a value of 15 per cent is a reasonable average. However, a proper traffic count and analysis should always be commissioned.

For a two-lane, two-way road in flat terrain with an hourly peak flow of 15 per cent of ADT, capacity occurs at the equivalent of an ADT of about 20,000 passenger cars per day. In rolling terrain the critical flow is equivalent to about 13,000 passenger cars per day; in mountainous terrain it is equivalent to about 7,000 passenger cars per day.

For the lower classes of road the volume of traffic is sufficiently low that congestion issues do not arise from traffic volume but from the disparity in speed between the variety of vehicles and other road users. In other words the traffic composition is the key factor.

Traffic volume for basic design purposes is based on the number of two (or more)-axled motorised vehicles. Consideration of other traffic (motor cycles, motor cycle-based taxis, non-motorised vehicles, pedestrians, etc) is taken into account by modifying the basic standards. This is done by combining the number of such road users using the PCU (passenger car unit) concept as shown in Table 5.1.

Motorcycle taxis (e.g. bajaj) are becoming popular in urban situations and it is only a matter of time before these spread to more rural areas and become adapted for freight as well as for passenger transport.

Table 5-1: PCU Values for non-4-wheeled motorised vehicles

Vehicle	PCU value
Pedestrian	0.15
Bicycle	0.2
Motor cycle	0.25
Bicycle with trailer	0.35
Motor cycle taxi (bajaj)	0.4
Motor cycle with trailer	0.45
Small animal-drawn cart	0.7
Bullock cart	2.0
All based on a passenger car = 1.0	

If the number of PCUs exceeds certain limits on some classes of road, the geometric design is modified to cater for them by increasing the width of the shoulders (Table 2.5).

Roads are designed to provide good service for many years and therefore the traffic level to be used in the design process must take into account traffic growth. Designing for the current traffic will invariably lead to inadequate standards in the future unless the traffic growth rate is extremely low.

To deal with these uncertainties it is generally expected that there is a strong correlation between traffic level, traffic growth rates and the functional classification of a road. Such classification is therefore often seen as a suitable alternative to represent traffic. However, although traffic levels often increase in line with the functional classification, this is not always true and, furthermore, the traffic levels and growth rates are likely to differ considerably between different areas and different regions of the country. For example, the traffic on a 'collector' road in one area of the country might be considerably more than on a 'main access' road in another area. The design of the road, and therefore the standards adopted, should reflect the *traffic level*.

In general it is expected that growth rates on roads that do not have 'through' traffic (essentially feeder roads) will be lower than the growth rates on the higher classes of road, but each situation should be treated on its own merits taking into account any expected future developments.

For geometric design it is the daily traffic that is important. The approach recommended for estimating the traffic for geometric design purposes is based on the estimated traffic level at the middle of the design life period and this therefore requires an estimate of the traffic growth rate. This method eliminates the risk of under-design that may occur if the initial traffic is used and the risk of over-design if traffic at the end of the design life is used. A design life of 15 to 20 years is recommended for paved roads and 10 years for unpaved roads.

Care is required if the traffic levels are such that the *capacity* of the road is likely to be exceeded before the end of the design period. Traffic congestion is costly and constructing extra capacity in the future by adding an additional lane is often difficult and also 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 in the beginning.

Normally a general growth rate is assumed or is provided by government based on the growth in registered vehicles during previous years. However, local development plans may indicate higher growth rates in some places.

Where there is no existing road, estimating the initial traffic is difficult and estimating future traffic even more so. However, in many cases where a new road is proposed, there is likely to be pedestrian traffic and therefore some information on the likely vehicular traffic after the road is constructed. In some cases an economic evaluation may have been carried out to justify the road in the first place. This will have provided an estimate of the amount of goods transported by pedestrians and the likely amount that will be carried by vehicles. In the unlikely event that there is no information available, the lowest class of engineered road (DC1) should be designed. Historical growth rates of similar roads in any specific area should be considered if available.

It should be noted that the issue of road classification to determine the standards to be applied is not difficult. A maximum of eight different standards are defined (DC1-DC8) and each will be applicable over a specific traffic range. These ranges are therefore quite wide and little difficulty should normally be experienced in assigning a suitable standard to a new road project. Where the expected traffic is near to a traffic boundary, it is prudent to use the higher classification.

5.3 The Design Vehicle

The size of the largest vehicle that is expected to use the road dictates many aspects of the geometric design. Such vehicles must be able to pass each other safely and to negotiate all aspects of the horizontal and vertical alignment. Thus the vehicle characteristics and dimensions affecting design include power to weight ratio, minimum turning radius and travel path during a turn, and vehicle height and width. The road elements affected include the maximum gradient, lane width, horizontal curve widening, and junction design. Trucks of different sizes are usually used for different road standards – the driver of a large 6-axle truck would not expect to be able to drive through roads of the lowest standards.

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. In Ethiopia four different design vehicles have been used, as shown in Tables 5.2 and 5.3, but there is very little difference between design vehicles DV2 and DV3. Roads designed for the single unit truck are suitable for the bus provided the front and rear overhangs of the bus are taken into account when designing curves; and this can be done with suitable curve widening where required as described later. The standard for the lowest class of road (DV1) is insufficient for DV2 and DV3.

The maximum turning path for a single unit truck, a single unit bus, and a semi-trailer combination are shown in Figures 5.2, and 5.3.

Table 5-2: Design Vehicle Characteristics

Design vehicle	Code	Height (m)	Width (m)	Length (m)	Front overhang (m)	Rear overhang (m)	Wheelbase (m)	Minimum turning radius (m)
4x4 Utility	DV1	1.3	2.1	5.8	0.9	1.5	3.4	7.3
Single unit truck	DV2	4.1	2.6	11.0	1.5	3.0	6.5	12.8
Single unit bus	DV3	4.1	2.6	12.1	2.1	2.4	7.6	12.8
Truck + semi-trailer	DV4	4.1	2.6	15.2	1.2	1.8	4.8+8.4 =13.2	13.7

Table 5-3: Design Vehicle for Each Road Class

Design standard	Design vehicle
DC8, DC7, DC6, DC5	DV4
DC4	DV4
DC3	DV3
DC2	DV3
DC1	DV1

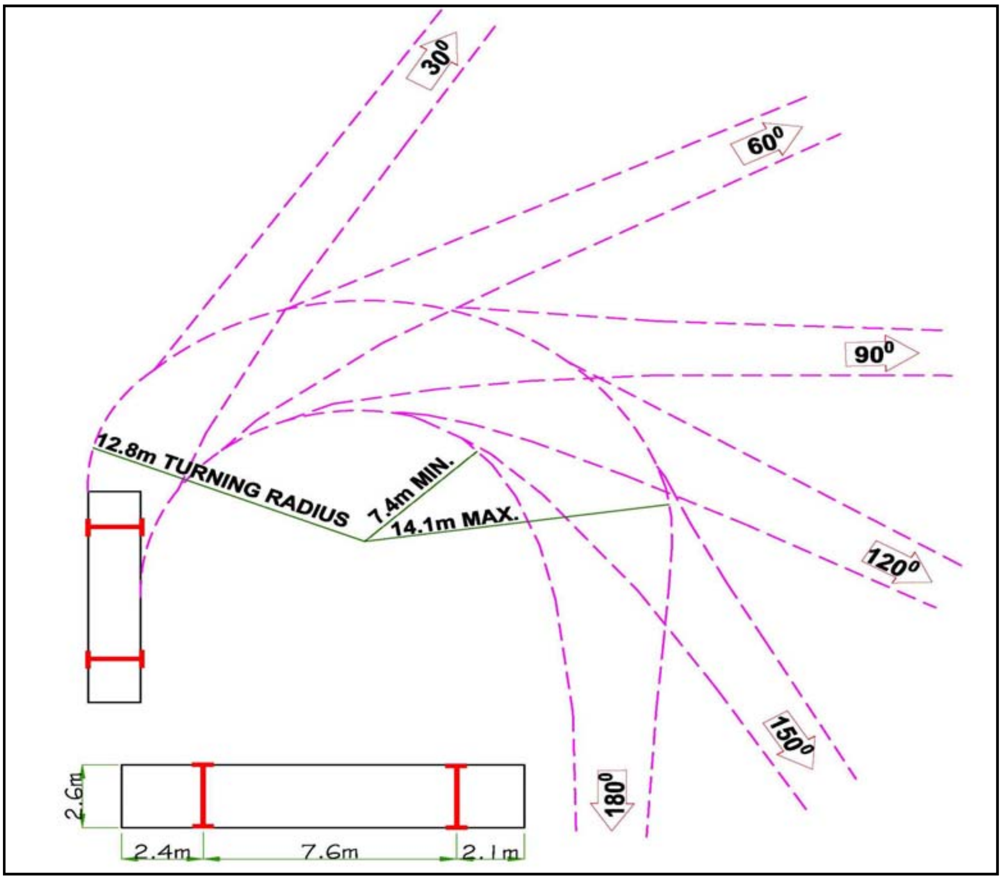


Figure 5-2: Dimensions and Turning Radius Path for Single Unit Bus (DV3)

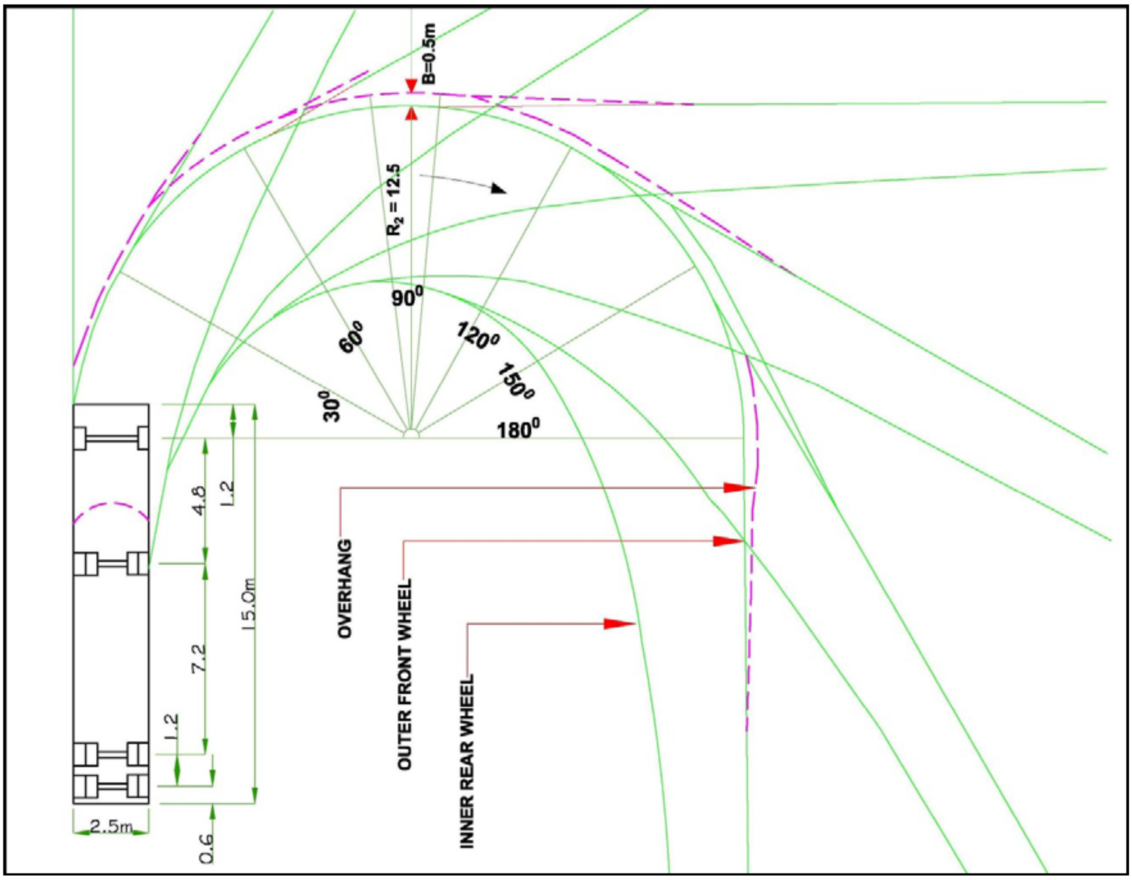


Figure 5-3: Dimensions and Turning Radius for a Semi-Trailer Combination

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

5.4 Design Speed

The design speed is used as an index which links 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. In practice, most roads will only be constrained to minimum parameter values over short sections or on specific geometric elements.

Design elements such as lane and shoulder widths, horizontal radius, super-elevation, sight distance and gradient are directly related to design speed. The design speeds given in Table 2.1 have been determined in accordance with the following guidelines:

i) On local roads whose major function is to provide access, high speeds are undesirable.

ii) Drivers usually adjust their speed to physical limitations and prevailing traffic conditions. Where a difficult location is obvious to the driver, he/she is more apt to accept a lower speed of operation.

iii) Economic considerations (road user savings vs. construction costs) may justify a higher design speed for a road carrying large volumes of traffic than for a less heavily trafficked road in similar topography.

iv) Change in design speed, if required due to a change in terrain class, should not occur abruptly but over sufficient distances to enable drivers to change speed gradually. The change in design speed should not be greater than one design speed step (10 or 15 km/hr) and the section with the lower geometric standards should be long enough to be clearly recognizable by drivers (not, for example, just one single curve).

v) It is often found that the physical terrain changes by two steps, typically from mountainous to flat terrain. Where possible in such circumstances, a transition section of road should be provided with limiting parameters equivalent to the rolling terrain type. Where this is not possible, i.e. a Departure from Standards, special attention shall be given to the application of warning signs and/or rumble strips to alert the driver to the changing conditions.

vi) The design of a road in accordance with a chosen design speed should ensure a safe design. The various design elements have to 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.

There are several limitations of the design speed concept that should be considered during design:

1. Selection of dimensions to accommodate a specified design speed does not necessarily ensure a consistent alignment design. Design speed is significant only when physical road characteristics limit the speed of travel. Thus, a road can be designed with a constant design speed, yet have considerable variation in achievable speeds and therefore appear to a driver to have a wide variation in character. For example, the radii of curves within a section should be consistent, not merely greater than the minimum value.
2. For horizontal alignments, design speed applies only to curves, not to the connecting tangents. Design speed has no practical meaning on tangents. As a result, the operating speed on a tangent, especially a long one, can significantly exceed the design speed of the road as a whole.
3. The design speed concept 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. For 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 with 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.
4. Vehicle operating speed is not necessarily synonymous with design speed. Drivers normally adjust speed according to their desired speed, posted speed, traffic volumes and perceived alignment hazards. The perception of hazard presented by the alignment may vary along a road designed with a constant design speed. The speed adopted by a driver tends to vary accordingly and may exceed the design speed.
5. Different alignment elements may have quite different levels of perceived hazard. Entering a horizontal curve too fast will almost certainly result in loss of control, so drivers adjust their speed accordingly. However, the possibility of a curtailed sight

distance concealing a hazard is considered a remote occurrence. Unfortunately drivers do not generally adjust their speed to compensate for sight distance restrictions.

To help overcome these weaknesses in the use of design speed to design individual geometric elements, speed profiles are used. A speed profile is a graphical depiction showing how the 85th percentile operating speed varies along a length of road. This profile helps to identify undesirably large differentials in the 85th percentile operating speed between successive geometric elements, e.g. a curve following a tangent, thereby also identifying where improvements should be made. In the common case in flat terrain where speeds considerably higher than the design speed occur on the straight sections of the lower standards of road, designing adjacent curves with a higher radius and higher design speed than the standard requires, will usually result in a slightly shorter, and therefore less expensive, section. However, the entire design should be based on the revised design speed to ensure the appropriate levels of safety are maintained.

For urban or peri-urban conditions, the design speed is influenced by other factors. In such areas, speed controls are frequently included. Traffic speeds are influenced by the presence of other vehicles travelling in and across the through lanes, physical and right-of-way constraints together with pedestrian and safety considerations. The current speed limit through villages is 30 km/h but it is possible that this limit will be increased in the future. However, a design speed through peri-urban or urban areas of 50 km/h is used for design although such segments are posted presently at 30 km/h. Legal speed limits should not be used as design parameters.

Traffic calming measures to reduce vehicle speeds in populated areas are discussed in Chapter 13.

5.5 Traffic Composition – Proportion of Heavy Vehicles

The density of roads in Ethiopia is quite low and one of the consequences of this is that the proportion of heavy vehicles in the traffic stream on relatively low volume roads is often quite high. Design standards DC2, DC3 and DC4 include a modification to cater for this.

For DC4, if the number of 'large' vehicles, defined as 3-axled (or more) trucks with GVWs (Gross Vehicle Weights) potentially greater than 12 tonnes, is greater than 40 the width of the paved surface is increased to 7.0m. If there are more than 80 Large Heavy Vehicles then the standard for DC5 should be used instead of DC4.

For DC3, if the number of Large Heavy Vehicles is greater than 25, design standard DC4 should be used and, for DC2, if the number of Large Heavy Vehicles exceeds 10 then DC3 should be used.

5.6 Terrain

5.6.1 Terrain Class

Terrain has the greatest effect on road costs therefore it is not economical to apply the same standards in all terrains. Fortunately drivers of vehicles are familiar with this and lower standards are expected in hilly and mountainous terrain. Four categories have been defined which apply to all roads and are illustrated in Figures 5.4 to 5.11.

Terrain class needs to be established before a road is designed hence it needs to be defined independently of the alignment that is finally selected for the road. It is determined by counting the number of 5-metre contours crossed by a straight line connecting the two ends of the road section in question according to the following definitions:

Flat	0-10 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally below 3%;
Rolling	11-25 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally between 3% and 25%.
Mountainous	26-50 five-metre contours per km. The transverse ground slopes perpendicular to the ground contours are generally above 25%.
Escarpment	Escarpments are geological features that require special geometric standards because of the engineering problems involved. They are characterised by more than 50 five-metre contours per km and the transverse ground slopes perpendicular to the ground contours are generally greater than 50%.

It is important to note that a road in mountainous terrain can be designed to follow a reasonably direct route involving considerable earthworks or it can follow contour lines more closely. This will require less earthworks but the road will be considerably longer. Whichever option is chosen, the classification of the terrain remains the same irrespective of road gradients, cross slopes or any other feature of the road itself.

5.6.2 Rolling Terrain

An important aspect of geometric design concerns the ability of vehicles to ascend steep hills. Roads that need to be designed for very heavy vehicles or for animal drawn carts require specific standards to address this, for example, special climbing lanes. Fortunately the technology of trucks has improved greatly over the years and, provided they are not grossly overloaded (which is a separate problem) or poorly maintained, they do not usually require special treatment. On the other hand, animal drawn vehicles are unable to ascend relatively low gradients and catering for them in rolling and mountainous terrain is rarely possible. Climbing lanes cannot be justified on LVRs and nor can the provision of very low maximum gradients.

5.6.3 Mountainous and Escarpment Terrain

In mountain areas the geometric standard takes account of the constraints imposed by the difficulty and stability of the terrain. This design standard may need to be reduced locally in order to cope with exceptionally difficult terrain conditions. Every effort should be made to design the road so that the maximum gradient does not exceed the standards shown in Tables 2.6 to 2.17; but where higher gradients cannot be avoided, they should be restricted in length. Gradients greater than 12% should not be longer than 250m and relief gradients are also required as indicated in the Tables. Horizontal curve radii of as little as 13m may be unavoidable, even though a minimum of 15m is specified.

Figure 5-4: Flat Terrain; Flat Roadway Alignment

Figure 5-5: Rolling Terrain; Flat Roadway Alignment

Figure 5-6: Rolling Terrain; Flat to Rolling Roadway Alignment

Figure 5-7: Rolling Terrain; Rolling Roadway Alignment

(NB: This design of bridge approach is not recommended because the bridge parapet is unprotected creating a hazardous situation.)

Figure 5-8: Mountainous Terrain; Flat Roadway Alignment

Figure 5-9: Mountainous Terrain; Mountainous Roadway Alignment

Figure 5-10: Escarpment Terrain; Mountainous Roadway Alignment

Figure 5-11: Escarpment Terrain; Escarpment Roadway Alignment

5.7 Roadside Population and Adjoining Land Use

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

- A wider cross section;
- Specifically designed lay-bys for passenger vehicles to pick up or deposit passengers;
- 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 Wereda seat or a larger populated area, an extra carriageway of 3.5m width is provided in each direction for parking and for passenger pick-up and a 2.5m pedestrian footpath is also specified. The latter is essentially the shoulder. In addition, the main running surface is paved and is at least 7.0m wide. Thus the road in such areas is similar to Class DC4 but with an additional wide parking/activities carriageway and a footpath on each side.

When passing through a Kebele seat, a 2.5m paved shoulder is specified but no additional footpath; although one could easily be provided if required. The carriageway is also increased to 7.0m and therefore the standard is very similar to DC4 but with wider shoulders.

These standards are not justified for the lower traffic levels of DC2, which is a single carriageway, unless the road is passing through a particularly well populated area that is not classified as a Kebele or Wereda seat but where additional traffic may be expected. In such circumstances the shoulders should be widened to 2.5 metres for the extent of the populated area.

5.8 Pavement Type

For a similar ‘quality’ of travel there is a difference between the geometric design standards required for an unsealed road (gravel or earth) and for a sealed road. This is because of the very different traction and friction properties of the two types of surfaces and the highly variable nature of natural materials. Higher geometric standards are generally required for unsealed roads. A road that is to be sealed at a later date should be designed to the higher, unsealed, geometric road standards.

5.9 Soil Type and Climate

Soil type affects the ideal geometric design, principally in terms of cross-section rather than in terms of the width of the running surface or road curvature. With some problem soils the cross-section can be adjusted to minimise the severity of the problem by, for example, minimising the speed of water flow; minimising the likelihood of excessive water inundation or penetration into the carriageway; and/or moving problems areas further away from the carriageway itself.

Ideally maximum gradients for unpaved roads should also depend on soil types but this is usually impracticable because, in most climatic regions, almost any gradient causes problems for unpaved roads. Recent research has demonstrated that gravel-surfaced roads are unsustainable in many more situations than has been thought previously and this applies equally to earth roads. Consequently every effort is being made to introduce or to develop more sustainable surfacings for use where unpaved roads deteriorate too quickly.

Such surfacings cannot usually be justified for long stretches of road where they are not essential hence the concepts of spot improvements and environmentally optimised design (EOD) are being developed and refined.

5.10 Construction Technology

In a labour-abundant economy it is usually beneficial to maximise the use of labour rather than rely predominantly on equipment-based methods of road construction. In such a situation the choice of technology might affect the standards that can be achieved, especially in hilly and mountainous areas. This is because:

- maximum cuts and fills need to be small;
- economic haul distances are limited to those achievable using wheel-barrows;
- mass balancing is achieved by transverse rather than longitudinal earth movements;
- maximum gradients follow the natural terrain gradients;
- horizontal alignments may be less direct.

The standards in hilly and mountainous terrain are always lower than in flat terrain but this reduction in standards need not necessarily be greater where labour-based methods are used. Following the contour lines more closely will make the road longer but the gradients can be less severe. Every effort should be made to preserve the same standards in the particular terrain encountered irrespective of construction method.

5.11 Administrative Function

It is sometimes necessary to take account of the administrative or functional classification of roads because a certain standard may be expected for each functional class of road irrespective of the current levels of traffic. Generally the hierarchy of administrative classification broadly reflects the traffic levels observed but anomalies are common where, for example, traffic can be lower on a road higher in the hierarchy. It is recommended that the standards selected should be appropriate to the task or traffic level of the road in question, but a minimum standard for each administrative class can also be defined if it is policy to do so.

The classification in Ethiopia comprises five functional classes.

Trunk Roads (Class I)

Centres of international importance and roads terminating at international boundaries are linked with Addis Ababa by trunk roads. They are numbered with an ‘A’ prefix: an example is the Addis-Gondar Road (A3). Trunk roads usually have a design AADT ≥1000, although they can have volumes as low as 150 AADT (see Table 2.1).

Link Roads (Class II)

Centres of national or international importance, such as principal towns and urban centres are linked to each other by link roads (see Table 2.1). A typical link road has an AADT greater than 300 although values can range between 75 and 10,000 AADT. They are numbered with a ‘B’ prefix. An example of a typical link road is the Woldiya - Debre Tabor - Woreta Road (B22), which links Woldiya on Road A2 with Bahir Dar on Road A3.

Main Access Roads (Class III)

Centres of provincial importance are linked by main access roads (see Table 2.1). The AADTs are typically between 150 and 300 but can range from 25 - 1000. They are numbered with a ‘C’ prefix.

Collector Roads (Class IV)

Roads linking locally important centres to each other, to a more important centre, or to higher class roads, are classified as collector roads. AADTs are between 25 and 300. They are numbered with a ‘D’ prefix (see Table 2.1).

Feeder Roads (Class V)

Any road linking a minor centre such as a market to rural communities is classified as a feeder road. AADTs are less than 150. They are numbered with an ‘E’ prefix.

Roads of the highest classes, trunk and link roads have, as their major function to provide mobility, while the primary function of lower class roads is to provide access. The roads of intermediate classes have, for all practical purposes, to provide both mobility and access.

The classification and description of all existing trunk, link and main access roads within the country, including road name, distance, type of road and road numbering, are given in Appendix A.

5.12 Safety

Experience has shown that simply adopting ‘international’ design standards from developed countries will not necessarily result in acceptable levels of safety on rural roads. The main reasons include the completely different mix of traffic, including relatively old, slow-moving and usually overloaded vehicles; a large number of pedestrians, animal drawn carts and, possibly, motorcycle-based forms of transport; poor driver behaviour; and poor enforcement of regulations. In such an environment, methods to improve safety through engineering design assume paramount importance.

Although little research has been published on rural road safety in Ethiopia, the following factors related to road geometry are known to be important:

- Vehicle speed;
- Horizontal curvature;
- Vertical curvature;
- Width of shoulders.

These factors are all inter-related and part of geometric design. In addition, safety is also affected by:

- Traffic level and composition;
- Inappropriate public transport pick-up/set-down areas;
- Poor road surface condition (e.g. potholes);
- Dust (poor visibility);
- Slippery unsealed road surfaces.

The last three factors are related to structural design and dealt with in the ERA *Pavement Design Manuals.*

Conflicts between motorised vehicles and pedestrians are often a major safety problem on many rural roads where separation is generally not economically possible. There are convincing arguments based on safety considerations for keeping traffic speeds low in mixed traffic environments rather than aiming for higher design speeds, as is the case for major roads. The use of wider shoulders is also suggested. These considerations have been incorporated into this manual.

Traffic level and composition are important. A considerable number of conflict situations can arise when the number of PCUs of non-motorised traffic is large even though the number of two (or more)-axled motorised traffic is quite low. Furthermore, the proportion of heavy vehicles on the LVRs of Ethiopia can be high, leading to more serious conflict situations. The overall traffic class standards are based on the number of two (or more)-axled motorised vehicles but additional safety features are based on:

- the number of PCUs of non 2-axled motorised vehicles and pedestrians; and
- the proportion of heavy vehicles in the motorised stream.

Pedestrians (and draft animals) find it uncomfortable to walk on poorly graded gravel shoulders containing much oversized material, especially in bare feet. They usually choose to walk on a paved running surface, if available, despite the greatly increased safety risk. Thus, provision of a wider unsurfaced shoulder does not ensure greater safety. On the approaches to market villages, where the pedestrian traffic increases greatly on market days, provision of a separate footpath is the best solution provided that the soil is suitable.

The following factors should be considered when designing for safety:

i) Wherever possible, non-motorised traffic should be segregated by physical barriers, such as raised kerbs (through villages and peri-urban areas).

ii) Designs should include features to reduce speeds in areas of significant pedestrian activity, particularly at crossing points. Traffic calming may need to be employed (see Chapter 13).

iii) To minimize the effect of a driver who has lost control and left the road, the following steps should be taken.
   - Steep open side-drains should be avoided since these increase the likelihood that vehicles will overturn. (Chapter 13).
   - Trees should not be planted immediately adjacent to the road.

iv) Guard rails should only be introduced at sites of known accident risk because of their high costs of installation and maintenance.

v) Junctions and accesses should be located where full safe stopping sight distances are available (see Chapter 11).

5.13 Matrix of Standards

There are eight basic standards based on traffic level DC1-DC8 (Table 2.1) and for each of these there are four standards to cope with terrain (flat, rolling, mountainous and escarpment). Additional standards for some road classes are provided to cater for roadside population/activities, traffic composition (essentially the number of PCUs of non-motorised traffic, including pedestrians (Table 2.5)), the percentage of heavy vehicles in the traffic stream (Table 2.3) and the type of road surface (paved or unpaved). The additional standards for traffic composition and roadside activities are essentially standards to enhance safety.

Once these factors have been taken into account, safety alone no longer affects the number of road standards because an acceptable level of safety must be applied to each road class. This will differ *between* classes (greater safety features for higher traffic) but not within classes. The administrative classification does not add to the number of standards either. If the traffic level indicates that a lower standard than would normally be acceptable based on administrative classification is sufficient, the road can be built to the minimum standard appropriate to its administrative classification.

In contrast to the judgements required for quantifying traffic, the standards themselves are largely dictated by the selected design speed and form a continuous range as design speed increases.

6 CROSS SECTION ELEMENTS

6.1 Introduction

A cross-section will normally consist of the carriageway, shoulders or curbs, drainage features, and earthwork profiles.

1. Carriageway- the part of the road constructed for use by moving traffic, including traffic lanes, auxiliary lanes such as acceleration and deceleration lanes, climbing lanes, and passing lanes, and bus bays and lay-bys.
2. Roadway- consists of the carriageway and the shoulders, parking lanes and viewing areas
3. Earthwork profiles- includes side slopes and back slopes

For urban cross-sections, cross-section elements may also include facilities for pedestrians, cyclists, or other specialist user groups. These include curbs, footpaths, and islands. It may also provide for parking lanes. For dual carriageways, the cross-section will also include medians. Typical Cross Sections are illustrated in Appendix E of this manual. Bus lay-bys, parking lanes, passing lanes, and viewing areas are discussed in Chapter 14.

Lane and shoulder widths should be adjusted to traffic requirements and characteristics of the terrain (Tables 2.1, 2.2, 2.3, 2.5). The cross-section may vary over a particular route because these controlling factors vary. The basic requirements are, however, that changes in 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. However, all narrow structures must be widened or replaced when the width across the structure is actually less than the carriageway width for the road standard in question.

6.2 Lane Widths

The width of the running surface and shoulders of a road largely define its cost, other things being equal, hence defining width standards that are acceptable both to the highway authority and to the travelling public is vital.

The width of the carriageway and shoulders has a great influence on road safety; wide roads generally being safer than narrower ones unless speeds on narrow roads are low. However, quantifying this in economic terms is inherently very difficult.

Wider roads require less shoulder maintenance because fewer vehicles drive over the vulnerable edge between the running surface and shoulder. Less maintenance is also needed because wheelpaths are wider, wheel loads less concentrated and less damage is caused to surfacings and road bases. Also, the weak area near to the edge of the running surface receives less traffic and deteriorates less rapidly. On the other hand, the area of surface that requires maintenance is larger on wide roads hence when re-surfacing is required the cost is higher.

There is insufficient quantifiable evidence to justify carriageway and shoulder widths purely on economic grounds using whole life cost principles. This requires knowledge about the costs of accidents and their causes, knowledge about the road deterioration and the effects of maintenance and much more. Knowledge about these issues has been collected for many years and is contained in the publications related to the Highway Development and Management Model HDM-4 and its predecessors. Such a model is very useful for calculating the optimum standards for various aspects of the design of a road but calculating optimum standards for widths remains a difficult problem. Because of this, road widths are based on the long term international evolution of such standards modified by local considerations. Judgement rather than precise calculations are required and the final standards are based on consensus. A primary principle, however, is that higher standards are required for higher levels of traffic. Standards for carriageway widths are shown in Table 2.1 for all road design standards.

Auxiliary lanes at intersections often help to facilitate traffic movement. Such added lanes are discussed in the Chapters 11 and 12.

6.3 Shoulders

A shoulder is the portion of the roadway contiguous to the carriageway for the accommodation of stopped vehicles; traditional and intermediate non-motorised traffic, animals, and pedestrians; emergency use; the recovery of errant vehicles; and lateral support of the pavement layers. Shoulder widths for the different design standards, terrain type, and urban/rural environment are shown in Table 2.2. Shoulders are not specifically defined for gravel and earth roads because, in practice, the road material spreads across the trafficable area with no distinct demarcation. On paved roads, shoulders vary from a minimum of 0.5m up to 3.0m depending on the terrain and design classification.

The basic shoulder widths shown in Table 2.2 are increased on some of the lower road standards if the number of Large Heavy Vehicles in the traffic stream is high (Table 2.3) and if the number of motorcycles, non-motorised vehicles and pedestrians (measured in terms of Passenger Car Units) exceeds 300 (Table 2.5). At the present time this limit is based on average daily conditions. On market days the road within several kilometres of an urban centre can contain a very high volume of non-motorised traffic and this can be sufficient for the average over the week to exceed 300 and justify wider shoulders. However many markets, although very busy, may not be quite large enough to exceed this average. The design engineer should be aware of the local situation and, if he/she feels that it is necessary (e.g. on the basis of safety and/or congestion) to widen the shoulders, he/she should be encouraged to request a modification to the standards.

Where the carriageway is paved, the shoulder should also be sealed with a bituminous surface treatment. This has several advantages;

i) prevents edge ravelling and the maintenance problems associated with parking on an unpaved shoulder;
ii) controls ingress of moisture into the upper pavement layers;
iii) provides paved space for vehicular parking outside of the traffic flow;

iv) provides a better surface for vehicles experiencing emergency repairs;
v) caters for the very heavy pedestrian traffic observed in the villages, traffic that would otherwise use the roadway.

The road layout in terms of widths in urban and peri-urban areas is shown in Table 2.2. The sealed shoulder width may increase to 3.5 metres where provision for a parking lane is required and the shoulders should be paved rather than sealed. In fact, it is not strictly a shoulder but an additional lane.

In cases where terrain is severe, the existing roadway width is narrow, and where the shoulder width can only be maintained through an excessive volume of earthwork e.g. at escarpment conditions, standards can be reduced through the Departure from Standard process presented in Chapter 2.

6.4 Normal Crossfall

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

The normal crossfall should be 3.0 percent on paved roads. Shoulders having the same surface as the roadway should have the same normal crossfall. Unpaved shoulders on a paved road should be 1.5 percent steeper than the crossfall of the roadway.

The crossfall or camber on unpaved roads does not remain the same for very long because of movement of the gravel and its loss. Research has shown that the rate of deterioration of a gravel road is highly dependent on the camber and the ability of the surface to shed water effectively. The higher the initial camber, the longer it takes for the surface to deteriorate to a shape that fails to shed water adequately, thereby causing accelerated deterioration. In most circumstances, crossfalls/cambers as high as 6.0 percent should be used, although this value may need to be modified for some types of gravel. .

For shoulder crossfall in super-elevated conditions, refer to Chapter 8.

6.5 Side Slopes and Back Slopes

Side slopes should be designed to ensure the stability of the roadway and to provide a reasonable opportunity for recovery of an out-of-control vehicle.

Three regions of the roadside are important when evaluating the safety aspects:
i) top of the slope (hinge point),
ii) side slope, and
iii) toe of the slope (intersection of the fore slope with level ground or with a back slope, forming a ditch).

Figure 6.1 illustrates these three regions.

Research has found that rounding at the hinge point can significantly reduce the hazard potential. Similarly, rounding at the toe of the slope is also beneficial.

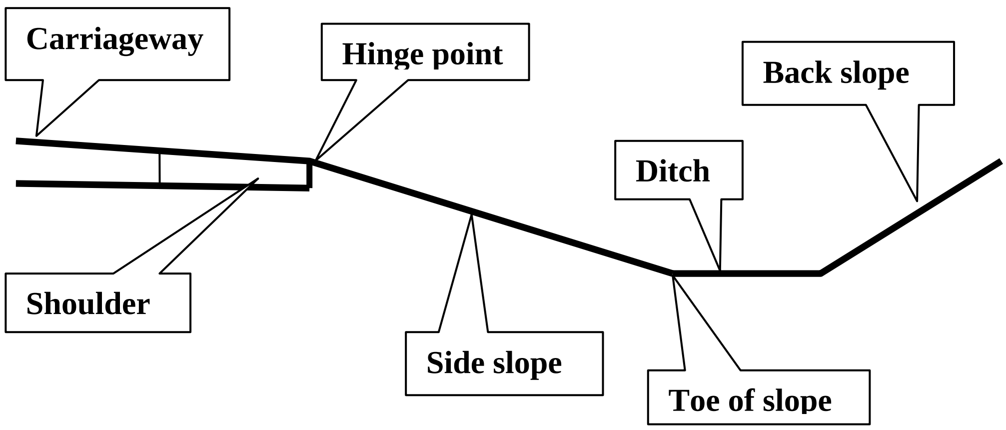


Figure 6-1: Designation of Roadside Regions

Refer to Table 6.1 for details of side slopes and back slopes.

Embankment or fill slopes parallel to the flow of traffic may be defined as recoverable, non-recoverable, or critical. Recoverable slopes include all embankment slopes 1:4 or flatter. Motorists who encroach on recoverable slopes can generally stop their vehicles or slow them enough to return to the roadway safely. Fixed obstacles such as culvert head walls should not extend above the embankment within the clear zone distance.

A non-recoverable slope is defined as one which is traversable, but from which most motorists will be unable to stop or to return to the roadway easily. Typically, vehicles on such slopes can be expected to reach the bottom. Embankments between 1:3 and 1:4 generally fall into this category. Since a high percentage of encroaching vehicles will reach the toe of these slopes, the clear zone distance extends beyond the slope, and a clear run-out area at the base is desirable.

A critical slope is one on which a vehicle is likely to overturn. Slopes steeper than 1:3 generally fall into this category.

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

Table 6.1 indicates the side slope ratios recommended for use in the design according to the height of fill and cut, the material and practical experience in Ethiopia 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 too steep to meet the recommendations for adequate safety. Achieving a good safety design is clearly a function of overall cost and at the present time is only likely to be viable for the highest classes of road.

Table 6-1: Slope Ratio Table – Vertical to Horizontal

Material	**Height of Slope (m)**	**Side Slope (V:H)**		**Back Slope**
		Fill	Cut	
Earth Soil	0.0 – 1.0	1:3	1:2	1:3
	1.0 – 2.0	1:2		1:2
	>2.0	2:3		2:3
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 – 2.0	1:2		1:2
	>2.0	2:3		2:3
Black Cotton Soil (expansive clays)(1)	0.0 – 2.0	1:6	-	-
	>2.0	1:4		

Note 1 Move ditch away from fill as shown in Figure 6.2

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. Slope configuration and treatments in areas with identified slope stability problems should be addressed as a final design issue.

6.6 Roadside Ditches

For the detailed design of roadside ditches, the ERA *Drainage Design Manual* should be used.

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

Under normal circumstances the adoption of a trapezoidal cross-section will facilitate maintenance and will be acceptable from the point of view of traffic safety. It is much easier and appropriate to dig and clean a trapezoidal drain with hand tools and the risk of erosion is lower. The minimum recommended width of the side drain is 500mm. This shape has high flow capacity and, by carefully selecting the gradients of its side slopes, it will resist erosion.

The V-shape is the standard shape for a drainage ditch constructed by a motor-grader. It can be easily maintained by heavy equipment but it has relatively low capacity necessitating more frequent structures for emptying it. Furthermore the shape concentrates flow at the invert and encourages erosion. Using a V-shaped ditch the minimum depth should be 0.6m in mountainous and escarpment terrain, and 1.0m elsewhere.

The side slope and back slope of ditches should generally be no less than 1:2; however, these slopes should conform to the slopes given in Table 6.1.

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 (6m for trunk roads), as shown in Figure 6.2. The ditch in this instance should have a trapezoidal, flat-bottom configuration.

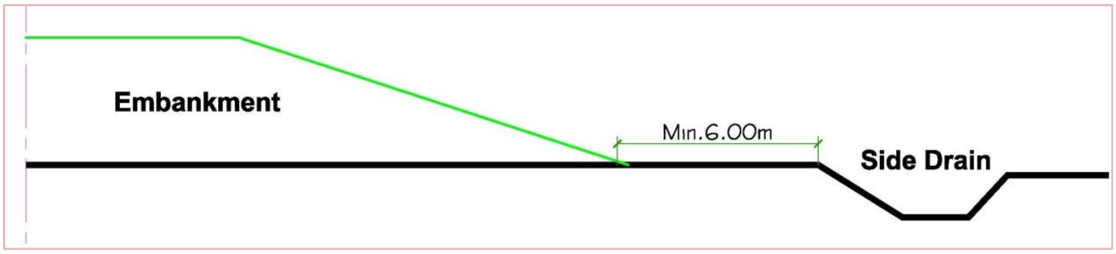


Figure 6-2: Side Drain Ditch Location in Expansive Soils

6.7 Clear Zone

Once a vehicle has left the roadway, an accident may occur. The end result of an encroachment depends upon the physical characteristics of the roadside environment. Flat, traversable, stable slopes minimize overturning accidents, which are usually severe. Elimination of roadside furniture or its relocation to less vulnerable areas is an option in the development of safer roadsides. If a fixed object or other roadside hazard cannot be eliminated, relocated, modified, or shielded, for whatever reason, consideration should be given to delineating the feature so it is readily visible to a motorist.

For adequate safety, it is desirable to provide an unencumbered roadside recovery area that is as wide as practical on a specific highway section. The cleared width should be a minimum of 15 metres each side from the edge of the roadway for the higher road standards.

For lower standard roads, the clear zone can be reduced. It 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 metres. 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-metre clearance from the edge of the shoulder.

Horizontal clearance to road signs, marker posts, etc. must be a minimum of 1.0m from the edge of the carriageway.

6.8 Right-of-Way

Right-of-ways, or road reserves, are provided in order to accommodate road width and to enhance the safety, operation and appearance of the roads. The width of the right-of-way depends on the cross section elements of the highway, topography and other physical controls together with economic considerations. Although it is desirable to acquire sufficient right-of-way to accommodate all elements of the cross section and appropriate border areas, right-of-way widths should be limited to a practical minimum in both rural and developed areas affecting the economy of the inhabitants.

Right of ways will be equidistant from the centreline of the road to the left and to the right of the carriageway. They should always be determined and shown on the final design plans of road projects.

Road reserve widths applicable for the different road classes are shown in Tables 2-6 through to 2.16. In mountainous or escarpment terrain, a cut section may be of such depth that the right-of-way width is exceeded from the top of cut on one side to the other top of cut.

Additional areas required for outlets etc., should be provided in a manner that will not endanger the future integrity of the drainage facility and will provide adjoining land owners restricted use of this land after completion of the road.

Reduced widths should be adopted only when these are found necessary for economic, financial or environmental reasons in order to preserve valuable land, resources or existing development or when provision of the desirable width would incur unreasonably high costs because of physical constraints. In such cases, it is recommended that the right-of-way should extend a minimum of a nominal 3 metres from the edge of the road works. However, where this occurs, it is advisable to restrict building activity along the road to prevent overcrowding, to preserve space for future improvements, and to provide for sight distances at curves. The distance across the carriageway from building line to building line should be a minimum of 15m.

For dual carriageway roads it may be necessary to increase the road reserve width above the given values.

6.9 Four-Lane and Divided Roads

For such roads a planning horizon exceeding the 15 or 20 years used for rural roads is required. Four-lane and divided roads are necessary when the traffic volume is sufficient to justify their use and, in urban/peri-urban areas, this may be anticipated in the foreseeable future. Indeed, some cities and towns have assumed that they will eventually be inevitable and have included four-lane roadways in their master plans.

A minimum median width of 5.0 metres is required to allow the provision of left-turning lanes outside of the adjacent carriageway, and to avoid having a turning passenger vehicle from the minor road protrude into the through lanes. Geometric standards for four lane roads are given in Tables 2.1, 2.2, and 2.3.

6.10 Medians

The median is the total area between the inner edges of the inside traffic lanes of a divided road, and includes the inner shoulders and central islands. The purpose of the median is to separate opposing streams of traffic hence reducing the possibility of vehicles crossing into the path of opposing traffic. This is accomplished by the selection of the width of the median or by a physical barrier such as a guardrail.

Medians are also used to reduce the nuisance of headlight glare by the planting of shrubs on the central island. The shrubs should not grow so tall that sunlight could fall into the driver's eyes in bands - the stroboscopic effect encountered in avenues of trees in the early morning or late afternoon. In addition, the stems of the shrubs should not grow so thick as to become a further possible hazard to the motorist; a maximum stem thickness of 100 mm is recommended. Medians should not, as far as possible, be obstructed by street furniture.

Median width depends not only on traffic volume but also on the function of the road and on traffic composition. For example, a median functioning purely as a pedestrian refuge could be much narrower than one protecting a turning vehicle (which could be semi-trailer plus trailer).

A median width of 9.2 m eliminates most cross-median accidents, and this width is recommended where no barriers are provided between opposing traffic flows. Where a road is to be constructed in stages, the median should be wide enough to accommodate future lanes, without falling below the recommended width in the final stage. Operational difficulties may arise at intersections with very wide medians because of the duration of turning movements. The median should, however, be wide enough to provide refuge to the design vehicle.

With severe space limitations, it is possible to use medians that are as little as 1.5 m wide. These would, however, serve only to accommodate back-to-back guardrails to ensure vehicular separation. A median that is 5.0 m wide is able to accommodate a right turn lane with provision for a pedestrian refuge but would also require guardrail protection to separate the opposing flows of traffic.

It is suggested that the median island should be depressed rather than raised, because a raised or kerbed median island, will automatically require the inner shoulder to be 3.0 m wide to allow sufficient space for emergency manoeuvres, including stopping. A depressed median also facilitates drainage.

The purpose of an outer separator is most frequently to separate streams of traffic flowing in the same direction but at different speeds and also to modify weaving manoeuvres. In general, the standards applied to medians are equally appropriate to outer separators.

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 is 10 per cent. Slopes flatter than this may lead to ponding and to water flowing from the median to the carriageway. Slopes steeper than 1:4 make control of an errant vehicle difficult, leading to a greater possibility of cross-median accidents. If surface drainage requires a median slope steeper than 1:4, this aspect of road safety might justify replacing surface drainage by an underground drainage system.

Differential, or split, grading requires the median to be sloped to absorb the height difference between the carriageways. This is achieved, in the case of small height differences, by locating the low point of the median eccentrically, retaining the maximum permissible slope. The limit is reached when the low point is adjacent to the lower carriageway and functions as a side drain. If a steeper slope is required, the carriageways have to be designed as completely independent roadways, with full-width shoulders, guardrails if necessary, and sufficient distance between shoulder breakpoints, with the side slope appropriate to the in-situ material, to accommodate the height difference between carriageways.

The design of the ends of medians is described in Section 11.9.2.

6.11 Single Lane Roads

For low traffic volume roads (<75ADT) single lane operation is adequate because the probability of vehicles meeting each other from opposite directions is small and the few passing manoeuvres can be undertaken at very reduced speeds using either the shoulder or passing bays. Provided sight distances are adequate for safe stopping, these manoeuvres can be performed without hazard, and the overall loss in efficiency brought about by the reduced speeds is small.

The lowest design standards (DC1 and DC2) are not sufficiently wide for passing and overtaking to occur on the carriageway and passing bays must be provided. The increased width at passing bays should be such as to allow two design vehicles to pass, i.e. a minimum of 5.0 m width, and vehicles are expected to stop or slow to a very low speed.

Normally, passing bays should be located every 300 to 500 metres depending on the terrain and geometric conditions. However, adjacent passing bays must be visible from each other. Account should be taken of sight distances, the likelihood of vehicles meeting between passing bays and the potential difficulty of reversing. In general, passing bays should be constructed at the most economic locations as determined by terrain and ground conditions such as transitions from cuttings to embankment, rather than at precise intervals.

The length of individual passing bays varies with local conditions and the type of design vehicle but, generally, a length of 20 metres including tapers caters for most commercial vehicles.

Significant cost savings may be realized in mountainous and escarpment terrain by incorporating short lengths of DC1 standard within a DC2 road. This Departure from Standard may be economically justified, especially in escarpment terrain, for design traffic flows of less than 75 vehicles per day. However, appropriately placed, inter-visible passing bays are essential to ensure the free flow of traffic.

6.12 Typical Cross Sections and Standard Cross Sections

Typical cross sections are illustrated in *ERA Standard Detail Drawings,* and cross sections for the standard classes of roads are illustrated in Appendix E of this manual.

7 DESIGN SPEED AND SIGHT DISTANCES

7.1 Design Speed

Design speed is defined as the maximum (actually the 85th percentile) safe speed that can be maintained over a specified section of road when conditions are so favourable that the design features of the road govern the speed. To ensure that a driver is presented with a consistent speed environment, design speed is used as an index that essentially defines the geometric standard of a road, linking many of the factors that determine the road's service level, namely traffic volume; terrain; pavement type; safety/population density; and road function.

The concept of design speed is very useful because it allows the key elements of geometric design to be selected for each standard of road in a consistent and logical way. For example, design speed is relatively low in mountainous terrain to reflect the necessary reductions in standards required to keep road costs to manageable proportions. The speed is higher in rolling terrain and highest of all in flat terrain.

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.

In view of the mixed traffic that occupies the rural roads of Ethiopia and the cost benefit of selecting lower design speeds, it is prudent to select values of design speed towards the lower end of the internationally acceptable ranges as shown in Table 7.1.

Table 7-1: Design Speeds

Design standard	Design speed (km/h)				
	Flat	Rolling	Mountain	Escarpment	Urban
DC 8	120	100	85	70	50
DC 7	120	100	85	70	50
DC 6	100	85	70	60	50
DC 5	85	70	60	50	50
DC 4	70	60	50 (46) (1)	25	50
DC 3	70	60	50 (46) (1)	25	50
DC 2	60	50	40(37) (1)	20	50
DC 1	50	40	30	20	40

Note 1 The design speeds in mountainous terrain for unpaved roads has been adjusted slightly so that the minimum radii of curvature are the same for both the paved and unpaved option. This ensures that when a road is upgraded to paved standard, the existing curves are not already 'over' designed.

Changes in design speed, if required because of a change in terrain, should be made over distances that enable drivers to change speed gradually. Thus changes should never be more than one design step at a time and the length of the sections with intermediate standards (if there is more than one change) should be long enough for drivers to realise there has been a change before another change in the same direction is encountered. In general, a particular design speed should be used for a minimum distance of five kilometres (i.e. considerably more than one single bend). Where this is not possible, warning signs should be provided to alert drivers to the changes.

7.2 Stopping Sight Distance

In order to ensure that the design speed is safe, the geometric properties of the road must meet certain minimum or maximum values to ensure that drivers can see far enough ahead to carry out normal manoeuvres such as overtaking another vehicle or stopping if there is an object in the road.

The distance a vehicle requires to stop safely is called the stopping sight distance. It mainly affects the shape of the road on the crest of a hill (vertical alignment) but if there are objects near the edge of the road that restrict a driver's vision on approaching a bend, then it also affects the horizontal curvature.

The driver must be able to see any obstacle in the road, hence on a crest curve, the stopping sight distance depends on the size of the object and the height of the driver's eye above the road surface. The driver needs time to react and then the brakes of the vehicle need time to slow the vehicle down. Hence stopping sight distance is dependent on the speed of the vehicle and the efficiency of its brakes. The surface characteristics of the road also affect the braking time so the values for unpaved roads differ from those of paved roads, although the differences are small for design speeds below 60km/h.

The stopping distance also depends on the gradient of the road; it is harder to stop on a downhill gradient than on a flat road because a component of the weight of the vehicle acts down the gradient in the opposite direction to the frictional forces that are attempting to stop the vehicle.

The stopping sight distance is given by the following formula;

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

where,

d = distance (metres)
t = driver reaction time, generally taken to be 2.5 seconds
V = initial speed (km/h)
f = coefficient of friction between tyres and roadway (see Table 7.2)
g = gradient of road as a percentage (downhill is negative)

On a flat road the value of g is zero. On a 5 percent downhill gradient the stopping distance at 120 km/hr is typically 16 percent longer. At a 10 percent gradient it is nearly 40 percent longer, as shown in Table 7.2. The Table also shows that for speeds above 50 km/hr, the gradient of the road makes a significant difference and must be taken into account in establishing safe sight distances.

Table 7-2: Stopping and Passing Sight Distances for Paved Roads

Design Speed (km/h)	Coefficient of Friction (f)	Stopping Sight Distance (m)			Minimum Passing Sight Distance (m) (from formulae)	Passing Sight Distance to allow manoeuvre to be aborted (m)
		g = 0	g = 5%	g = 10%		
20	.42	18	18	19	160	-
25	.41	23	24	25	190	50
30	.40	30	32	33	220	80
40	.37	45	47	50	285	135
50	.35	65	70	75	350	180
60	.33	85	90	105	415	230
70	.315	110	120	140	480	270
80	.305	140	155	180	545	310
85	.295	155	175	205	575	330
90	.29	170	195	230	610	345
100	.285	210	240	285	675	375
110	.28	245	285	340	740	405
120	.28	285	330	400	805	425

The coefficient of friction values shown in Table 7.2 have been determined from test results such as those shown in Figure B.1 of Appendix B, using the lowest results of the friction tests. The values shown in the main third column of Table 7.2 for minimum stopping sight distance are calculated from the above formula.

Table 7.3 is similar to Table 7.2 but is for unpaved roads where the coefficients of friction are lower and much more variable, depending on the properties of the gravel or soil.

It is important to note that the values in the Tables are for dry weather conditions. Stopping sight distances are much longer in unfavourable wet conditions.

Full adherence to the required sight distances is essential for safety reasons. On the inside of horizontal curves it may be necessary to remove trees, buildings or other obstacles to obtain the necessary sight distances. If this cannot be done, the alignment must be changed. In rare cases where it is not possible and a change in design speed is necessary, adequate and permanent signage must be provided.

7.2.1 Stopping Sight Distances for Trucks

Trucks with conventional braking systems require longer stopping distances from a given speed than do passenger cars. However, a truck driver is able to see the vertical features of the obstruction from substantially further because of the higher driver eye height. This is particularly important on crest curves (Chapter 9) where object height and the driver's eye height dictate the sight distance available. In addition, posted speed limits for trucks are often considerably lower than for passenger vehicles.

Separate stopping sight distances for trucks and passenger cars are, therefore, not generally used in highway design. However, there is evidence that the sight distance advantage provided by the higher driver eye level in trucks does not always compensate for their inferior braking. Some reasons for the longer truck braking distances include:

- Poor braking characteristics of empty trucks. The problem relates to the suspension and tyres that are designed for maximum efficiency under load;
- Uneven load between axles;
- Propensity of truck drivers not to obey posted speed limits;
- Inefficient brakes of articulated trucks, and
- Effect of curvature where some of the friction available at the road/tyre interface is used to hold the vehicle in a circular path.

To balance between the costs and benefits in designing for trucks, truck stopping sight distances should be checked at potentially hazardous locations. In general, the deceleration rate for trucks is 1.5 m/s^2 which is about half that of cars and is equivalent to a coefficient of friction of half that shown in Table 7.2 and 7.3. This increases the stopping distance by 40% at lower speeds, increasing to 70% at 120 km/h. Where required (e.g. for vertical alignment – see Chapter 9) the driver's eye height is taken as being at 1.8 m and the object height is as defined in Tables 9.1 and 9.2.

Table 7-3: Stopping Sight Distances for Unpaved Roads

Design Speed (km/h)	Coefficient of Friction (f)	Stopping Sight Distance (m)		
		g = 0	g = 5%	g = 10%
20	.34	19	19	21
25	.33	25	25	30
30	.32	30	35	37
40	.30	50	55	60
50	.28	70	80	90
60	.26	95	110	130
70	.25	125	145	175
80	.24	160	185	235
85	.24	180	210	270
90	.23	200	240	305
100	.23	240	290	375

7.3 Stopping Sight Distance for Single Lane Roads (Meeting Sight Distance)

For single lane roads, adequate sight distances must be provided to allow vehicles travelling in the opposite direction to see each other and to stop safely if necessary. 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 metres is also sometimes added. 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. However, single lane roads have a relatively low design speed, hence meeting sight distances should not be too difficult to achieve.

7.4 Intersection Sight Distance

Intersection sight distance is similar to stopping sight distance, Table 7.2, 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. This topic is dealt with in Section 11.3 and summarised in Tables 11.1 and 11.2. Further details are also provided in Appendix C.

7.5 Decision Sight Distance

Stopping sight distances are usually sufficient to allow reasonably competent and alert drivers to stop under ordinary circumstances. However, these distances are often inadequate when:

- A situation arises that does not require an emergency stop but drivers need to make complex decisions;
- Information is difficult to perceive; or
- Unexpected or unusual manoeuvres are required.

Limiting sight distances to those provided for stopping may also preclude drivers from performing evasive manoeuvres, which are often less hazardous and otherwise preferable to an emergency stop. Even with an appropriate complement of standard traffic control devices, stopping sight distances may not provide sufficient visibility for drivers to perceive and understand complex situations and to perform the necessary manoeuvres.

It is evident that there are many locations such as exits from freeways, or where lane shifts or weaving manoeuvres are performed, where it would be prudent to provide longer sight distances. In these circumstances, decision sight distance provides the greater length that drivers need. If the driver can see what is unfolding far enough ahead, he or she should be able to handle almost any situation. Decision sight distance, sometimes termed 'anticipatory sight distance', is the distance required for a driver to:

- detect an unexpected or otherwise 'difficult-to-perceive' information source or hazard in a roadway environment that may be visually cluttered;
- recognize the hazard or its potential threat;
- select an appropriate speed and path; and
- initiate and complete the required safety manoeuvre safely and efficiently.

Because decision sight distance gives drivers additional margin for error and affords them sufficient length to manoeuvre their vehicles at the same or reduced speed rather than to just stop, it is substantially longer than stopping sight distance. Drivers need decision sight distances whenever there is likelihood for error in information reception, decision-making, or control actions. Critical locations where these kinds of errors are likely to occur, and where it is desirable to provide decision sight distance include:

- Approaches to interchanges and intersections;
- Changes in cross-section such as at toll plazas and lane drops;
- Design speed reductions; and
- Areas of concentrated demand where there is likely to be ‘visual noise’, e.g. where sources of information, such as roadway elements, opposing traffic, traffic control devices, advertising signs and construction zones, compete for attention.

The minimum decision sight distances that should be provided for specific situations are shown in Table 7.4. If it is not feasible to provide these distances because of horizontal or vertical curvature or if relocation is not possible, special attention should be given to the use of suitable traffic control devices for advance warning.

Although a sight distance is suggested for the left side exit, the designer should bear in mind that exiting to the left on a main road is in conflict with driver expectancy and is highly undesirable. The only reason for providing this value is to allow for the possibility that a left side exit has to be employed.

Table 7-4: Decision Sight Distances (metres)

Design Speed km/h	**Situations**				
	Interchanges. Sight distance to nose		**Lane drop, merge. Sight distance to taper area**	**Lane shift. Sight distance to beginning of shift**	**Intersections. Sight distance to turn lane**
	Right exit	**Left 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

7.6 Control of Sight Distance

Sight distances should be checked during design and adjustments made to meet the minimum requirements. The following values should be used for the determination of sight lines. Details of crest and sag curve design are to be found in Chapter 9.

a) Driver's eye height: 1.05 metres
b) Object height for stopping sight distance: 0.2 metres
c) Object height for passing sight distance: 1.30 metres
d) Object height for decision sight distance 0.00 metres

On the inside of horizontal curves, it may be necessary to remove buildings, trees or other sight obstructions or widen cuts on the insides of curves to obtain the required sight distance (see Figure 7.1).

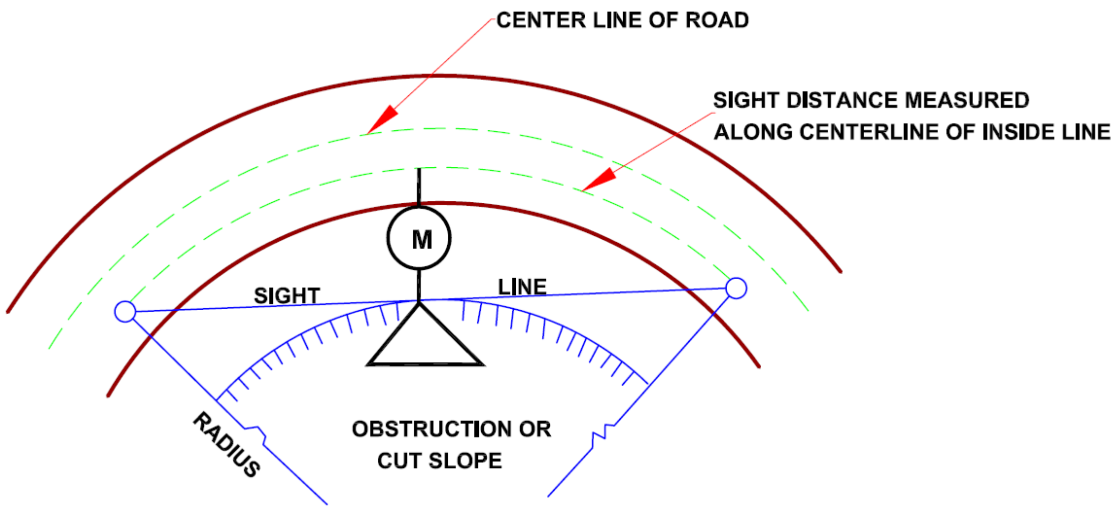


Figure 7-1: Sight Distance for Horizontal Curves

Relevant formulae are as follows:

Length of Sight Line (S) = 2R sin(Δ/2) where Δ = Deflection angle (^{o})

Length of Middle Ordinate (M) = R(1-cos(Δ/2)

Example:

Radius = 1000 metres, Δ = 20^o;

S = 2R sin(Δ/2)   M = R (1 – cos(Δ/2)

= 2(1000)(sin(10^o)   = 1000(1- cos(10^o)

= 347 metres   = 15.2 metre

The available sight distance needs to be checked separately for both stopping and passing sight distance, for each direction of travel.

7.7 Passing Sight Distance

The Passing Sight Distance is the minimum sight distance on a two-way road that must be available to enable the driver of one vehicle to pass another vehicle safely without interfering with the speed of an oncoming vehicle travelling at the design speed. Hence factors affecting the safe sight distances required for overtaking are complicated because they involve the capability of a vehicle to accelerate and the length and speed of the vehicle being overtaken. Assumptions also need to be made about the speed differential between the vehicle being overtaken and the overtaking vehicle. In view of all these assumptions many road authorities have simply based their standards on empirical evidence.

Within the sight area, the terrain should be at the same level or a level lower than the roadway, otherwise, for horizontal curves, it may be necessary to remove obstructions and widen cuttings on the insides of curves to obtain the required sight distance. Care must be exercised in specifying passing/no-passing zones in areas where the sight distance may be obscured in the future due to vegetative growth.

The passing sight distance is generally determined by a formula with four components, as follows:

d_1 = initial manoeuvre distance, including a time for perception and reaction

d_2 = distance during which passing vehicle is in the opposing lane

d_3 = clearance distance between vehicles at the end of the manoeuvre

d_4 = distance traversed by the opposing vehicle

The formulae for these components are as indicated below:

$$d_1 = 0.278\, t_1\, (v - m + a.t_1/2)$$

Where

t_1 = time of initial manoeuvre, s

a = average acceleration, km/h/s

v = average speed of passing vehicle, km/h

m = difference in speed of passed vehicle and passing vehicle, km/h

$$d_2 = 0.278\, v.t_2.$$

Where

t_2 = time passing vehicle occupies left lane, s

v = average speed of passing vehicle, km/h

d_3 = safe clearance distance between vehicles at the end of the manoeuvre, and is dependent on ambient speeds as per Table 7.5:

d_4 = distance traversed by the opposing vehicle, which is approximately equal to d_2 minus the portion of d_2 whereby the passing vehicle is entering the left lane, estimated as:

d_4 = $2d_2/3$

Table 7-5: Clearance Distance (d_3) for Different Ambient Speeds

Speed Group (km/h)	**50-65**	**66-80**	**81-100**	**101-120**
d_3 (m)	30	55	80	100

The time t_1 for the initial perception and manoeuvre is particularly variable and also depends on the vehicle speed – more time is required for assessment when the overtaking manoeuvre is considered to be dangerous.

The *minimum* Passing Sight Distance (PSD) for design is

$$PSD = d_1 + d_2 + d_3 + d_4$$

The resulting *minimum* sight distances for passing are as indicated in the sixth main column of Table 7.2. For the reasons outlined herein, however, the preferable (or desirable) PSD at 40km/h is 15% greater than the values quoted in the Table, rising to 40% greater at 120km/h.

A method of measuring and recording sight distances on plans is given in Appendix C.

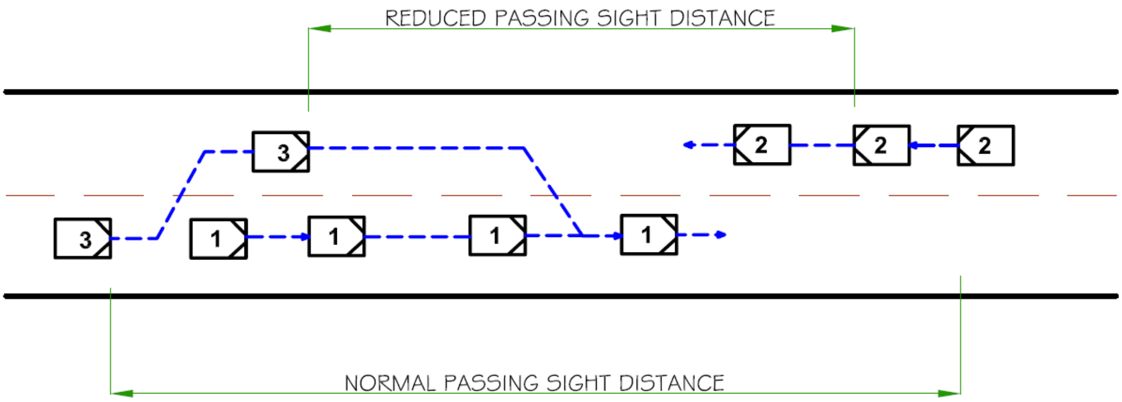


Figure 7-2: Passing Sight Distance

7.8 Minimum Provision of Passing Sight Distance

An alternative design strategy is to base the passing sight distances on providing enough sight distance for a vehicle to safely abort a passing manoeuvre if another vehicle is approaching. The recommended values are shown in the seventh column of Table 7.2.

Sight distance records are also useful on two-lane highways for determining the percentage of length of highway on which sight distance is restricted to less than the minimum needed for passing. This is important in evaluating capacity. With recorded sight distance, as in the lower part of Figure C-1 of Appendix C, it is a simple process to determine the percentage of length of highway with a given sight distance or greater.

Passing Sight Distance is a desirable requirement for two-way single roadway roads. Sufficient visibility for passing increases the capacity and efficiency of a road and should be provided for as much of the road length as possible within financial limitations.

Table 7.6 gives guide values for the extent to which passing sight distance should be provided.

Table 7-6: Guide Values for the Minimum Provision of Passing Sight Distance

<table>
<tr><th rowspan="2">Design Standard</th><th colspan="5">Percent Passing Opportunity and Terrain</th></tr>
<tr><th>Flat</th><th>Rolling</th><th>Mountainous</th><th>Escarpment</th><th>Urban/Peri-Urban</th></tr>
<tr><td>DC8</td><td rowspan="2">50</td><td>50</td><td rowspan="2">25</td><td rowspan="6">0</td><td rowspan="6">20</td></tr>
<tr><td>DC7</td><td>33</td></tr>
<tr><td>DC6</td><td rowspan="2">25</td><td rowspan="2">25</td><td rowspan="4">15</td></tr>
<tr><td>DC5</td></tr>
<tr><td>DC4</td><td rowspan="2">20</td><td rowspan="2">20</td></tr>
<tr><td>DC3</td></tr>
</table>

8 HORIZONTAL ALIGNMENT

8.1 General

The horizontal alignment consists of a series of straight sections (tangents), circular curves, transition curves (spirals) and super-elevation. The horizontal curves are designed to ensure that vehicles can negotiate them safely. The alignment design should be aimed at avoiding sharp changes in curvature, thereby achieving a safe uniform driving speed. Transition curves between straight sections of road and circular curves whose radius changes continuously from infinity (tangent) to the radius of the circular curve (R) are used to reduce the abrupt introduction of centripetal acceleration that occurs on entering the circular curve. They are not required when the radius of the horizontal curve is large and are normally not used on the lower classes of road. In Ethiopia their use is confined to roads where the design speed is 80km/hr or greater.

In order for a vehicle to move in a circular path, an inward radial force is required to provide the necessary centripetal acceleration or, in other words, to counteract the centrifugal force. This radial force is provided by the sideways friction between the tyres and the road surface assisted by the cross-fall or super-elevation.

In order to calculate the minimum horizontal radius of curvature, R_{min}, for a particular design speed, the equation is:

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

Where

V_D = design speed (km/h)
e = maximum super-elevation (%/100)
f = side friction coefficient

The design speed is thus one of the main design parameters. Values for each class of road under each of its operating conditions are shown in Table 2.1.

Values of the minimum radii of curvature for different design speeds and super-elevations based on this formula are shown in Tables 8.1 for paved roads and Table 8.2 for unpaved roads. For unpaved roads the super-elevation will not be constant but will vary as the gravel is worn away. A value of 4 percent has been used in the calculations as a reasonable compromise during the life of the gravel surface, assuming an initial maximum value of 6 percent.

Side friction coefficients are dependent on

i) vehicle speed;
ii) type, condition and texture of roadway surface;
iii) weather conditions; and
iv) type and condition of tyres.

The coefficient is considerably less than the longitudinal friction coefficient. Its value decreases as speed increases but there is considerable disagreement about representative values, especially at the lower speeds. Tables 8.1 and 8.2 were developed based on the results of several studies (Appendix B). For paved roads the coefficient ranges from between 0.18 and 0.25 at 20km/h down to between 0.09 and 0.16 at 120km/h.

Table 8-1: Minimum Radii for Horizontal Curves for Paved Roads

Design speed (km/h)	20	25	30	40	50	60	70	80	85	100	120
Side Friction Factor (f)	0.23	0.22	0.21	0.19	0.17	0.16	0.14	0.13	0.12	0.11	0.10
Super-elevation = 4%	15	19	30	55	95	145	215	300	350	515	780
Super-elevation = 6%	15	18	27	50	85	135	195	270	310	455	685
Super-elevation = 8%	15	17	25	50	80	120	175	240	280	410	610
Super-elevation = 10%	15	16	25	45	75	110	160	220	255	375	555

Table 8-2: Minimum Radii for Horizontal Curves for Unpaved Roads

Design speed (km/h)	20	25	30	40	50	60	70	80	85	90	100
Side Friction Factor	0.19	0.17	0.165	0.15	0.14	0.12	0.11	0.10	0.10	0.10	0.09
Super-elevation = 4%	15	25	35	65	115	175	255	355	410	475	610

For unpaved roads the friction is usually considerably less. 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 design speed could be maintained. A poorly bound surface with many loose particles has a very low value of friction and it has to be assumed that vehicles will be driven on such a surface at a speed that is much lower than the nominal design speed dictated by the sight distances and radii of curvature.

The Tables above indicate the minimum radii of curvature for different design speeds and road surfaces. In general, these radii should be used only under the most critical conditions. The deviation angle of each curve should be as small as the physical conditions permit. The deviation should be absorbed in the flattest possible curve so that passing opportunities are not unduly restricted.

Changes in design speed (see Section 5.4), if required due to changes in terrain class, should not be made abruptly but over sufficient distance to enable drivers to change speed gradually. The change in design speed should not be greater than one design speed step (usually 10 or 15 km/h) and the section with the lower geometric standards should be long enough to be clearly recognizable by drivers and not, for example, just a single curve.

The physical terrain sometimes changes by two terrain classes, typically from mountainous to flat. Where possible a transition section of road should be provided with limiting parameters equivalent to the intermediate terrain type namely rolling terrain. Where this is not possible, adequate warning signs must be provided to alert drivers to the changes in geometric standards.

Under normal circumstances sections of road will contain many curves that are larger than the minimum radii specified in the design standards. For reasons of safety and driver comfort it is inadvisable for consecutive curves to differ in radius by a large amount even though they are both greater than the minimum. Indeed, all the various design elements must be combined in a balanced way, avoiding the application of minimum values for one or a few elements at a particular location when other elements are considerably above the minimum requirements. Figure 8.1 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 into the 'useable' category some discomfort or inconvenience will be felt because of the increase in centripetal force when entering the tighter curve.

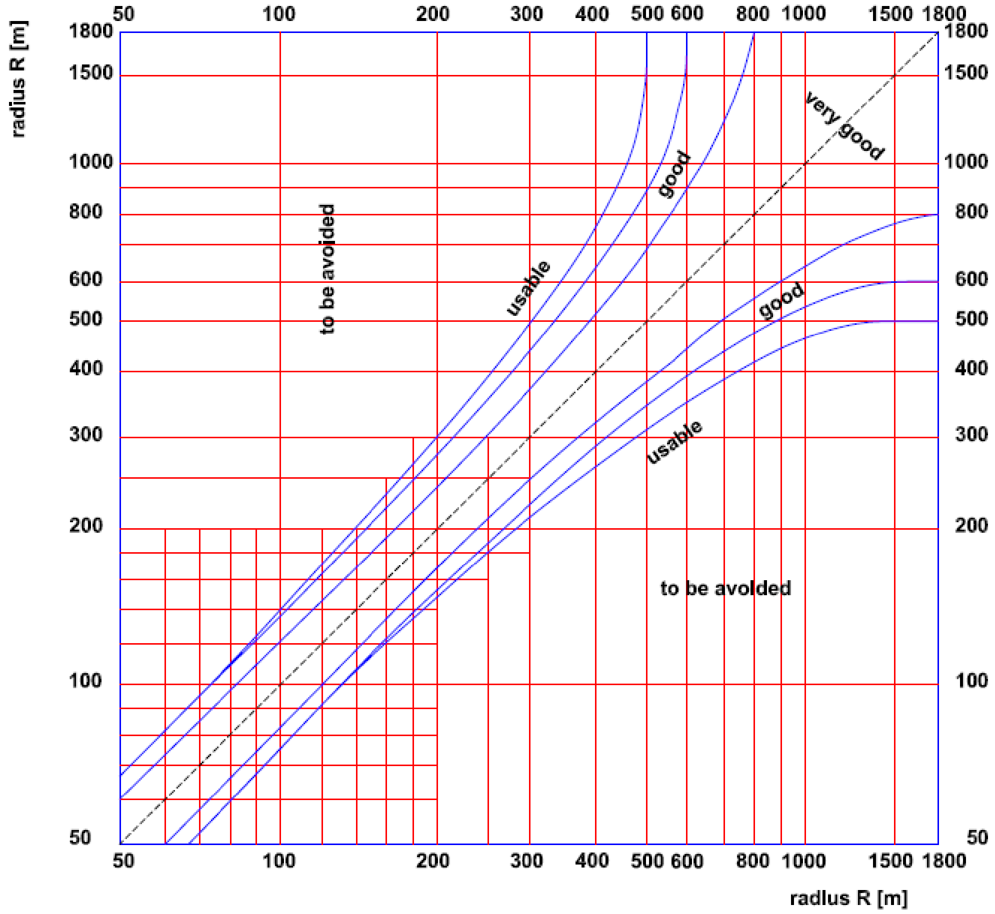


Figure 8-1: Ratio of radii of consecutive horizontal curves

8.2 Cross-fall

For both paved and unpaved roads there are constraints on the maximum cross-fall, as summarised in Tables 2.6 to 2.17. These constraints translate directly into minimum values of horizontal radii of curvature.

8.3 Elements of a Circular Curve

The elements of circular curves are shown in Figure 8.2

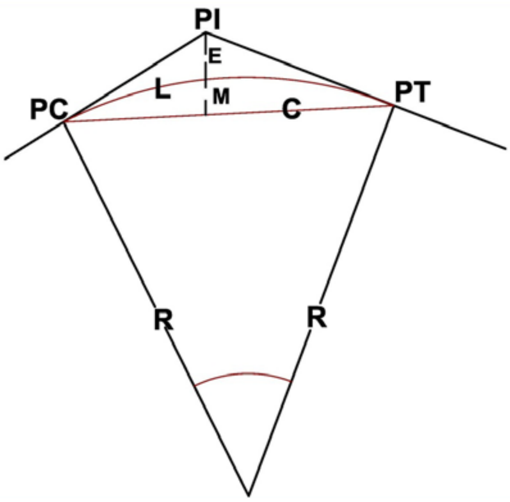


Figure 8-2: Curve Elements

1) Δ is the *Deflection Angle* (in degrees).

2) R is the *Radius* of the curve.

3) T is the *Tangent Distance* PC to PI; T = R.tan(Δ/2)

4) E is the *External Distance*: E = R.[sec(Δ/2) – 1]

5) L is the *Curve Length*. L = 2.π.R.Δ/360

6) M is the *Middle Ordinate* M = R.[1 – cos(Δ/2)]

7) C is the *Chord Length* from PC to PT C = 2.R.sin(Δ/2)

8) Point PC is the *Point-of-Curvature* Station PC = PI – T

9) PT is the *Point-of-Tangency* Station PT = PC+ L

Example:

A curve has a deflection angle of Δ = 23° 18’ 02”, and a radius of 1432.6m. The Point of Intersection (PI) is 5+053.87. Calculate the tangent distance (T), external distance (E), curve length (L), Point of Curvature (PC), and Point of Tangent (PT).

$$T = R\tan\frac{\Delta}{2} = (1432.6)\tan\frac{23°18'02'}{2} = 1432.4(.2026) = 295m$$

$$L = \Delta \times \frac{R \times 2\pi}{360} = 23.3\frac{1432.6 \times 2 \times 3.14}{360} = 582m$$

$$E = R\left|\sec\frac{\Delta}{2} - 1\right| = 1432.6\left|\left[\frac{1}{\cos\frac{\Delta}{2}}\right] - 1\right| = 1432.4(.02103) = 30m$$

PC = PI – T = 5+053.87 – 295.35 = 4 + 758

PT = PC + L = 4 + 758.49 + 582.51 = 5+341

8.4 Minimum Turning Radii

Buses, trucks, trucks with trailers and 4x4 utility vehicles require minimum design turning radii of 12.8m, 13.7m and 7.3m respectively (Table 5.1). 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 minimum horizontal curve radius of 15m is specified in Tables 8.1 and 8.2 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;

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

8.5 Isolated Curves

The horizontal curvature over a particular road section should be as consistent as possible. Long tangent roadway segments joined by an isolated curve designed at or near the minimum radius are unsafe. Long straight sections encourage drivers to drive at speeds in excess of the design 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 radius as shown in Tables 2.6 through 2.16 should be increased by 50 percent. This will usually result in the ability to negotiate the curve at a speed approximately 10 km/hr higher than the design speed.

8.6 Length of Tangent Sections and Curves

There are conflicting views about curve lengths. One school of thought maintains that the horizontal alignment should maximise the length of road where adequate sight distances are provided for safe overtaking. Overtaking is difficult on curves of any radius and hence the length of curved road should be minimised. This requires curve radii to be relatively close (but not too close) to the minimum for the design speed to maximise the length of straight sections.

The alternative view is that very long straight sections should be avoided. Long tangent sections increase the danger from headlight glare and usually lead to excessive speeding. For example, a long tangent causes speeds to creep up to about 120 km/h or even higher and the driver then has to reduce speed to negotiate the following curve, thereafter accelerating again. *Ideally, drivers should be encouraged to maintain a speed which is close to the design speed to reduce the possibility* of an error of judgment leading to an accident. It has been found that a maximum tangent length, measured in metres, of 20 times the design speed in km/h, achieves this effect. For example, a design speed of 100 km/h suggests that tangents should, ideally, not be longer than about 2.0 km. Thus a safer alternative is obtained by a winding alignment with tangents deflecting 5 to 10 degrees alternately from right to left. Straight sections should have lengths (in metres) less than 20 x design speed in km/h. Such ‘flowing’ curves restrict the view of drivers on the inside carriageway and reduce safe overtaking opportunities, *therefore such a winding alignment should only be adopted where the straight sections are very long*. In practice this only occurs in very flat terrain. The main aspect is to ensure that there are sufficient opportunities for safe overtaking and therefore, provided 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.

For small changes of direction it is often desirable to use a large radius of curvature. This improves the appearance 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. A widely adopted guideline is that, on minor roads, curves should have a minimum length of 150 metres for a deflection angle of 5^{o} and that this length should be increased by 30 metres for every 1^{o} decrease in deflection angle. On major roads and freeways, 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.

The minimum length of tangent must allow for the run-off of the super-elevation of the preceding curve followed by the development of that for the following curve. This distance should be calculated during detailed design but, as a rule of thumb, a tangent length of less than 200 m is likely to prove inadequate.

8.7 Reverse Curves, Broken-Back Curves, and Compound Curves

Curves are more frequent in rugged terrain. Tangent sections are shortened, and a stage may be reached where successive curves can no longer be dealt with in isolation. Three cases of successive curves are (see Figure 8.3):

i) Reverse Curve: a curve followed by another curve in the opposite direction.

ii) Broken-back Curve: a curve followed by another curve in the same direction but with only a short tangent in-between.

iii) Compound curve: curves in the same direction but of different radii, and without any intervening tangent section.

The occurrence of abrupt reverse curves (having a short tangent between two curves in opposite directions) should be avoided. Such geometrics make it difficult for the driver to remain within his lane. It is also difficult to super-elevate both curves adequately, and this may result in erratic operation.

The broken-back arrangement of curves (having a short tangent between two curves in the same direction) should also be avoided except where very unusual topographical or right-of way conditions dictate otherwise. Drivers do not generally anticipate successive curves in the same direction hence safety is compromised. Problems also arise associated with super-elevation and drainage.

The use of compound curves affords 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 he has entered a curve, hence safety is compromised. Their use should also be avoided where curves are sharp.

Compound curves with large differences in curvature introduce the same problems as are found at the transition from a tangent to a small-radius curve. Where the use of compound curves cannot be avoided, the radius of the flatter circular arc should not be more than 50 percent greater than the radius of the sharper arc; ie. R_1 should not exceed $1.5.R_2$. A compound arc on this basis is suitable as a form of transition from either a flat curve or a tangent to a sharper curve, although a spiral transition curve is preferred (see Section 8.10).

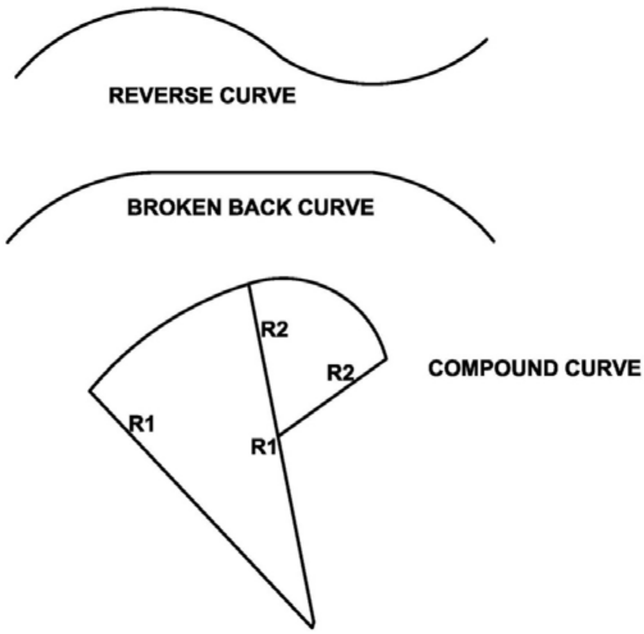


Figure 8-3: Reverse Curves, Broken-Back Curves, and Compound Curves

8.8 Widening on Curves and Embankments

The use of long curves of tight radii should be avoided where possible because drivers following the design speed will find it difficult to remain in the traffic lane. Widening of the carriageway where the horizontal curve is tight is usually necessary to ensure that the rear wheels of the largest vehicles remain on the road when negotiating the curve; and, on two lane roads, to ensure that the front overhang of the vehicle does not encroach on the opposite lane. Widening is therefore also important for safety reasons.

Vehicles need to remain centred in their lane to reduce the likelihood of colliding with an oncoming vehicle or driving on the shoulder. Sight distances should be maintained as discussed above.

Widening on high embankments is recommended for design classes DC8 through to DC4. 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 cases apply.

Curve widening is required on all standards of roads and should be sufficient to cater for the design vehicle. Table 8.3 gives the values to be adopted in the design.

Table 8-3: Widening on Curves and High Fills

Radius of Curve (m)	Curve Widening: Single Lane (m)	Curve Widening: Two Lanes (m)	Fill Widening	
			Height of fill (m)	Amount (m)
>250	0.0	0.0	0.0-3.0	0.0
120- 250	0.0	0.6	3.0- 6.0	0.3
60-120	0.0	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 8.10: Switchbacks			

Curve widening is generally not applied to curves with a radius greater than 250 metres regardless of the design speed or the lane width. Widening should transition gradually on the approaches to the curve so that the full additional width is available at the start of the curve. Although a long transition is desirable to ensure that the whole of the travelled way is fully usable, this results in narrow pavement slivers that are difficult, and correspondingly expensive, to construct. In practice, curve widening is thus applied over no more than the length of the super-elevation runoff preceding the curve. 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.

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

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, there was a considerable need for widening on curves. Because of the inconvenience attached to widening the surfacing of a lane, it follows that the required widening may not always have been provided. Where a 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.

8.9 Switchback Curves

Switchback or hairpin curves are used where necessary in traversing mountainous and escarpment terrain. Employing a radius of 20m or less, with a minimum of 10m, they are generally outside of the standards for all road designs and are specified using the guidelines listed in the Departure from Standards section in Chapter 2.

Switchback 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 Figures 5.2 through to 5.4. These figures show that the minimum outer radii for design vehicles DV2 through DV4 are 12.5m, 14.1m, and 12.5m, respectively. Minimum inner radii are 8m, 7.4m, and 6m, respectively.

Switchback requirements can be determined which allow for:

- Passage of two opposing DV4 vehicles. This is recommended for Design Standards DC8, DC7 and DC6
- Passage of a single DV4 and a DV1. This is recommended for Design Standards DS4- DC5
- Passage of only a single DV4. This is recommended for Design Standards DC3, DC3, DC2 and DC1.

Figure 8.4 illustrates a switchback curve.

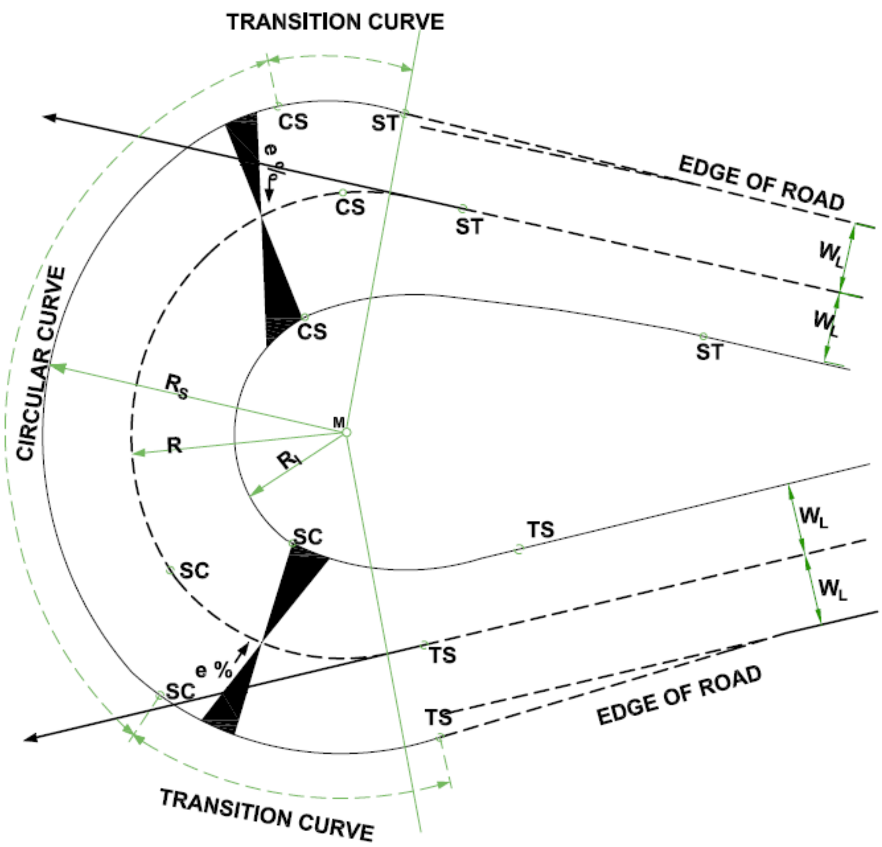


Figure 8-4: Switchback Curve

For a design example, consider road standard DC3 which allows for only the passage of a single DV4 vehicle. By superimposing Figure 3.11 for design vehicle DV4 over Figure 8.4 at the same scale, it can be shown that the requirements are (see Figure 8.5):

$R = 10m \qquad R_i = 6m \qquad R_s = 14m$

Thus although the normal carriageway width for a paved DC3 is 6.0m, at the switchback curve 8m is required.

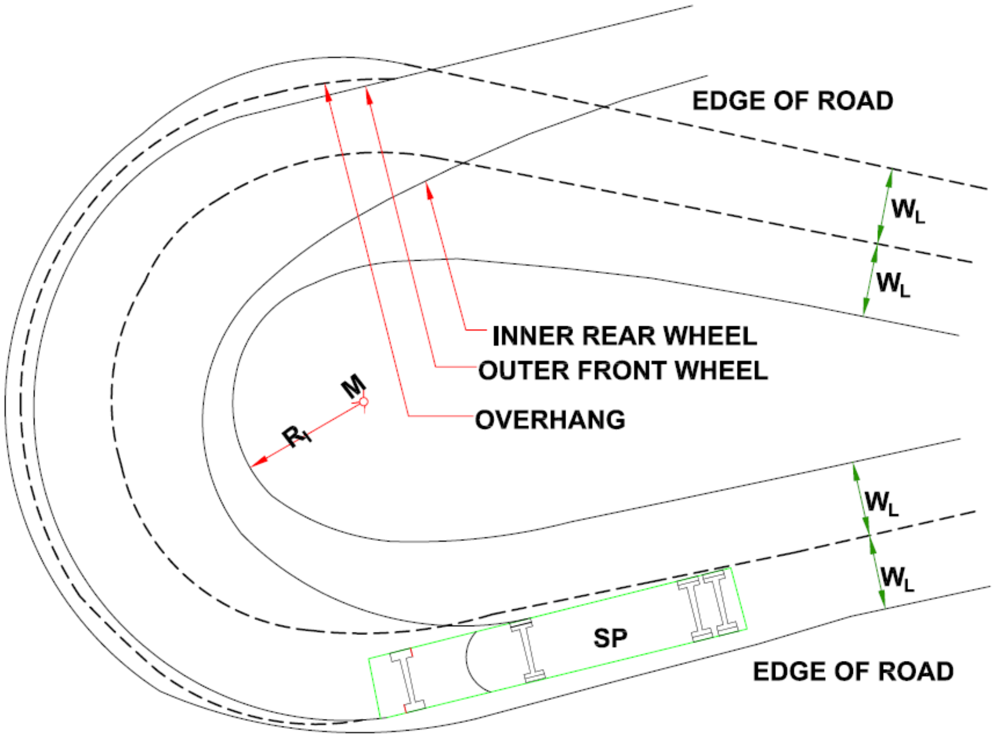


Figure 8-5: Switchback Curve for the Passage of Single DV4 Vehicle

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 switchback.

It is important to provide relief from a severe gradient through the switchback. Gradient parameters associated with a switchback curve are discussed in Section 9.5.

8.10 Transition Curves

The characteristic of a transition curve is that it has a constantly changing radius. Transition curves may be inserted between tangents and circular curves to reduce the abrupt introduction of lateral acceleration and therefore to enhance safety. They may also be used between two circular curves.

For large radius curves, the rate of change of lateral acceleration is small and transition curves are not normally required. It can also be argued that transition curves are not a requirement for certain roads, particularly those of lower classification. Another possible strategy would be to consider transitions for roads where a significant portion of the curves have a super-elevation in excess of 60 percent of the maximum super-elevation. For Ethiopian roads, transition curves are a requirement for trunk and link road segments having a design speed of equal to or greater than 80 km/hr. They are also required if the radius of the circular curve is less than the values shown in Table 8.4.

Table 8-4: Transition Curve Requirements

Design Speed (km/hr)	Transition Required if Radius of Curve is less than:
80	380
85	428
90	480
100	590
110	720
120	850

If a transition curve is required, the Euler spiral, which is also known as the clothoid, should be used. The radius varies from infinity at that tangent end of the spiral to the radius of the circular arc at the circular curve end. By definition, the radius at any point of the spiral varies inversely with the distance measured along the spiral.

In the case of a combining spiral connecting two circular curves having different radii, there is an initial radius rather than an infinite value.

8.11 Super-elevation

A tighter curve can be designed if higher values of super-elevation are used, but high values of super-elevation are not recommended if the friction is low, such as in locations where mud is likely to contaminate the road surface regularly. High values are also not recommended where mixed traffic and/or roadside development severely limit the speed of vehicles. In urban areas an upper limit of 4 percent should be used. Similarly, either a low maximum rate of super-elevation or no super-elevation is employed within important intersection areas or where there is a tendency to drive slowly because of turning and crossing movements, warning devices, and signals.

Super-elevation is, however, a requirement for all standards of roads.

8.12 Super-elevation Runoff

In alignment design with spirals, the super-elevation runoff is provided over the whole of the transition curve. The length of runoff is the spiral length, with the tangent to spiral (TS) transition point at the beginning and the spiral to curve (SC) transition point at the end. The change in cross slope begins by removing the adverse cross slope from the lane or lanes on the outside of the curve on a length of tangent just ahead of TS (the tangent runout). Between the TS and SC (the super-elevation runoff) the travelled way is rotated to reach the full super-elevation at the SC. This procedure is reversed on leaving the curve. By this design the whole of the circular curve has full super-elevation, as shown in Figure 8.6.

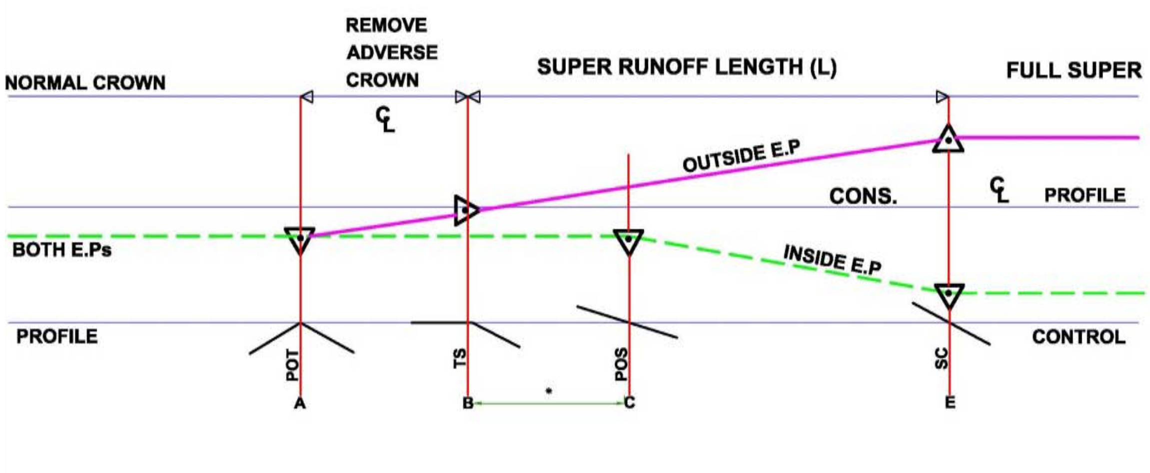


Figure 8-6: Spiral Curve Transition.

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

Current design practice is to place approximately two-thirds of the runoff on the tangent approach and one-third on the curve, as shown in Figure 8.7.

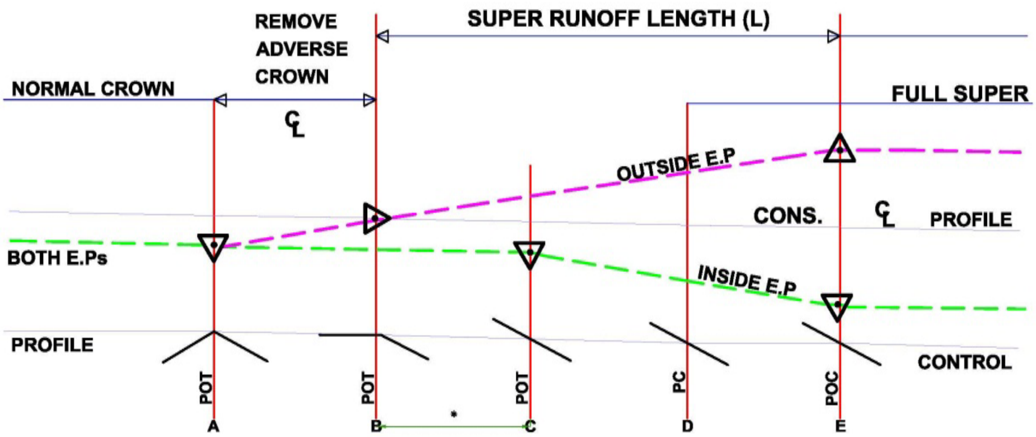
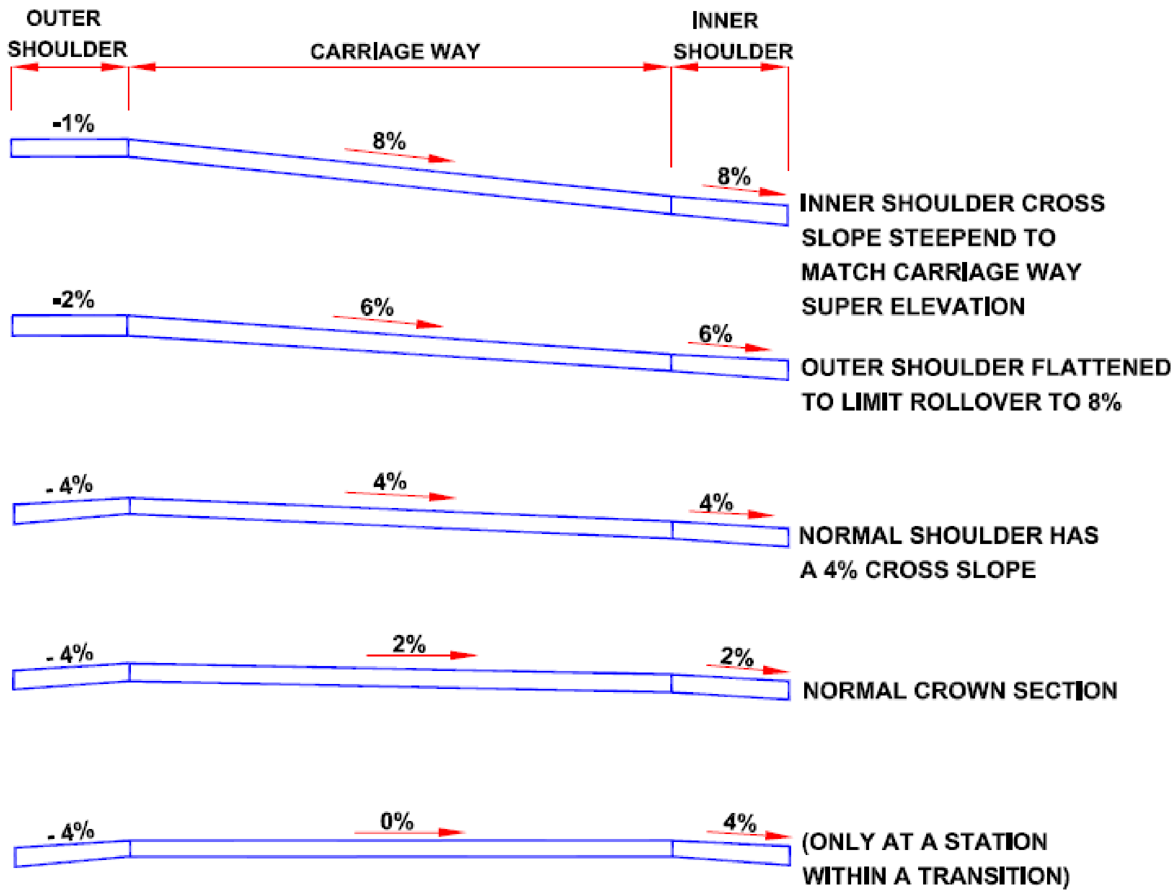


Figure 8-7: Circular Curve Transition

Table 8.5 gives super-elevation rates and length of runoff for horizontal curves at different speeds for a maximum super-elevation of 8 percent. Table 8.6 gives super-elevation rates and length of runoff for horizontal curves at different speeds for a maximum super-elevation rate of 4 percent.

8.13 Shoulder Super-elevation

Figure 8.8 depicts shoulder super-elevation rates corresponding to carriageway super-elevation rates. The figure shows that on the low side (inner shoulder) of super-elevated curves, the shoulder super-elevation matches the roadway super-elevation. On the high side (outer shoulder), the super-elevation is set such that the grade break between the roadway and the shoulder is 8 percent. An exception to this occurs at a maximum super-elevation of 8 percent, where the resultant shoulder super-elevation would be an undesirable flat configuration. Here the super-elevation is set at -1% to drain the shoulder.



Note: For design classes DC5 and lower the shoulder may be sloped with the carriageway, but the shoulder should then be surfaced on the outside of the curve.

Figure 8-8: Shoulder Super-elevation (for Surfaced Roads)

Table 8-5: Super elevation Rates and Length of Run-Off: 8%

	V_d=30km/h		V_d=40 km/h		V_d=50 km/h		V_d =60 km/h		V_d=70 km/h		V_d=85 km/h		V_d=100 km/h		V_d= 120 km/h	
R (m)	e (%)	L(m)	e (%)	L (m)	e (%)	L (m)	e (%)	L(m)	e (%)	L(m)	e (%)	L(m)	e (%)	L(m)	e (%)	L(m)
7000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
5000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
3000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	56	2.4	101
2500	NC	0	NC	0	NC	0	NC	0	NC	0	RC	47	2.1	56	2.9	101
2000	NC	0	NC	0	NC	0	NC	0	NC	0	2.2	47	2.6	56	3.5	101
1500	NC	0	NC	0	NC	0	NC	0	RC	39	2.5	47	3.4	56	4.6	101
1400	NC	0	NC	0	NC	0	RC	33	2.1	39	2.6	47	3.6	56	4.9	101
1300	NC	0	NC	0	NC	0	RC	33	2.2	39	2.8	47	3.8	56	5.2	101
1200	NC	0	NC	0	NC	0	RC	33	2.4	39	3.0	47	4.1	56	5.6	101
1000	NC	0	NC	0	RC	28	2.2	33	2.8	39	3.5	47	4.8	56	6.5	101
900	NC	0	NC	0	RC	28	2.4	33	3.1	39	4.2	47	5.2	56	7.1	101
800	NC	0	NC	0	RC	28	2.7	33	3.4	39	4.6	47	5.7	56	7.6	103
700	NC	0	RC	22	2.2	28	3.0	33	3.8	39	5.1	47	6.3	56	8.0	108
600	NC	0	RC	22	2.6	28	3.4	33	4.3	39	6.5	47	6.9	56	R_{min} = 665	
500	NC	0	2.2	22	3.0	28	3.9	33	4.9	39	7.2	47	7.8	56		
400	RC	17	2.7	22	3.6	28	4.7	33	5.7	39	7.8	51	8.0	64		
300	2.1	17	3.4	22	4.5	28	5.6	34	6.7	44	8.0	55	R_{min} = 395			
250	2.5	17	4.0	22	5.1	28	6.2	37	7.3	48	R_{min} = 270					
200	3.0	17	4.6	24	5.8	31	7.0	42	7.9	52						
175	3.4	17	5.0	26	6.2	33	7.4	44	8.0	52						
150	3.8	18	5.4	28	6.7	36	7.8	47	R_{min} = 175							
140	4.0	19	5.6	29	6.9	37	7.9	47								
130	4.2	20	5.8	30	7.1	38	8.0	48								
120	4.4	21	6.0	31	7.3	39	R_{min} = 125									
110	4.7	23	6.3	32	7.6	41										
100	4.9	23	6.5	33	7.8	42										
90	5.2	25	6.9	36	7.9	43										
80	5.5	26	7.2	37	8.0	43										
70	5.9	28	7.5	39	R_{min} = 80											
60	6.4	31	7.8	40												
50	6.9	33	8.0	41												
40	7.5	36	R_{min} = 50													
30	8.0	38														
	R_{min}=30															

e_{max}	= 8.0%
R	= radius of curve
V	= assumed design speed
e	= rate of superelevation
L	= minimum length of runoff(does not include tangent runout)
NC	= normal crown section
RC	= remove adverse crown, super-elevation at normal crown slope

Note : Lengths rounded in multiples of 10m to permit simpler calculations.

Table 8-6: Super elevation Rates and Length of Run-Off: 4%

	V_d=30km/h		V_d=40 km/h		V_d=50 km/h		V_d =60 km/h		V_d=70 km/h		V_d=85 km/h		V_d=100 km/h	
R (m)	e (%)	L(m)	e (%)	L (m)	e (%)	L (m)	e (%)	L(m)	e (%)	L(m)	e (%)	L(m)	e (%)	L(m)
7000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
5000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0
3000	NC	0	NC	0	NC	0	NC	0	NC	0	NC	0	RC	56
2500	NC	0	NC	0	NC	0	NC	0	NC	0	RC	47	2.2	56
2000	NC	0	NC	0	NC	0	NC	0	NC	0	2.1	47	2.6	56
1500	NC	0	NC	0	NC	0	NC	0	RC	39	2.2	47	2.7	56
1400	NC	0	NC	0	NC	0	NC	0	RC	39	2.3	47	2.8	56
1300	NC	0	NC	0	NC	0	RC	33	RC	39	2.5	47	2.9	56
1200	NC	0	NC	0	NC	0	RC	33	RC	39	2.7	47	3.2	56
1000	NC	0	NC	0	NC	0	RC	33	2.2	39	2.9	47	3.4	56
900	NC	0	NC	0	RC	28	2.1	33	2.4	39	3.2	47	3.5	56
800	NC	0	NC	0	RC	28	2.3	33	2.5	39	3.4	47	3.7	56
700	NC	0	NC	0	RC	28	2.5	33	2.7	39	3.5	47	3.9	56
600	NC	0	RC	22	2.1	28	2.7	33	2.9	39	3.7	47	4.0	56
500	NC	0	RC	22	2.3	28	2.9	33	3.1	39	3.9	47	R_{min} = 490	
400	NC	0	2.1	22	2.5	28	3.3	33	3.4	39	4.0	47		
300	RC	17	2.3	22	2.8	28	3.6	33	3.8	39	R_{min} = 285			
250	RC	17	2.6	22	3.0	28	3.8	33	3.9	39				
200	2.3	17	2.8	22	3.3	28	3.9	33	R_{min} = 215					
175	2.4	17	2.9	22	3.5	28	4.0	33						
150	2.5	17	3.1	22	3.7	28	R_{min} = 150							
140	2.6	17	3.2	22	3.8	28								
130	2.6	17	3.3	22	3.8	28								
120	2.7	17	3.4	22	3.9	28								
110	2.8	17	3.5	22	4.0	28								
100	2.9	17	3.6	22	4.0	28								
90	3.0	17	3.7	22	R_{min} = 100									
80	3.2	17	3.8	22										
70	3.3	17	3.9	22										
60	3.5	17	4.0	22										
50	3.7	18	R_{min} = 60											
40	3.9	19												
	R_{min}=35													

e_{max} = 4.0%
R = radius of curve
V = assumed design speed
e = rate of superelevation
L = minimum length of runoff(does not include tangent runout)
NC = normal crown section
RC = remove adverse crown, super-elevation at normal crown slope

Note : Lengths rounded in multiples of 10m to permit simpler calculations.
Use of e_{max} = 4.0% should be limited to urban conditions.

Source: AASHTO

9 VERTICAL ALIGNMENT

9.1 Introduction

Vertical alignment is the combination of parabolic vertical curves and tangent sections of a particular slope. The selection of rates of grade and lengths of vertical curves is based on assumptions about characteristics of the driver, the vehicle and the roadway. Vertical curvature may impose limitations on sight distance, particularly when combined with horizontal curvature.

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 describes the mathematical concepts for defining the vertical curvature of the road; defines the limiting characteristics for each road class; recommends maximum and minimum gradients; indicates gradient requirements through villages; develops the criteria for incorporation of a climbing lane; and provides vertical clearance standards.

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.

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 'roller coaster' or 'hidden dip' type of profile should be avoided.

9.2 Vertical Curve Formula

Vertical curves are required to provide smooth transitions between consecutive gradients. The simple parabola is specified for these because the parabola provides a constant rate of change of curvature and, hence, acceleration and visibility, along its length. Equations relating the various aspects of the vertical curve are as follows (Figure 9.1):

$$Y(x) = r.X^2/200 + X.g_1/100 + Y_{BVC} \qquad \text{Equation 9.1}$$

$$r = (g_2 - g_1)/L = G/L$$

Where

- BVC = Beginning of the vertical curve. The coordinates are normally (0, Y(0)),
- EVC = End of the vertical curve. The coordinates are normally (L, Y(L)),
- Y(X) = Elevation of a point on the curve (metres)
- X = Horizontal distance from the (BVC) (metres)
- g_1 = Starting gradient (%),
- g_2 = Ending gradient (%),
- r = Rate of change of grade per section (% per metre),
- L = Length of curve (horizontal distance) in metres,
- G = g_1 - g_2 (%),

K = L/G = horizontal distance required to achieve a 1% change in grade (metres),

Z = vertical distance from the tangent to the curve (metres)

Useful relationships are;

Equation of tangent g_1 is $Y(X) = Y(0) + g_1.X/100$

Equation of tangent g_2 is $Y(X) = Y(L) + g_2.(X-L)/100$

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

The Intersection Point always occurs at an x coordinate of 0.5L hence the elevation is always;

$$Y(IP) = (g_2+3.g_1)L/800 + Y(0)$$

Examples of crest and sag vertical curves are shown in Figures 9.1 and 9.2, respectively.

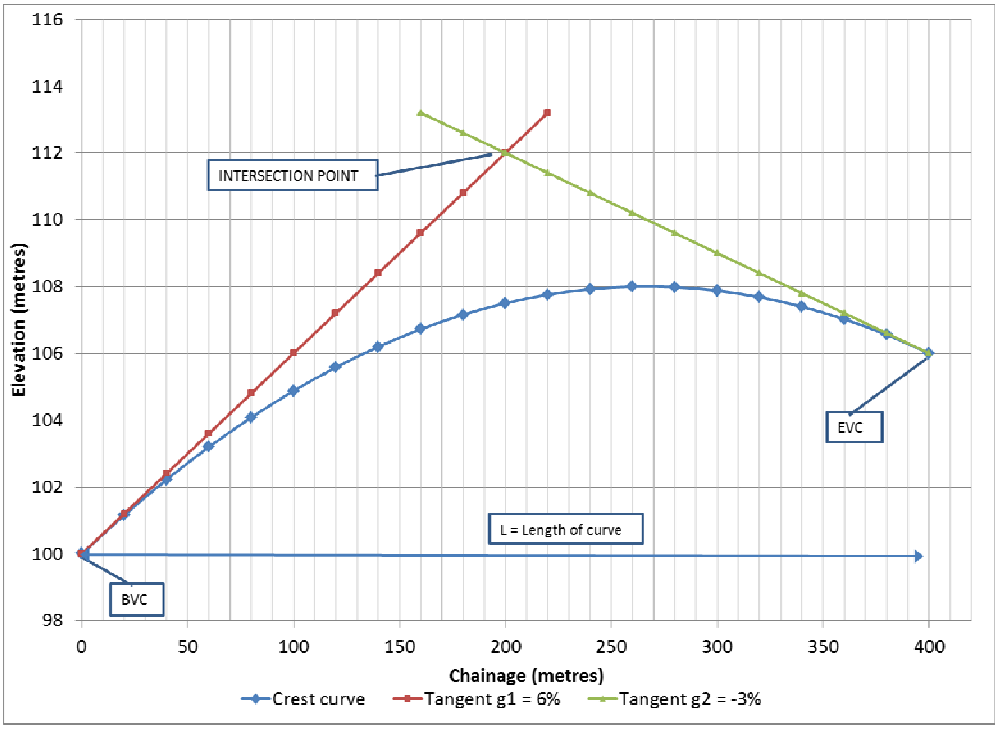


Figure 9-1: Crest Curve

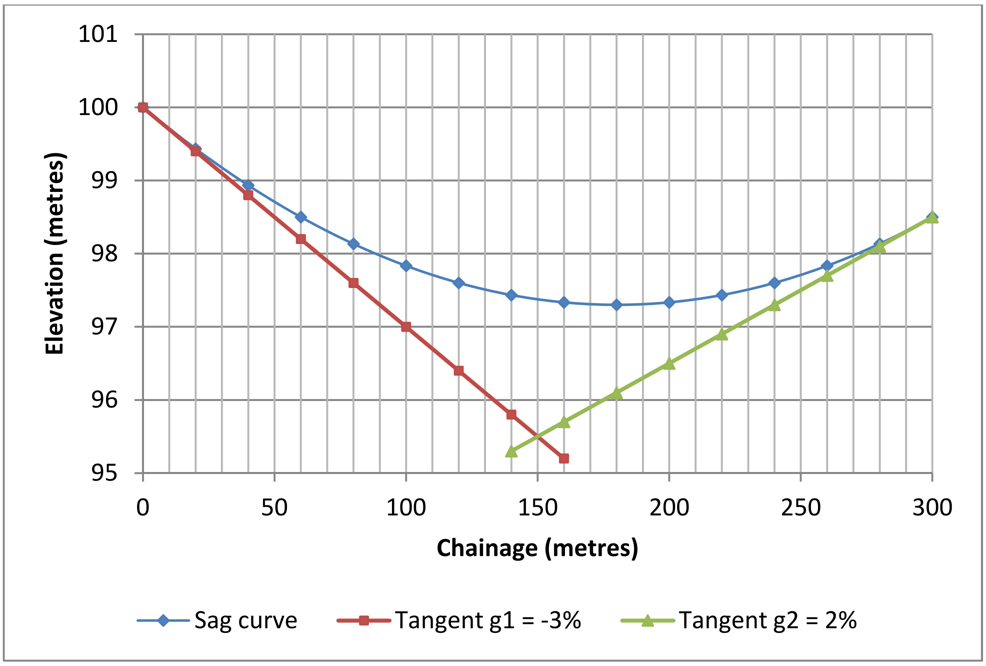


Figure 9-2: Sag Curve

Example:

For the crest curve shown in Figure 9.1 the two tangent grade lines are +6% and -3%. The Beginning of the Vertical Curve is at chainage 0.000 and its elevation 100.0m. The length of the vertical curve is 400m. Compute the End of Vertical Curve and the coordinates of the Intersection Point.

The y coordinate of the EVC is	$Y(L)$	$= (g_1+g_2)L/200 + Y(0)$
		$= (6 - 3).400/200 + 100.0 = 106.0$
The x coordinate is	$X(L)$	$= 400.0$
The coordinates of the IP are	$X(IP)$	$= L/2 \quad = 200.0$ and
	$Y(IP)$	$= (g_2+3.g_1)L/800 + Y(0)$
		$= (-3 + 3x6).400/800 + Y(0) = 107.5m$

9.3 Crest curves

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

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

$$L_m = (G.S^2)/[200(h_1^{0.5} + h_2^{0.5})^2] \qquad \text{and therefore}$$

$L_m = K.G$

where

L_m = minimum length of vertical crest curve (metres)

S = required sight distance (metres)

h_1 = driver eye height (metres)

h_2 = object height (metres)

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

$$L_m = 2S - [200.(h_1^{0.5} + h_2^{0.5})^2]/G$$

Eye height (h_1) has been taken as 1.05 metres, and object heights h_2 of 0.2 metres and 0.6 metres above the road surface.

In choosing which object height to adopt it is useful to know that in *open* country there is very little evidence of vehicles being involved in accidents over crest curves because of small objects in the carriageway. This may, of course, be because accident records are very imperfect in many countries but evidence from the USA and Canada supports this conjecture. In relatively flat terrain the most likely objects are another vehicle, a human being crossing the road and a pothole. The first two are more than 0.6m high and the third is at road surface level. An object height equal to the road surface itself is also applicable for sag curves on the approach to a ford or drift where a driver may have to stop because of the presence of surface water. For crest curves it has become more common to adopt an object height of 0.6m to cater for night time conditions when the object is likely to be the tail lights of a vehicle. However, in hilly and mountainous terrain the most likely object is a fallen rock and in forested area, the branch of a tree. In such conditions an object height of 0.2 metres is the best compromise.

The minimum lengths of crest curves have been designed to provide sufficient sight distance during daylight conditions. Longer lengths would be needed to meet the same visibility requirements at night on unlit roads but, even on a level road, low beam headlight illumination may not show up small objects at the design stopping sight distances. There is no point in providing a suitable sight distance if headlights are not bright enough to make use of it, therefore these longer lengths of curve are not justified. From a safety point of view, high objects such as vehicles and their tail lights will be adequately illuminated at the required stopping sight distances. Approaching vehicles will be identified by the approaching illumination and drivers should be more alert at night and/or be travelling at reduced speed.

Similar calculations can be carried out based on passing sight distance. High values of K result so that, in the situation where the crest of the curve is in cut, the increase in volume of excavation will be significant. Although the designer should seek to provide as much passing sight distance as possible along the length of the road, it may be useful to shorten the crest curve in order to increase the lengths of the grades on either side rather than to attempt achieving passing sight distance over the crest curve itself.

Minimum values of K for crest curves are shown in Tables 9.1 and 9.2.

Table 9-1: Minimum Values for Crest Vertical Curves (Paved Roads)

Design Speed (km/h)	K for Stopping Sight Distance			K for Passing Sight Distance
	h_2 = 0m	h_2 = 0.2m	h_2 = 0.6m	
20	2	1	1	10
25	3	1	1	30
30	4	2	1	50
40	10	5	3	90
50	20	10	7	130
60	35	17	11	180
70	60	30	20	245
80	95	45	30	315
85	115	55	35	350
90	140	67	45	390
100	205	100	67	480
110	285	140	95	580
120	385	185	125	680

Table 9-2: Minimum Values for Crest Vertical Curves (Unpaved Roads)

Design Speed (km/h)	K for Stopping Sight Distance			K for Passing Sight Distance
	h_2 = 0m	h_2 = 0.2m	h_2 = 0.6m	
20	2	1	1	10
25	3	1	1	30
30	5	2	2	50
40	11	6	4	90
50	25	11	8	135
60	45	20	15	185
70	75	35	25	245
80	120	58	40	315
85	150	72	50	350
90	185	90	60	390
100	270	130	88	480

9.4 Sag Curves

It is assumed that adequate sight distance will be available on sag curves in daylight. However, at night, visibility is limited by the distance illuminated by the headlamp beams.

Assumptions concerning the brightness of the headlights, their height above the road and the divergence of the beams have been made and minimum sag curve lengths for this condition have been computed. However the results lead to unrealistically long vertical curves, especially at higher speeds, and the required sight distances may be in excess of the effective range of the headlamp beam. Thus, the only likely situation when the calculations are useful is on the approaches to fords and drifts and other similar locations where flowing or standing water may be present on the road surface. Most of these structures occur on low speed roads where headlamp illumination is more likely to reach the full sight distances.

It is therefore recommended that, for most situations, sag curves are designed using a driver comfort criterion of vertical acceleration. A maximum acceleration of 0.3m/sec^2 is often used. This translates into

$$K > V^2/395$$

Where V is the velocity in km/hr.

The resulting curve length values are shown in Table 9.3.

Table 9-3: Minimum Values of K for Sag Curves

Design Speed (km/h)	K for driver comfort
20	1.0
25	1.5
30	2.5
40	4.
50	6.5
60	9
70	12
80	16
85	18
90	20
100	25
110	30
120	36

9.5 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 above formulae, 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 9.3. For this reason, where the algebraic difference in gradient is less than 0.5 percent, 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 metres or longer, except in mountainous or escarpment terrain.

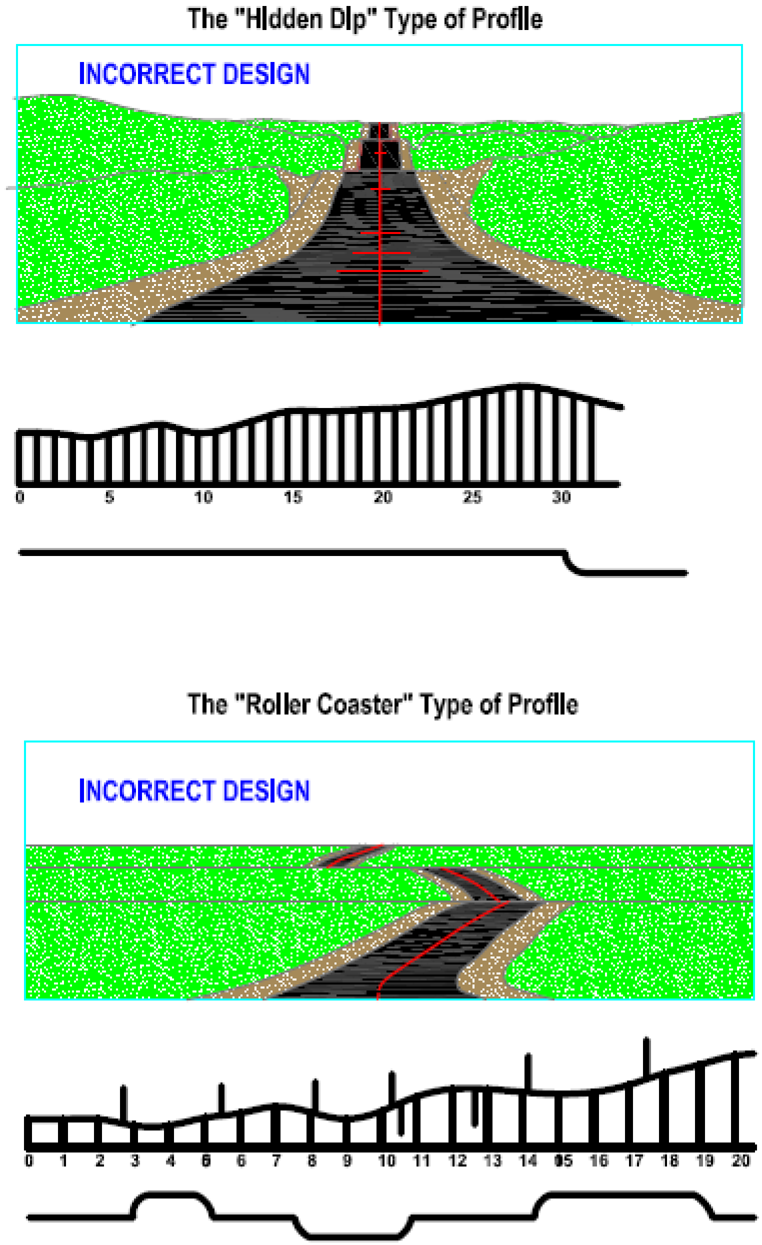


Figure 9-3: Hidden Dip and Roller Coaster Profiles

Where a crest curve and a succeeding sag curve have a common beginning and end, 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

For lower standard roads (DC1, DC2 and DC3), no minimum curve length is specified. In these cases, the curve lengths should be kept to a minimum to enhance drainage capabilities, and the curve lengths should match the K values given in Tables 2.6 to 2.17 for stopping sight distance. Where the difference in grade is less than 0.5 percent, the vertical curve is often omitted.

9.6 Maximum Gradients

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

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

However, under normal operating conditions the level of service and considerations of safety the geometric design should aim at achieving grades which will not reduce the speed of heavy vehicles to such an extent as to cause intolerable conditions for following drivers. It has been found that the frequency of truck accidents increases sharply when truck speed is reduced by more than 15 km/h. A speed reduction of 20 km/h is recommended as representing intolerable conditions. 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 lanes for the slower-moving vehicles

The vehicle fleet in Ethiopia is composed of a high percentage of vehicles that are under-powered and poorly maintained. Some existing roads are avoided and under-utilised by traffic because of an inability to ascend the existing grades. ERA has little choice but to limit gradients based on the existing fleet, although this translates into an added cost to develop the road infrastructure.

Recent research has quantified the costs of using earth or gravel surfaces on steep gradient. It has shown that spot improvements whereby the steep sections are surfaced with one of a variety of appropriate surfacings built by labour-based methods are very cost effective at providing all year access and reducing maintenance costs and whole life costs. Thus the limiting gradients on gravel and earth roads reflect this and it is expected that a spot improvement strategy will become standard practice in Ethiopia.

Maximum ‘absolute’ gradient and maximum ‘desirable’ gradient are therefore extremely important criteria that greatly affect both the serviceability and cost of the road. Performance considerations have formed the basis for defining the limiting criteria as summarised in Table 9.4 and shown in the design standards in Tables 2.6 to 2.17.

When gradients of 7 percent or greater are reached, consideration should be given to paving the steep sections (spot improvements) to enable sufficient traction to be achieved as well as to minimise maintenance requirements. As traffic increases, the economic dis-benefits of severe gradients, measured as increased vehicle operating and travel time costs, will justify reducing the severity and/or length of a gradient or paving the steep sections. On the higher design classes of road, the lower maximum recommended gradients reflect these economics. However, an economic assessment of alternatives to long or severe gradients should be undertaken where possible.

Table 9-4: Maximum Gradients for Paved Sections

Topography	Maximum Gradient (%), for Paved Sections									
	DC8, DC7, DC6		DC5, DC4		DC3, DC2		DC1		Basic Access	
	D	A	D	A	D	A	D	A	D	A
Flat	3	5	4	6	6	8	6	10		
Rolling	4, 5	7	6	8	7	9	7	10		
Mountainous	6, 7	9	8	10	10	12	10	12	NA	NA
Escarpment	6, 7	9	8	10	10	12	10	12		
Urban	6	8	7	9	7	9	7	9		

Note: D is the desirable value, A is the absolute value.

Standards for desirable maximum gradients were set to assure user comfort and to avoid severe reductions in the design speed. If the occasional terrain anomaly is encountered that requires excessive earthworks to reduce the vertical alignment to the desirable standard an absolute maximum gradient can be used. Employment of a gradient in excess of the desirable maximum can only be authorized through a Departure from Standard (see Section 2.2).

9.7 Maximum Gradients at Switchbacks

Where switchback 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 switchback curve is 4 percent for road standards DC8 to DC5, and 6 percent for DC4 to DC1. The minimum allowable gradient is 0.5%.

Corresponding crest and sag curves approaching the switchback curve must meet the requirements of Sections 9.3 and 9.4, and the transitions must be completed outside of the switchback curve. The sag curve above the switchback shall be made as long as possible to allow ascending vehicles to accelerate at the flatter grade when leaving the switchback.

9.8 Minimum Gradients

The minimum gradient for the usual case is 0.5 percent. However, flat and level gradients on uncurbed paved highways are acceptable when the cross slope and carriageway elevation above the surrounding ground is adequate to drain the surface laterally. With curbed highways or streets, longitudinal gradients should be provided to facilitate surface drainage.

9.9 Gradients through Villages

In many instances the natural grade level is flat through villages. The adjacent roadside ditches in such circumstances can readily become clogged and ineffective. Sometimes they are deliberately blocked to provide access to adjacent property or to channel flow for agricultural use. These practices lead to saturation of the subgrade and hence pavement failure, and should be avoided.

9.10 Critical Length of Gradient

The critical length of gradient is the maximum length of a designated upgrade upon which a loaded truck can operate without unreasonable reduction in speed. It is, to some extent, dependent on the gradient of the approach because a downhill approach will allow vehicles to gain momentum and thereby to increase the critical length. The critical length of gradient also decreases, as gradient increases. This is shown in Table 9.5. Where it is necessary to exceed the critical length of gradient on heavily trafficked roads, it is desirable to provide either safe passing distances on the rise, or a climbing lane for heavy vehicles.

9.11 Climbing Lanes

A climbing lane is an effective means of reducing the impact of a steep gradient. A climbing lane is an auxiliary lane added outside the continuous lanes and has the effect of reducing congestion in the through lanes by removing slower vehicles from the traffic stream. It also enhances road safety by reducing the speed differential in the through lane. The requirements for climbing lanes are therefore based on road standard, speed and traffic volume.

Benefits from the provision of a climbing lane accrue because faster vehicles are able to overtake more easily, resulting in shorter average journey times, reduced vehicle-operating costs, and increased safety. Benefits increase with increases in gradient, length of gradient, traffic flow, the proportion of trucks, and reductions in overtaking opportunities. The effect of a climbing lane in breaking up queues of vehicles held up by a slow moving truck will continue for some distance along the road.

Climbing lanes must be considered for roads when present traffic volumes are greater than 400 ADT. Thus the application of climbing lanes is limited particularly to trunk and link roads. Table 9.5 was prepared according to the criteria that a 20 km/h speed reduction is expected for a truck. It is used in the design to indicate locations where climbing lanes are recommended.

A climbing lane layout is shown in Figure 9.4. Climbing lanes must be clearly marked and, where possible, should end on level or downhill sections where speed differences between different classes of vehicles are lowest to allow safe and efficient merging manoeuvres. The introduction and termination of a climbing lane shall be effected by 100 metre long tapers. The tapers shall not be considered as part of the climbing lanes.

The starting point of the grade can be approximated as a point halfway between the preceding vertical point of intersection and the end of the vertical curve.

In escarpment terrain the carriageway and shoulder widths may have been reduced, hence a climbing lane, which will increase the width considerably, may not be economically justified. Consideration must be given to a balance between the benefits to traffic and the initial construction cost. In sections requiring heavy side cut, the provision of climbing lanes may be unreasonably costly in relation to the benefits. Reduced level of service over such sections is an alternative.

The climbing lane is sometimes not effectively utilised, especially when traffic flows are heavy, because the drivers of slower vehicles fear that they will not be allowed to merge with the faster vehicles where the climbing lane ends. The preferred layout forces faster vehicles to merge with the slower, thus allaying this fear to some extent. This layout is preferred based purely on that fact that a vehicle can merge more readily with a slower- than with a faster-moving stream of traffic (see Figure 9.4).

A slow moving vehicle should be completely clear of the through lane by the time its speed has dropped by 20 km/h, and remain clear of the through lane until it has accelerated again to a speed which is 20 km/h less than its normal speed.

Table 9-5: Climbing Lanes

Design Class	Gradient (%)	Critical length of gradient above which a climbing lane is required (m)	Maximum desirable length of gradient (m)
DC7 and DC6	4	300	900
DS7, DC6, DC5	5	240	800
	6	200	700
	7	170	600
	8	150	500
	9	130	400
	10	Required	400
DC4	11	Required	400
	12	Required	400

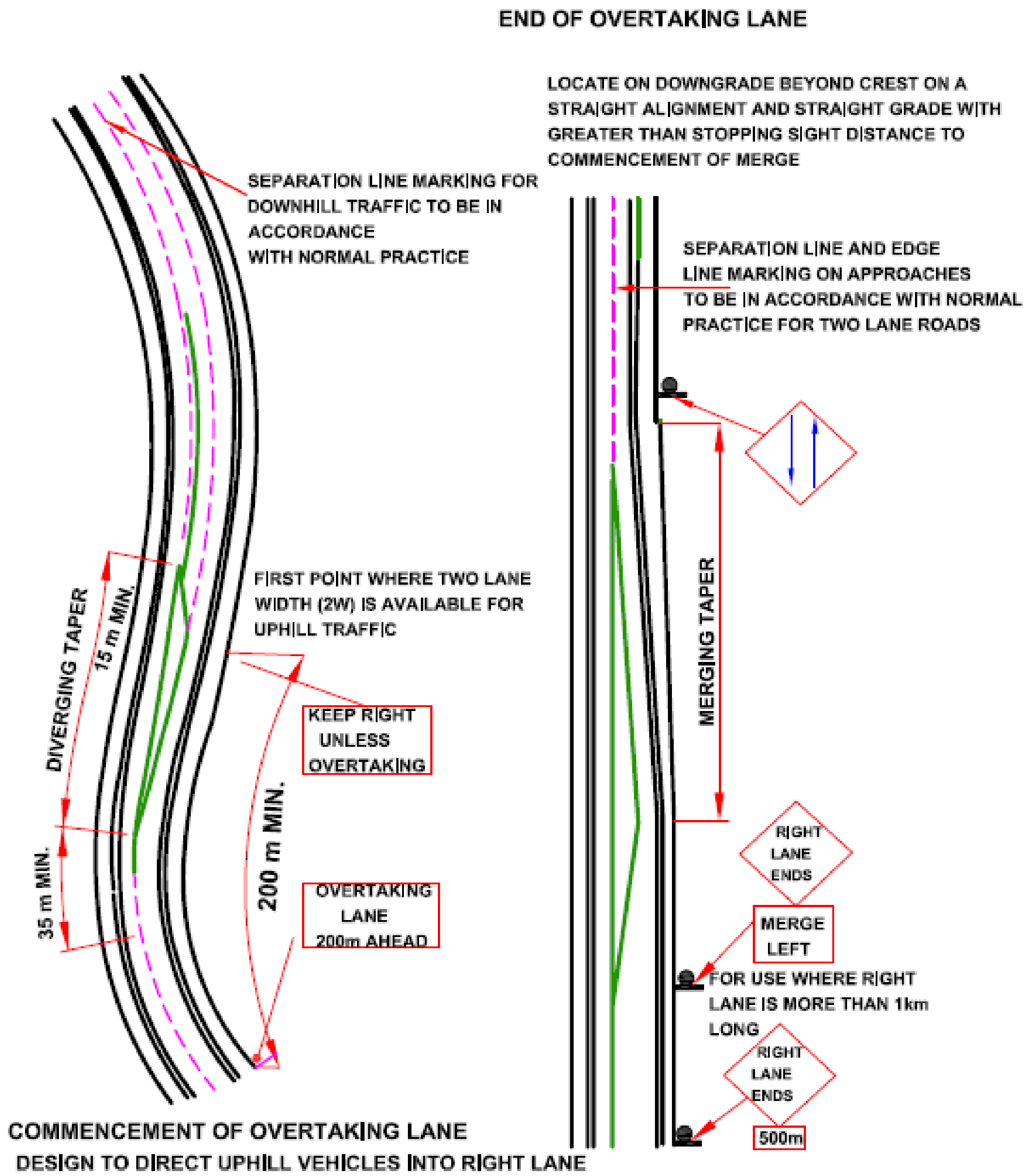


Figure 9-4: Layout for Climbing Lane

The performance characteristics of a heavy vehicle are such that, for a particular gradient, the vehicle speed will decrease to a final ambient speed that can be maintained by that vehicle on that grade. This limits, in most references, any discussion on the maximum length allowable at a given grade even considering the employment of a climbing lane. However, in the interests of factors such as vehicle operating costs and travel time losses, the absolute recommended maximum lengths at any given grade are also indicated in the last column of Table 9.5. When these distances are reached, it is necessary to design a relief gradient of less than 6 percent between steep sections. The relief gradient must extend a minimum of 100 metres.

These values have also taken into consideration the safety factors associated with the increase in speed resulting in the descent of steep grades. Although they may mitigate the safety hazard, they do not eliminate it. For example, a non-braking heavy truck will accelerate from 0 km/hr to 90 km/hr over a distance of about 500 metres at a descending grade of 5 percent. This emphasizes the need to provide warning signs for such vehicles at all long continuous grades. The use of ‘escape lanes’ is discussed in Section 13.9.

9.12 Vertical Clearances

Bridges over water shall normally have a minimum clearance height according to Table 9.6 unless a refined hydraulic analysis has been made. The standard minimum headroom or clearance under bridges or tunnels shall be 5.1m 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 0.1m may be provided. Light superstructures (e.g. timber, steel trusses, steel girders, etc) over roadways shall have a clearance height of at least 5.3m. See ERA's *Bridge Design Manual* for further reference.

Table 9-6: Vertical Clearance from Superstructure to Design Flood Level (DFL)

Design Flow at Bridge (m^3/s)	Vertical Clearance (m)
5 to 30	0.6
30 to 300	0.9
>300	1.2

Source: ERA Bridge Design Manual

Underpasses for pedestrians and bicycles shall not be less than 2.4m. For cattle and wildlife, underpasses shall be designed as the normal height of the actual kind of animal plus 0.5m, and for horse-riding the clear height shall be not less than 3.4m. Bridges above railways shall have a clearance height of at least 6.1m- 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 *ERA Drainage Design Manual.*

10 PHASING OF HORIZONTAL AND VERTICAL ALIGNMENT

10.1 Alignment Defects Due to Mis-phasing

Phasing of the vertical and horizontal curves of a road implies their coordination so that the line of the road appears to a driver to flow smoothly, avoiding the creation of hazards and visual defects. It is particularly important in the design of high-speed roads on which a driver must be able to anticipate changes in both horizontal and vertical alignment well within the safe stopping distance. It becomes more important with small radius curves than with large.

Defects may arise if an alignment is mis-phased. Defects may be purely visual and do no more than present the driver with an aesthetically displeasing impression of the road. Such defects often occur on sag curves. When these defects are severe, they may create a psychological obstacle and cause some drivers to reduce speed unnecessarily. In other cases, the defects may endanger the safety of the user by concealing hazards on the road ahead. A sharp bend hidden by a crest curve is an example of this kind of defect.

10.2 Types of Mis-phasing and Corresponding Corrective Action

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

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.

10.3 Vertical Curve Overlaps One End of the Horizontal Curve

If a vertical curve overlaps either the beginning or the end of a horizontal curve, a driver’s perception of the change of direction at the start of the horizontal curve may be delayed because his sight distance is reduced by the vertical curve. This defect is hazardous. The position of the crest is important because vehicles tend to increase speed on the down gradient following the highest point of the crest curve, and the danger due to an unexpected change of direction is consequently greater. If a vertical sag curve overlaps a horizontal curve, an apparent kink may be produced, as indicated in Figures 10.1b and c.

The defect may be corrected in both cases by completely separating the curves. If this is uneconomic, the curves must be adjusted so that 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.

10.4 Insufficient Separation between the Curves

If there is insufficient separation between the ends of the horizontal and vertical curves, a false reverse curve may appear on the outside edge-line at the beginning of the horizontal curve. This is a visual defect, illustrated in Figure 10.1d.

Corrective action consists of increasing the separation between the curves, or making the curves concurrent, as in Figure 10.1a.

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

If both ends of a crest curve lie on a sharp horizontal curve, the radius of the horizontal curve may appear to the driver to decrease abruptly over the length of the crest curve. If the vertical curve is a sag curve, the radius of the horizontal curve may appear to increase. An example of such a visual defect is shown in Figure10.1e. The corrective action is to make both ends of the curves coincident as in Figure 10.1a, or to separate them.

10.6 Vertical Curve Overlaps Both Ends of the Horizontal Curve

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

The corrective action is to make both ends of the curves coincident. If the horizontal curve is less sharp, a hazard may still be created if the crest occurs off the horizontal curve. This is because the change of direction at the beginning of the horizontal curve will then occur on a downgrade (for traffic in one direction) where vehicles may be increasing speed.

The corrective action is to make the curves coincident at one end so as to bring the crest on to the horizontal curve.

No action is necessary if a vertical curve that has no crest is combined with a gentle horizontal curve.

If the vertical curve is a sag curve, an illusory crest or dip, depending on the “hand” of the horizontal curve will appear in the road alignment.

The corrective action is to make both ends of the curves coincident or to separate them.

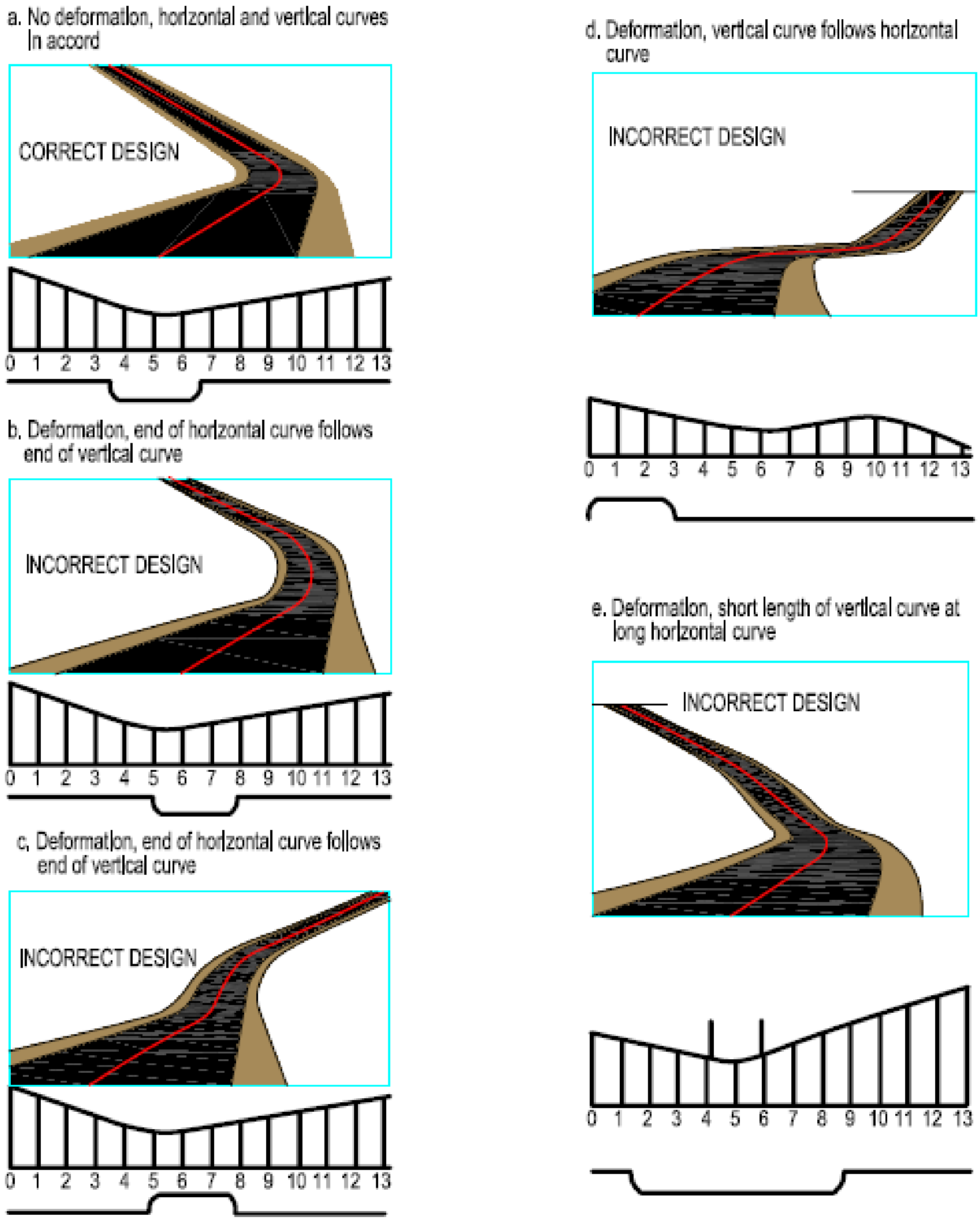
10.7 Other Mis-Phasing

Other types of mis-phasing are indicated in Figure 10.1:

- A sag curve occurs between two horizontal curves in the same direction in Figure 10.1g. This illustrates the need to avoid broken back curves in design (see Chapter 8: Horizontal Alignment).
- A double sag curve occurs at one horizontal curve in Figure 10.1h. This illustrates the effect in this case of a broken back vertical alignment on design (see Chapter 9: Vertical Alignment).
- Figure 10.1i shows a lack of phasing of horizontal and vertical curves. In this case, the vertical alignment has been allowed to be more curvilinear than the horizontal alignment.

10.8 The Economic Penalty Due to 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. He will normally accept curves that have to be phased for reasons of safety. In cases when the advantage due to phasing is aesthetic, the designer will have to balance the costs of trial alignments against their elegance.



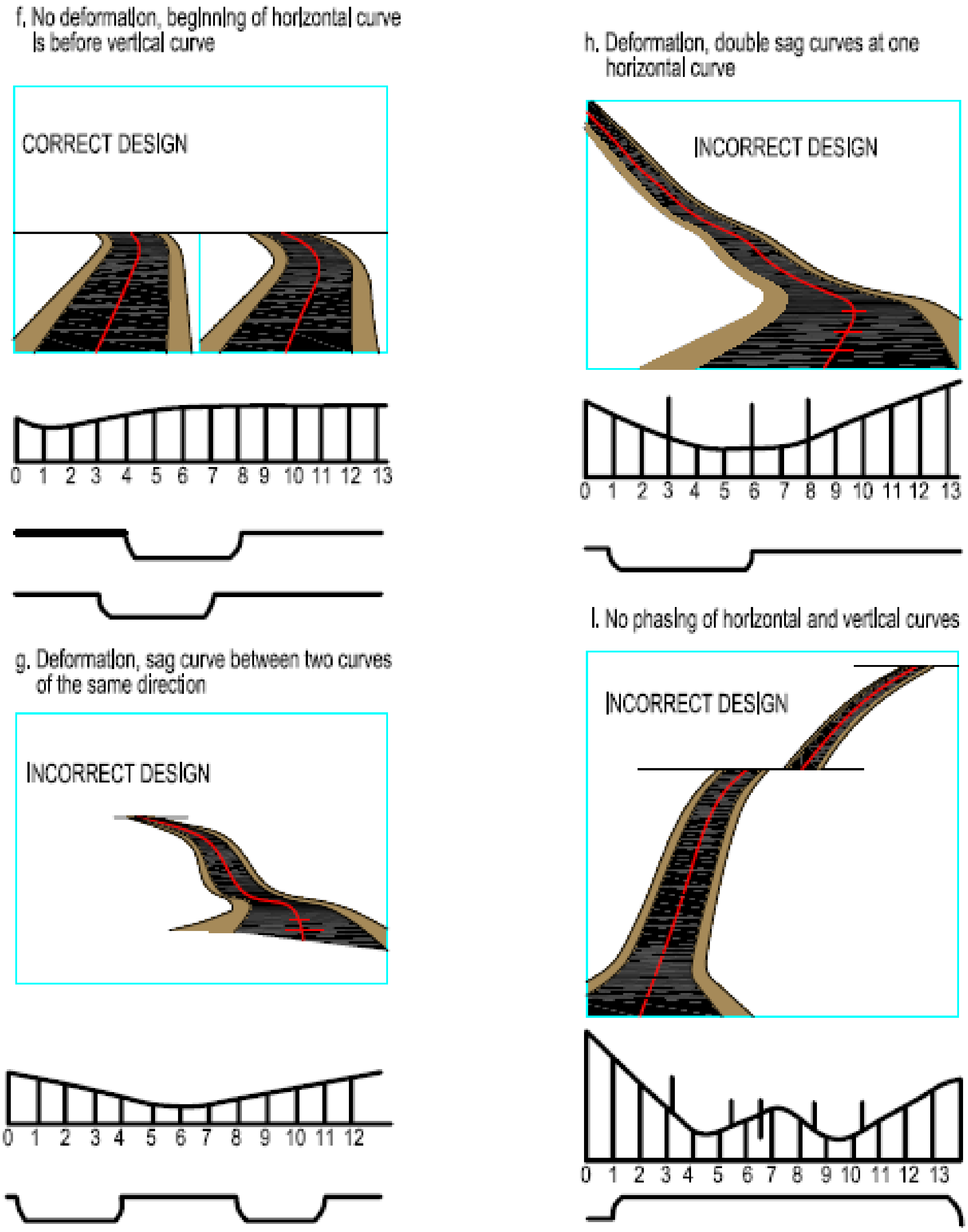


Figure 10-1: Phasing of Horizontal and Vertical Curves

11 AT-GRADE JUNCTIONS

11.1 Introduction

A junction, or intersection, is the general area where two or more roads join. A disproportionate amount of traffic accidents occur at junctions, and thus, from a traffic safety aspect, junctions require attention and careful design. Good junction design should allow transition from one route to another or through movement on the main route and intersecting route with minimum delay and maximum safety. To accomplish this, the layout and operation of the junction should be obvious to the driver, with good visibility between conflicting movements.

Differing junction types will be appropriate under different circumstances depending on traffic flows, speeds, and site limitations. Types of junctions are illustrated in Figure 11.1 and include:

i) Three-legged T and Y-Junctions.
ii) Four-legged Cross-Junctions.
iii) Multi-legged (>4) Junctions.
iv) Roundabouts.
v) Grade Separation (discussed in Chapter 12).

Intersections vary greatly in their design and complexity based primarily on the traffic volume that they have to accommodate. The important factors to be considered in the selection of an intersection type are:

i) Volumes of traffic to be accommodated.
ii) Proportion of traffic on each approach.
iii) Approach speeds.
iv) Land use and land availability.
v) Local environment.
vi) Cost of construction.

Safety at intersections is of key importance. For example, the National Safety Council in the USA has estimated that 56 per cent of all urban accidents and 32 per cent of all rural accidents occur at intersections. Poor design, e.g. inadequate sight distances, leads to significantly higher injury and total accident rates, hence good design is vital if accident rates are to be minimised. It should be noted that roundabouts have considerable safety advantages over other types of at-grade intersections.

This chapter describes the design for all at-grade junctions. A checklist for junction design is presented in Figure 11.19 at the end of the chapter.

11.2 Design Requirements

Intersections are of two basic types, those where traffic speed is uncontrolled and those where some control is provided. Control may consist of a mandatory ‘stop line’ or considerably more complex signal control or traffic management system.

Uncontrolled intersections have no control at all, or simply provide advice to drivers to ‘give-way’ or ‘yield’ on the basis of priority to the first to arrive at the junction or to traffic on the major road. Such intersections are used where traffic volume is low. When the traffic is similar on all legs, the recommended capacity of such a junction is about 250 vehicles per hour in each direction on each leg. This increases to about 750 vehicles per hour in each direction on the major road when traffic on the minor road is 10 per cent of this. When traffic exceeds these values, additional features need to be included as described in the following Sections.

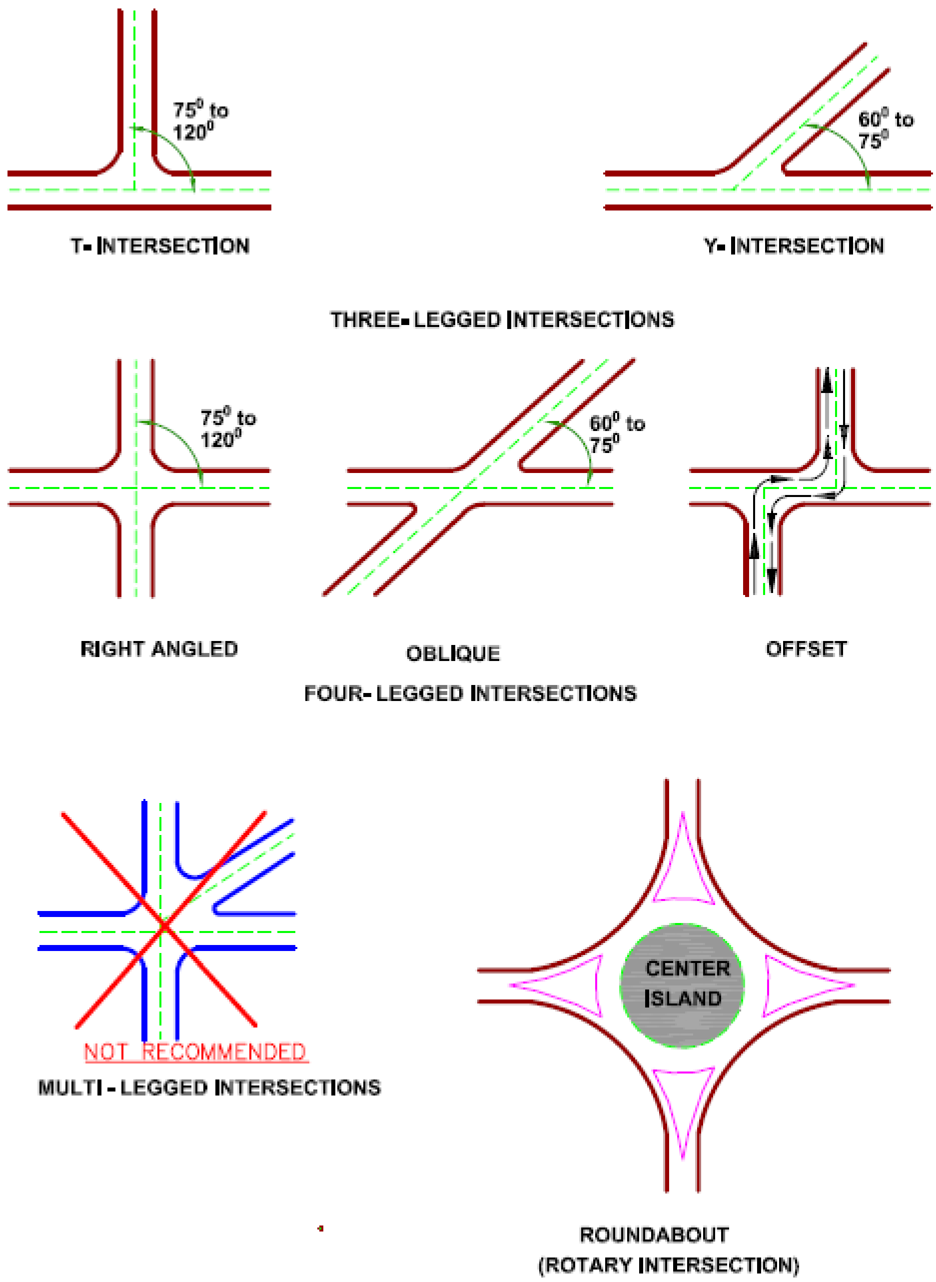


Figure 11-1: Types of at-grade junctions

The design of junctions must take account of the following basic requirements:

i) Safety.
ii) Operational comfort.
iii) Capacity.

iv) Economy.

A junction is considered safe when it is visible, comprehensible, and manoeuvrable. The right of way should follow naturally and logically from the junction layout and the types of junctions used throughout the whole road network should be similar. The use of road signs may be necessary and road markings and other road furniture will almost always be required.

The basic objectives of good intersection design are:

- Reduction of the number of points of potential conflict to the minimum compatible with efficient operation;
- Reduction of the complexity of conflict areas whenever possible;
- Limitation of the frequency of actual conflicts; and
- Limitation of the severity of those conflicts that do occur.

The tools available to apply these principles are interrelated:

i) Defining and arranging traffic lanes;
ii) Use of traffic islands of all sizes and types;
iii) Median islands;
iv) Horizontal and vertical approach geometry;
v) Pavement tapers and transitions; and
vi) Traffic control devices.

The first five elements are a range of physical features while traffic control devices are an integral part of any intersection.

11.3 Selection of Junction Type

11.3.1 Junction Choice

The choice of a junction type requires knowledge of traffic demand, intersection performance and accident prediction. It is often difficult to determine the best intersection type for any particular situation, taking into account capacity, delay, safety and physical layout factors. Several alternatives may be possible at a junction. A guide for the selection of junction type based on different combinations of traffic flows is shown in Figure 11.2. For lower volume junctions, the applicable junction type depending on traffic flow is shown in further detail in Figure 11.3.

The basic advantages and disadvantages of different junction types, including grade separation, are as follows:

Priority (T-Junction, Cross-Junction). For low flows. These can cause long delays. They require sufficient stopping sight distance. Delays can be improved by signal installation.

Roundabouts. These are best for low to medium flows. They provide for minimum delays at lower flows. They have been shown to be safer than priority junctions. They require attention to pedestrian movements and the accommodation of slow-moving traffic.

Grade-Separation. This is only for high flows because they are expensive but they result in minimum delays. Pedestrian movements also need special consideration.

These options are discussed in more detail below.

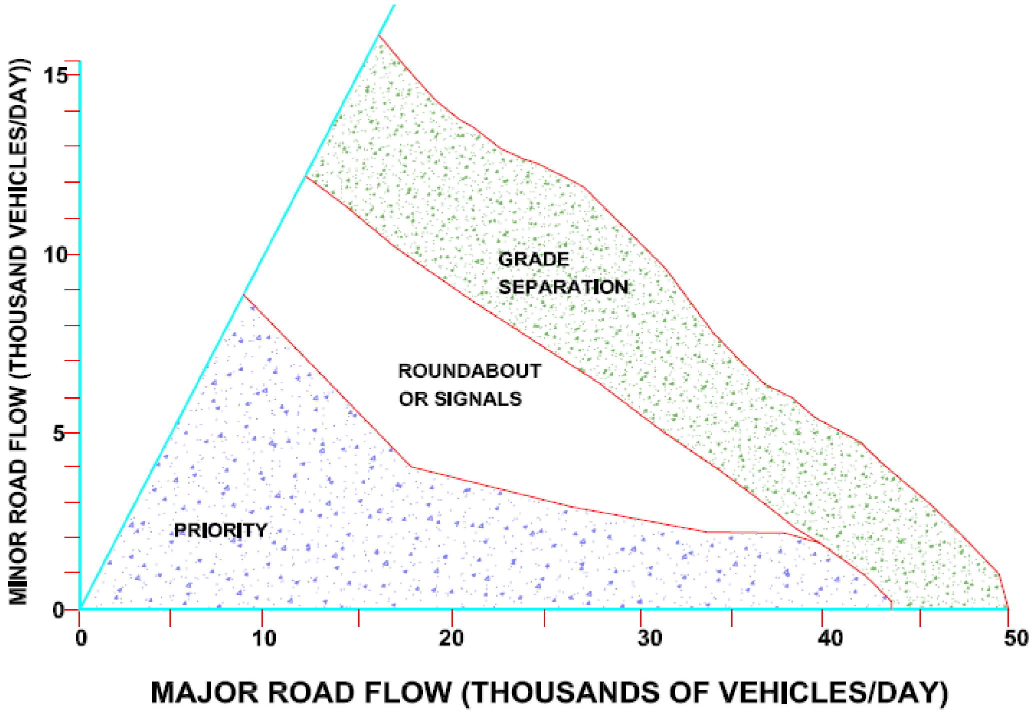


Figure 11-2: Junction Selection Based on Traffic Flows

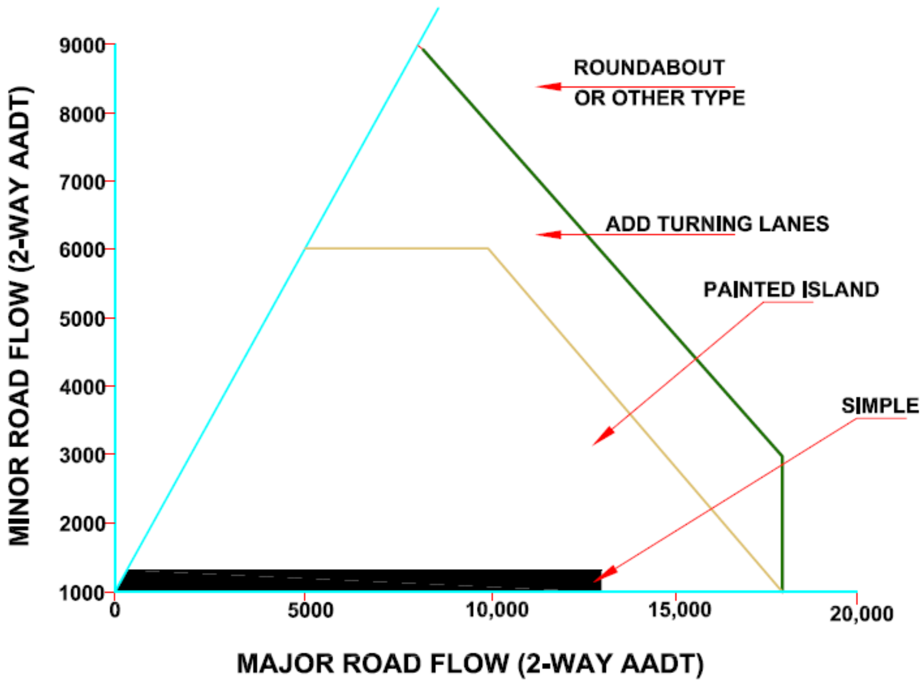


Figure 11-3: T-Junction Selection for Various Major and Minor Road Traffic Flows

11.3.2 T-Junctions

The basic junction layout for rural roads is the T-junction with the major road traffic having priority over the minor road traffic. Applications of T-junctions include the staggered T-junction, which caters to cross-traffic. Staggered T-junctions are often the result of a realignment of the minor route to improve the angle of the skew of the crossing, as shown in Figure 11.4. Where such staggered T-junctions are used, the left-right stagger is preferred to the right-left stagger. The reason for this is that, in the latter case, a crossing vehicle must re-enter the minor road by making a left turn on the major roadway. In other words the number of turning manoeuvres that require traffic to cross a traffic stream increases from 2 to 4. In such cases, the inclusion of a left-turning lane between the staggers should be considered. The minimum distance between the T-junctions is shown in Table 11.1.

Figure 11-4: Staggered X-Junctions

11.3.3 Cross Junctions

A cross junction is required where two highways cross each other (i.e. a crossroads). It therefore has four legs. The overall principles of design, island arrangements, use of turning lanes, and other parameters are similar to those used in T-junctions.

Experience in some countries has shown that converting crossroads into roundabouts can reduce accident costs by more than 80 per cent.

Where more complex junction layouts involving the intersection of four or more roads are encountered, these should be simplified by redesign to two junctions, or a roundabout should be used.

Having selected the basic junction layout, it is necessary to adapt this basic layout in accordance with the following principles to ensure that a safe, economic and geometrically satisfactory design will be produced.

11.4 Angle of Intersection

The angle of intersection of two roadways influences the operation and safety of a junction. Large skews increase the pavement area and thus the area of possible conflict. Operationally they are undesirable because:

- Crossing vehicles and pedestrians are exposed for longer periods;
- The driver's sight angle is more constrained and gap perception becomes more difficult;
- Vehicular movements are more difficult;

- Large trucks require more pavement area; and
- Defining vehicle paths by channelization is more difficult.

For new intersections the crossing angle should preferably be in the range 75° to 120°. The absolute minimum angle of skew is 60° because drivers, particularly of trucks with closed cabs, have difficulty at this angle of skew in seeing vehicles approaching from one side. The designer should be able to specifically justify using an angle of skew less than 75°. In the remodelling of existing intersections, the accident rates and patterns will usually indicate whether a problem exists and provide evidence on any problems related to the angle of skew.

11.5 Distance between Adjoining Junctions

Level of service and driver perception is affected by the spacing of junctions. In certain cases it may be necessary to limit the number of junctions for reasons of safety and serviceability. Table 11.1 gives a guide to the minimum spacing for each road design standard, and should be used for the design of new roads or when reviewing junction layouts.

Table 11-1: Access Control

Design Standard	Minimum Spacing of Junctions (m)
DC8	1000
DC7	500
DC6- DC5	300
DC4- DC3	100
DC2- DC1	20

11.6 Sight Distances and Visibility

The key to good design and maximum safety is to provide adequate sight distances for the manoeuvres that are required. On a basic cross-road intersection these manoeuvres are left turns and right turns from both the minor road and the major road and crossing manoeuvres across the junction. 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 time required to carry out the manoeuvre depends upon:

i) Whether the vehicle is in motion and at what speed when it reaches the junction (yield control – Figure 11.5) or begins from a stopped position (stop control – Figure 11.6);
ii) The type and power of the vehicle;
iii) The length of the vehicle;
iv) The distance the vehicle needs to travel (number of lanes plus median, if present);
v) The gradient of the road which the vehicle has to negotiate;
vi) The perception and reaction time of the driver;

Thus calculating the time *required* is complex and varies considerably for different conditions.

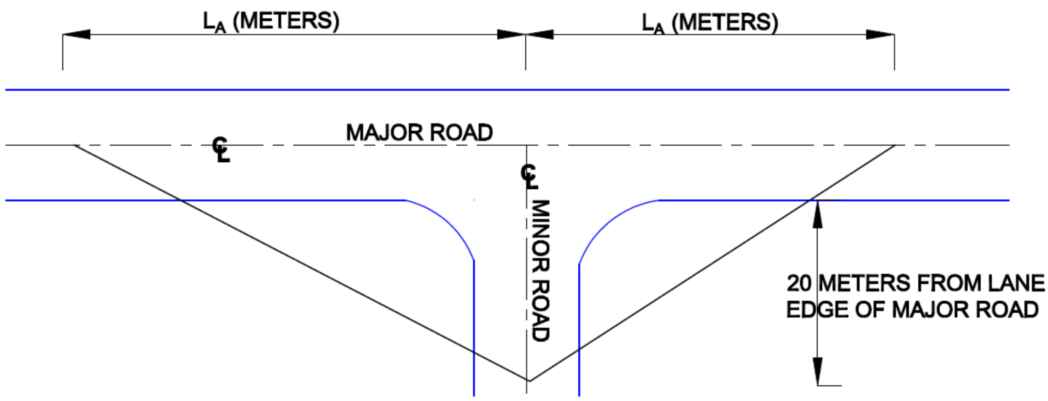


Figure 11-5: Visibility Splay for 'Yield' Conditions

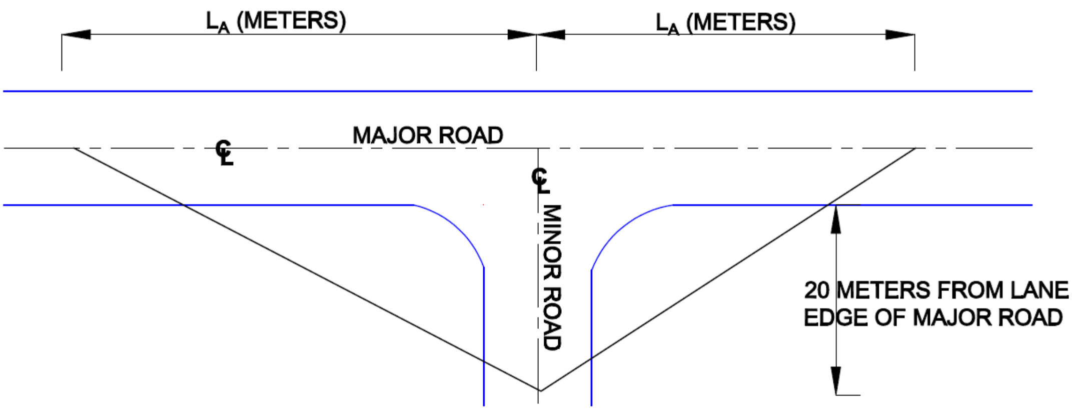


Figure 11-6: Visibility Splay for 'Stop' Conditions

The time *available* to carry out the manoeuvre depends on the speed of traffic in the lanes to be crossed. This speed is not the design speed of the road because drivers tend to slow down when approaching junctions, even when on the major road. However 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.

Models have been developed for carrying out these calculations but require many assumptions and are not reliable. The best information is obtained from empirical data, but this is primarily based on research in western countries. As has been emphasised elsewhere in this manual, the mix of traffic, its age spectrum, overloading practices etc. are entirely different in Ethiopia. Furthermore, it is apparent from the discussion above that the required sight distances also depend on driver behaviour. Nobody can doubt that driver behaviour in western countries is significantly different to driver behaviour in Ethiopia. In summary, it is not a simple task to calculate the optimum or minimum sight distances applicable to different junction designs, different road classes and different mixes of traffic in Ethiopia. A pragmatic approach is to utilise the available empirical data but to select conservative options for safety. Appendix C summarises the empirical approach and shows how sight distances can be calculated for the majority of manoeuvres and for different design vehicles.

The greatest sight distances are needed for the manoeuvres that take the longest to execute (required time) and involve joining fast traffic (available time). This inevitably means that heavy truck and trailer combinations require the greatest sight distances when joining a main road. Catering for this situation is not always possible. The methods described in Appendix C can be used to calculate sight distances for different design vehicles but it is prudent to use DV4 for most designs.

Practical sight distances are shown in Tables 11.2 and Table 11.3 below and are generally more conservative than the more precise values obtained using Appendix C. However, when the gradients on any of the legs exceed 3 per cent and when multiple lanes are involved, prudent use of Appendix C should be made, bearing in mind that driver behaviour is likely to be different in Ethiopia and therefore conservative options should be used.

Table 11-2: Minimum Sight Distances for 'Yield' 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 11-3: Minimum Sight Distances for 'Stop' 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

11.7 Lane Width and Manoeuvrability

All traffic lanes should be of adequate width and radius for the appropriate vehicle turning characteristics. To accommodate truck traffic, turn radii shall be a minimum of 15 metres.

Where intersecting roadways have shoulders or sidewalks, the main road shoulder should be continued through the intersection. Lane widths should normally be the same as that of the major road so that approaches from the minor road are usually widened if necessary.

Where conditions are severely constrained, lane widths as low as 3.3 metres can be considered, provided that approach speeds are below 80 km/h. In constricted urban conditions on low speed-roadways, lane widths of 3.0 metres should be the minimum adopted.

Offsets from the edge of a turning roadway to kerb lines should be 0.6 to 1.0 metres.
The edges of traffic lanes should be clearly indicated by road markings.

11.8 Horizontal and Vertical Alignment

The horizontal and vertical alignments through and approaching an intersection are critical features. Simple alignment design allows for early recognition of the intersection and timely focus on the intersecting traffic and manoeuvres that must be prepared. The following are specific operational requirements at intersections:

- The alignments should not restrict the required sight distance;
- The alignments should allow for the frequent braking and turning associated with intersections; and
- The alignments should not require a driver's attention to be detracted from the intersection manoeuvres and conflict avoidance.

As a general guide, horizontal curve radii at intersections should not be less than the desirable radius for the design speed on the approach roads. For high-speed roads with design speeds in excess of 80 km/h, approach gradients should not be greater than -3 per cent. For low-speed roads in an urban environment this can be increased to -6 per cent.

11.9 Channelisation

11.9.1 Purpose of Channelisation

The operation of the junction depends principally upon the frequency of gaps that naturally occur between vehicles in the main road flow. These gaps should be of sufficient duration to permit vehicles from the minor road to merge with, or cross, the major road flow. In consequence, junctions are limited in capacity, but this capacity may be optimised by, for example, channelisation or the separation of manoeuvres.

At-grade intersections with large paved areas, such as those with large corner radii or with angles of skew differing greatly from 90^o, permit unpredictable vehicle movements, require long pedestrian crossings and have unused pavement areas. Even at a simple intersection there may be large areas on which vehicles can wander from natural or expected paths. Under these circumstances it is usual to resort to channelisation.

The purpose of channelisation is to manage the conflicts that are inherent in any intersection. There are eight principles of channelisation:

i) Undesirable or wrong-way movements should be discouraged or prohibited;
ii) Vehicle paths should be clearly defined;
iii) The design should encourage safe vehicle speeds;
iv) Points of conflict should be separated whenever possible;
v) Traffic streams should cross at close to right angles and merge at flat angles;
vi) High priority flows should have the greater degree of freedom;
vii) Decelerating, slow-moving or stopped vehicles should be separated from higher-speed through lanes; and
viii) Refuge for pedestrians and the handicapped should be provided where appropriate.

Thus channelisation is the process whereby a vehicle is guided safely through the intersection area from an approach leg to the selected departure leg. Guidance is offered by lane markings that clearly define the required vehicle path and also indicate auxiliary lanes for turning movements. Various symbols are also used as road markings to indicate that turns, either to the left or to the right, from selected lanes are mandatory. At intersections that are complex or have high volumes of turning traffic, it is usually necessary to reinforce the guidance offered by road markings by the application of:

- Channelising islands;
- Medians and median end treatments;
- Corner radii;
- Approach and departure geometry;
- Pavement tapers and transitions;
- Traffic control devices including signs and signals; and
- Arrangement and position of lanes.

11.9.2 Channelising Islands

A traffic island is a defined area between traffic lanes for the control of vehicle movements and which may also be used as a pedestrian refuge. Traffic islands may take the form of an area delineated by barrier curbs or a pavement area marked by paint or a combination of these. Traffic islands may be included in the design of junctions for one or more of the following purposes:

i) Separation of conflicts.
ii) Control of angle of conflict.
iii) Reduction of excessive pavement areas.
iv) Regulation of traffic and indication of proper use of junction.
v) Arrangements to favour a predominant turning movement.
vi) Protection of pedestrians.
vii) Protection and storage of turning and crossing vehicles.
viii) Location of traffic signs.

Islands are either elongated or triangular in shape and are situated in areas not normally used as vehicle paths, the dimensions depending upon the particular junction or bus stop layout.

The layout of an island is determined by the edges of the through traffic lanes, turning vehicles and the lateral clearance to the island sides. Island curbs should be offset a minimum of 0.3 metres from the edge of through traffic lanes even if they are mountable.

Typical island shapes are illustrated in Figure 11.7. The designer should bear in mind that islands are hazards and should be less hazardous than whatever they are replacing.

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. 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 limited.

In rural areas, kerbs are not common and, at the speeds prevailing 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 may be preceded by a painted island.

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.

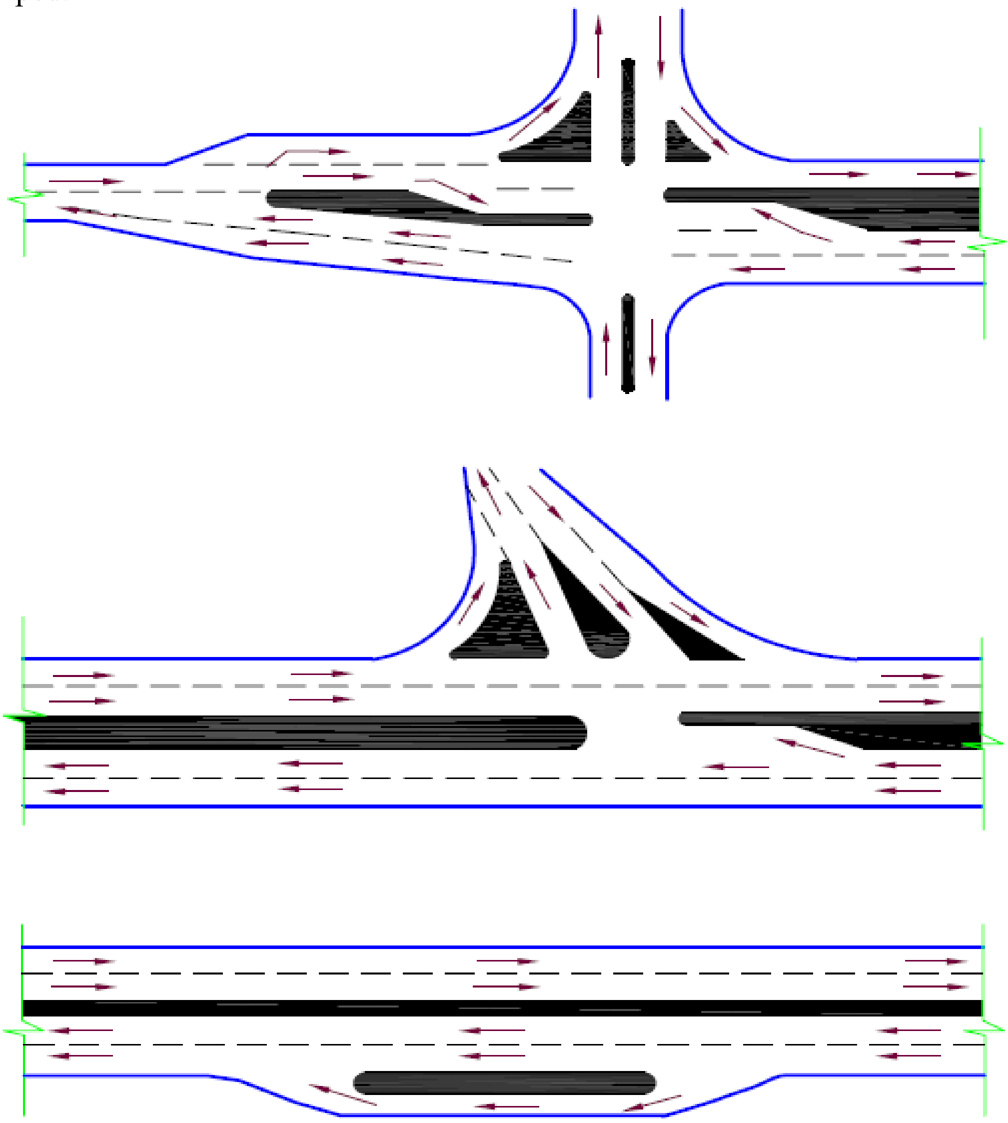


Figure 11-7: Traffic Islands

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.

Islands should not be less than 5 m^2 in area to ensure that they are easily visible to approaching drivers.

Islands are generally either long or triangular in shape, 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:

- The corner radii and associated tapers;
- The angle of skew of the intersection; and

- The turning path of the design vehicle.

A typical triangular island is illustrated in Figure 11.8. The approach ends of the island usually have a radius of about 0.6 metres and the offset between the island and the edge of the travelled way is typically 0.6 to 1.0 metres 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 one metre from the edge of the usable shoulder, the side adjacent to the through lane being 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 metres from the edge of the travelled way, with the side adjacent to the through lane being tapered back to terminate 0.6 metres from the edge of the through lane.

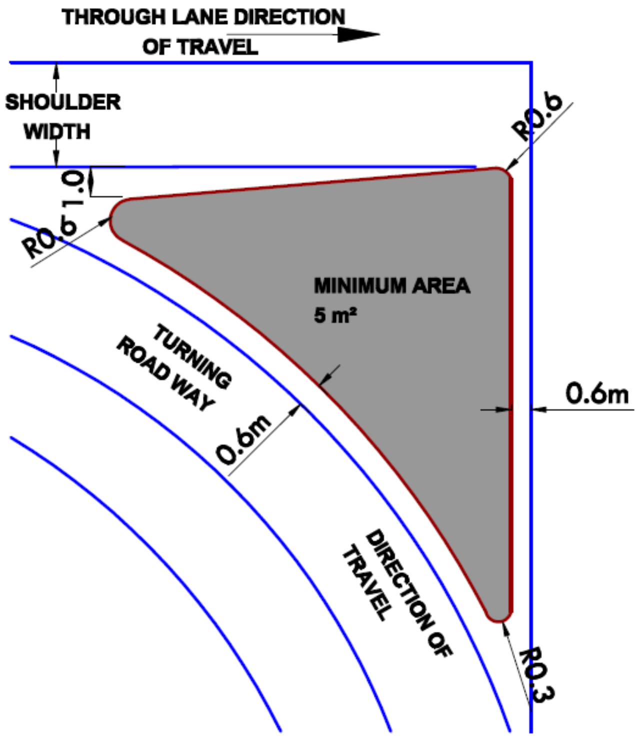


Figure 11-8: Typical Triangular Island

Dividing, or splitter, islands usually have a teardrop shape (Figure 11.9). 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. They are also often employed on the minor legs of an intersection where these legs have a two-lane, two-way or four-lane undivided cross-section.

The principle function of a dividing island is to warn the driver of the presence of the intersection. This can be achieved if, at the widest point of the island, its edge is in line with the edge of the approach leg. To the approaching driver, it appears as though the entire lane had been blocked off by the island. If space does not permit this width of island, a lesser blocking width must be applied, but anything less than half of the approach lane width is not effective.

Splitter islands are also used in the approach to roundabouts where there is a need to redirect vehicles entering a roundabout through an angle of not more than 30°.

Dividing islands are usually kerbed to ensure that the island is visible within normal stopping sight distance. However, it may be advisable to draw the driver's attention to the island by highlighting the kerbs with paint or reflective markings. As in the case of the triangular island, the nose of the dividing island should be offset by 0.6 m from the centreline of the minor road. For the sake of consistency, the radius of the nose should be of the order of 0.6 m.

The balance of the shape of the island is defined by the turning paths of vehicles turning both from the minor road to the major road and from the major road to the minor.

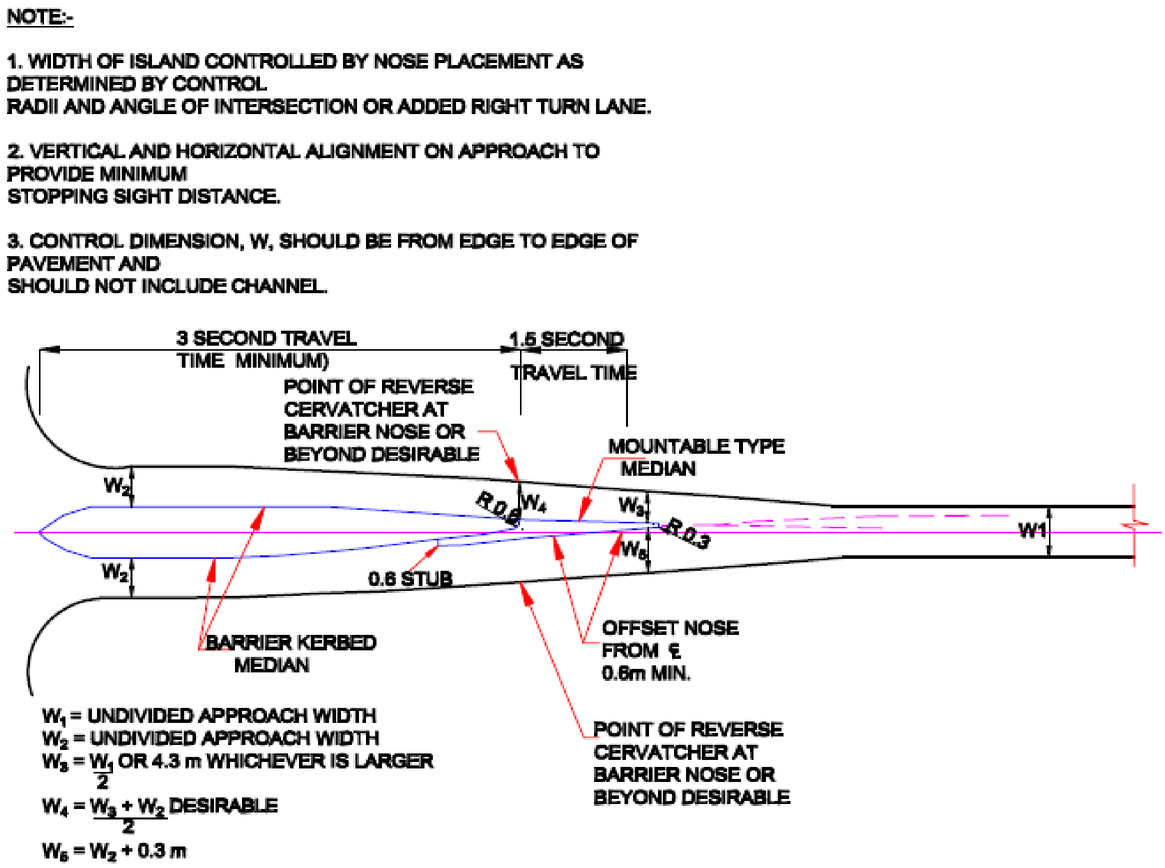


Figure 11-9: Splitter Island

11.9.3 Medians

Median islands are discussed in detail in Section 6.10. The general layout of median openings at intersections is normally dictated by wheel-track templates. However, median openings should not be shorter than:

- The surfaced width of the crossing road plus its shoulders.
- The surfaced width of the crossing road plus 2.5 m (if kerbing is provided).
- 12.4 metres.

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.

Above a median width of 5 m, 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 and parallel to the centreline of the minor road, is recommended. These end treatments are illustrated in Figure 11.10.

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 is better placed to serve as a refuge for pedestrians crossing the dual carriageway road.

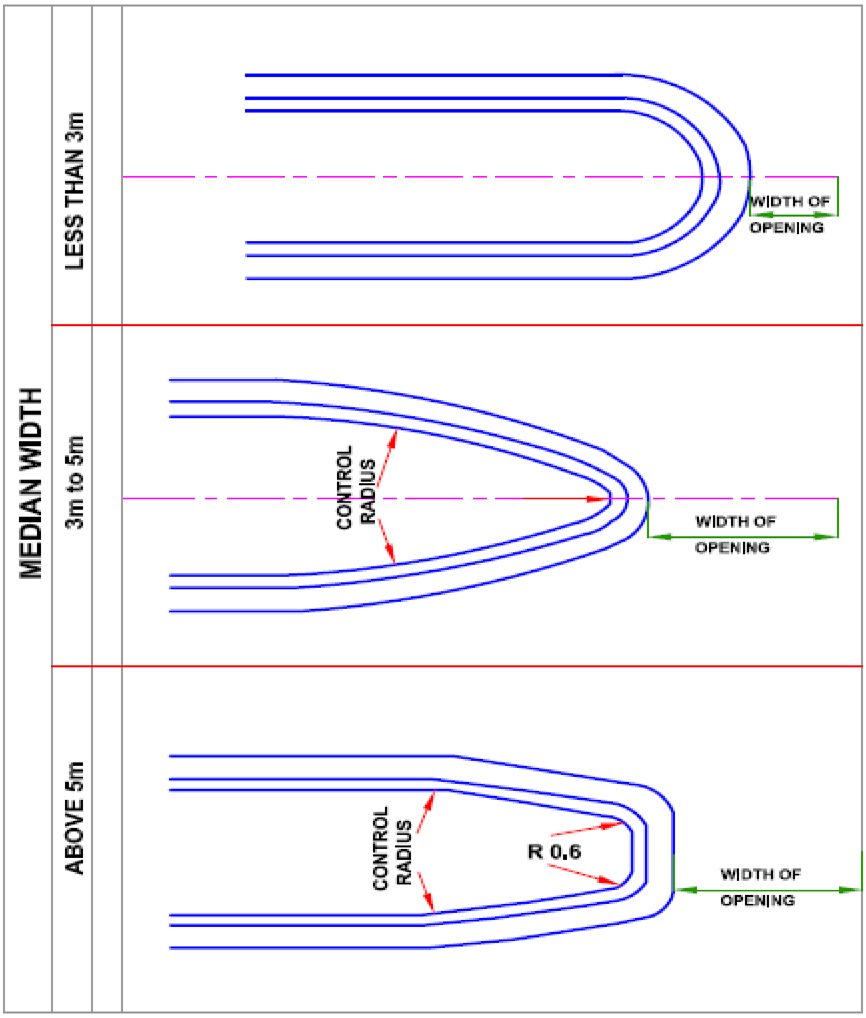


Figure 11-10: Median End Treatment

(Criteria for width of opening are described in the text)

11.10 Speed-Change Lanes

11.10.1 Purpose

Deceleration lanes for vehicles turning left or right from a major road are of particular value 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 desirable. These are incorporated into the Standard Detail Drawing for all junctions on trunk and link roads.

The length of such speed-change lanes are based on acceptable levels of discomfort for decelerating and for accelerating. These lengths are greater than stopping sight distances because the latter are concerned with emergency braking.

11.10.2 Right Turn Lane

Right turn lanes, comprising a taper section and deceleration lane, shall be provided for all trunk and link road DC8, DC7, and DC6 junctions, and for other road standards meeting any of the following conditions:

i) On four or more lane roads and divided highways.

ii) When the major road design speed is 100 km/hr or greater, and the present year AADT on the major road is greater than 1500 AADT.

iii) When the present year AADT of the right-turning traffic is greater than 750 AADT.

A detail of the layout for the Right Turn Lane is given in Figure 11.11. The length of the right turn lane including the taper, measured as shown in the Figure, is related to design speed as indicated in Table 11.4. The width of the major approach lane shall be the same as the width of the traffic lanes.

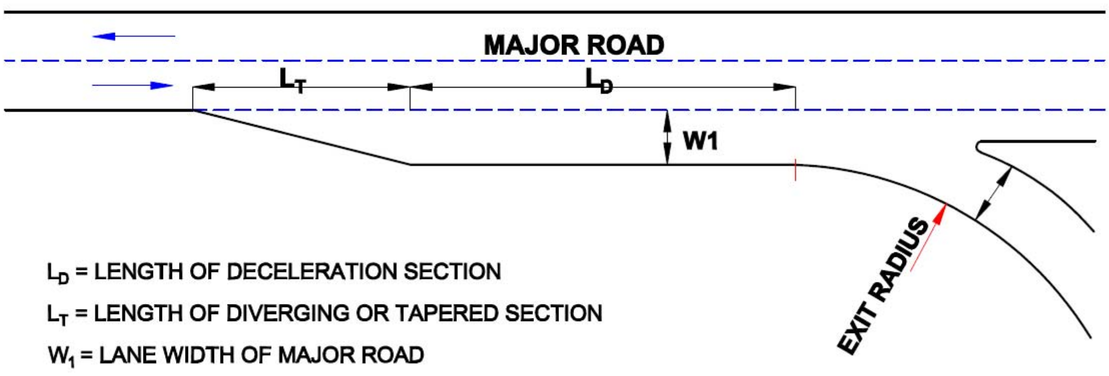


Figure 11-11: Layout for Right Turn Lane

Table 11-4: Length of Right Turn Lane

Main road design speed (km/hr)	Length of diverging section (LT) (m)	Length of deceleration section (LD)					
		Exit control speed (km/hr)					
		40	50	60	70	85	100
60	65	60	20	-	-	-	-
70	75	85	45	25	-	-	-
85	80	130	90	70	45	-	-
100	90	185	145	125	100	55	-
120	110	270	230	210	180	145	85

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 per cent for every 1 per cent change in grade. Thus, for example, for a down-hill grade of 4 per cent the length should be increased by 20 per cent.

11.10.3 Left Turn Lanes

Warrants are the same as for a right-turning lane. A separate lane for left turning traffic (traffic turning left from the major road into the minor road) shall be provided for all trunk and link road DC8, DC7, and DC6 junctions. Warrants for inclusion of left turn lanes for other road standards are under any of the following conditions:

i) On four or more lane roads and divided highways.
ii) When the major road design speed is 100 km/hr or greater, and the present year AADT on the major road is greater than 1500 AADT.
iii) When the present year AADT of the left-turning traffic is greater than 750 AADT.

A left turn lane consists of a taper section, a deceleration section and a storage section. The minimum lengths for taper sections are as for right turn lanes (Table 11.4). A detail of the layout for a Left Turn Lane for a single carriageway is given in Figure 11.12; the configuration for dual carriageways is shown in Figure 11.13. The length of the storage section is as indicated in Table 11.5.

Table 11-5: Lengths of Storage Sections for Left Turn Lanes

Left-Turning Traffic (AADT)	Length of Storage Section (L_S) (m)
0-1500	20
1500-3000	40
>3000	60

Provision of left turn lanes can be made for both the major and minor road. On single roadway roads where a left turn lane is to be provided, a painted central reserve shall always be used.

In order to accommodate a left turn lane on a single roadway road the roadway has to be widened to provide the required width. The widening shall be designed so that the through lanes are given smooth and optically pleasing alignments. The width of the through lanes at the junction shall be the same as the approach lanes.

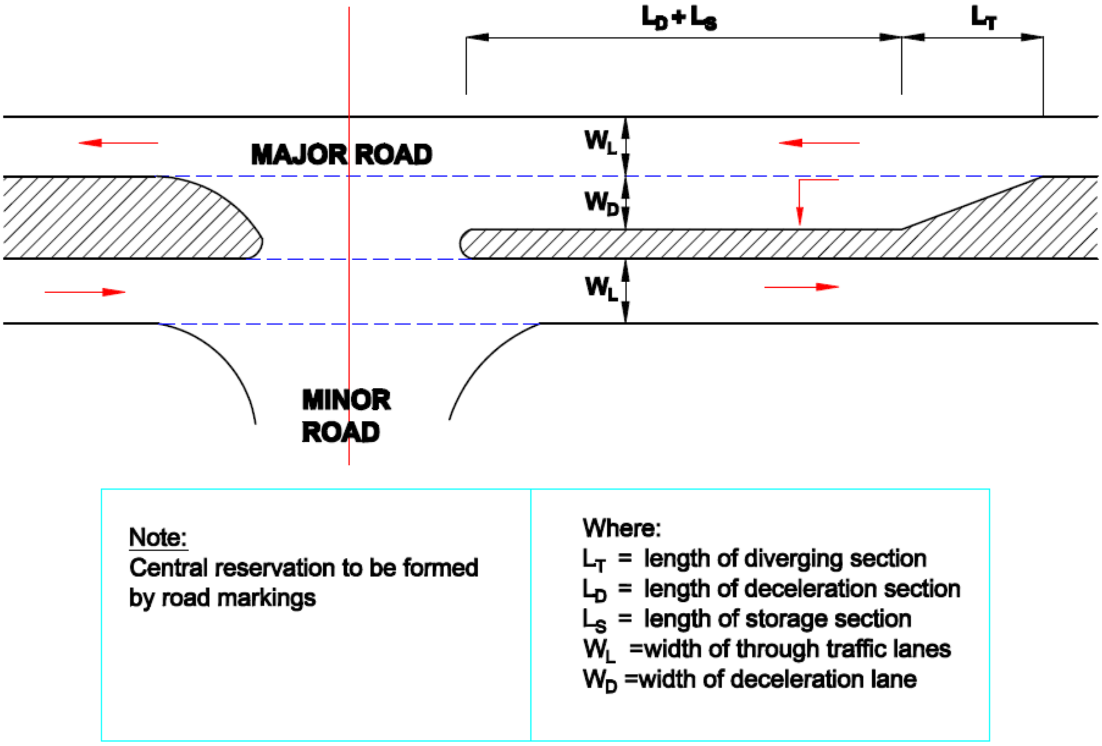


Figure 11-12: Layout for Left Turn Lane, Single Carriageway

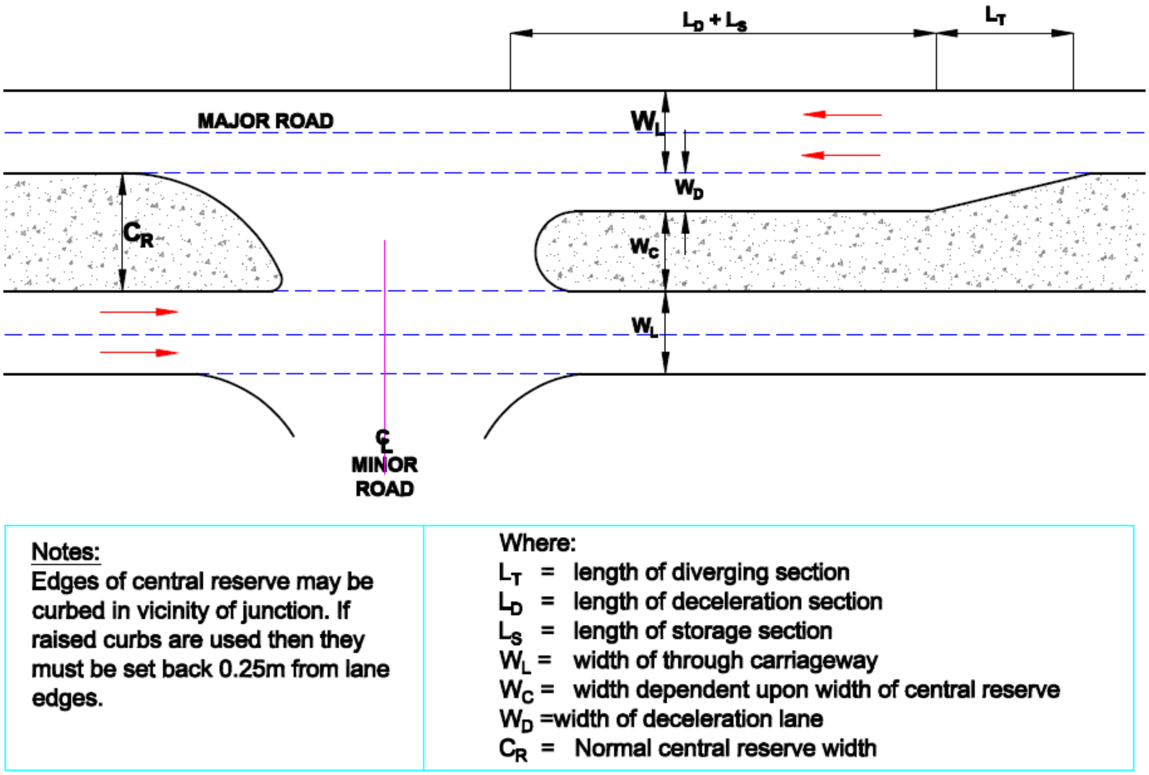


Figure 11-13: Layout for Left Turn Lane, Dual Carriageway

The widening shall be provided by the deviation of both through lanes from the centreline. This shall be achieved by introducing a taper of 100-metres length at the beginning and end of the widening.

The total length of an acceleration lane (i.e. not including the merging taper) is shown in Table 11.6.

Table 11-6: Length of Acceleration/Merging Lane

Main road design speed (km/hr)	Length of merging taper (m)	Length of Acceleration/Merging Lane					
		Entrance control speed (km/hr)					
		40	50	60	70	85	100
60	65	110	60	-	-	-	-
70	75	180	130	70	-	-	-
85	80	315	265	205	130	-	-
100	90	460	415	350	280	150	-
120	110	700	655	595	525	400	240

11.11 Turning Roadways

Turning 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 the passing of stalled vehicles.
- Case 2 One-lane one-way with provision for the passing of stalled vehicles.
- Case 3 Two-lane one-way operation.

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. It is reasonable to assume, even in the absence of traffic counts, that there will be enough trucks in the traffic stream to warrant consideration, and Condition B is normally adopted for design purposes. Widths of turning roadway for the various cases and conditions are given in Table 11.7. The radii in the Table refer to the inner edge of the pavement.

The simplest design is a semicircle which is adequate for medians of up to 3.0 m wide. For medians wider than 3.0 m, a bullet-nose end treatment is preferred. The bullet-nose is formed by two portions of control radius arc and an assumed small radius, e.g. 0.6 m. The bullet-nose closely follows the path of the inner rear wheel of the design vehicle and results in less intersection pavement and a shorter length of opening than the semi-circular end. For wider medians, a bullet-nose end requires shorter lengths of opening.

Above a width of 5.0 m the minimum lengths to provide for cross-traffic become the controlling factor. At this stage the bullet nose end should be replaced by a flattened bullet nose, the flat end being parallel to the centre line of the crossing road.

Table 11-7: Turning roadway widths

Inner Radius (m)	Case 1			Case 2			Case 3		
	Condition			Condition			Condition		
	A	B	C	A	B	C	A	B	C
15	4.0	5.5	7.9	6.1	8.8	13.4	7.9	10.7	15.2
20	4.0	5.2	6.7	5.8	8.2	11.0	7.6	10.1	12.8
30	4.0	4.9	6.4	5.8	7.6	10.4	7.6	9.4	12.2
40	3.7	4.9	6.4	5.5	7.3	8.8	7.3	9.1	10.7
60	3.7	4.9	5.2	5.5	7	8.2	7.3	8.8	10.1
80	3.7	4.6	5.2	5.5	6.7	7.6	7.3	8.6	9.4
100	3.7	4.6	4.9	5.2	6.7	7.3	7.0	8.4	9.1
150	3.7	4.3	4.6	5.2	6.7	7.3	7.0	8.2	9.1

Figure 11.14 shows the design of a typical cross road junction illustrating turning sections, channelization islands, deceleration lanes, tapers, medians and mountable kerbs.

11.12 Private Access

A private access is defined as the intersection of an unclassified road with a classified road. An access shall have entry and exit radii of 6 metres or greater, depending upon the turning characteristics of the expected traffic. The minimum width shall be 3m. A typical access is shown in Figure 11.15; dotted lines show the possible edge of the corresponding shoulders. The location of the access must satisfy the visibility requirement for ‘stop conditions’ described in Section 11.6. A drainage pipe shall be placed as required. The access shall be constructed back to the right-of-way line, with a taper to match the existing access.

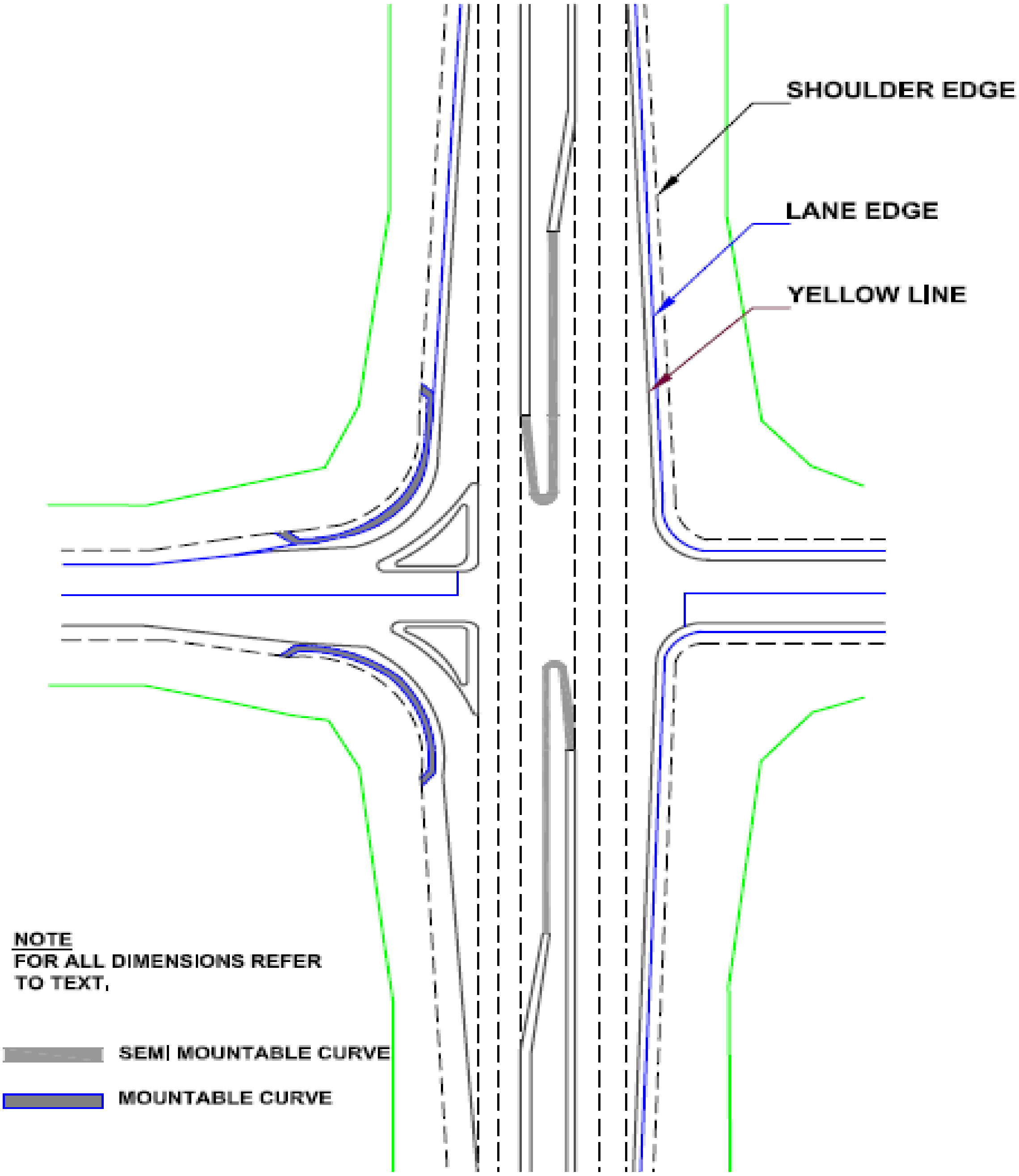


Figure 11-14: Typical Cross-road Intersection

(showing most features described in the text)

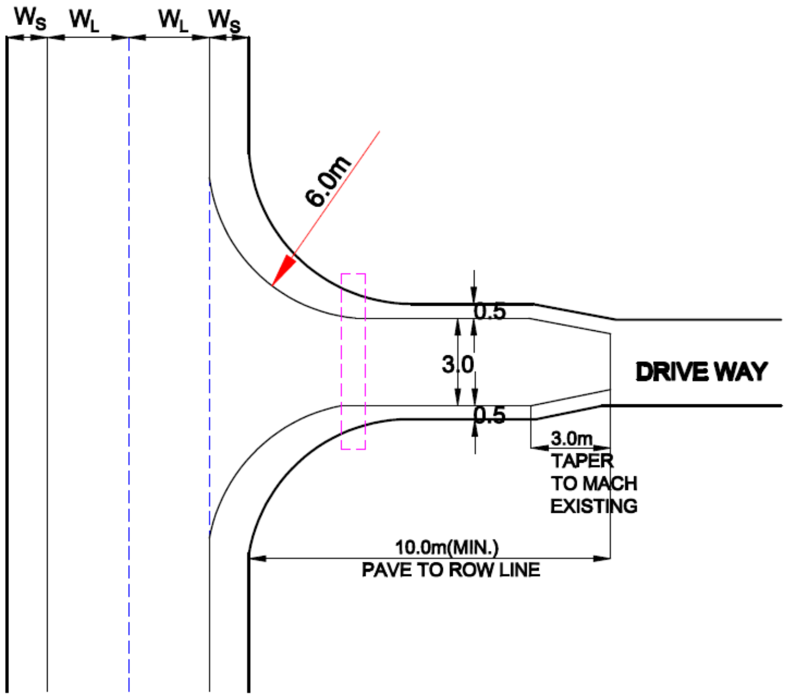


Figure 11-15: Typical Access

11.13 Roundabouts

11.13.1 Typical Features

A roundabout is a one-way circulatory system around a central island, entry to which is controlled by markings and signs. Priority is given to traffic already on the roundabout. Roundabouts thus operate by deflecting the vehicle paths to slow the traffic and promote yielding. Figure 11.16 illustrates the features of a typical roundabout.

Roundabouts provide high capacity and minimum delay. They also have a good safety record largely because traffic speeds are low and the number of potential traffic conflicts is greatly reduced, for example, from 32 at a cross-roads to just 8 at an equivalent four-legged roundabout.

Despite their advantages, roundabouts are not appropriate in every situation. They may be inappropriate:

a) Where spatial restraints (including cost of land), unfavourable topography or high construction costs make it impossible to provide an acceptable geometric design;

b) Where traffic flows are unbalanced, with high flows on one or more approaches leading to serious delays to traffic on the major road;

c) Where there are substantial pedestrian flows;

d) As an isolated intersection in a network of linked signalised intersections;

e) In the presence of reversible lanes;

f) 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; and

g) Where signalised traffic control situated down-stream of the roundabout could cause a queue to back-up through the roundabout.

Roundabouts should be considered when:

a) Intersection volumes do not exceed 3,000 vph (vehicles per hour) at three-legged or 4,000 vph at four-legged intersections;

b) The proportional split between the volumes on the major and minor road does not exceed 70/30 where, on three-legged intersections, the flow is less than 1,500 vph or, on four-legged intersections, is less than 2,000 vph;

c) The proportional split between the volumes on the major and minor road does not exceed 60/40 where; on three-legged intersections, the intersection flow is greater than 1,500 vph or; on four-legged intersections, is greater than 2,000 vph.

A typical roundabout is shown schematically in Figure 11.16.

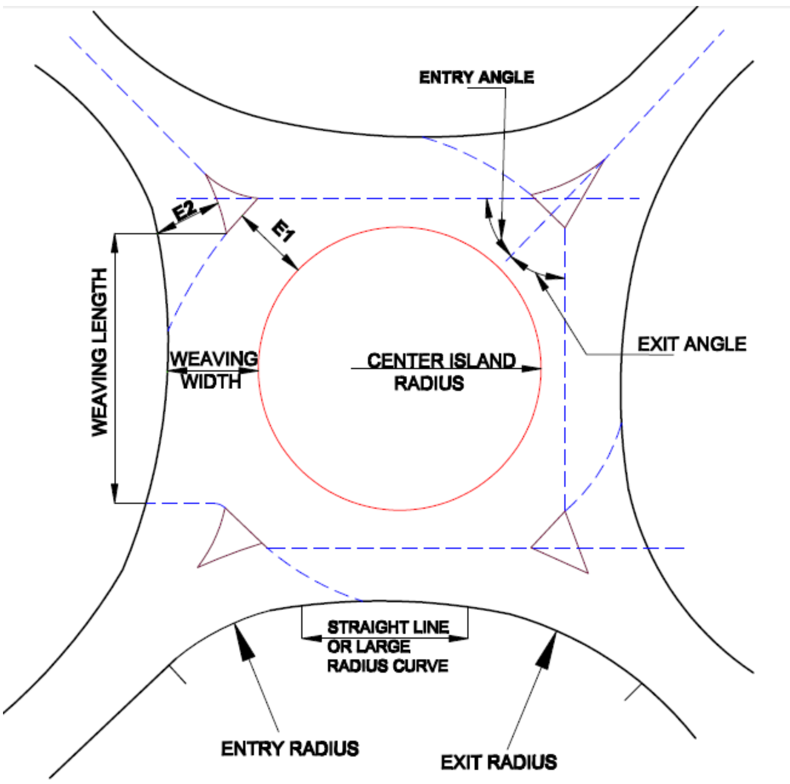


Figure 11-16: Roundabout Layout

11.13.2 Design Speed and Safety

The design speed within the roundabout should, ideally, be between 40 and 50 km/h. Unfortunately this suggests a radius of between 60 and 90 metres hence requiring an overall diameter of the roundabout of the order of 175 metres. Very often, the space for this size of intersection is not available and a lower design speed must be accepted.

Roundabouts should not be introduced directly on rural roads where the design speeds of adjacent sections are 90 km/h or greater. Where the design speeds on the approaches are high, i.e. more than 15 km/h faster than the design speed within the facility, it may be necessary to consider forcing a reduction in vehicle speed. This could be by means of horizontal reverse curvature. The ratio between the radii of successive curves should be of the order of 1.5:1.

Consideration should be given to the use of rumble strips and warning signs at the approaches to warn drivers to anticipate the roundabout. Speed humps should not be employed as speed-reducing devices on major roads or on bus routes and are usually unacceptable on approaches to major roundabouts. They are, however, sometimes acceptable in urban environments for lower flows and lower speeds.

Roundabouts are usually more difficult for pedestrians to cross than normal junctions hence arrangements should be made to provide adequate directions.

11.13.3 Sight distance

Approaching drivers should have a clear view of the 'nose' of the separator (or splitter) island. At the yield line and while traversing the roundabout, they should have an uninterrupted view of the opposing legs of the intersection at all times. This requirement suggests that the elaborate landscaping schemes sometimes placed on the central islands of roundabouts are totally inappropriate to the intended function of the layout.

Sight distance for intersections as described in Section 11.6 should be provided on each approach to the roundabout to ensure that drivers can see the nose of the splitter island. It follows that roundabouts should not be located on crest curves.

11.13.4 The General Layout

The various components of a roundabout are illustrated in Figure 11.16. The general layout of a roundabout should provide for the following:

i) Deflection of the traffic to the right on entry to promote movement and ensure low traffic speeds.
ii) Adequate entry widths.
iii) Suitable visibility at any entry of each adjacent entry.
iv) Adequate circulation space compatible with entry widths.
v) Central islands of diameter sufficient only to give drivers guidance on the manoeuvres expected.
vi) A simple and clear layout.
vii) Entry and exit deflection angles and central island radius should prevent through speeds in excess of 50 km/h. This is accomplished by maximising the difference between the shortest route a driver can 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 metres (see Figure 11.17).

Deflection

An important component is the deflection forced on vehicles on the approach to the roundabout. The intention is to reduce the speed of vehicles so that, within limits, the greater the deflection the better. The limit is the minimum acceptable angle of skew at an intersection of 60°. This corresponds to a deflection on entry of 30°. The approach radius should not exceed 100 metres, which corresponds to the recommended design speed of 40 to 50 km/h.

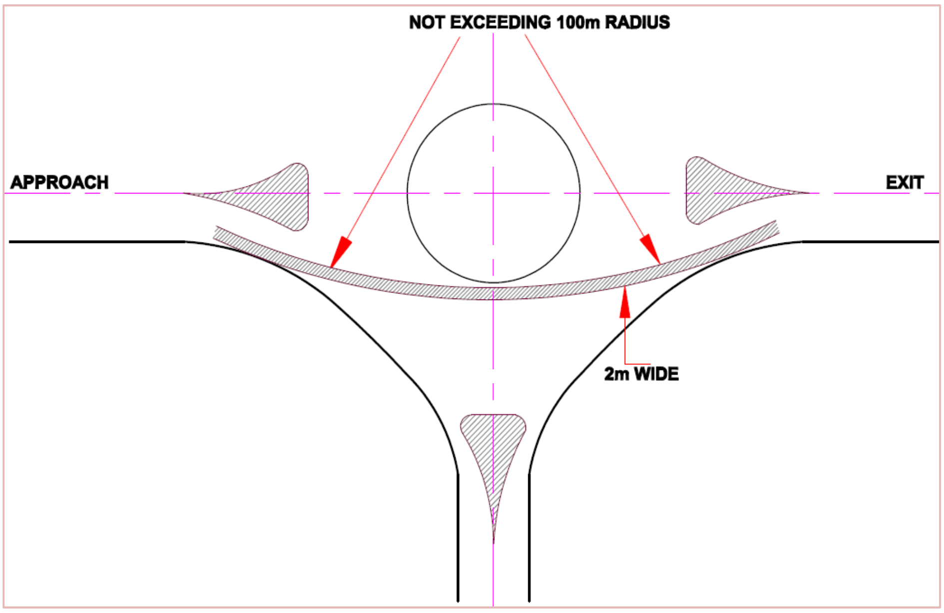


Figure 11-17: Vehicle Path through Roundabout

Entries and exits

The widths of single lane approaches to roundabouts are typically of the order of 3.4 to 3.7 metres. 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. The recommended minimum width for a single-lane entry is 5 metres. If demanded by high approach volumes, the flaring may add a full lane to the entry to increase capacity. The width of a two-lane entry should be of the order of 8 metres. A variation on the two-lane entry is to have a single-lane circulatory roadway with an auxiliary lane provided for the benefit of vehicles turning right at the roundabout. The auxiliary lane is shielded from traffic approaching from the left by moving the end of the splitter island forward to provide a circulatory road width adequate only for single-lane travel. This approach could be adopted with advantage when right-turning traffic represents 50 percent or more of the entry flow or more than 300 vph during peak hours.

Circulatory roadway

The circulatory roadway width is a function of the swept path of the design vehicle and of the layout of the exits and entries and generally should be either equal to, or slightly greater (1.2 times) than the width of the entries. The width should be constant throughout the circle. In the construction of the swept path of the design vehicle, it should be noted that drivers tend to position their vehicles close to the outside kerbs on entry to and exit from the roundabout and 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 metres, the offset should be not less than 1.6 metres, with 2.0 metres being preferred.

To ensure that vehicles do not travel faster than the design speed, the maximum radius on the vehicle path should be kept to 100 metres or less (Figure 11.17). As a general guideline, the circulatory roadway should be sufficiently wide to allow a stalled vehicle to be passed. The minimum roadway width for single-lane operation under these circumstances is of the order of 6.5 metres between kerbs. Two-lane operation requires a roadway width of about 9.0 metres. If trucks are present in the traffic stream in sufficient numbers to influence design, the circulatory road width should be increased by 3 metres both in the single-lane and in the two-lane situation. A significant proportion of semi-trailers requires the width of the circulatory road width to be increased to 13 metres and 16 metres in the single-lane and the two-lane situation respectively. A circulatory road width of 13 metres 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 below.

The cross-slope on the roadway should be away from the central island and equal to the camber on the approaches to the intersection.

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 sight lines across the island. 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 9.5m. 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 percent is recommended.

Splitter Islands

Splitter islands should be provided on the approaches to roundabouts to:

a) Allow drivers to perceive the upcoming roundabout and to reduce entry speed;
b) Provide space for a comfortable deceleration distance;
c) Physically separate entering and exiting traffic;
d) Prevent deliberate and highly dangerous wrong-way driving;
e) Control entry and exit deflections; and
f) Provide a refuge for pedestrians and cyclists and 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 square metres so as to ensure their visibility to the oncoming driver. 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 to 1 metre.

The roundabout depicted in Figure 11.18 and in the Standard Detail Drawings is acceptable for traffic volumes of up to 15,000 (based on empirical evidence rather than gap-acceptance theory).

The following steps may be followed in laying out trial geometry for a roundabout:

1. Select the general design criteria to be used.

2. Select the appropriate design vehicle for the site. This will generally be the DV4 for all design standards.
3. Adopt a minimum design vehicle turning radius. This will generally be 15m radius. Check the design using the template in Chapter 5.
4. Determine from traffic flows the number of lanes required on entry, exit and circulation.
5. Identify the needs of pedestrians.
6. Identify the location of controls such as right-of-way boundaries, utilities, access requirements, and establish the space available.
7. Select a trial central island diameter and determine the width needed of the circulating carriageway.
8. Draw the roundabout.
9. Check that the size and shape is adequate to accommodate all intersecting legs with sufficient separations for satisfactory traffic operations.
10. Lay out the entrance/exit islands.
11. Check the achievement of adequate deflection. Adjust as required.
12. Check site distances at approaches and exits.
13. Layout lane and pavement markings.
14. Layout lighting plan
15. Layout sign plan.

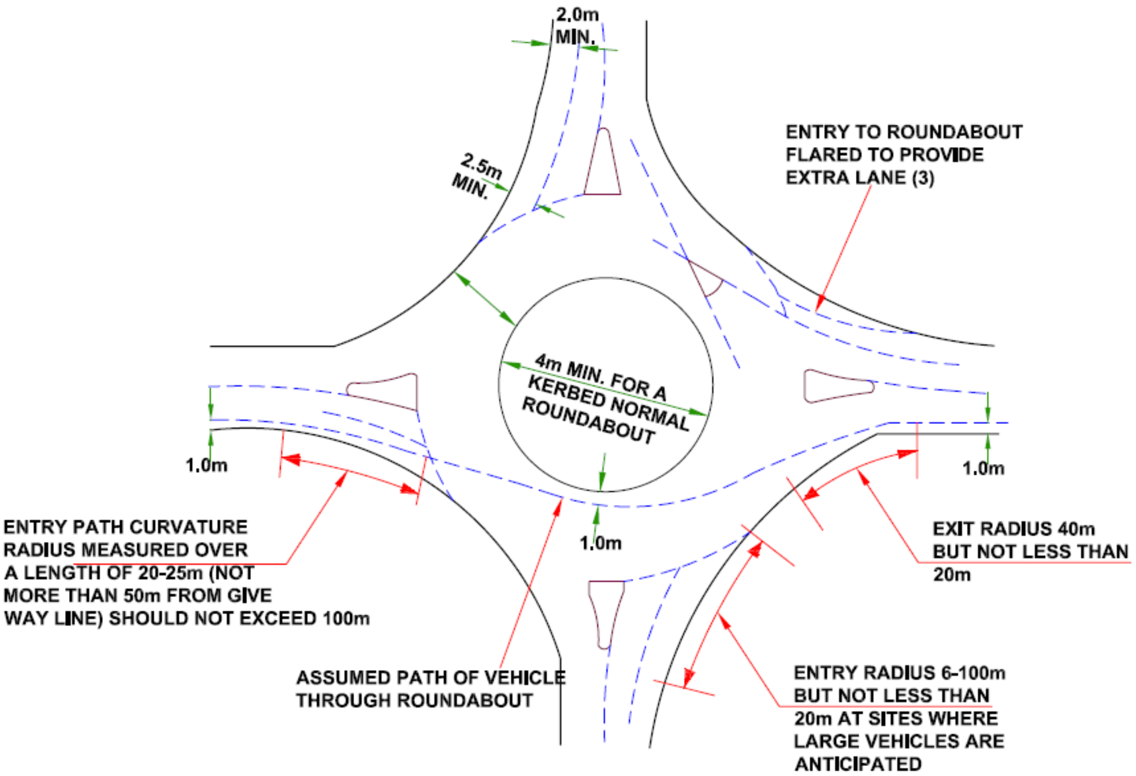


Figure 11-18: Roundabout Dimensions

11.14 Checklist for Junction Design

The following is a checklist of factors that need to be considered in the design of junctions.

- ❑ Will the junction be able to carry the expected/future traffic levels without becoming overloaded and congested?
- ❑ Have the traffic and safety performance of alternative junction designs been considered?
- ❑ Is the route through the junction as simple and clear to all users as possible?
- ❑ Is the presence of the junction clearly evident at a safe distance to approaching vehicles for all directions?
- ❑ Are warning and information signs placed sufficiently in advance of the junction for a driver to take appropriate and safe action given the design speeds on the road?
- ❑ On the approach to the junction, is the driver clearly aware of the actions necessary to negotiate the junction safely?
- ❑ Are turning movements segregated as required for the design standard?
- ❑ Are drainage features sufficient to avoid the presence of standing water?
- ❑ Is the level of lighting adequate for the junction, location, pedestrians, and the design standard?
- ❑ Are the warning signs and markings sufficient, particularly at night?
- ❑ Have the needs of pedestrian and noon-motorized vehicles been met?
- ❑ Are sight lines sufficient and clear of obstructions including parked and stopped vehicles?
- ❑ Are accesses prohibited a safe distance away from the junction?
- ❑ Have adequate facilities such as footpaths, refuges, and crossings, been provided for pedestrians?
- ❑ Do the design, road marking and signing clearly identify rights of way and priorities?
- ❑ Is the design of the junction consistent with road types and adjacent junctions?
- ❑ Are the turning lanes and tapers where required of sufficient length for speeds and storage?

Date: .... Designer...

Date: .... Responsible Engineer............................

Figure 11-19: Checklist for Junction Design

12 GRADE – SEPARATED JUNCTIONS AND INTERCHANGES

12.1 General

Grade-separated interchanges are usually associated with freeways. The fundamental difference between a freeway and any other road is that it is subject to rigid control of access. Entrance to and exit from a freeway may take place only at specified points, typically remote from each other, and then only at very flat angles of merging and diverging. As such, the freeway is characterised by the fact that all intersections along its length are interchanges.

However, at any road junction, the flow of traffic, expressed in terms of a level of service, can be enhanced by an interchange. Where two main roads intersect, it may be found that traffic volumes are too high to be accommodated at an at-grade intersection, regardless of the level of sophistication of the provision made for turning movements by means of channelization, signalization and auxiliary lanes for through traffic. Generally, if an intersection is likely to become a bottleneck, and all possibilities for improving its capacity have been exhausted, an interchange is warranted. In the planning of a new road under circumstances where close spacing of heavily trafficked intersections is anticipated, it may be necessary to consider the provision of interchanges at points where the design level of service cannot be achieved with at-grade intersections.

Interchanges are divided into two functional classes, referred to as Access (or Minor or Service) interchanges, and Systems (or Major) interchanges. Access interchanges serve local areas by providing access to freeways whereas systems interchanges are the nodes of the freeway network, linking the individual freeways into a cohesive unit. These two fundamentally different applications require different types of interchange layout.

An Access interchange provides free flow conditions for the main freeway with suitable ramps to ensure that entering and exiting traffic do not affect traffic speeds on the freeway. However the associated intersections with the minor road (or roads) are at-grade and include the normal yield, stop and traffic controlled options described in Chapter 11.

In contrast, Systems (or Major) interchanges provide free flow conditions for more than one freeway and ensure unhindered travel for all vehicles on them. Such interchanges are very much more complex to design and build.

Thus the use of grade separation results in the separation of traffic movements between the intersecting roads so that only merging or diverging movements remain. The extent to which individual traffic movements should be separated from each other depends mainly upon capacity requirements and traffic safety aspects; it also depends upon the extent to which important traffic movements should be given free flow conditions.

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

i) An at-grade junction has insufficient capacity.
ii) The junction is justified economically from the savings in traffic delays and accident costs.

iii) Grade separation is cheaper on account of topography or on the grounds that expensive land appropriation can be avoided by its construction.
iv) For operational reasons.
v) Where roads cross freeways/motorways.

12.2 Choice of Scheme

In deciding on the location of a grade-separated junction, the following factors should be taken into account:

i) Predicted traffic volumes.
ii) Cost of junction.
iii) Congestion control.
iv) Trip length (travel distance).
v) Size of urban areas.

From a study of conflicting traffic movements, it will generally be apparent which traffic streams must be grade separated, leaving the other streams to be dealt with by junctions 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 a number of alternative designs should be prepared. The final choice of scheme must satisfy capacity requirements, geometric standards, and operational needs, and represent an economical design. In some instances the choice of a particular design will be determined by the adoption of two-stage construction, e.g. constructing an at-grade junction first and providing grade separation later.

12.3 Types of Junction

12.3.1 Common Types

Figure 12.1 illustrates the common types of interchange. Grade-separated junctions 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.

Each category is discussed briefly below with reference, where appropriate, to the basic line diagram layouts shown in Figure 12-1.

12.3.2 Three-Way Junctions (Layouts A and B)

For some Y-junctions where grade separation of only one traffic stream is required, layout A may be appropriate. The movements associated with the missing leg must be channelled to another location and this is only appropriate if the traffic volumes on the missing leg are low and capable of being served by an at-grade junction elsewhere.

Layout B shows a typical three-leg junction. It is appropriate for traffic where the major road is DC8 and the minor road is DC7 - DC3. 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. Higher loop traffic requires multiple loop lanes.

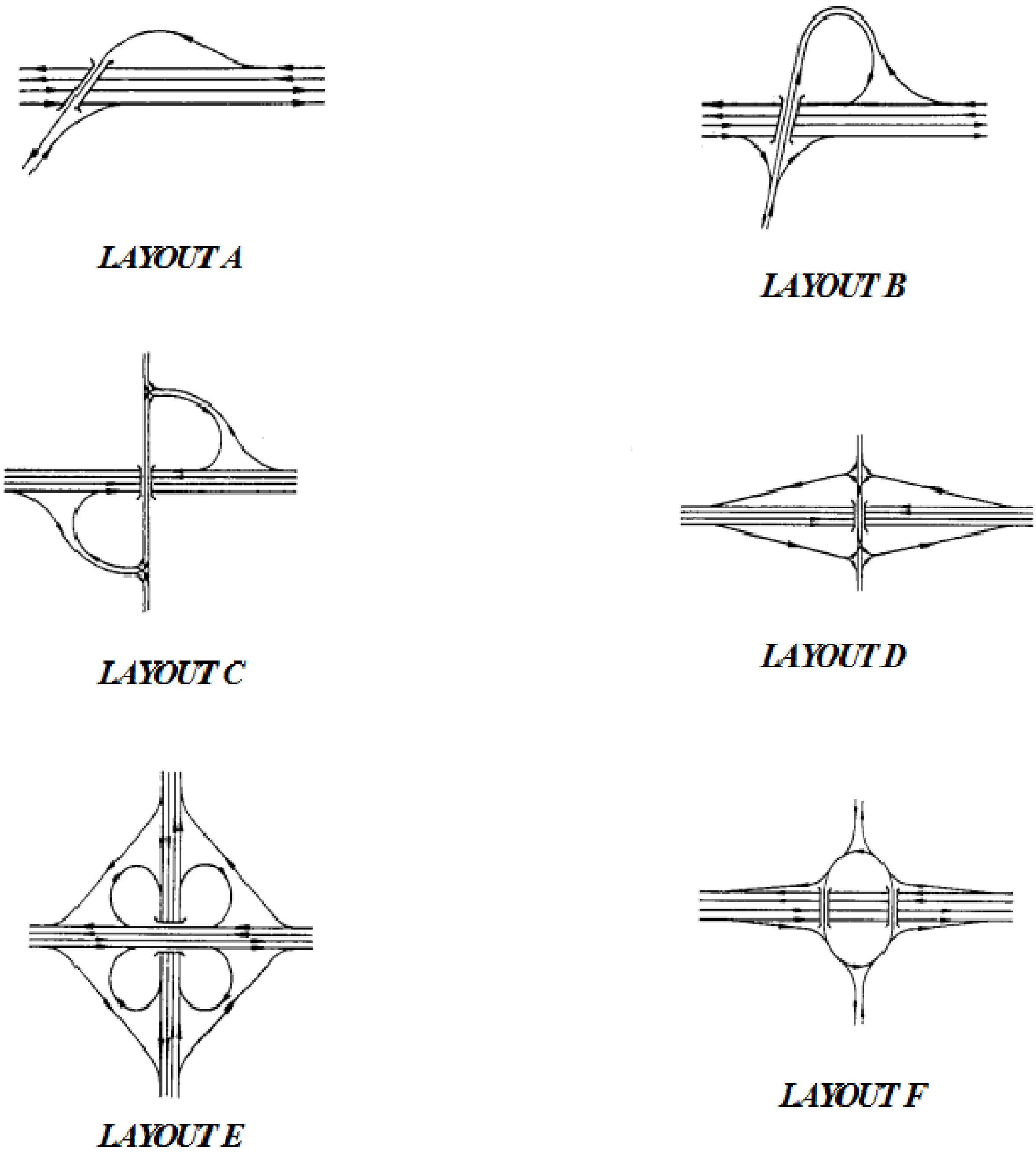


Figure 12-1: Typical Layouts for Grade-Separated Junctions

12.3.3 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 junction, which has the advantage of being easily adapted to meet difficult site conditions.

Layout D shows the normal 'diamond' junction, which requires the least land appropriation. The choice between these options is generally dependent on land requirements.

These configurations are appropriate for traffic volumes of up to 30,000 AADT on the four-lane major road (3,000 vehicles per hour), with traffic of up to 10,000 ADT on the minor road. They are appropriate for traffic where the major road is DC8 and the minor road is DC7 - DC3. With a single loop lane, it is appropriate for loop traffic of 1,000 vehicles per hour. Higher loop traffic would require multiple loop lanes.

12.3.4 Junctions of Two Major Roads (Layouts E and F)

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' junction involving only one bridge but requiring a large land appropriation. Layout F shows a typical roundabout interchange involving two bridges. This layout 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.

12.3.5 Junctions of more than Two Major Roads

Junctions of more than two main roads are difficult to design, occupy large areas of land and, requiring numerous bridges, are extremely expensive. This type of junction, although unlikely to be required on rural roads in Ethiopia, can often be reduced by changes in the major road alignments (which will simplify the traffic pattern) to a combination of the simpler and economic layouts described above.

12.4 Geometric Standards

12.4.1 Design Speed

The geometric standards given in this manual for roads and at-grade junctions also apply to grade separated junctions. However, the low design speeds of loops and other ancillary roads necessitate further standards to be given.

The design speed for the through traffic movements is determined in accordance with Chapter 5. Stopping sight distances appropriate for the design speed should always be provided.

The design speed for loops and ramps is dependent 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/hr. Higher design speeds require higher radii of curvature and longer loops and therefore have a significant cost implication. However the design speed should not be so low that it is requires drivers who are leaving the freeway to reduce speed too quickly hence either compound curves are required suitable for an entry speed of 65% of the design speed of the freeway or a deceleration lane must be provided on the freeway.

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. Deceleration and acceleration lanes must also be provided on the freeway (Section 11.10).

Where a dual carriageway intersects with another dual carriageway (Major Interchange), the interchange between the facilities must be designed so that the loop roads do not entail any significant reduction in the design speeds of the crossing carriageways.

12.4.2 Acceleration and Deceleration Lanes

The minimum standards to be applied for right turn deceleration lanes are the same as for at-grade junctions (Section 11.10). The total length of the acceleration lane (i.e. not including the merging taper) shall never be less than 150 metres or more than 400 metres.

12.4.3 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 e = 8% which, at a design speed of 50 km/h, leads to a minimum radius of 80 metres. 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.

12.4.4 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 9 to provide safe stopping sight distances.

12.4.5 Widths and Gradients of Loops

The minimum carriageway width for loops on straight sections and horizontal radii greater than 150m shall be 4.0m with shoulders of 1.5 metres on the near side and 1.0 metre on the far side (widened by 0.5 metre where a guardrail is required). For loops on radii of 150 metres or less, the carriageway width shall be in accordance with Table 12.1.

The maximum up gradient should be 5% and the maximum down gradient should be 7%.

Table 12-1: Widths for Loops

Radius (metres)	25	30	40	50	75	100	150
Carriageway Width (m)	5.3	5.0	4.6	4.5	4.5	4.5	4.0

12.4.6 Clearances

The required vertical and horizontal clearances shall be in accordance with Chapters 6 & 9.

12.4.7 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. The capacity of each traffic lane, in DHV, is normally about 1000 vehicles per hour.

Thus, for example, Table 2.1 indicates a design traffic flow of 10,000 to 15,000 AADT for Design Class DC8. The expected DHV is therefore 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.

These DHV values are necessary in choosing the number of lanes for the loops corresponding to the junction.

12.4.8 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 and requirements for lane balance. Alternatively stated, the basic number of lanes is a constant number of lanes assigned to a route, exclusive of auxiliary lanes.

The number of basic lanes changes only when there is a significant change in the general level of traffic volumes on the route. Short sections of the route may thus have insufficient capacity. This problem can be overcome by the use of auxiliary lanes. In the case of spare capacity, reduction in the number of lanes is not recommended because this area could, at some future time, become a bottleneck. Unusual traffic demands, created by accidents, maintenance or special events, could also result in these areas becoming bottlenecks.

The basic number of lanes is derived from consideration of the design traffic volumes and capacity analyses. To promote the smooth flow of traffic there should be a proper balance of lanes at points where merging or diverging manoeuvres occur. In essence, there should be one lane where the driver has the choice of a change of direction without the need to change lanes.

At merges, the number of lanes downstream of the merge should be one less than the 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 freeway, the above rule indicates that the number of freeway lanes beyond the diverge is reduced by one. This can be used to drop a basic lane to match anticipated flows beyond the diverge. Alternatively, it can be an auxiliary lane that is dropped.

Basic lanes and lane balance are brought into harmony with each other by building on the basic lanes, adding or removing auxiliary lanes as required. The principle of lane balance should always be applied in the use of auxiliary lanes. Operational problems on existing roadways can be directly attributed to a lack of lane balance and failure to maintain route continuity. The application of lane balance and coordination with basic number of lanes is illustrated in Figure 12.2.

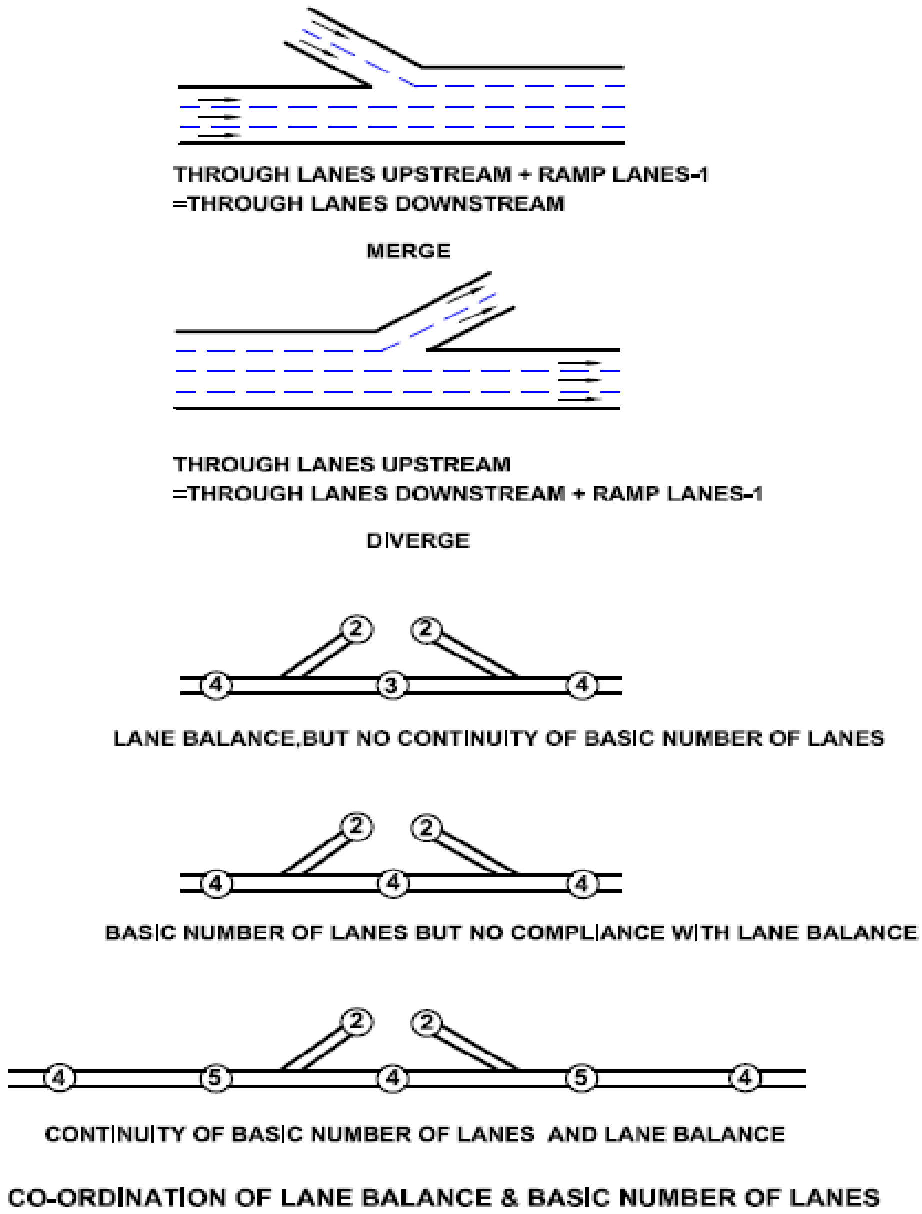


Figure 12-2: Principles of Lane Balance

12.4.9 Minimum Spacing of Successive Interchanges

The distance between two successive grade-separated junctions is an element of great importance in ensuring the desired level of service. In suburban zones it is necessary to establish a minimum distance between successive grade-separated junctions. The recommended minimum distance is 2.0km.

12.5 Design Principles

Special design principles apply to grade-separated junctions and must be considered when comparing the characteristics of alternative designs. The main principles and described below:

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 a large number of grade-separated junctions, a consistent design speed is desirable for loops. This speed shall be not less than 65% of the speed of the adjoining major road.
4. As a general rule, left-turning movements that are grade separated should be made through a right-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 loops, entry contrary to the one way movement can be restricted by the use of suitably shaped traffic islands to supplement the traffic signs.

13 ADDITIONAL SAFETY AND MISCELLANEOUS DESIGN ITEMS

13.1 Introduction

Road user safety has economic consequences in terms of property damage, loss of earnings or production and hospital costs resulting from physical injury, in addition to the emotional consequences of pain, suffering and death. Safety and economy are the foundations on which competent design rests. Inadequate consideration of either will automatically result in inadequate design.

Miscellaneous design items in this chapter also include bus lay-bys and parking bays, parking lanes, safety barriers, emergency escape ramps, safety rest areas and scenic overlooks, public utilities, and railway grade crossings.

13.2 The Road Accident Situation

The road accident statistics in Ethiopia, in common with many other countries, show that death rates from road accidents are 30 to 50 times higher than in the countries of Western Europe. The numbers of serious injuries resulting from road accidents are equally alarming. Economic analysis has shown conclusively that this high level of road accidents has economic consequences for the country that is equivalent to a reduction of 2-3% of GDP. This is a very significant drain on the economy. Furthermore, the consequences of the road accidents 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 accidents.

The geometric design of roads has an important part to play in this endeavour and road safety aspects have been highlighted throughout this manual. Road and shoulder widths have been increased to accommodate pedestrians, NMTs, and intermediate forms of transport (IMTs); moderate design speeds have been used for elements of road alignment; parking places and lay-bys for buses have been included in populated areas; account has been taken of reduced friction on unpaved roads; adequate sight distances have been provided; and much more.

However there are a number of other steps that could be taken to improve safety. These include:

i) Traffic calming measures to reduce speeds in populated area;
ii) Road markings, signage and lighting;
iii) Segregating pedestrians and motorised vehicles in populated areas;
iv) Providing crash barriers at dangerous locations;
v) Providing a professional safety audit at the design stage.

A checklist of engineering design features that affect road safety is given in Figure 13.1. Many of these have been discussed in the appropriate chapters of this manual but for more detail the TRL publication *'Towards Safer Roads'* is recommended.

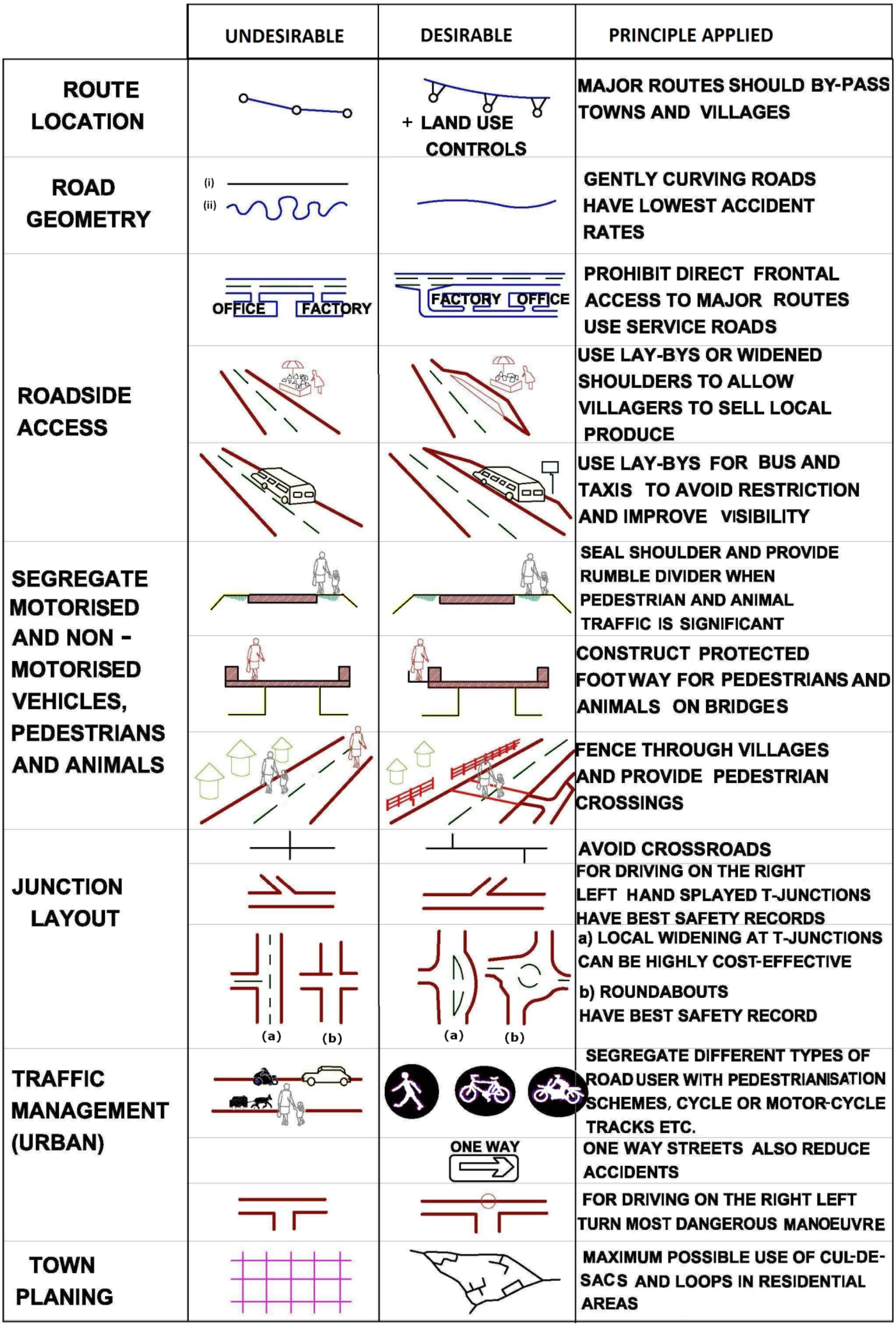


Figure 13-1: Engineering Design Features Affecting Road Safety

13.3 Traffic Calming

13.3.1 Purpose

The seriousness of road accidents increases dramatically with speed and hence very significant improvements to road safety are possible if traffic can be slowed down. This process is called traffic calming. All such methods have their 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 needs to be carried out in Ethiopia to identify the most cost effective approaches.

The effect of any traffic calming measure on all the road users should be carefully considered before they are installed. Some are unsuitable if large buses are part of the traffic stream; some are very harsh on bicycles, motorcycles and motor cycle taxis; and some are totally unsuitable when there is any animal drawn transport.

The three most common methods are:

i) Chicanes;
ii) Rumble strips; and
iii) Speed reduction humps.

13.3.2 Chicanes

These are designed to produce artificial congestion by reducing the width of the road to one lane for a very short distance (3-5m) at intervals (typically 300m) 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 is causes will be severe. For safety, they must be illuminated at night.

13.3.3 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, where 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 4m but at intervals along the road of typically 50 to 200 metres. They are uncomfortable to drive across at speed hence they are effective in slowing down the traffic. They do not need to be illuminated at night.

13.3.4 Speed Reduction Humps and Cushions

These are probably the most familiar measures used to slow traffic. They are essentially bumps in the road extending uniformly from one side to the other. Unlike rumble strips, speed reduction humps are quite high and, if they are designed badly, they can cause considerable vehicle damage. They are often used in villages where they are placed at intervals of between 50m and 200m. They are effective but usually unpopular with drivers.

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. The height of the bump is usually 75 or 100 mm but the width should be at least 2.5 metres and the change in slope from the roadway onto the hump should be gradual. The top of the hump can be rounded or flat.

Speed humps should not be employed as speed-reducing devices on major roads or on bus routes. Where design speeds are of the order of 100 km/h or more, the speed hump would have to be long and the height low to ensure that the vertical acceleration caused by the speed hump does not cause the driver to lose control.

Based on a similar principle to the speed hump, speed reducing cushions are more versatile. They are essentially very similar to the speed hump but the hump is not continuous across the road. The width of a two lane road is usually covered by two or three cushions with considerable gaps between them. The idea is that Large Heavy Vehicles will not be able to pass without at least one wheel running over one of the humps but bicycles and motorcycles can pass between them without interference. If suitably designed, the wheels of animal drawn carts can also avoid the humps.

13.4 Bus Lay-Bys and Parking Bays

Rural bus lay-bys serve to remove buses from the traffic lanes. Parking bays are provided for taxis and other vehicles to stop outside of the roadway. The location and design of lay-bys should provide ready access in the safest and most efficient manner possible. Providing lay-bys clear of the lanes for through traffic can considerably reduce the interference between buses, taxis and other traffic.

To be fully effective, lay-bys should incorporate;

i) A deceleration lane or taper to permit easy entrance to the loading area.
ii) A standing space sufficiently long to accommodate the maximum number of vehicles expected to occupy the space at one time.
iii) 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. A taper of 5:1, longitudinal to transverse, is a desirable minimum.

A loading area should provide 15 metres of length for each bus. The width should be at least 3.5 metres and preferably 4.0 metres. The merging or re-entry taper may be somewhat more abrupt than the deceleration taper but, preferably, should not be sharper than 3:1.

The total length of lay-bys for a two-bus loading area of minimum design should be as shown in Figure 13.2 and in the Standard Detail Drawings. These lengths of lay-bys expedite bus manoeuvres, encourage full compliance on the part of bus and taxi drivers, and lessen interference with through traffic. Sufficient footpaths should be provided at bus lay-bys.

The standard detail drawings show a bus turnout at a mid-block location. They also depict a parking bay (see also Figure 13.2).

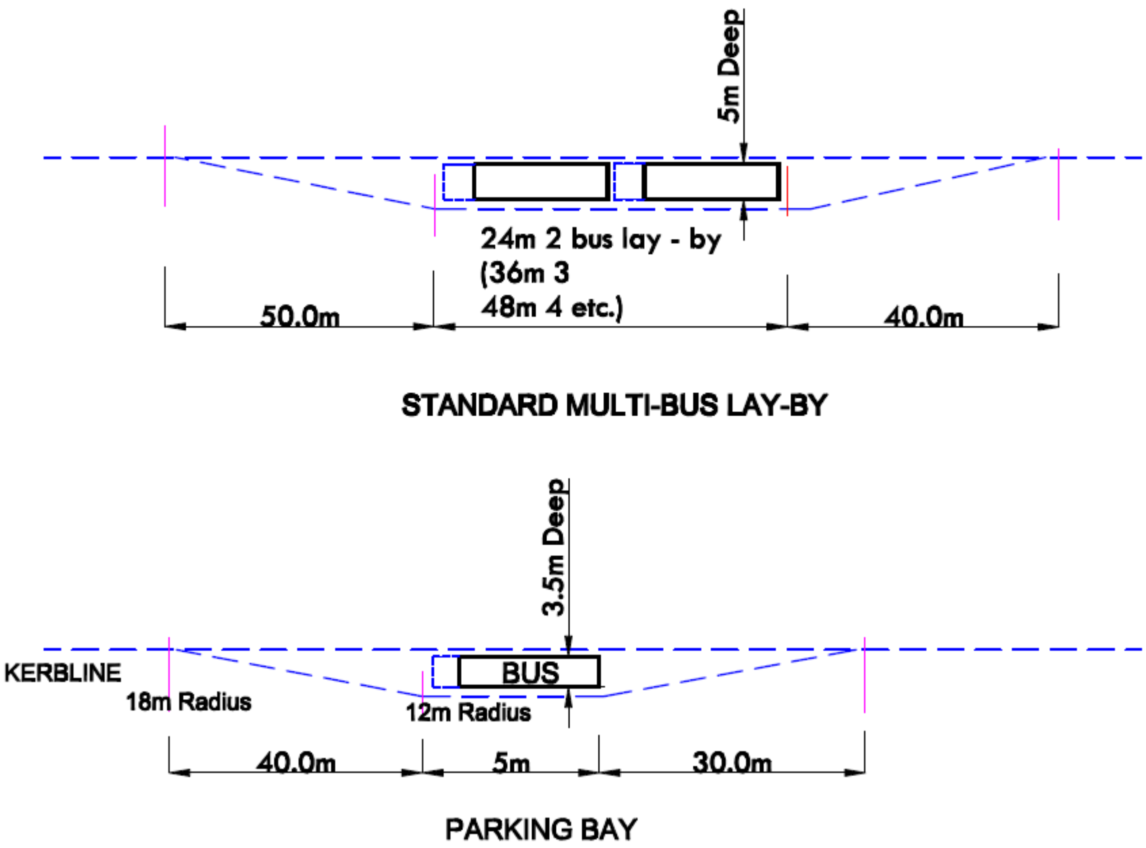


Figure 13-2: Bus Lay-Bys and Parking Bays

Locating bus lay-bys and parking bays on the near side of junctions is to be discouraged. Where possible, bus turnouts should be positioned subsequent to junctions. This location minimizes congestion and delays at the junction. It is preferable that they are located at least 75m past an intersection.

13.5 Parking Lanes

Parking lanes differ from parking bays in that they allow for parking of vehicles rather than solely buses and taxis. They should be provided at all congested business and shopping areas. The parking lane width for parallel parking is 3m, which may be reduced to 2.5m where available space is limited. Where additional parking capacity is desired and sufficient carriageway width is available, angle parking may be adopted.

13.6 Passing Points

Single lane roads do not allow most vehicles to pass in opposite directions or to overtake, hence passing places have to be provided. A clear distinction should be drawn between passing points and lay-bys. Lay-bys are provided for specific purposes, such as parking or bus stops. A passing point is a short length of widened road with a taper at each end. It appears similar to an elongated bus stop or a lay-by. The structure and surface of the passing point is the same as the main carriageway. The increased width provided by a passing place should allow two vehicles to pass at slow speed and hence depends on the design vehicle.

The most important feature of passing points is the frequency at which they are constructed along the road. The frequency depends upon the following factors:

a) Meeting sight distances
b) Traffic volume and mix
c) Acceptable reversing distance for vehicles
d) Terrain
e) Strength of surrounding ground.

Meeting sight distance refers to the length of road where drivers in vehicles travelling towards each other begin to see each other. Ideally, at least one passing point should be placed within each sight distance and the next passing point should always be visible from the one before.

In hilly terrain the spacing of passing points must be more flexible and responsive to both sight distance and the constraints of the surrounding landscape. As a general rule the optimal spacing should be equal to the sight distance, up to a maximum of 500 metres. An alternate approach is to locate passing places at regular intervals (say every 500m).

Drivers of heavy or wide vehicles may be unwilling to reverse long distances hence where trucks are travelling in both directions, it may be necessary to reduce the passing point spacing.

After determining the spacing and location of passing points, the length and width should be set. The length is primarily dependent on the traffic volume. If the traffic volume is high, the passing point should be long enough to accommodate several vehicles waiting for oncoming traffic to pass. A general guide for length (including tapers) of passing points is shown in Table 13-1.

Table 13-1: Length (including tapers) of Passing Points

Traffic volume (vehicles per day)	Length of Passing Point (m)
< 20	25
20 to 30	50
More than 30	75

A suitable width depends upon the width of the road itself. The criterion is to provide enough overall width for two design vehicles to pass each other safely at low speed hence a total trafficable minimum width of 5.5 metres is required. Allowing for vehicle overhang when entering the passing bay, a total width of 6.5 metres is suitable.

13.7 Safety Rest Areas and Scenic Overlooks

Safety rest areas and scenic overlooks are desirable elements of the complete highway development and are provided for the safety and convenience of the highway user.

A safety rest area is a roadside area with parking facilities separated from the roadway, provided for vehicle drivers to stop and rest for short periods. The rest area should provide the user with an opportunity to stop in an atmosphere that affords a distinct change from the monotony of driving.

A scenic overlook is a roadside area provided for drivers to park their vehicles, beyond the shoulder, primarily for viewing the scenery or for taking photographs in safety. The attraction of such a facility depends upon the presence of scenic and historical points of interest. The facilities should be designed so as to avoid marring the landscape.

Site selection for safety rest areas and scenic overlooks should consider the scenic quality of the area, accessibility, and adaptability to development. Site plans should be developed that should include proper and safe location of entrances and exits, road signs and markings, acceleration and deceleration lanes as required, and parking areas for cars and trucks. They may also include certain types of rest facilities (benches, tables, shelters, drinking fountains, rest rooms).

Where such facilities are specified, the average distance between rest areas should be 15 to 25 km. As far as possible, such facilities should be avoided where adjacent roadway gradients are in excess of 4%.

13.8 Safety Barriers

13.8.1 Purpose

Many accidents 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. The risk of these types of accidents can be reduced by the use of safety barriers (guardrails). Barriers may also protect roadside facilities from vehicle impact. The purpose of the barrier is to absorb or deflect the impact with as little severity as possible.

Safety barriers should be placed sufficiently far from the carriageway edge so as not to cause a hazard to vehicles, nor to reduce the effective width of the carriageway.

There is no standardization of the configuration of safety barriers at present. A description of each type of guardrail and a brief discussion of the positive and negative elements of each type follows.

13.8.2 Jersey Barriers

Of the guardrail types available, the Jersey barrier is the configuration classified as the safest. Constructed of concrete, this rigid barrier has the best chance of preventing the vehicle from proceeding beyond the barrier.

It also has the best chance of avoiding vehicle and occupant injury if impacted, because the profile allows the vehicle to ride up the barrier. However, the following problems have been noted:

- Jersey barriers must be continuous, because an opening, in addition to providing no protection, is in itself a hazard.

- The beginning and end of the barriers usually include no transition sections, and thus represent a hazard when hit head-on.

13.8.3 Grouted Rock Guardrail

This rigid barrier makes economic sense in that it employs materials available locally in its construction and also provides labour-intensive employment. However, the rail tends to be of a wider configuration than the others, and therefore requires a larger construction width. As it is of solid and substantial construction, it also represents a hazard of itself. This could be mitigated by the inclusion of end sections, and by the employment of a cross-section more closely approximating to that of a Jersey barrier.

13.8.4 Steel Rail Guardrail

This is a steel rail supported by concrete posts and is the most common worldwide configuration of guardrail. The barrier is slightly flexible and absorbs energy leading to less severe accidents. Its configuration is shown in Figure 13.6. The concrete posts must be built with good foundations. Special attention should be paid to the end sections, which should not be blunt.

13.8.5 Wire-Rope Barriers

This type of guardrail consists of two strands of cables fed through concrete posts. These guardrails are the least desirable configuration because:

1) If the cable is snapped due to an impact, the entire length of guardrail becomes ineffective. By comparison, if a steel rail configuration is hit, only one segment is ineffective.
2) The cable can be stolen, whether for use in towing a vehicle or for some other reason.

13.8.6 Use and Placement of Barriers

The routine employment of barriers is called into question for several reasons:

i) In addition to the construction cost of the guardrail itself, there are other related costs. These include the need to construct a wider roadway width to provide a platform for the construction of the guardrail. This is necessary, particularly in mountainous terrain and in rock cuts, and adds more to the construction costs than the cost of the guardrail itself.
ii) Traffic volumes are low on many road classes. It is likely that if the placement of guardrails were quantified on such roads using a cost/benefit analysis, they would not be cost effective.
iii) 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 the guardrail.

iv) Guidelines rather than standards usually govern the placement or non-placement of guardrails. Thus they are not an essential requirement for the road construction.
v) 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 a finding of liability are much greater.

The conclusion reached from consideration of the above is that guardrails should not routinely be constructed where long and steep side slopes are encountered. However, a compromise in the interest of safety is to provide delineators at all such sections.

Conversely, 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 on the *ERA Standard Detail Drawings*.

Where guardrails are employed, they should include reflectors to aid in the guidance of vehicles at night.

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.

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.

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 often the case that the additional width requirement for such installation cannot be achieved without significant earthwork costs, often comprising rock materials

Another factor is that 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.

Guardrails are only mandated at approaches to narrow bridges, being those of a width of 7.32 metres or narrower. A standard guardrail detail is shown in the *ERA Standard Detail Drawings* and in Figure 13.3.

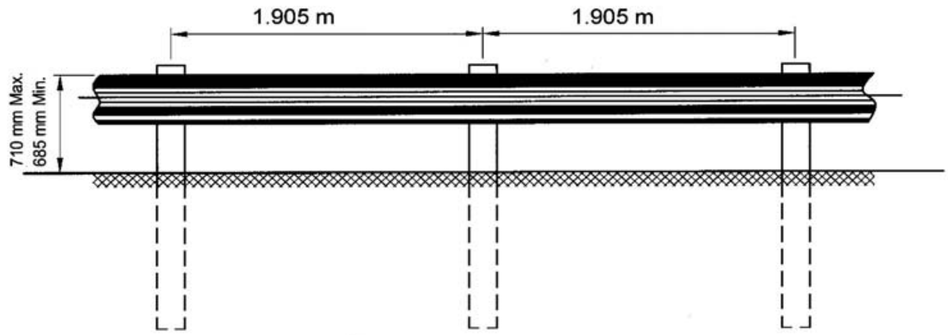


Figure 13-3: Guardrail

13.9 Emergency Escape Ramps

13.9.1 Need and Location

Where long, descending gradients exist, the provision of an emergency escape ramp at an appropriate location is desirable for the purpose of stopping an out-of control 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 the 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 runway vehicles, such as at intermediate points along the grade.

Escape ramps generally may be built at any feasible location where the main road alignment is tangent. They should be built in advance of curves that cannot be negotiated safely by a runaway vehicle, and in advance of populated areas.

13.9.2 Types

There are four types of emergency escape ramps. The first is a sand pile, the others are arrester beds, classified by grade; descending grade, horizontal grade, and ascending grade. They are illustrated in Figure 13.4. 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 metres 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 and the sand can be affected by weather. Because of these characteristics, 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. Ramp installations of this type use gradient resistance to advantage, supplementing the effects of the aggregate in the arrester bed, and generally reducing the length of ramp necessary to stop the vehicle. The loose material in the arresting bed increases the rolling resistance, and also 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.

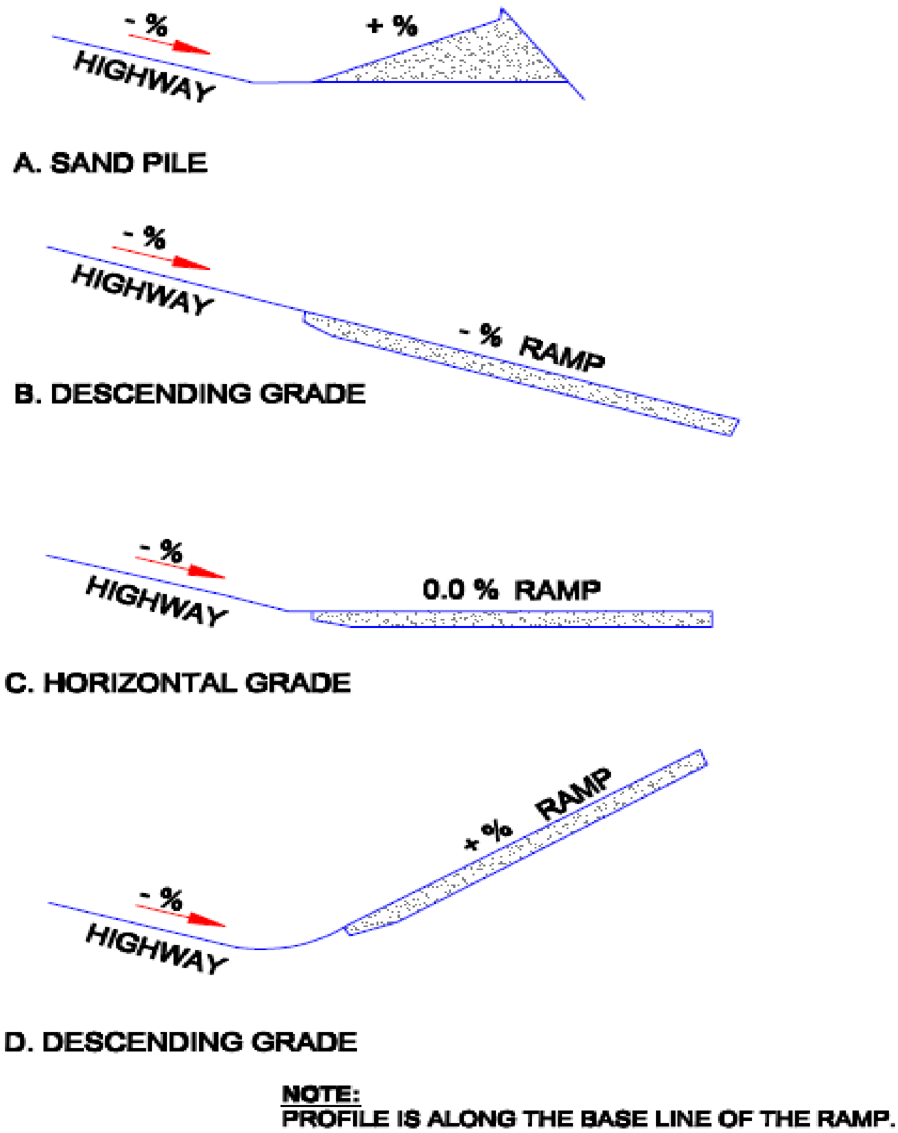


Figure 13-4: Basic Types of Emergency Escape Ramps (AASHTO)

13.9.3 Design Considerations

The design and construction of effective escape ramps involve a number of considerations as follows:

1. To safely stop an out-of-control vehicle, the length of the ramp must 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 undue 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 metres.
4. The in-fill material used in the arrester bed should be clean, not easily compacted, and have a high coefficient of rolling resistance. In-fill material should be single-sized natural or crushed coarse granular material or sand. Such material will maximize the percentage of voids, thereby providing optimum drainage and minimizing compaction. The use of single-size aggregate also minimizes 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 millimetres is recommended.
5. Contamination of in-fill material can reduce the effectiveness of the arrester bed by creating a hard surface layer at the bottom of the bed. Therefore, an aggregate depth up to 1.0m is recommended. To assist in decelerating the vehicle smoothly, the depth of the bed should be tapered from a minimum of 75 millimetres at the entry point to the full depth of aggregate in the initial 30 to 60 metres of the bed.
6. A positive means 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 so that the driver can enter safely 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 signing is required to inform a driver of the existence of an escape ramp and to prepare him well in advance so that he will have enough time to decide whether or not to use the escape ramp. 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:

$$L = \frac{V^2}{254(R \pm G)/100}$$

Where:

L	=	distance to stop (i.e. the length of the arrester bed), m,
V	=	entering velocity, km/h,
G	=	percent gradient of ramp,
R	=	rolling resistance expressed as equivalent percent gradient (Table 13.2).

For example, assume that topographic conditions at a site selected for an emergency escape ramp limit the gradient of an ascending ramp to 10 percent. The arrester bed is to be constructed with loose gravel for an entering speed of 140 km/h. Using Table 13.2, R is also determined to be 10 percent. The length necessary is determined from the above equation. For this case the length of the arrester bed is about 385 metres.

Table 13-2: Rolling Resistance of Roadway Surfacing Materials

Surfacing Material	Rolling Resistance (kg/100 kg GVM)	Equivalent Grade (%)[1]
Crushed aggregate, loose	50	5.0
Gravel, loose	100	10.0
Sand	150	15.0
Pea gravel	250	25.0

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 metre high with 1:1.5 slopes have been used at the end of ramps in several instances as the ‘last chance’ device.

13.9.4 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.

13.10 Safety Audits

The subject of road safety is remarkably complex in that, although many unsafe practices are glaringly obvious, there are many situations where it is difficult to identify what is likely to be unsafe, especially if the project is a new road and one is working from drawings. The history of road safety is full of ideas that were thought to improve road safety but often had no discernable effect or even made things worse. The problem has always been lack of reliable data; there is no substitute for a systematic method of recording the characteristics of road accidents and analysing the data when there is sufficient for reliable conclusions to be drawn.

Professional road safety auditing is the next best thing and is regularly undertaken on every road project in some countries in an attempt to improve the safety design from the very beginning. It is anticipated that this practice will become increasingly common in Ethiopia, especially for road projects located in populated areas.

13.11 Public Utilities

13.11.1 General

All highway improvements, whether upgraded within the existing right-of-way or entirely on new right-of-way, generally entail adjustment of utility facilities. The costs of utility adjustment vary considerably depending on the location of project. Utilities include:

a) Sanitary sewers.
b) Water supply lines.
c) Overhead and underground power and communications lines.
d) Drainage and irrigation lines.

The following factors should be considered in the location and design of utility installations.

1) Utility lines should be located to minimize 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 right-of-way line so as 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 normal to the highway alignment. Those utility crossings those 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 right-of-way limits 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 highway rights-of-way.
5) Sometimes attachment of utility facilities to highway structures, such as bridges, is a practical arrangement and may be authorized. Electric and Telephone Cables and water main placing in one trench should be done according to Figure 13.5 unless otherwise stated by the concerned institutions.

6) All utility installations on, over, or under highway right-of-way and attached structures should be of durable materials designed for long service-life expectancy, 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 aboveground 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. The clear zone dimension that is to be maintained for a specific functional classification is found in Chapter 6: Cross Section Elements.

13.11.2 Ethiopian Electric Light and Power Authority

The placement of light poles, power poles, wires, and underground cables, as per the Ethiopian Electric Light and Power Authority guidelines, is given in the Appendix D.

13.11.3 Addis Ababa Water and Sewerage Authority

The placement of water and sewerage pipes, as per the Addis Ababa Water and Sewerage Authority, is given in the Appendix D.

13.11.4 Ethiopian Telecommunications Corporation

The placement of telecommunications cables, as per the Ethiopian Telecommunications Corporation, is given in the Appendix D.

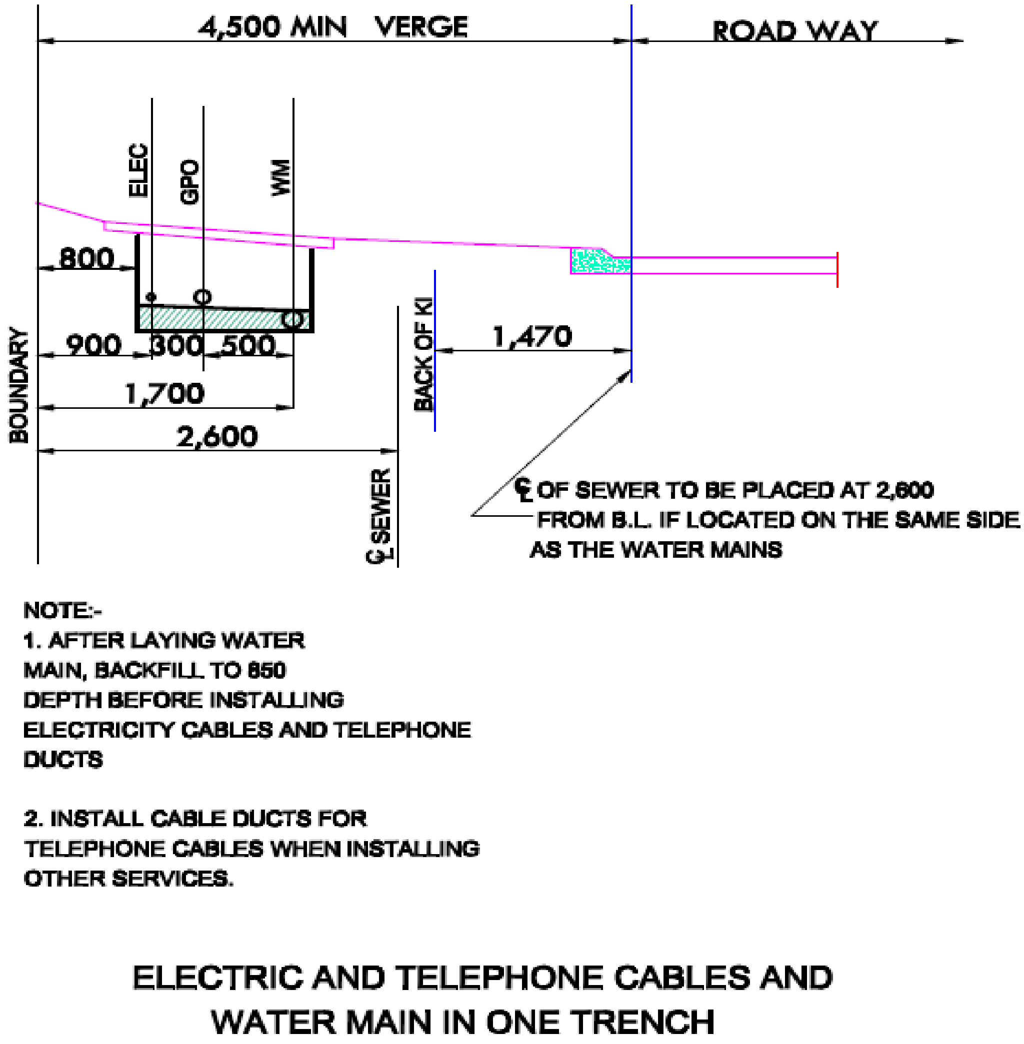


Figure 13-5: Utilities placement detail

13.12 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. 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 shall be not greater than 45º (Figure 13.6). 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.

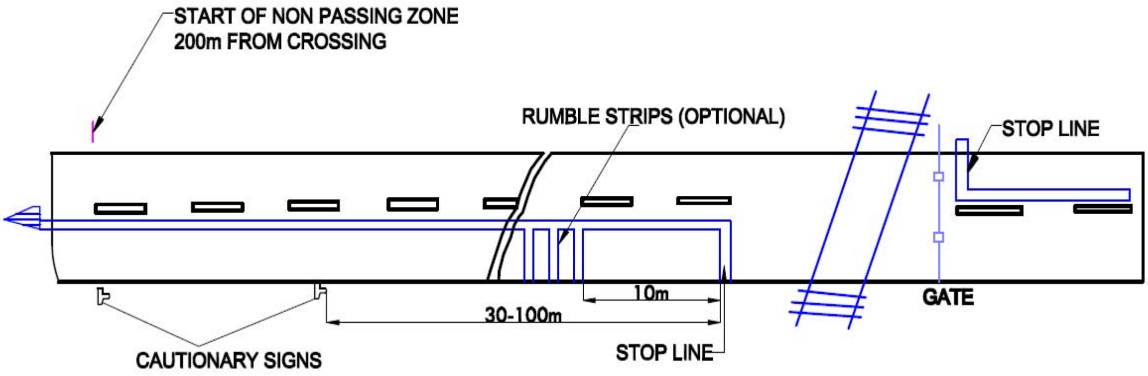


Figure 13-6: Railway Crossing Details with Rumble strips

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 (see Figure 13.7). Vertical curves should be of sufficient length to ensure an adequate view of the crossing, and crest and sag curves are the same as for the roadway design. The sight distance requirements down the tracks are similar to those for a roadway junction.

It is necessary to install signing to provide a safe crossing. Traffic control devices for railroad-highway grade crossings consist of signs and pavement markings. Standards for design and placement of these devices are covered in the Standard Detail Drawings.

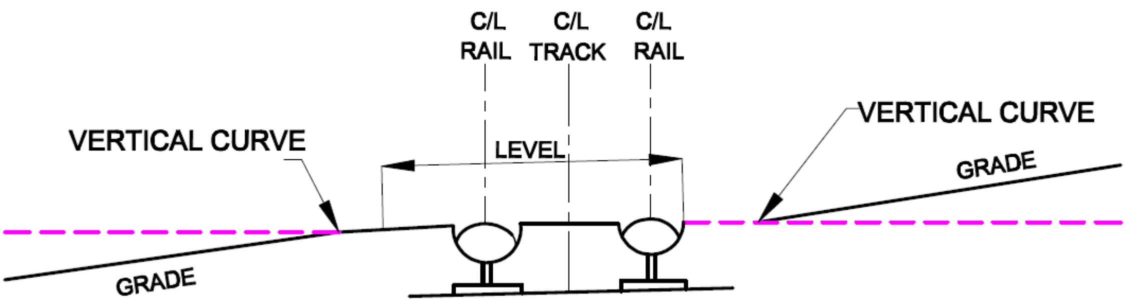


Figure 13-7: Railway Crossings Details on Vertical Curve

14 ROAD FURNITURE AND MARKINGS

14.1 Introduction

Road furniture and markings include the elements intended to improve the driver's perception and comprehension of the continually changing appearance of the road. Elements addressed herein include traffic signs, road markings, marker posts, traffic signals, and lighting.

Traffic signs provide essential information to drivers for their safe and efficient manoeuvring on the road. Road markings delineate the pavement edges and thereby clarify the paths that vehicles are to follow. Marker posts assist in a timely perception of the alignment ahead and, when equipped with reflectors, provide good optical guidance at night. Traffic signals are key elements for the efficient functioning of many urban roads and for some rural junctions. Finally, lighting is provided to improve the night time safety of a road.

Traffic signs, road markings, and marker posts must conform to ERA/RTA standards. Standards for traffic signs and road markings and their placement are provided in the *ERA Standard Detail Drawings.*

14.2 Traffic Signs

The extent to which signs and markings are required depends on the traffic volume, the type of road, and the degree of traffic control required for safe and efficient operation.

The safety and efficiency of a road depends to a considerable degree on its geometric design. However, physical layout must also be supplemented by effective traffic signing as a means of informing and warning drivers, and controlling drivers. Design of traffic signs and road markings is an intricate part of the design process.

Traffic signs are of three general types:

i) Regulatory Signs: indicate legal requirements of traffic movement
ii) Warning Signs: indicate conditions that may be hazardous to highway users
iii) Informatory Signs: convey information of use to the driver.

14.3 Road Markings

14.3.1 Purpose

The function of road markings is to encourage safe and expeditious operation of the road. Road markings either supplement traffic signs and marker posts or serve independently to indicate certain regulations or hazardous conditions. There are three general types of road markings - pavement markings, object markings and road studs.

14.3.2 Pavement markings

Pavement markings consist of centrelines, lane lines, no overtaking lines, edge lines, etc. Night time visibility of these markings can be markedly improved by mixing small glass beads into the paint or thermoplastic before applying it to the road surface. Other pavement markings such as stop and pedestrian crossings and various word and symbol markings may supplement pavement markings.

14.3.3 Object Markers

Physical obstructions in or near the carriageway should be removed in order to provide the appropriate clear zone. Where removal is impractical, such objects should be adequately marked by painting or by use of other high-visibility material. Where the object is in the direct line of traffic, the obstruction and marking thereon should be reflectorised.

14.3.4 Road Studs

Road studs are manufactured plastic objects incorporating reflectorised patches. Hybrid markings consisting of both reflective road markings and reflective studs can be useful for night-time driving in unlit areas. They are generally placed along the centreline of the road, in the middle of the "broken-line" portion of the marking, for added demarcation. The studs can also be used to give an audible and tactile warning of crossing any line that incorporates them, such as a pedestrian crossing.

The configuration for road markings is shown in the *ERA Standard Detail Drawings.*

All permanent pavement and object markings must be formed in thermoplastic materials and must be reflectorized.

14.4 Marker Posts

Marker posts have the function of controlling traffic to encourage safe and expeditious operation. There are two types of marker posts in use – guideposts and kilometre posts.

Guideposts are intended to make drivers aware of potential hazards such as abrupt changes in shoulder width, abrupt changes in the alignment, approaches to structures etc. For changes in shoulder width and approaches to structures, guide posts should be placed at 50m intervals. For spacing of guideposts at curves, see Table 14.1.

Table 14-1: Spacing of Guide Posts at Curves

Curve Radius (m)	Guide Post Spacing (m)
500	35
200	20
100	12
50	8
30	5

Kilometre posts are a requirement for all trunk and link roads. The kilometre posts must be numbered as indicated in Appendix A beginning at Addis Ababa or beginning at the trunk road-link road intercepts. Kilometre posts must be placed every 1 km, past the edge of the shoulder.

Marker posts shall be constructed from either concrete or plastic. Plastic marker posts may have the following advantages:

i) Lower initial costs.
ii) Lower maintenance cost due to their ability to absorb an impact and remain intact.
iii) Minimal damage to impacting vehicle.
iv) Safer highway environment.
v) Unlikely to be removed by local population for alternative use.

The incorporation of reflective panels into the marker post greatly improves their visibility at night.

14.5 Traffic Signal

Traffic signals control vehicular and pedestrian traffic by indicating the priority of movement for certain predetermined or traffic-actuated intervals of time. They are key elements for the efficient functioning of many urban roads and for some rural junctions. The phasing of the signals at each road junction should be integrated to achieve optimum efficiency. In designing the road, careful consideration should be given to the junction location and geometry with respect to traffic signal visibility and pedestrian requirements.

The layout of traffic lanes at signal-controlled junctions determines the functioning of the junction. Adequate provision should be made for right and left turning lanes and signals must be phased accordingly. Consideration should also be given to the provision of pedestrian signals at major junctions.

14.6 Lighting

Lighting is provided to improve the safety of a road. Statistics indicate that the night-time accident rate is higher than during daylight hours, which, to a large degree, may be attributed to impaired visibility. In urban areas, where there are concentrations of pedestrians and junctions, fixed source lighting tends to reduce accidents. However, lighting of rural highways is seldom justified except at junctions, intersections, and railway level crossings, narrow or long bridges, tunnels, sharp curves, and areas where there is activity adjacent to the road (e.g. markets).

To minimize the effect of glare and to provide the most economical lighting installation, luminaries should be mounted at a height of at least 9 metres. High mounted luminaries provide greater uniformity of lighting and mounting heights of 10 to 15 metres are frequently used. High mast lighting (special luminaries on masts of 30 metres) is used to illuminate large areas such as intersections. This type of lighting gives a uniform distribution of light over the whole area and thus illuminates the layout of the intersection.

Lighting columns (poles) should be placed behind vertical kerbs whenever practical. The appropriate distance is 0.5m behind the kerb for roads with a design speed of 50 km/h or less, and 1.2m or greater for roads with a design speed of 80 km/h or greater. Where poles are located within the clear zone, regardless of distances from the edge of the carriageway, they should be designed to include a frangible impact attenuation feature. However, these types of poles should not be used on roads in densely populated areas, particularly with footways. When struck, these poles may collapse and cause injury to pedestrians or damage adjacent property. Because of lower speeds and parked vehicles on urban roads, there is much less chance of injuries to vehicle occupants from striking fixed poles compared to higher speed roads.

On dual carriageways, lighting may be located either in the median or on the right hand side of each carriageway. However, with median installation the cost is generally lower and illumination is higher on the high-speed outer lanes. On median installations, dual mast arms should be used, for which 12-15 metre mounting heights are favoured.

These should be protected with a suitable safety barrier. On narrow medians, it is preferable to place the lighting poles so that they are integral with the median barrier.

When it is intended to install highway lighting in the future, providing the necessary conduits/ducts as part of the initial road construction can give rise to considerable savings.

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Appendix A CLASSIFICATION AND DESCRIPTION OF ROADS

The following are the classified roads in Ethiopia under the five classes. This classification should always be updated and the information should be used after getting confirmation from the relevant ERA Division by the designer.

Table A-1: Numbering of Trunk Roads

Number	Road Section	Length (km)
A1	Addis- Assab	853
A2	Addis- Axum	1071
A3	Addis- Gondar	737
A4	Addis- Gimbi	445
A5	Addis- Metu	510
A6	Jima-Mizan Teferi	554
A7	Mojo- Arba Minch	432
A8	Shashemene- Agere Maryam	214
A9	Nazareth- Asela	77
A10	Awash- Dengego- Degehabur	572
A10a	Dengego- Dire Dawa	20
	Total	5485

Table A-2: Numbering of Roads

Number	Road Section	Length (km)	Surfacing Type
	I. Trunk Roads		
A1	**Addis- Assab**	**853**	**Paved**
A1-1	Addis – Modjo	71	
A1-2 A1-2	Modjo- Nazreth	25	
A1-3	Nazreth - Metehara	95	
A1-4	Metehara- Awash Junction	46	
A1-5	Awash Junction –Gewane	153	
A1-6	Gewane – Mille	150	
A1-7	Mille - Semera	75	
A1-8	Semera – Serdo	30	
A1-9	Serdo - Dobi	50	
A1-10	Dobi - Burie	130	
A1a	Dobi - Galafi	28	
	II. Link Roads		
B11	**Mille - Kombolcha**	**130**	
B11-1	Mille – Bati	88	Unpaved
B11- 2	Bati – Kombolcha	42	Paved
	III. Main Access Roads		

	IV. Collector Roads		
D11	Assaita junc. - Assaita	50	Unpaved
	V. Feeder Roads		
E11	Modjo - Ejere - Arerti	60	Unpaved

	I. Trunk Roads		
A2	**Addis- Axum**	**1071**	
A2-1	Addis – Debreberehan	130	Paved
A2-2	Debreberehan – Debresina	60	Paved
A2-3	Debresina – Efeson/Ataye	110	Paved
A2-4	Efeson/Ataye-Kemisse-Kombolcha	105	Paved
A2-5	Kombolcha- Dessie	25	Paved
A2-6	Dessie – Woldiya	120	Paved/Unpav.
A2-7	Wodiya – Waja	66	Unpaved
A2-8	Waja - Maichew	80	Unpaved
A2-9	Maichew- Mekele	120	Unpaved
A2-10	Mekele - Adigrat	125	Paved
A2-11	Adigrat - Adiabun	108	Unpaved
A2-12	Adiabun – Axum	22	Unpaved
	II. Link Roads	**35**	
B20	**Addigrat - Zalanbesa**	**240**	Paved
B21	**Dessei - Gundowoin**	180	Unpaved
B21-1	Dessei – Mekane Selam	60	Unpaved
B21-2	Mekane Selam - Gundowoin	**299**	Unpaved
B22	**Weldiya -Woreta**	120	Unpaved
B22-1	Weldiya- Gashena	22	
B22-2	Gashena – Filakit	21	
B22-3	Filakit – Debrezebit	96	
B22-4	Debrezebit – Debre Tabour	40	
B22-5	Debre Tabour - Woreta		
	III. Main Access Roads	86	
C21	D/Berehan – Deneba – Jihur		Unpaved
	IV. Collector Roads	135	
D21	Tarmaber - Mehalemeda	40	Unpaved
D22	Robit - Awash		Unpaved
	V. Feeder Roads	85	
E21	Senbo - Kesem	42	Unpaved
E22	D/Berhan - Ankober	22	Unpaved
E23	Tarmaber - Seladingay		Unpaved

	I. Trunk Roads		
A3	**Addis- Gondar**	**737**	
A3-1	Addis – Commando	113	Paved
A3-2	Commando – Abay River	95	Paved
A3-3	Abay River – Dejen	22	Paved
A3-4	Dejen – Debre Markos	72	Paved
A3-5	Debre Markos – Bure Junction	110	Unpaved
A3-6	Bure Junction – Dangla	73	Unpaved
A3-7	Dangla – Bahir Dar	80	Unpaved
A3-8	Bahir Dar – Werota	53	Unpaved
A3-9	Werota – Azezo Airport	101	Unpaved
A3-10	Azezo Airport - Gondar	18	Paved
	II. Link Roads		
B30	**Gondar – Axum**	**356**	Unpaved
B30-1	Gondar – Debark	103	
B30-2	Debark – Adi Arkay	73	
B30-3	Adi Arkay – Boya River	20	
B30-4	Boya River – Inda Selassei	97	
B30-5	Inda Selassei - Axum	63	
B31	**Tik – Bahir Dar**	**259**	Unpaved
B31-1	Tik - Mota	139	
B31-2	Mota –Bahir Dar	120	
B32	**Bure Junction – Nekempte**	**257**	Unpaved
B32-1	Bure Junction – Abay River	60	
B32-2	Abay River - Gida	89	
B32-3	Gida – Ander Guten	46	
B32-4	Ander Guten - Nekempte	62	
	III. Main Access Roads		
C31 **C32**	**Mukautre - Alemketema**	**105**	Unpaved
C33	**Mankussa - Birsheleko**	**26**	Unpaved
C33-1	**Kosober – Bambudi**	**323**	Unpaved
C33-2	Kosober – Chagni	56	
C33-3	Chagni - Mambuk	56	
C33-4	Mambuk - Guba	131	
C34	Guba junc. - Bambudi	80	
C35	**Azezo - Metema**	**185**	Unpaved
	Gondar - Humera	**250**	Unpaved
	IV. Collector Roads		
D31		250	
	Dogolo – Aksta – Tenta – Dawnt-Gashe		Unpaved
D32		53	
	Dibate junc. - Dibate		Unpaved
D33		22	
	Tissabay junc. – Tissabay		Unpaved
		23	
	V. Feeder Roads		
E31		52	
	Chancho - Deneba		Unpaved
E32			
	Azezo - Gorgora		Unpaved

	I. Trunk Roads		
A4	**Addis- Gimbi**	**441**	
A4-1	Addis- Holeta	40	Paved
A4-2	Holeta - Ambo	85	Paved
A4-3	Ambo – Gedo	65	Paved
A4-4	Gedo - Bako	57	Paved
A4-5	Bako - Nekempte	81	Paved
A4-6	Nekempte - Gimbi	113	Unpaved
	II. Link Roads		
B40	**Gimbi – Assosa**	**233**	Unpaved
B40-1	Gimbi – Nejo	70	
B40-2	Nejo – Mendi	70	
B40-3	Mendi – Bambasi	51	
B40-4	Bambasi - Assosa	42	
B41	**Ambo – Giyon (Weliso)**	**60**	Unpaved
B42	**Nekempte Junction – Bedele**	**92**	Unpaved
B42-1	Nekempte Junction – Dedessa Bridge	62	
B42-2	Dedessa Bridge – Bedele	30	
B43	**Gimbi(Mekenajo) – Dembidolo**	**197**	Unpaved
B43-1	Gimbi(Mekenajo) –Alem Teferi	111	
B43-2	Alem Teferi – Dembidolo	86	
	III. Main Access Roads		
C40	Asosa - Kurmuk	96	Unpaved
C41	Holeta - Muger	85	Unpaved
C42	Gedo - Fincha	134	Unpaved
C44 C44 -	**Bambasi - Gambela**		Unpaved
1	Bambasi - Begi	70	
C44 -2	Begi - Mugi	114	
C44 -3	Mugi - Shebel	94	
C44-4	Shebel - Gambela		
C44a	**Mugi - Dembidolo**	43	Unpaved
	IV. Collector Roads	79	
D41	Nedjo – Jarso – Shemel Toke		Unpaved
	V. Feeder Roads	51	
E41	Asossa junc. - Dabus		Unpaved

	I. Trunk Roads		
A5	**Addis- Metu**	**580**	
A5-1	Addis (Alemgena) – Giyon (Weliso)	94	Paved
A5-2	Giyon (Weliso) – Walkite	42	Paved
A5-3	Walkite – Gibe River	33	Paved
A5-4	Gibe River - Saja	65	Paved
A5-5	Saja - Jimma	87	Paved
A5-6	Jimma - Bedele	143	Paved/Unpav.
A5-7	Bedele- Metu	116	Paved/Unpav.
	II. Link Roads		
B50	**Metu – Gambella**	**164**	Unpaved
B50-1	Metu - Gore	18	
B50-2	Gore – Gambella	146	
B51	**Alemgena - Sodo**	**339**	Unpaved
B51-1	Alemgena - Butajira	130	
B51-2	Butajira -Hosaina	100	
B51-3	Hosaina - Areka	69	
B51-4	Areka - Sodo	40	
B52	**Jima -Sodo**	**241**	Unpaved
B52-1	Jima - Chida	82	
B52-2	Chida - Waka	74	
B52-3	Waka - Sodo	85	
B53	**Gore – Mizan Teferi**	**179**	Unpaved
B53-1	Gore – Gecha	73	
B53-2	Gecha - Tepi	55	
B53-3	Tepi – Mizan Teferi	51	
	III. Main Access Roads		
C50	**Gambela - Jikawo**	**120**	Unpaved
C51	**Welkite - Hosaina**	**150**	Unpaved
C52	**Indibir – Ziway**	**108**	Unpaved
C52-1	Indibir – Butajira	60	
C52-2	Butajira - Ziway	48	
C53	**Bole junc. - Tolay**	**65**	Unpaved
C54	**Limmu Junc.- Sintu - Atnago**	**80**	Unpaved
	IV. Collector Roads		
D51	Gambela - Abobo	45	Unpaved
D52	Metu - Alge	50	Unpaved
	V. Feeder Roads		
E51	II. Tulubolo - Arbuchulule	27	Unpaved
E52	Atat junc. - Kose	47	Unpaved
E53	Gubre junc.-Bojobar	62	Unpaved
E54	Agaro – Gera Agriculture devt.	72	Unpaved
E55	Metu –Sor Hydro Electric Power	32	Unpaved

	I. Trunk Roads		
A6	**Jima- Mizan Teferi**	**216**	Unpaved
A6-1	Jima- Bonga	101	
A6-2	Bonga - Mizan Teferi	115	
	V. Feeder Roads		
E61	Bonga – Chida	80	Unpaved
E62	Shishinda - Tepi	74	Unpaved
	I. Trunk Roads		
A7	**Mojo- Arba Minch**	**446**	Paved
A7-1	Mojo- Ziway	87	
A7-2	Ziway – Shashemene	89	
A7-3	Shashemene - Alaba	70	
A7-4	Alaba - Sodo	70	
A7-5	Sodo - Arba Minch	130	
	II. Link Roads		
	III. Main Access Roads		
C70	**Arbaminch - Kelem**		Unpaved
C70-1	Arbaminch - Konso	85	
C70-2	Konso - Woito	46	
C70-3	Woito - Turmi	130	
C70-4	Turmi – Kelem		
C71	**Sodo –Woito**		Unpaved
C71-1	Sodo –Sawla	139	
C71-2	Sawla – Jinka		
C71-3	Jinka –Keyafer – Woito	92	
C72	**Arbaminc junction – Gerese-Sawla**		Unpaved
C73	**Konso – Yabelo**	**135**	Unpaved
	IV. Collector Roads		
D70a	Meno – Fejej	135	Unpaved

	I. Trunk Roads		
A8a	**Shashemene – Wondo Genet**	**20**	Unpaved
A8	**Shashemene- Hgere Mariam**	**120**	Paved
A8-1	Shashemene- Awassa	20	
A8-2	Awassa - Aposto	35	
A8-3	Aposto - Dilla	54	
A8-4	Dilla - Hgere Mariam	11	
	II. Link Roads		
B81	**Shashemene-Dodola**	**73**	Unpaved
B82	**(Aposto) Wendo-Negele**	**279**	Unpaved
B82-1	(Aposto) Wendo-Kebre Mengeste	154	
B82-2	Kebre Mengeste – Negele	125	
	III. Main Access Roads		
C80	**Hageremariam – Moyale**	**300**	Paved
C80-1	Hageremariam – Yabelo	94	
C80-2	Yabelo – Mega	99	
C80-3	Mega – Moyale	107	
C81	**Agere Selam-Daye-Mejo-Soyema**	**100**	Unpaved
C82	**Mega – Bulbul**	**205**	Unpaved
C82-1	Mega – Wachile	85	
C82-2	Wachile– Bulbul	60	
C82-3	III. Bulbul – Negele	60	
C83	**Goba -Bitat**	**223**	Unpaved
C83-1	Goba - (Delo)Mena	112	
C83-2	(Delo)Mena - Bitat	111	
	IV. Collector Roads		
D81	**Wondo –Dila**	**37**	Unpaved
D82	**Negele –Dolo**	**314**	Unpaved
D82-1	Negele – Filtu	112	
D82-2	Filtu - Dolo	202	

	I. Trunk Roads		
A9	**Nazareth- Asela**	**77**	Paved
	II. Link Roads		
B90	**Asela -Gode**	**608**	Unpaved
B90-1	Asela-Dodola	120	
B90-2	Dodola-Robe	113	
B90-3	Robe-Ali-Ginir	138	
B90-4	Ginir-Imi	180	
B90-5	Imi -Gode	57	
B90a	**Robe- Goba**	**14**	Unpaved
B91	Dera- Chole	**123**	Unpaved
B91-1	Dera- Sire	23	
B91-2	Sire-Chole	100	
B92	**Iteya- Diksis - Robi**	**76**	Unpaved
	III. Main Access Roads		
C90	**Robe - Goro – Ginir**	**141**	Unpaved
C91	**Chole -Arberekti**	**164**	Unpaved
C91-1	Chole- Mechara	66	
C91-2	Mechara - Gelemso -Arberekti	98	
C92	**Robe - Shek Husen**	**1**	Unpaved
C92-1	Robe - Seru	**55**	
C92-2	Seru - Shek Husen	90	
		65	
	V. Feeder Roads		
E91	IV. Sodere – Nura era	60	Unpaved

	I. Trunk Roads		
A10	**Awash Junction – Degehabur**	**572**	
A10-1	Awash Junction – Arbereketi	106	Paved
A10-2	Arbereketi – Kulubi	130	Paved
A10-3	Kulubi - Dengego	31	Paved
A10-4	Dengego - Harar	31	Paved
A10-5	Harar - Jijiga	103	Unpaved
A10-6	Jijiga – Degehabur	171	Unpaved
A10a	**Dengego – Dire Dawa**	**20**	Paved
	II. Link Roads		
B100	**Degehbur-Gode**	**398**	Unpaved
B100-1	Degehbur-Kebridar	233	
B100-2	Kebridar-Gode	165	
B101	**Metehara- Chole**	**200**	Unpaved
B102	**Babile-Imi**	**365**	Unpaved
B102-1	Babile-Fik	160	
B102-2	Fik- Hamero	105	
B102-3	Hamero- Imi	100	
	III. Main Access Roads		
C100	Diredawa - Dewole	224	Unpaved
C101	Diredawa–Hurso-Erer	54	Unpaved
C102	Kebridar - Warder	120	Unpaved
C103	Gode - Hargele.	205	Unpaved
C104	Gode – Kelafo - Ferfer	135	Unpaved
	IV. Collector Roads		
D10 1	Kobo - Deder	12	Unpaved
D10 2	Harer - Jarso - Bombas	81	Unpaved

Appendix B COEFFICIENTS OF FRICTION

The coefficients of friction as determined by various authors are shown in Figure B-1. Longitudinal friction coefficients depend on vehicle speed, type, condition and texture of roadway surface, weather conditions, and type and condition of tyres. Its value decreases as speed increases but there is considerable disagreement about representative values, especially at the lower speeds.

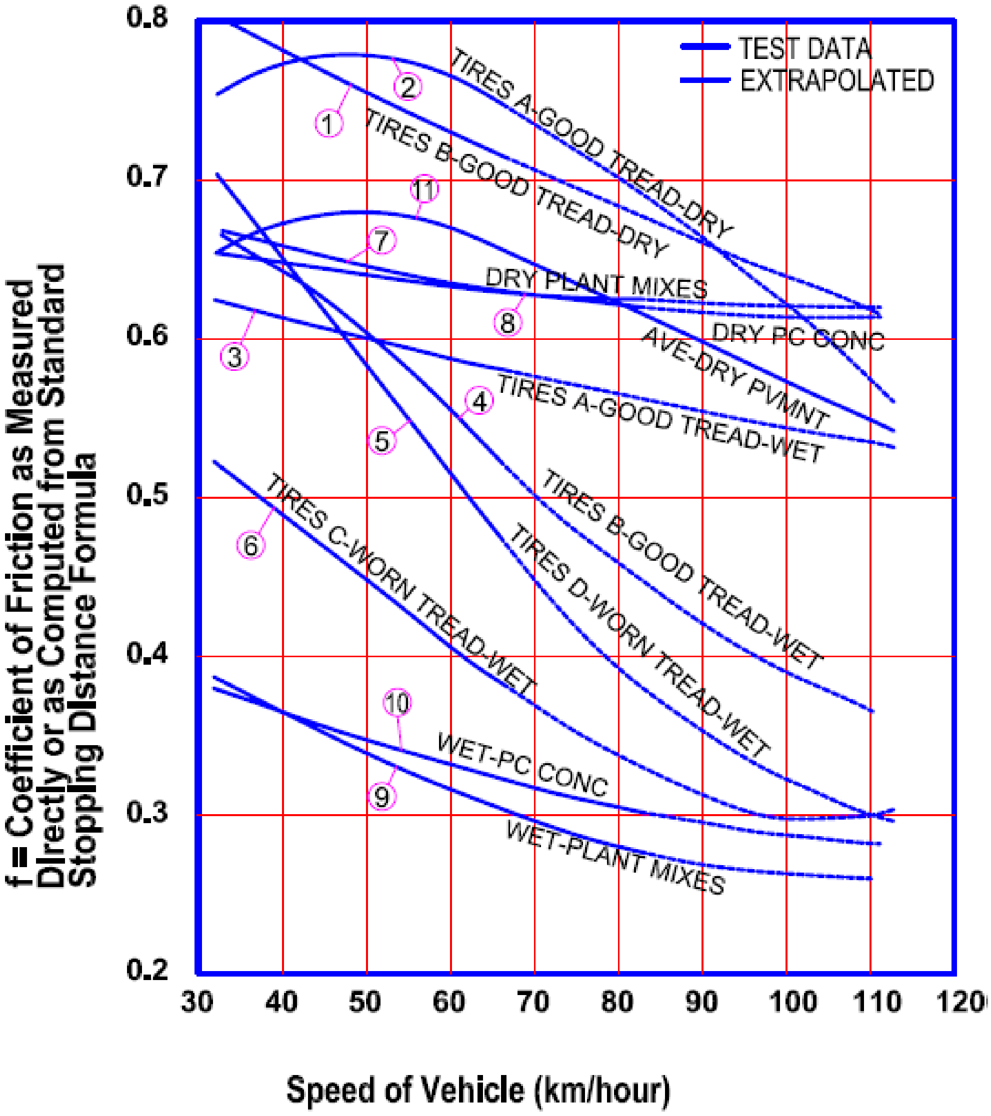


Figure B-1: Longitudinal Friction for Various Tyre and Pavement Conditions

It is therefore difficult to define representative values in a country such as Ethiopia where the conditions are so variable; worn tyres are common, gravel roads can have particularly low friction characteristics, and the climate varies from wet to arid.

Side friction coefficients are also dependent on vehicle speed, type, condition and texture of roadway surface, weather conditions, and type and condition of tyres. Figure B-2 illustrates some values obtained by various researchers.

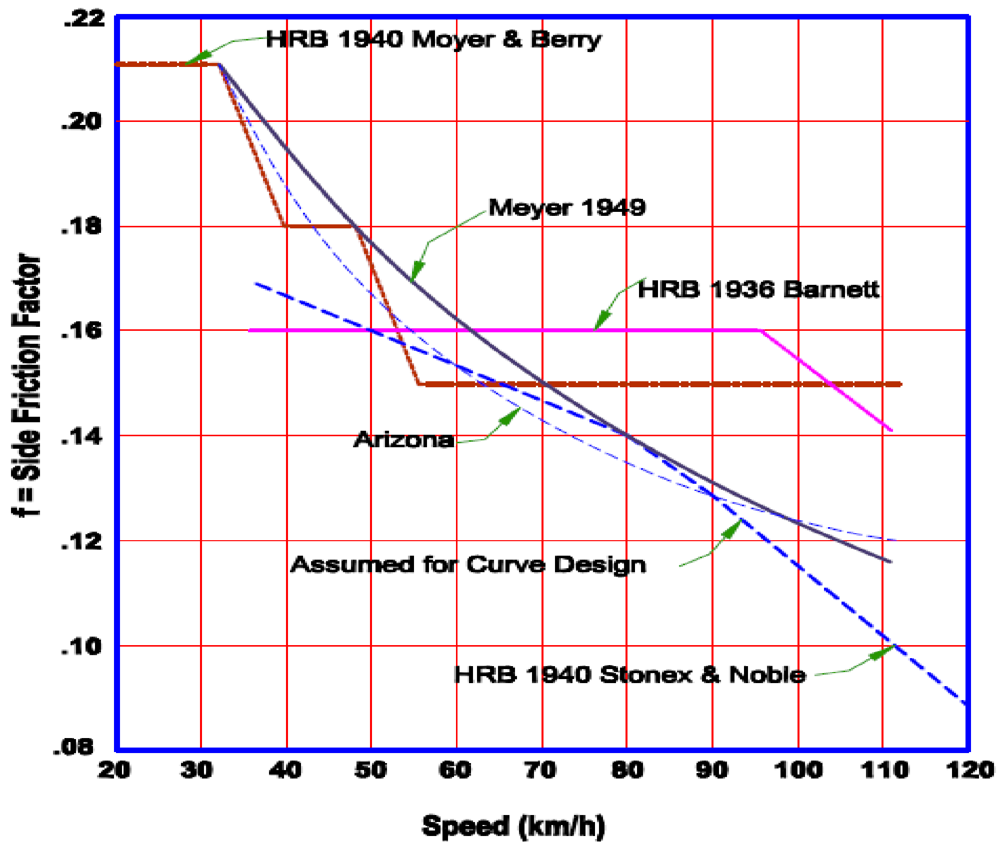


Figure B-2: Side Friction Factors for Rural Highways

The values used in this manual (Tables B-1 and B-2) allow a reasonable safety factor to cater for the wide range of conditions. For unpaved roads a systematic reduction in the values used for paved roads has been used.

It is reported that drivers in urban environments tolerate a higher degree of 'discomfort' than drivers in rural areas, hence it is sometimes advocated that higher coefficients of friction could be used for calculating minimum radii of curvature at the lower speeds in urban areas. In view of the general conditions in Ethiopia (see above) it is not considered prudent to do so.

Table B-1: Friction factors for paved roads

Design speed (km/h)	**20**	**25**	**30**	**40**	**50**	**60**	**70**	**85**	**100**	**120**
Longitudinal Factor	0.42	0.41	0.40	0.37	0.35	0.33	0.32	0.30	0.29	0.28
Side Friction Factor	0.23	0.215	0.205	0.185	0.17	0.155	0.145	0.13	0.125	0.105

Table B2 Friction factors for unpaved roads.

Design speed (km/h)	**20**	**25**	**30**	**40**	**50**	**60**	**70**	**80**	**85**	**90**	**100**
Longitudinal Factor	0.34	0.33	0.32	0.30	0.28	0.26	0.25	0.24	0.24	0.23	0.23
Side Friction Factor	0.185	0.17	0.165	0.15	0.135	0.125	0.12	0.11	0.105	0.10	0.09

Appendix C SIGHT DISTANCE AT INTERSECTIONS AND JUNCTIONS

C.1 Introduction

The provision of adequate sight distances and appropriate traffic controls is essential for safe intersection operation. Mathematical models have been developed for carrying out the required calculations for adequate sight distances at junctions of different types but require many assumptions and are not reliable. The best information is obtained from empirical data but this is primarily based on research in western countries. The mix of traffic, its age spectrum, overloading practices etc. are entirely different in Ethiopia. Furthermore, the required sight distances also depend strongly on driver behaviour. It is therefore not a simple task to calculate the optimum or minimum sight distances applicable to different junction designs, different road classes and different mixes of traffic. A pragmatic approach is to utilise the available empirical data but to select conservative options for safety.

Stopping sight distance should be provided continuously on all roadways including at the approaches to intersections. However, in rural areas or when approach speeds are in excess of 80 km/h, the decision sight clearance set out in Section 7.5 should be provided on all approaches to intersections for safe operation, particularly where auxiliary lanes are added to the intersection layout to accommodate the turning movements. This is the sight distance required by drivers entering the intersection to enable them to establish that it is safe to do so and then to carry out the manoeuvres necessary either to join or to cross the opposing traffic streams. The distances shown in this Appendix are derived from research into gap acceptance as reported in NCHRP Report 383 *Intersection Sight Distance*.

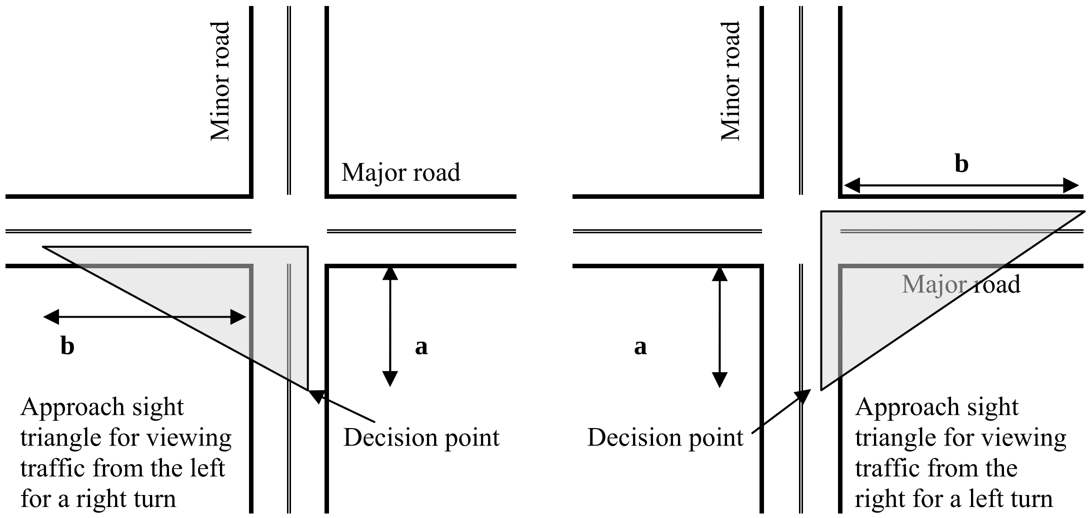
C.2 Sight Triangles

Each quadrant of an intersection should contain a clear sight triangle free of obstructions that may block a driver's view of potentially conflicting vehicles on the opposing approaches. Two different forms of sight triangle are required, *approach sight triangles* and *departure sight triangles* as shown in Figure C.1.

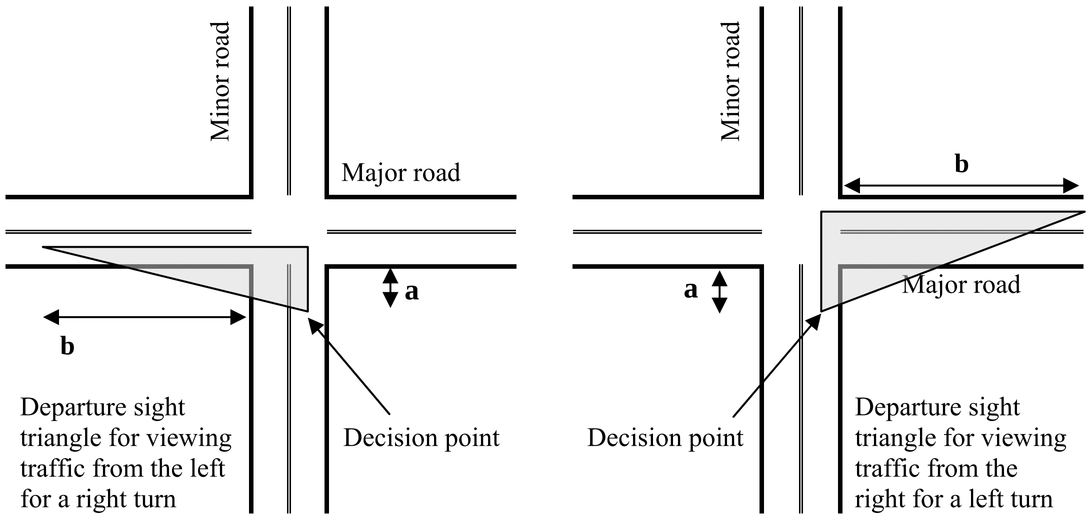
The approach triangle will have sides with sufficient lengths on both intersecting roadways such that drivers can see any potentially conflicting vehicle in sufficient time to *slow, or to stop* if need be, before entering the intersection.

For the departure sight triangle, the line of sight described by the hypotenuse of the sight triangle should be such that a vehicle just coming into view on the major road will, at the design speed of this road, have a travel time to the intersection corresponding to the *gap acceptable* to the driver of the vehicle on the minor road.

Both forms of sight triangle are required in each quadrant of the intersection. The line of sight assumes a driver eye height of 1.05 metres and an object height of 1.3 metres. The areas shown shaded in Figure C.1 should be kept clear of vegetation or any other obstacle to provide a clear line of sight. To this end, the road reserve is normally splayed to ensure that the entire extent of the sight triangle is under the control of the road authority. Furthermore, the profiles of the intersecting roads should be designed to provide the required sight distance. Where one or other of the approaches is in cut, the affected sight triangles may have to be 'day-lighted', i.e. the natural material occurring within the sight triangles may have to be excavated to ensure inter-visibility between the relevant approaches.



A Approach Sight Triangles



B Departure Sight Triangles

Figure C.1 Sight Triangles

Sight distance values are based on the ability of the driver of a passenger car to see an approaching passenger car. It is also necessary to check whether the sight distance is adequate for trucks. Because their rate of acceleration is lower than that of passenger cars and, as the distance that the truck has to travel to clear the intersection is longer, the gap acceptable to a truck driver is considerably greater than that required by the driver of a passenger car. For design purposes, the eye height of truck drivers is taken as 1.8 metres for checking the availability of sight distance for trucks.

C.3 Intersection Control

The recommended dimensions of the clear sight triangles vary with the type of traffic control used at an intersection because different types of control impose different legal constraints on drivers resulting in different driver behaviour. Sight distance policies for intersections with the following types of traffic control are shown below:

A. Intersections with no control;
B. Intersections with 'Stop' control on the minor road;
    a. Right turn from the minor road (Case B1);
    b. Left turn from the minor road (Case B2);
    c. Crossing manoeuvre from the minor road (Case B3);
C. Intersections with 'Yield' control on the minor road;
    a. Crossing manoeuvre from the minor road (Case C1);
    b. Left or right turn from the minor road (Case C2);
D. Intersections with traffic signal control;
E. Intersections with all-way Stop control.

C.3.1 Intersections with no control (Case A)

Uncontrolled intersections are not used in conjunction with the main road network but are common in rural networks and access roads to rural settlements. In these cases, drivers must be able to see potentially conflicting vehicles on intersecting approaches in sufficient time to stop safely before reaching the intersection. Thus sight triangles with legs at least equal to the *stopping sight distance* at the design speed of the road should be provided on all the approaches to uncontrolled intersections (Tables 7.2 and 7.3). However, the sudden appearance at the available sight distance of a vehicle about to cross a junction should not require an emergency stop by the vehicle on the other road (except in extreme circumstances) hence sight distances should exceed the stopping sight distance.

However, observations in the NCHRP study indicate that vehicles approaching uncontrolled intersections typically slow down. This occurs even when no potentially conflicting vehicles are present. Hence if sight triangles of the ideal size cannot be provided, approaching vehicles travelling at less than their normal running speed can still brake to a stop if required in an emergency situation. Table C.1 shows revised sight distances based on the NCHRP study.

If these sight distances cannot be provided, advisory speed signing to reduce speeds or installing Stop signs on one or more approaches should be investigated.

Uncontrolled intersections do not normally require departure sight triangles because they typically have very low traffic volumes. If a driver finds it necessary to stop at an uncontrolled intersection because of the presence of a conflicting vehicle, it is unlikely that another potentially conflicting vehicle will be encountered as the first vehicle departs the intersection.

Table C.1: Recommended Sight Distances for Intersections with no Traffic Control

Design Speed (km/h)	Sight Distance (m)			
	Approach gradient			
	0 to -3%	-4 %	-5%	-6%
30	25	25	25	30
40	30	35	35	35
50	40	45	45	45
60	50	55	55	55
70	65	70	70	70
80	80	90	90	95
90	95	105	105	115
100	120	130	130	145
110	140	155	170	170
120	165	180	200	200

Note. The stopping sight distances in Tables 7.2 and 7.3 should be used whenever possible

C.3.2 Intersections with Stop control on the minor road (Case B)

Departure sight triangles for intersections with Stop control on the minor road should be considered for three situations:

- Left turns from the minor road (Case B1);
- Right turns from the minor road (Case B2); and
- Crossing the major road from the minor road (Case B3).

Approach sight triangles, as shown in Figure C.1.A need not be provided at Stop-controlled intersections because all minor-road vehicles should stop before entering or crossing the major road.

Vehicles turning left from the minor road have to cross the stream of traffic approaching from the left and then merge with the stream approaching from the right. Right-turning vehicles need only merge with the stream approaching from the left. As the merging manoeuvre requires that turning vehicles should be able to accelerate approximately to the speed of the stream with which they are merging, it necessitates a gap longer than that for the crossing manoeuvre.

C3.2.1 Left turn from the minor road (Case B1)

A departure sight triangle for traffic approaching from the right as shown in Figure C.1.B should be provided for left turns from the minor road onto the major road for all Stop-controlled approaches. Field observations of the gaps accepted by the drivers of vehicles turning to the left onto the major road have shown that the values in Table C.2 provide sufficient time for the minor-road vehicle to accelerate from a stop and merge with the opposing stream without undue interference. These observations also revealed that major-road drivers reduce their speed to some extent to accommodate vehicles entering from the minor road. Where the gap acceptance values in Table C.2 are used to determine the length of the leg of the departure sight triangle along the major road, most major-road drivers need not reduce speed to less than 70 percent of their initial speed.

Table C.2 applies to passenger cars. However, for minor-road approaches from which substantial volumes of heavy vehicles enter the major road, the values for single-unit trucks or semitrailers should be applied. Table C.2 includes adjustments to the acceptable gaps for the number of lanes on the major road and for the approach gradient of the minor road. The adjustment for the gradient of the minor-road approach need be made only if the rear wheels of the design vehicle would be on an upgrade steeper than 3 per cent when the vehicle is at the stop line of the minor-road approach.

The length of the sight triangle along the major road (distance ***b*** in Figure C.1) is the product of the design speed of the major road in metres/second and the critical gap in seconds as listed in Table C.2. If these sight distances along the major road (including the appropriate adjustments) cannot be provided, consideration should be given to the installation of advisory speed signs on the major-road approaches.

Dimension ***a*** in Figure C.1.B depends on the context within which the intersection is being designed. In urban areas, drivers tend to stop their vehicles immediately behind the Stop line, which may be located virtually in line with the edge of the major road. A car driver would, therefore, be located about 2.4 metres away from the Stop line. In rural areas, vehicles usually stop at the edge of the shoulder of the major road. In the case of a 3-metre wide shoulder, for example, the driver would be approximately 5.4 metres away from the edge of the travelled way.

Table C.2: Travel Times Used to Determine the Leg of the Departure Triangle along the Major Road for Right and Left Turns from Stop-controlled Approaches

Vehicle	**Travel Time (seconds) at Design Speed of the Major Road**
Passenger car	7.5
Single unit truck	9.5
Semi trailer	11.5
Multi-lane highways. For left turns onto 2-way highways with more than 2 lanes, add 0.5 seconds for cars and 0.7 seconds for trucks for each additional lane (in excess of one) to be crossed by the turning vehicle. No adjustment is necessary for right turns.	
Adjustment for gradients. If the approach gradient on the minor road exceeds 3% add 0.1 second per gradient for right turns and 0.2 seconds per gradient for left turns	

Where the major road is a dual carriageway, two departure sight triangles have to be considered: a sight triangle to the left, as for the crossing movement and one using the acceptable gap as listed in Table C.2 for vehicles approaching from the right. This presupposes that the width of the median is sufficient to provide a refuge for the vehicle turning from the minor road. If the median width is inadequate, the adjustment in Table C.2 for multilane major roads should be applied with the median being counted as an additional lane. The departure sight triangle should be checked for various possible design vehicles because the width of the median may be adequate for one vehicle type and not for another so that two different situations have to be evaluated.

C3.2.2 Right turn from the minor road (Case B2)

A departure sight triangle for traffic approaching from the left, as shown in Figure C.1 should be provided for right turns from the minor road. The lengths of the legs of the departure sight triangle for right turns should generally be the same as those for the left turns used in Case B1. Specifically, the length of the leg of the departure sight triangle (dimension ***b***) along the major road should be based on the travel times in Table C.2, including appropriate adjustment factors.

Dimension ***a*** depends on the context of the design and can vary from 2.4 metres to 5.4 metres. Where sight distances along the major road based on the travel times from Table C.2 cannot be provided, it should be kept in mind that field observations indicate that, in making right turns, drivers generally accept gaps that are slightly shorter than those accepted in making left turns. The travel times in Table C.2 can be decreased by 1.0 to 1.5 seconds for right turn manoeuvres, where necessary, without undue interference with major-road traffic. When the recommended sight distance for a right-turn manoeuvre cannot be provided, even with this reduction, consideration should be given to the installation of advisory speed signs and warning devices on the major road approaches.

C3.2.3 Crossing manoeuvre from the minor road (Case B3)

In most cases it can be assumed that the departure sight triangles for right and left turns onto the major road, as described for Cases B1 and B2, will also provide more than adequate sight distance for minor-road vehicles crossing the major road. However, it is advisable to check the availability of sight distance for crossing manoeuvres:

- Where right and/or left turns are not permitted from a particular approach and crossing is the only legal manoeuvre;
- Where the crossing vehicle has to cross four or more lanes; or
- Where substantial volumes of heavy vehicles cross the highway and where there are steep gradients on the departure roadway on the far side of the intersection that might slow the vehicle while its rear is still in the intersection.

Table C.3 presents travel times and appropriate adjustment factors that can be used to determine the length of the leg of the sight triangle along the major road to accommodate crossing manoeuvres. At divided highway intersections, depending on the width of the median and the length of the design vehicle, sight distance may be needed for crossing both roadways of the divided highway or for crossing the near lanes only and stopping in the median before proceeding.

Table C.3: Travel Times Used to Determine the Leg of the Departure Triangle along the Major Road for Crossing Manoeuvres from Stop-controlled Approaches

Vehicle	Travel Time (seconds) at Design Speed of the Major Road
Passenger car	6.5
Single unit truck	8.5
Semi trailer	10.5
Multi-lane highways. For left turns onto 2-way highways with more than 2 lanes, add 0.5 seconds for cars and 0.7 seconds for trucks for each additional lane (in excess of one) to be crossed. In the case of dual carriageways with inadequate median width for refuge, count the median as another lane to be crossed.	
Adjustment for gradients. If the approach gradient on the minor road exceeds 3% add 0.2 second per percent gradient in excess of 3%.	

Notes: 1. For minor-road approach gradients that exceed +3 per cent, increase by the same factor as in Table C.1.
2. Travel time applies to a vehicle that slows before crossing the intersection but does not stop.

C.3.3 Intersections with Yield control on the minor road (Case C)

Vehicles entering a major road at a Yield-controlled intersection may, because of the presence of opposing vehicles on the major road, be required to stop. Departure sight triangles as described for Stop control must therefore be provided for the Yield condition. However, if no conflicting vehicles are present, drivers approaching Yield signs are permitted to enter or cross the major road without stopping. The sight distances needed by drivers on Yield-controlled approaches exceed those for Stop-controlled approaches because of the longer travel time of the vehicle on the minor road.

For four-legged intersections with Yield control on the minor road, two separate sets of approach sight triangles as shown in Figure C.1.A should be provided; one set of approach sight triangles to accommodate right and left turns onto the major road and the other for crossing movements. Both sets of sight triangles should be checked for potential sight obstructions.

C3.3.1 Crossing manoeuvres (Case C1)

The lengths of the leg of the approach sight triangle along the minor road to accommodate the crossing manoeuvre from a Yield-controlled approach (distance ***a*** in Figure C.1.A) are given in Table C.4. The distances are based on similar assumptions as those for Case A control. The distances and times in Table C.4 should be adjusted for the gradient of the minor road approach, using the same factors as used in Table C.1.

Table C.4: Leg of Approach Sight Triangle Along the Minor Road to Accommodate Crossing Manoeuvres from Yield-controlled Approaches

Design speed (minor road). (km/h)	Distance along minor road (m)	Travel time (t_a) from decision point to major road[(1)(2)]
30	30	3.4
40	40	3.7
50	50	4.1
60	65	4.7
70	85	5.3
80	110	6.1
90	140	6.8
100	165	7.3
110	190	7.8
120	230	8.6

Notes 1 For minor-road approach gradients that exceed +3 per cent, increase by the same factor as in Table C.1.

2 Travel time applies to a vehicle that slows before crossing the intersection but does not stop

The length of the leg of the approach sight triangle along the major road to accommodate the crossing manoeuvre (distance ***b*** in Figure C.1.A) should be calculated using the following equations:

$$t_c = t_a + (w + L_a)/(0.167.V_{minor})$$

$$b = 0.278.t_c.V_{major}$$

where:

t_c = travel time to reach and clear the major road in a crossing manoeuvre (sec)

b = length of leg of sight triangle along the major road (m)

t_a = travel time to reach the major road from the decision point for a vehicle that does not stop (sec) (use appropriate value for the minor road design speed from Table C.1, adjusted for approach grade, where appropriate)

w = width of intersection to be crossed (m)

L_a = length of design vehicle (m)

V_{minor}= design speed of minor road (km/h)

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

These equations provide sufficient travel time for the major road vehicle during which the minor-road vehicle can:

1. Travel from the decision point to the intersection, while decelerating at the rate of $1.5 m/s^2$ to 60 per cent of the minor-road design speed; and then
2. Cross and clear the intersection at the same speed.

Field observations did not provide a clear indication of the size of the gap acceptable to the driver of a vehicle located at the decision point on the minor road. If the required gap is longer than that indicated by the above equations, the driver would, in all probability, bring the vehicle to a stop and then select a gap on the basis of Case B. If the acceptable gap is shorter than that indicated by the above equations, the sight distance provided would, at least, provide a margin of safety.

If the major road is a divided highway with a median wide enough to store the design vehicle for the crossing manoeuvre, then only crossing of the near lanes need be considered and a departure sight triangle for accelerating from a stopped position in the median should be provided, based on Case B1.

C3.3.2 Left and right-turn manoeuvres (Case C2)

To accommodate left and right turns without stopping (distance ***a*** in Figure C.1.A), the length of the leg of the approach sight triangle along the minor road should be 25 metres. This distance is based on the assumption that drivers making right or left turns without stopping will slow to a turning speed of 15 km/h. The length of the leg of the approach sight triangle along the major road (distance ***b*** in Figure C.1.B) is similar to that of the major-road leg of the departure sight triangle for Stop-controlled intersections in Cases B1 and B2. For a Yield-controlled intersection, the travel times in Table C.2 should be increased by 0.5 seconds. [The minor-road vehicle requires 3.5 seconds to travel from the decision point to the intersection. These 3.5 seconds represent additional travel time that is needed at a Yield-controlled intersection (Case C). However, the acceleration time after entering the major road is 3.0 seconds less for a Yield sign than for a Stop sign because the turning vehicle accelerates from 15 km/h rather than from a stop condition. The net 0.5 seconds increase in travel time for a vehicle turning from a Yield-controlled approach is the difference between the 3.5 second increase in travel time on approach and the 3.0 second reduction in travel time on departure explained above].

Since approach sight triangles for turning manoeuvres at Yield-controlled sites are larger than the departure sight triangles used at Stop-controlled intersections, no specific check of departure sight triangles at Yield-controlled intersections should be necessary.

C.3.4 Intersections with traffic signal control (Case D)

In general, approach or departure sight triangles are not needed for signalised intersections. Indeed, signalisation may be an appropriate accident countermeasure for higher volume intersections with restricted sight distance and a history of sight-distance related accidents. However, traffic signals may fail from time to time. Furthermore, traffic signals at an intersection are sometimes placed on two-way flashing operation under off-peak or night time conditions. To allow for either of these eventualities, the appropriate departure sight triangles for Case B, both to the left and to the right, should be provided for the minor-road approaches.

C.3.5 Intersections with all-way Stop control (Case E)

At intersections with all-way Stop control, the first stopped vehicle on each approach would be visible to the drivers of the first stopped vehicles on each of the other approaches. It is thus not necessary to provide sight distance triangles at intersections with

All-way Stop control. All-way Stop control may be an option to consider where the sight distance for other types of control cannot be achieved. This is particularly the case if signals are not warranted.

C.3.6 Left turns from a major road (Case F)

Left-turning drivers need sufficient sight distance to enable them to decide when it is safe to turn across the lane(s) used by opposing traffic. At all locations, where left turns across opposing traffic are possible, there should be sufficient sight distance to accommodate these manoeuvres. Since a vehicle that turns left without stopping needs a gap shorter than that required by a stopped vehicle, the need for sight distance design should be based on a left turn by a stopped vehicle. The sight distance along the major road to accommodate left turns is the distance that would be traversed at the design speed of the major road in the travel time for the appropriate design vehicle given in Table C.5. This table also contains appropriate adjustment factors for the number of major-road lanes to be crossed by the turning vehicle.

Table C5: Travel Times Used to Determine the Sight Distance Along the Major Road to Accommodate Right Turns From the Major Road (Case F)

Vehicle	Travel Time (seconds) at Design Speed of the Major Road
Passenger car	5.5
Single unit truck	6.5
Semi trailer	7.5
Multi-lane highways. For left turns that have to cross more than one opposing lane add 0.5 seconds for cars and 0.7 seconds for trucks for each additional lane (in excess of one) to be crossed. In the case of dual carriageways where the median is not sufficiently wide to provide refuge for the turning vehicle, the median should be regarded as another lane to be crossed.	

If stopping sight distance has been provided continuously along the major road and if sight distance for Case B (Stop control) or Case C (Yield control) has been provided for each minor-road approach, sight distance should generally be adequate for left turns from the major road. However, at intersections or driveways located on or near horizontal or vertical curves on the major road, the availability of adequate sight distance for left turns from the major road should be checked. In the case of dual carriageways, the presence of sight obstructions in the median should also be checked.

At four-legged intersections, opposing vehicles turning left can block a driver's view of oncoming traffic. If left-turn lanes are provided, off-setting them to the right, to be directly opposite one other will provide left-turning drivers with a better view of oncoming traffic.

C.4 Effect of skew on sight distance

When two highways intersect at an angle outside the range of 75^o to 120^o and where realignment to increase the angle of intersection is not justified, some of the factors for determination of intersection sight distance will need adjustment. Each of the clear sight triangles described above is applicable to oblique-angle intersections. The legs of the sight triangle will lie along the intersection approaches and each sight triangle will be larger or smaller than the corresponding sight triangle would be at a right-angle intersection. The area within each sight triangle should be clear of sight obstructions, as described above. At skew intersections, the length of the travel paths for crossing manoeuvres will be increased. The actual path length for a crossing manoeuvre can be calculated by dividing the total width of the lanes (plus the median width, where appropriate) to be crossed by the sine of the intersection angle and adding the length of the design vehicle. The actual path length divided by the lane width applied to the major road cross-section gives the equivalent number of lanes to be crossed. This is an indication of the number of additional lanes to be applied to the adjustment factor shown in Table C3 for Case B3.

The sight distances shown for Case B can, regardless of the form of control, also accommodate turning movements from the minor road to the major road at skew intersections. In the obtuse angle, drivers can easily see the full sight triangle and, in addition, often accelerate from the minor road at a higher rate than when they have to negotiate a ninety-degree change of direction. In the acute-angle quadrant, drivers are often required to turn their heads considerably to see across the entire clear sight triangle. For this reason, it is suggested that Case A should not be applied to oblique-angle intersections. Stop or Yield control should be applied and the sight distances appropriate to either Case B or Case C provided. Even in a skew intersection it is usually possible for drivers to position their vehicles at approximately 90^{o} to the major road at the Stop line, offering added support for the application of Case B for skew intersections. When driving through a deflection angle greater than 120^{o}, the right turn to the minor road may be undertaken at crawl speeds. Allowance could be made for this by adding the time, equivalent to that required for crossing an additional lane, to the acceptable gap.

Appendix D MEASURING AND RECORDING SIGHT DISTANCE ON PLANS

By determining graphically the sight distances on the plans and recording them at frequent intervals, the designer can appraise the overall layout and provide a more balanced design by minor adjustments in the plan of profile. Once the horizontal and vertical alignments are tentatively established, the practical means of examining sight distances along the proposed highway is by direct scaling on the plans. Methods for scaling sight distances are demonstrated in Figure D-1. The figure also shows a typical sight distance record that could be shown on the final plans.

Because the view of the highway ahead may change rapidly in a short distance, it is desirable to measure and record sight distance for both directions of travel at each station. Both horizontal and vertical sight distances should be measured and the shorter lengths recorded. In the case of two-lane highways, passing sight distance in addition to stopping sight distance should be measured and recorded.

Horizontal sight distance on the inside of a curve is limited by obstructions such as buildings, hedges, wooded areas, high ground, or other topographic features. These generally are plotted on the plans. Horizontal sight is measured with a straightedge, as indicated at the upper left in Figure D-1. The cut slope obstruction is shown on the worksheets by a line representing the proposed excavation slope at a point 600 millimeters (approximate average of 1070 millimeters and 150 millimeters) above the road surface for stopping sight distance and at a point about 1100 millimeters above the road surface for passing sight distance. The position of this line with respect to the centerline may be scaled from the plotted highway cross sections. Preferably, the stopping sight distance should be measured between points on the one traffic lane, and passing sight distance from the middle of one lane to the middle of the other lane. Such refinement on two-lane highways generally is not necessary and measurement to the centerline or traveled way edge is suitable. Where there are changes of grade coincident with horizontal curves that have sight-limiting cuts slopes on the inside, the line-of-sight intercepts the slope at a level either lower or higher than the assumed average height. In measuring sight distance the error in the use of the assumed 600-or 1100-millimeters height usually can be ignored.

Vertical sight distance may be scaled from a plotted profile by the method illustrated at the right center of Figure D-1. A transparent strip with parallel edge 1300 millimeters apart and with scratched lines 150 millimeters and 1070 millimeters from the upper edge, in accordance with the vertical scale, is a useful tool. The 1070-millimeter line is placed on the station from which the vertical sight distance is desired, and the strip is pivoted about this point until the upper edge is tangent to the profile. The distance between the initial station and the station on the profile intersected by the 150 millimeters line is the stopping sight distance. The distance between the initial station and the station on the profile intersected by the lower edge of the strip is the passing sight distance.

A simple sight distance record is shown in the lower part of Figure D-1. Sight distances in both directions are indicated by arrows and figures at each station on the plan and profile sheet of the proposed highway. Sight distance less than 500 m may be scaled to the nearest 10 meters and those greater than 500 meters to the nearest 50 meters. Sight distances can easily be determined also where plans and profiles are drawn using computer-aided design and drafting systems (CADD), although such programs presently do not automatically make this determination.

Sight distance records for two-lane highways may be used to advantage to tentatively determine the marking of no-passing zones. No-passing zones thus established serve as a guide for markings when the highway is completed. The zone so determined should be checked and adjusted by field measurements before actual markings are placed.

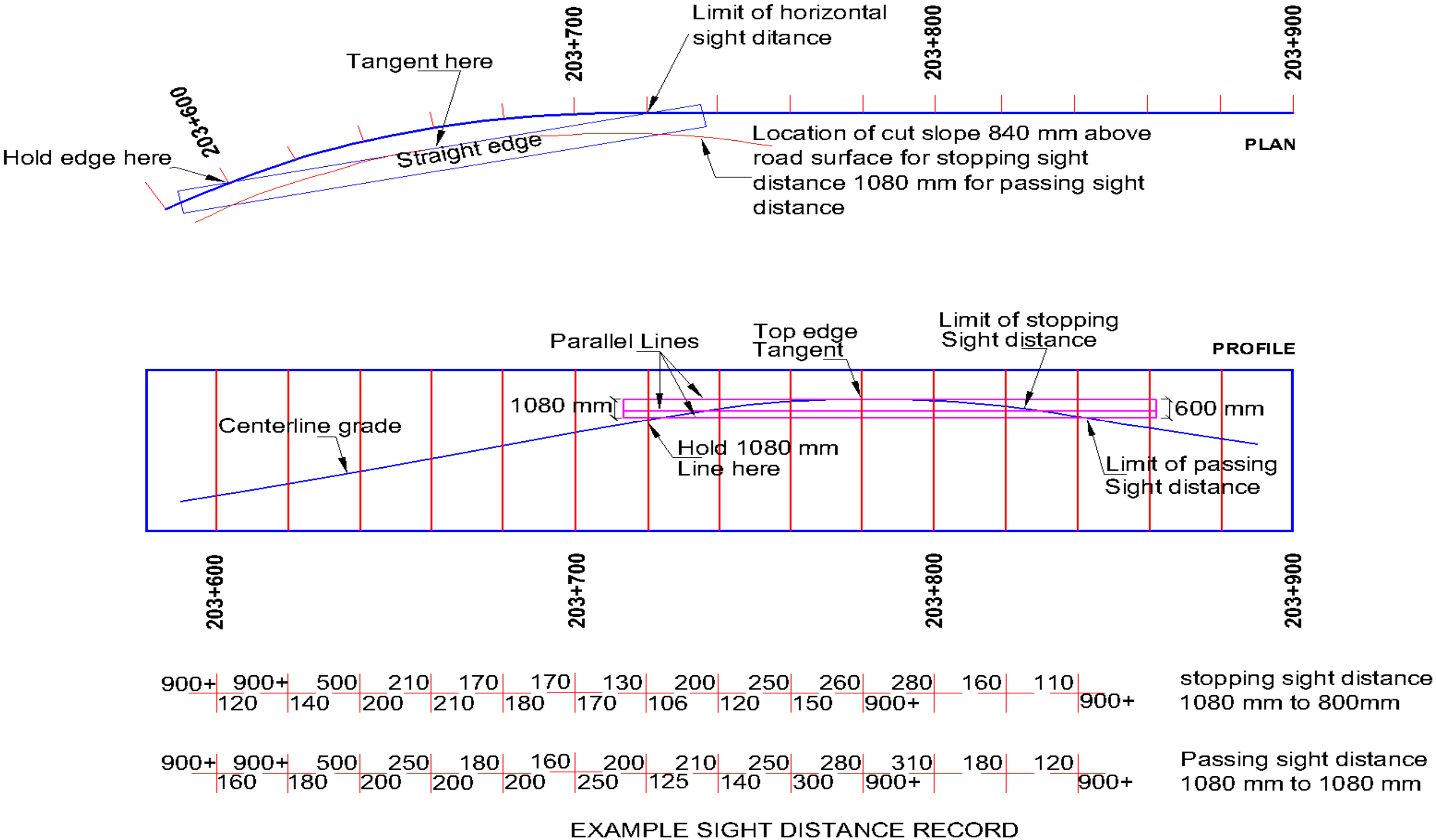


Figure D-1: Scaling and Recording Sight Distances on Plans

Appendix E UTILITIES

E.1 Ethiopian Electric Light and Power Authority

The Ethiopian Electric Light and Power Authority Distribution Manual, dated January 1975, shows the following parameters:

Minimum vertical clearance of wires above roads:

The manual generally gives no guidance on the placement of the utility within the road reserve. A drawing shows that street lighting is to be placed 1 meter behind a curb on a main road. The manual does not state placement for un-curbed roads, and such placement should not be allowed. Similarly, power poles are placed 60 cm behind curbs; in rural areas, the poles shall be placed at least 15 meters from the center of the road.

For underground cables crossing the road, unprotected cables are placed 80 cm below a gravel road; or cables encased with concrete pipe are placed 100 cm below an asphalt road or paved sidewalk. The EELPA shows vertical clearances as per Table E-1.

Table E-1: Minimum Vertical Clearance of Wires above Road Surface in Centimetres

Type of Location	Guys, Messengers, Etc.	Open-Supply Wires, Voltage to Ground		
		0 to 750	750 to 15,000	15,000 to 50,000
When crossing above:				
Railroads	820	820	850	910
Streets, alleys, and roadways	550	550	610	670
Private driveways	310	310	610	670
Walkways for pedestrians only	240	460	460	520
When wires are along:				
Streets or alleys	550	550	610	670
Roads in rural districts	430	460	550	610

E.2 Addis Ababa Water and Sewerage Authority

The Addis Ababa Water and Sewerage Authority reports that they generally follow the proposals indicated in the Master Plan for pipe laying. This consists of laying pipes under the sidewalks. However, in situations where the sidewalk is narrow or does not exist, pipes are laid in the median, if there is one, or in the edge of the asphalt road.

For pipes up to 150 millimeters diameter, pipes are laid at a depth as indicated in Table E-2. For larger pipes, the depth is generally greater than 1.5 meters.

Table E-2: Trench Dimensions for Water and Sewerage Pipe Culverts

Diameter (mm)	Depth (cm)	Width cm)	Bedding thickness (cm)
150	85	70	10
125	72.5	70	10
100	70	50	10
75	57.5	40	-
≤50	55	40	-

E.3 Ethiopian Telecommunications Corporation

The Ethiopian Telecommunications Corporation reports that for directly buried cable installation, the ETC uses a depth of 1.0-1.2 meters for primary cable and 0.6-0.8 meters for secondary cable. ETC does not have a standard location plan within the road right-of-way at the time of the preparation of this manual.

Appendix F TYPICAL CROSS SECTIONS AND STANDARD CROSS SECTIONS

Standard Cross Sections for road classes from DC1 to DC8 are shown in Figures F-1 to F-8 and typical town sections from F-9 to F-13.

Note that many towns have master plans that will show a slightly different configuration from the town sections shown. In such cases, our typical section should represent the minimal requirements.

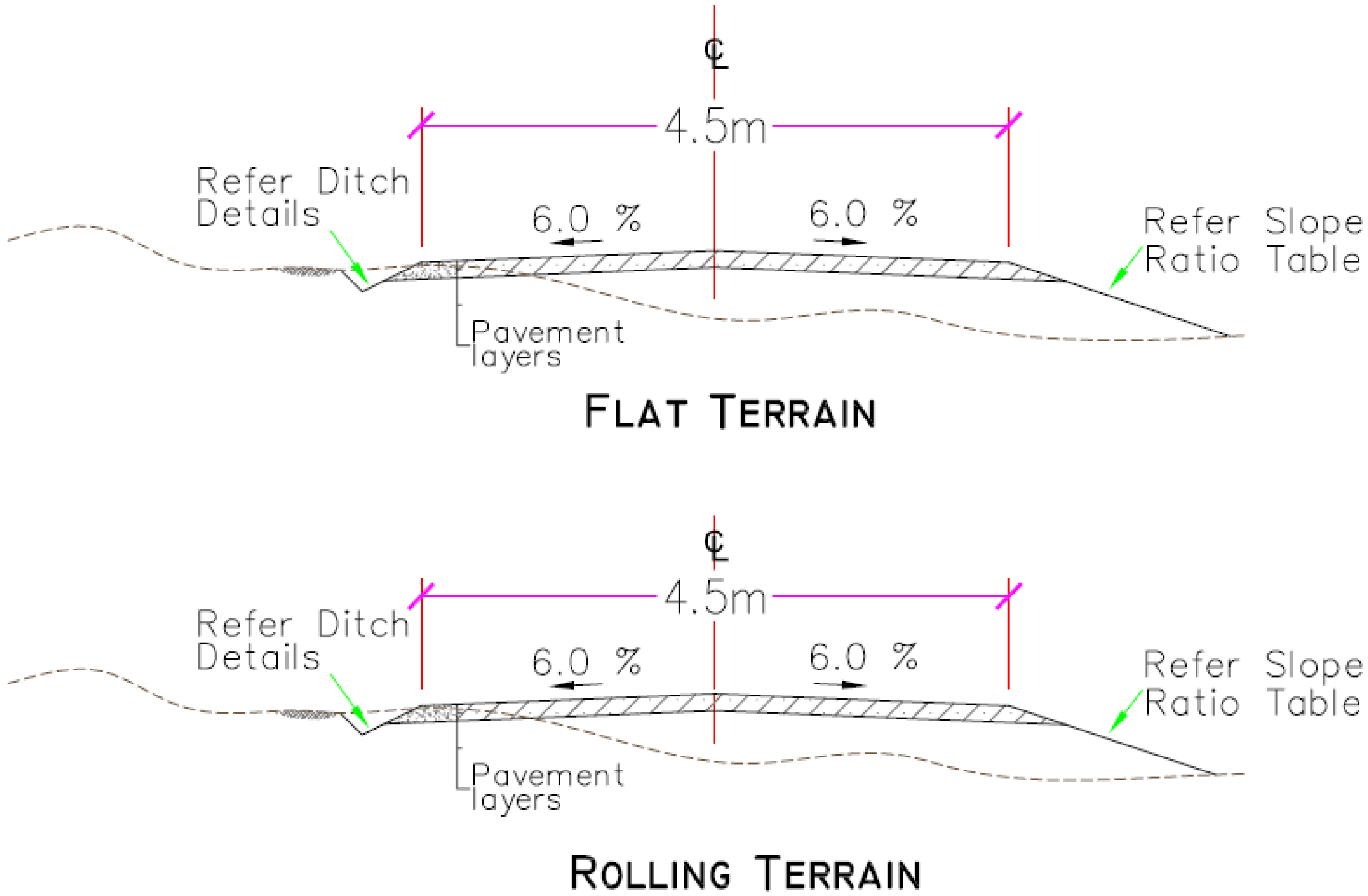
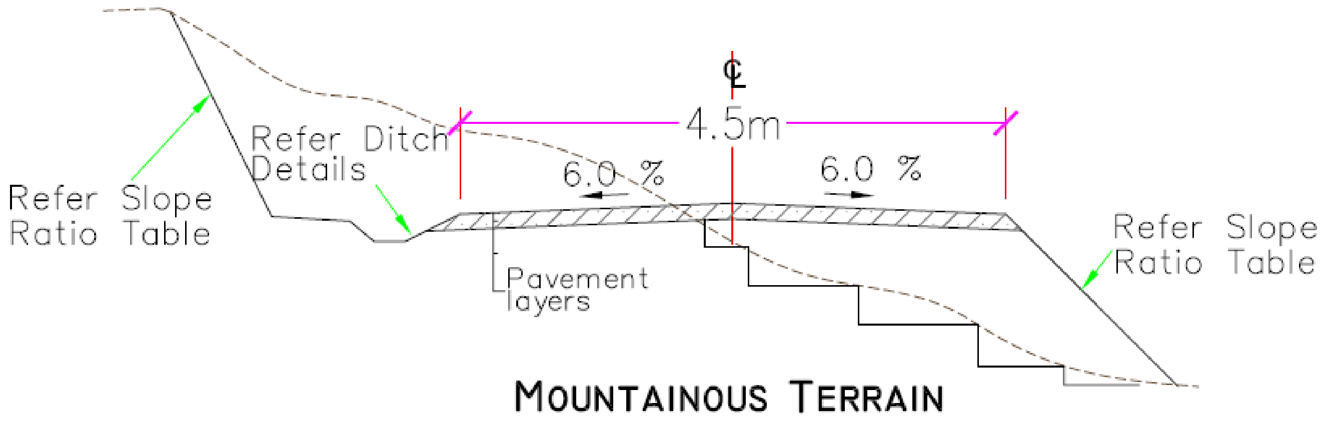


Figure F-1(a): DC1 Unpaved Flat and Rolling Terrain Typical Cross Section



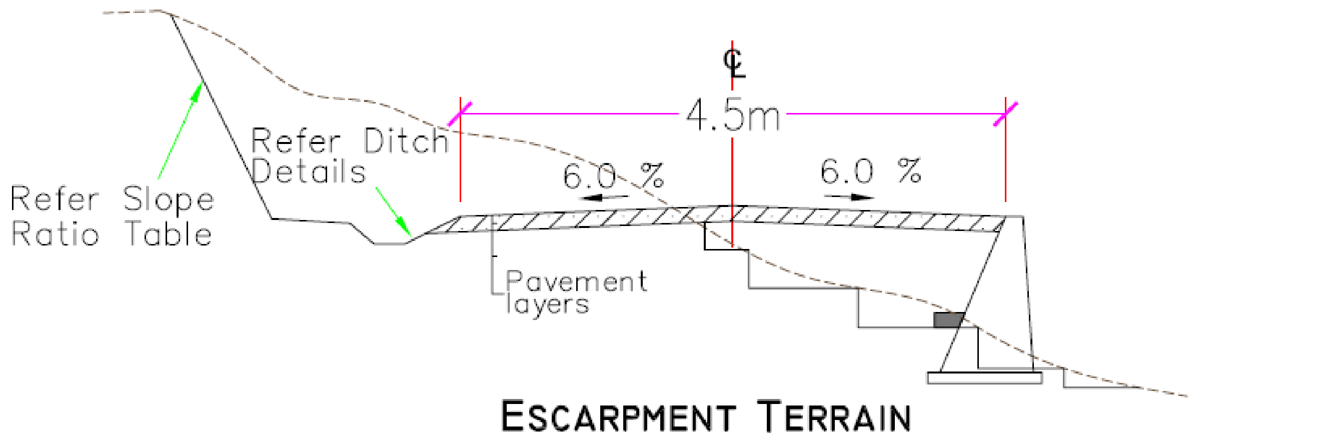


Figure F-1(b): DC1 Unpaved Mountainous and Escarpment Terrain Typical Cross Section

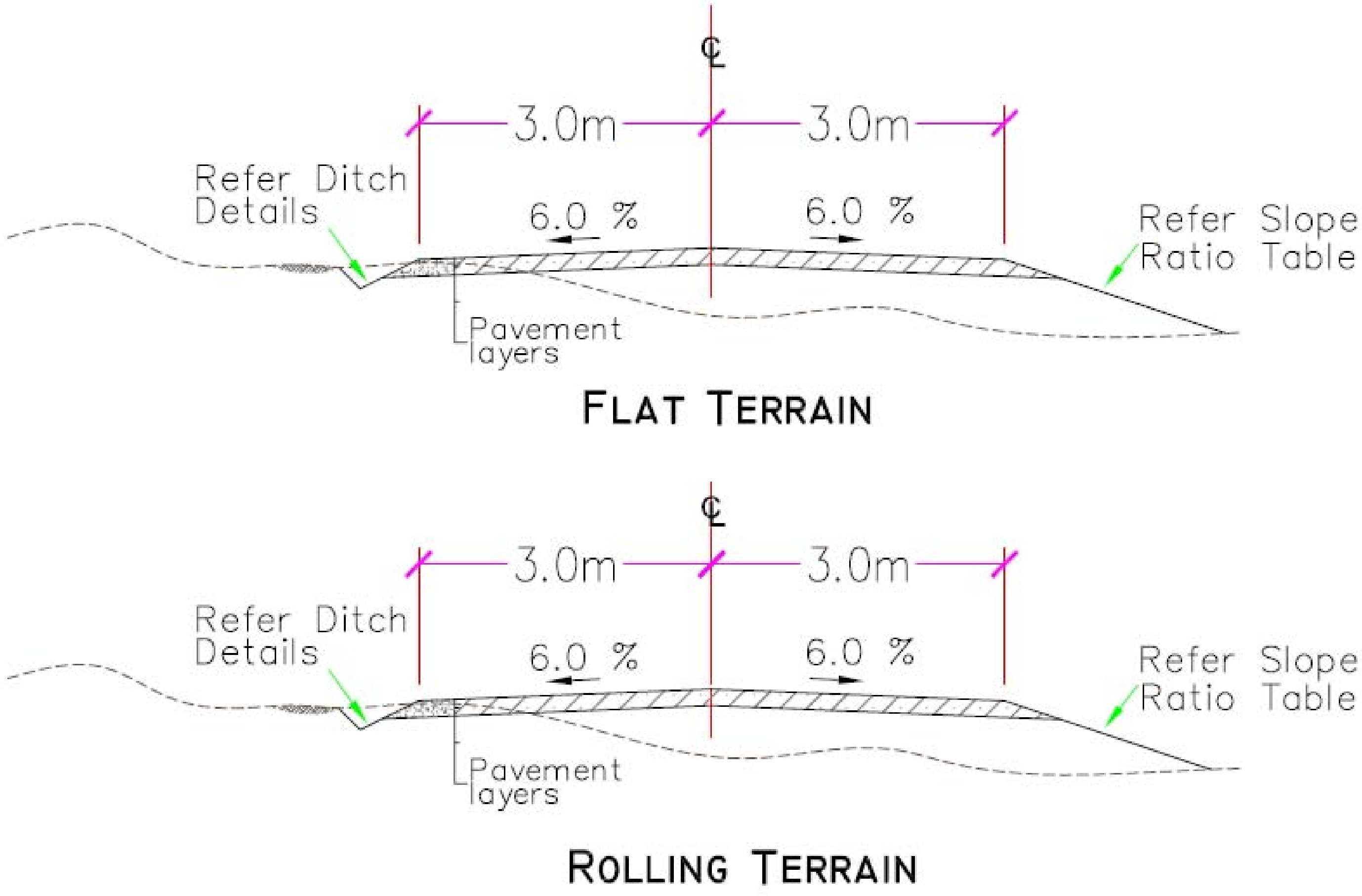


Figure F-2(a): DC2 Unpaved Flat and Rolling Terrain Typical Cross Section

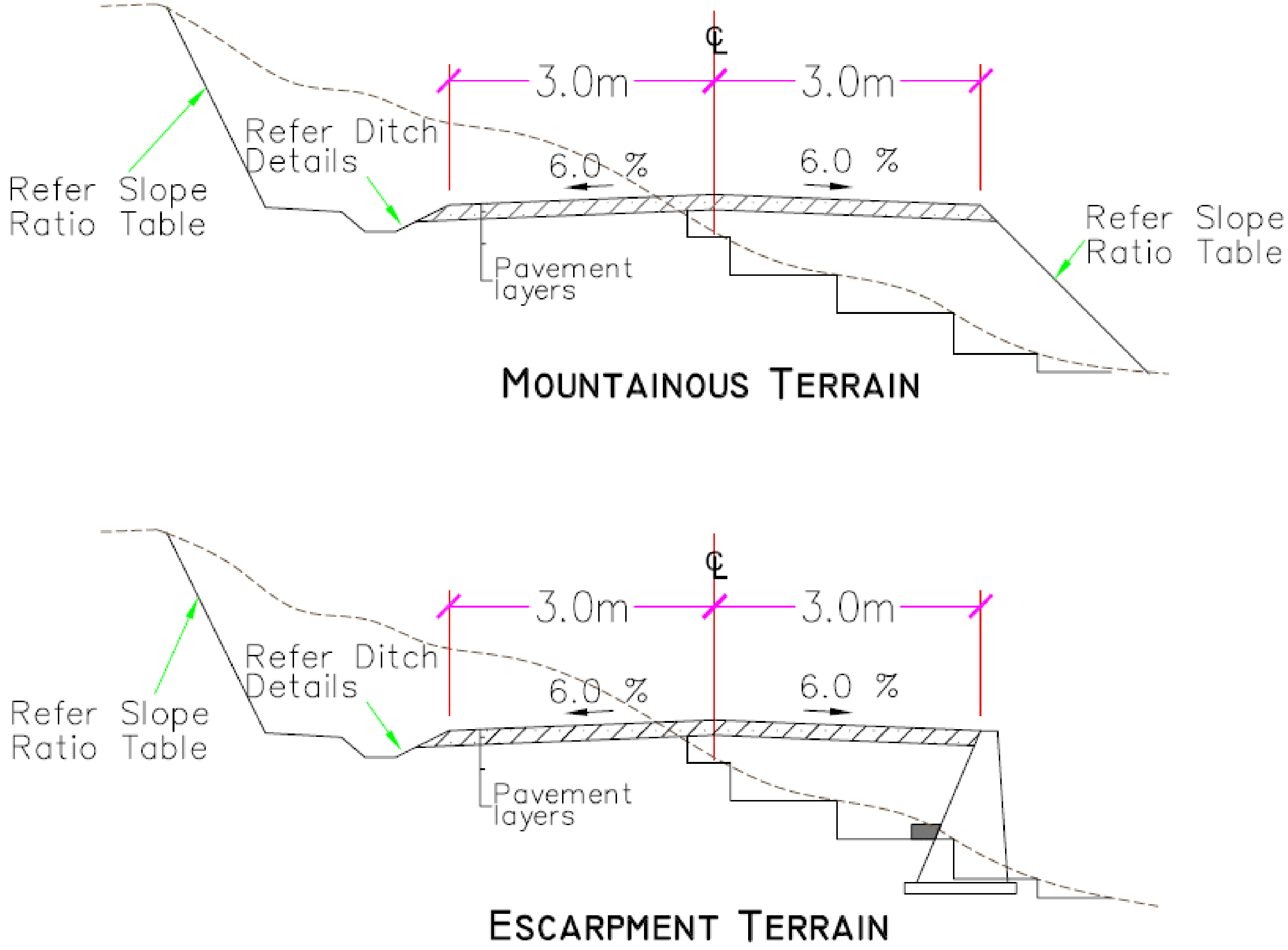
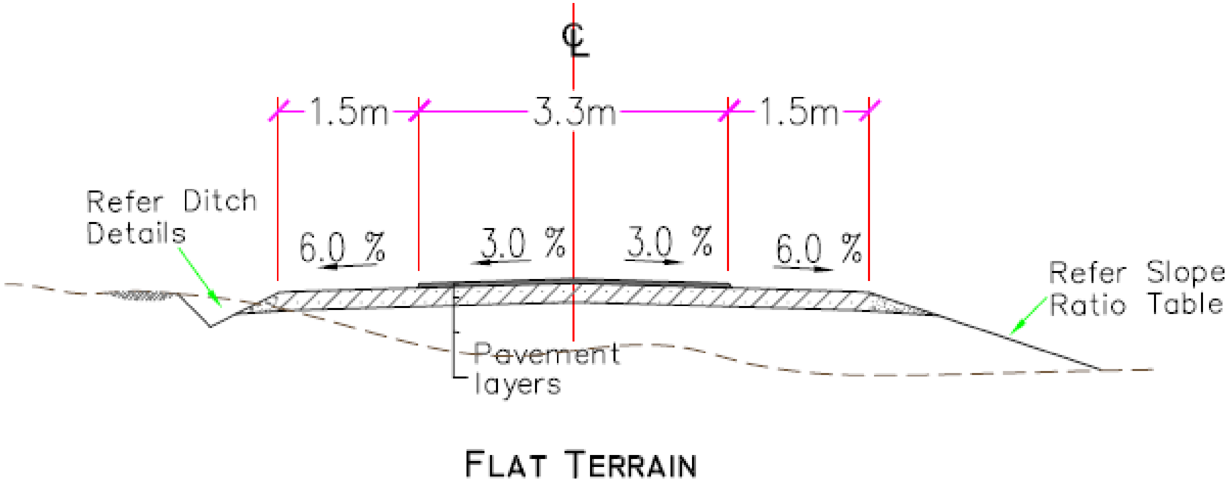


Figure F-2(b): DC2 Unpaved Mountainous and Escarpment Terrain Typical Cross Section



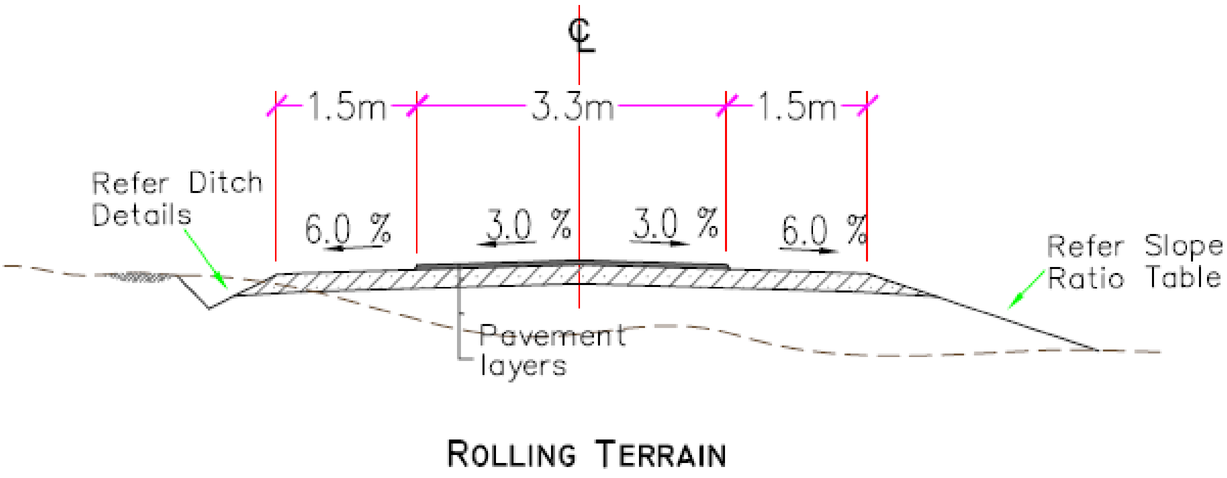


Figure F-2(c): DC2 Paved Flat and Rolling Terrain Typical Cross Section

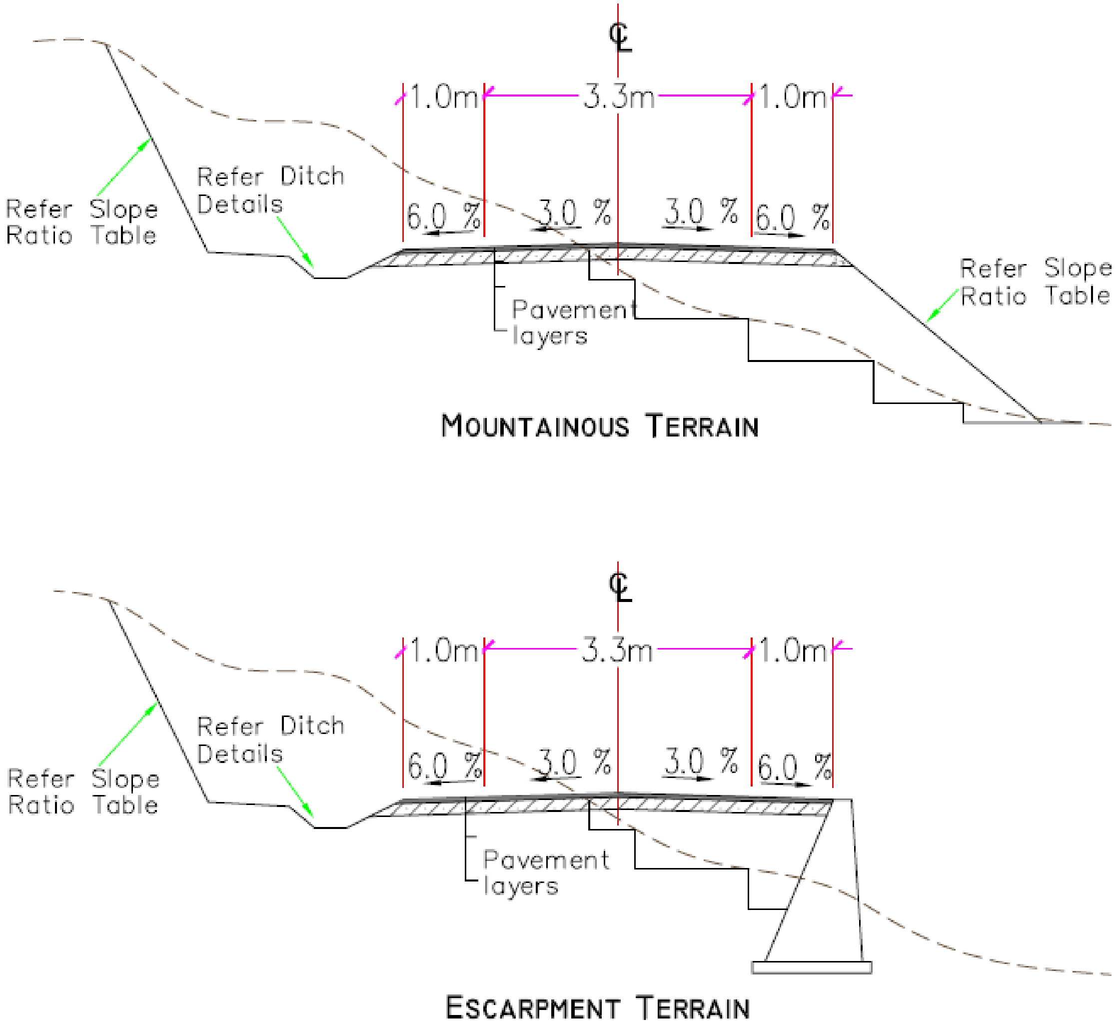


Figure F-2(d): DC2 Paved Mountainous and Escarpment Terrain Typical Cross Section

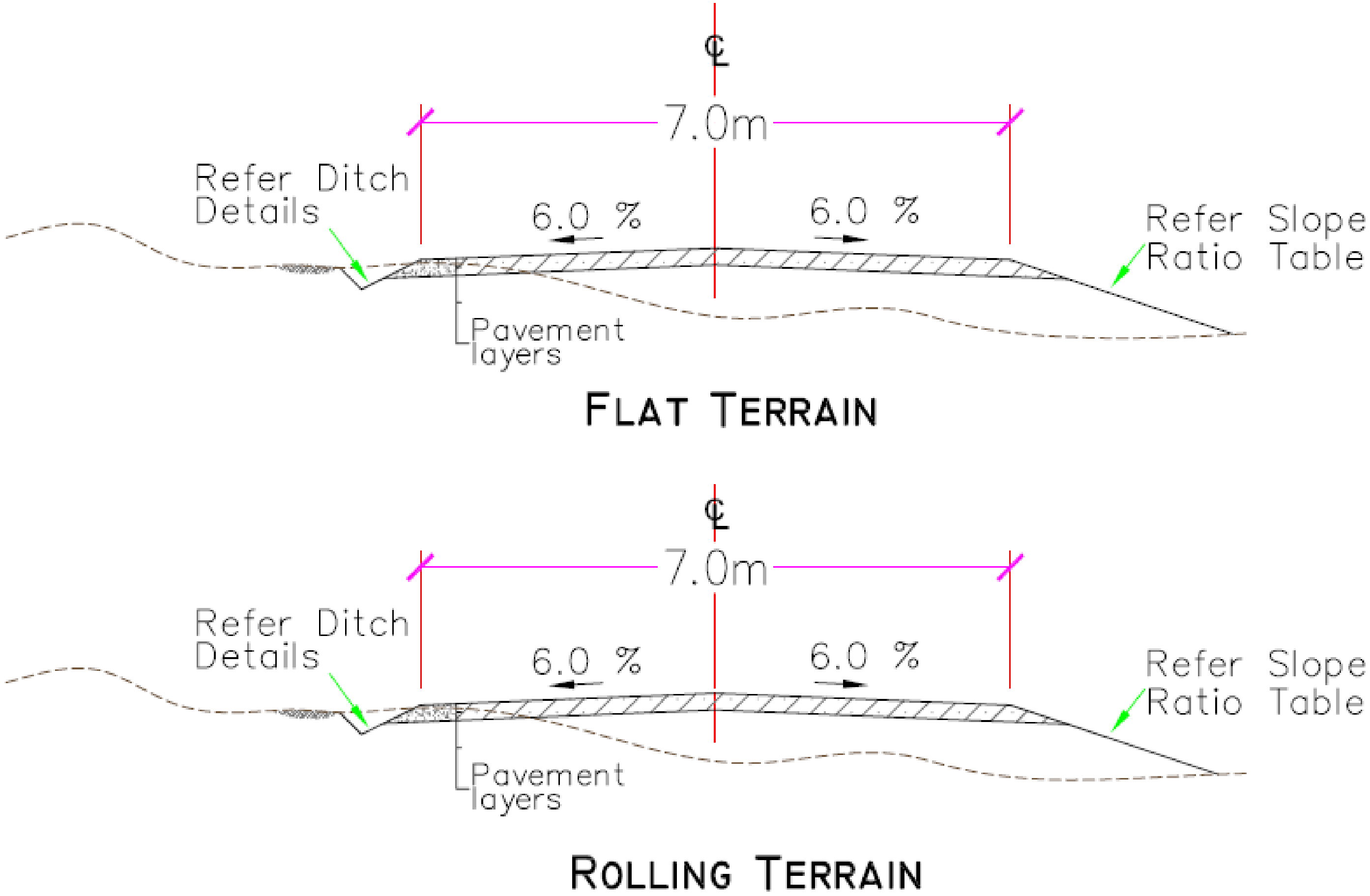


Figure F-3(a): DC3 Unpaved Flat and Rolling Terrain Typical Cross Section

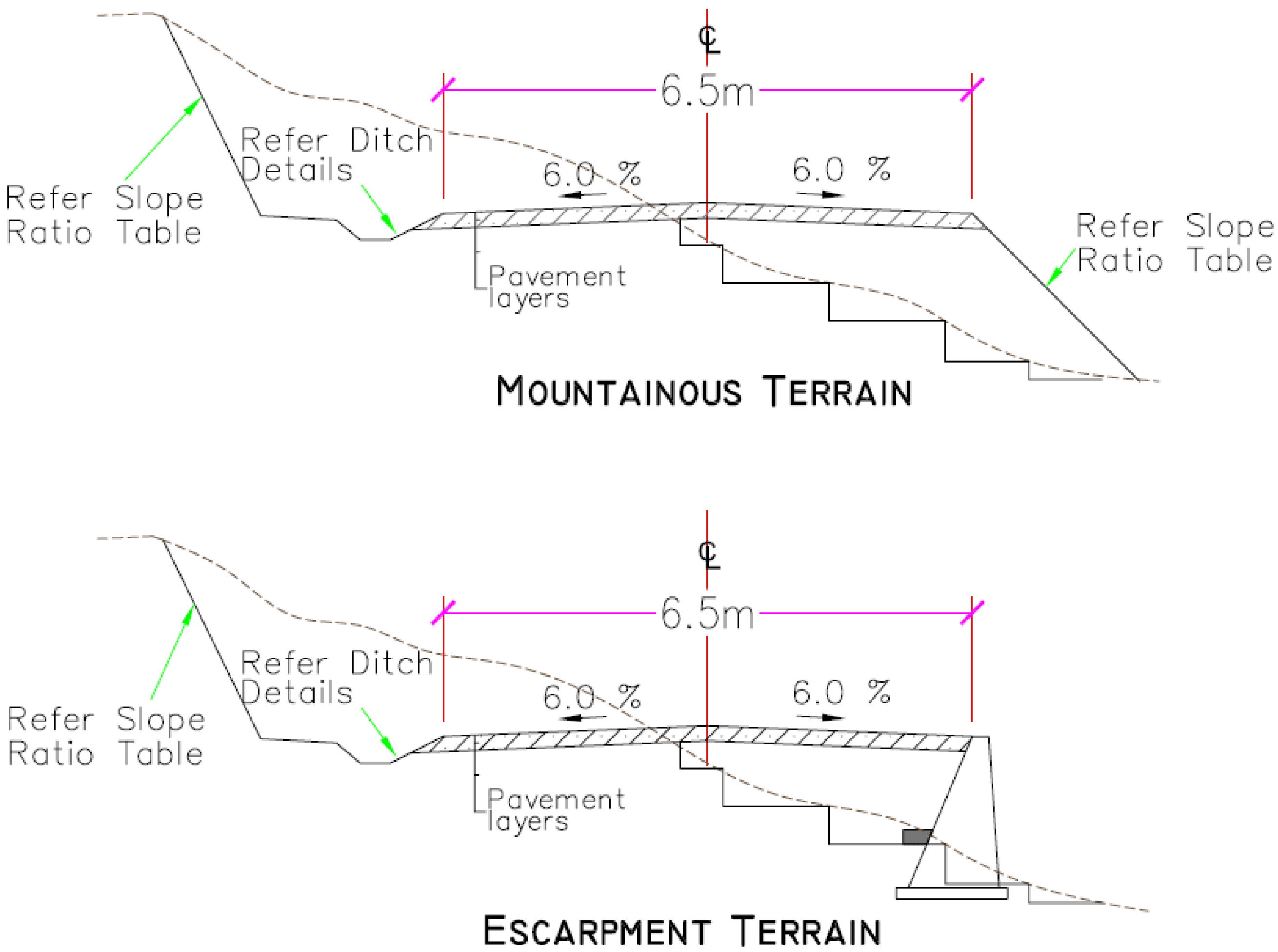
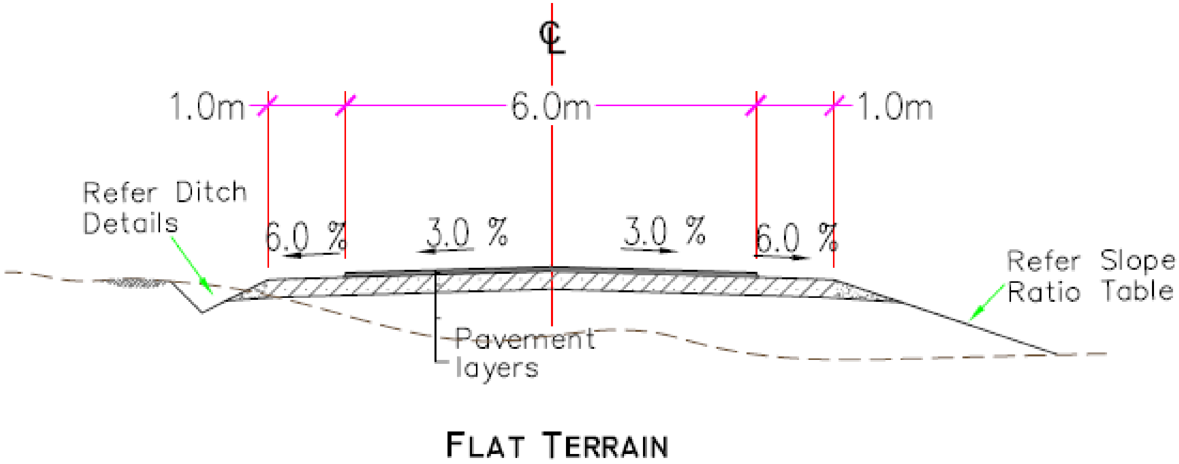


Figure F-3(b): DC3 Unpaved Mountainous and Escarpment Terrain Typical Cross Section



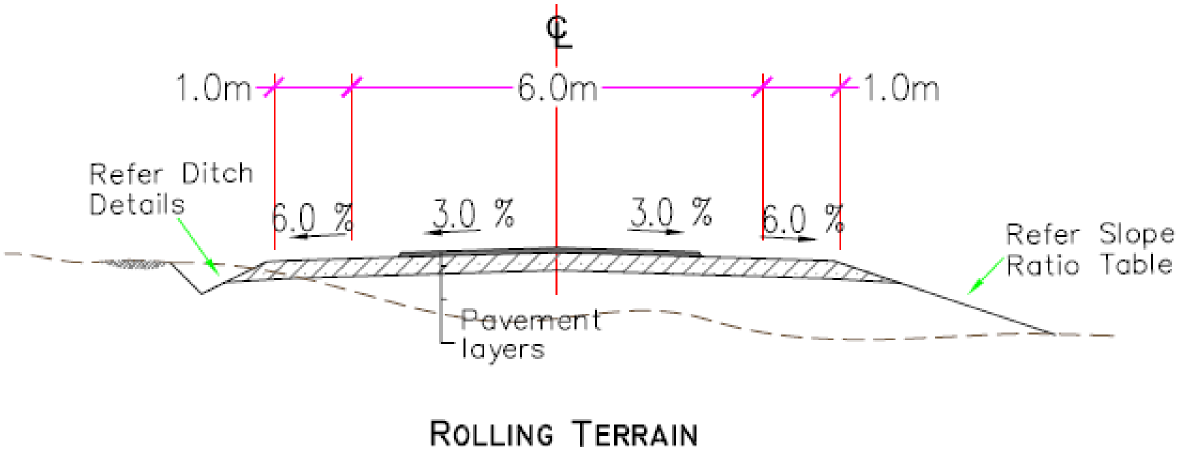
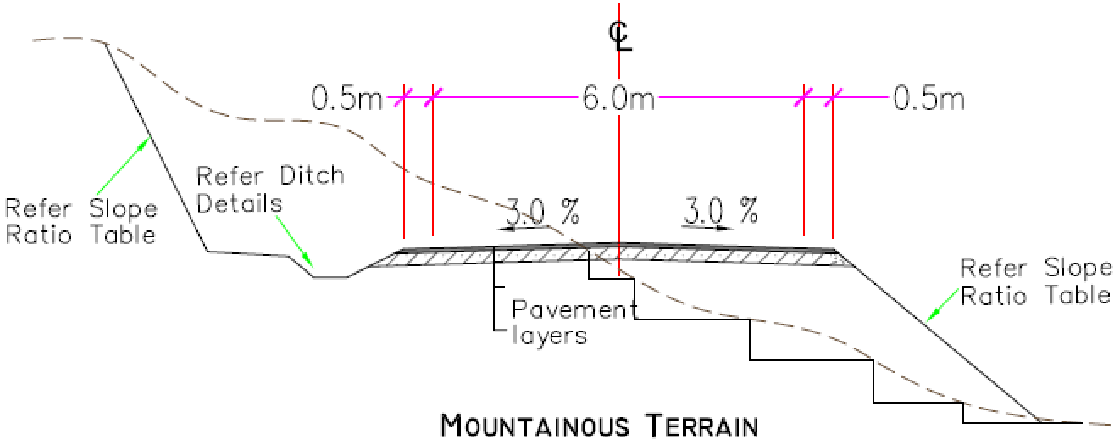


Figure F-3(c): DC3 Paved Flat and Rolling Terrain Typical Cross Section



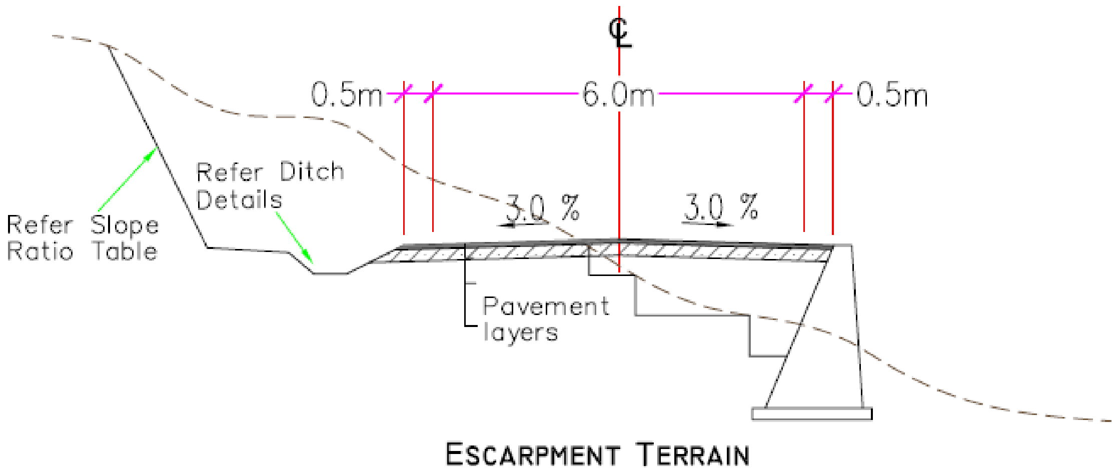


Figure F-3(d): DC3 Paved Mountainous and Escarpment Terrain Typical Cross Section

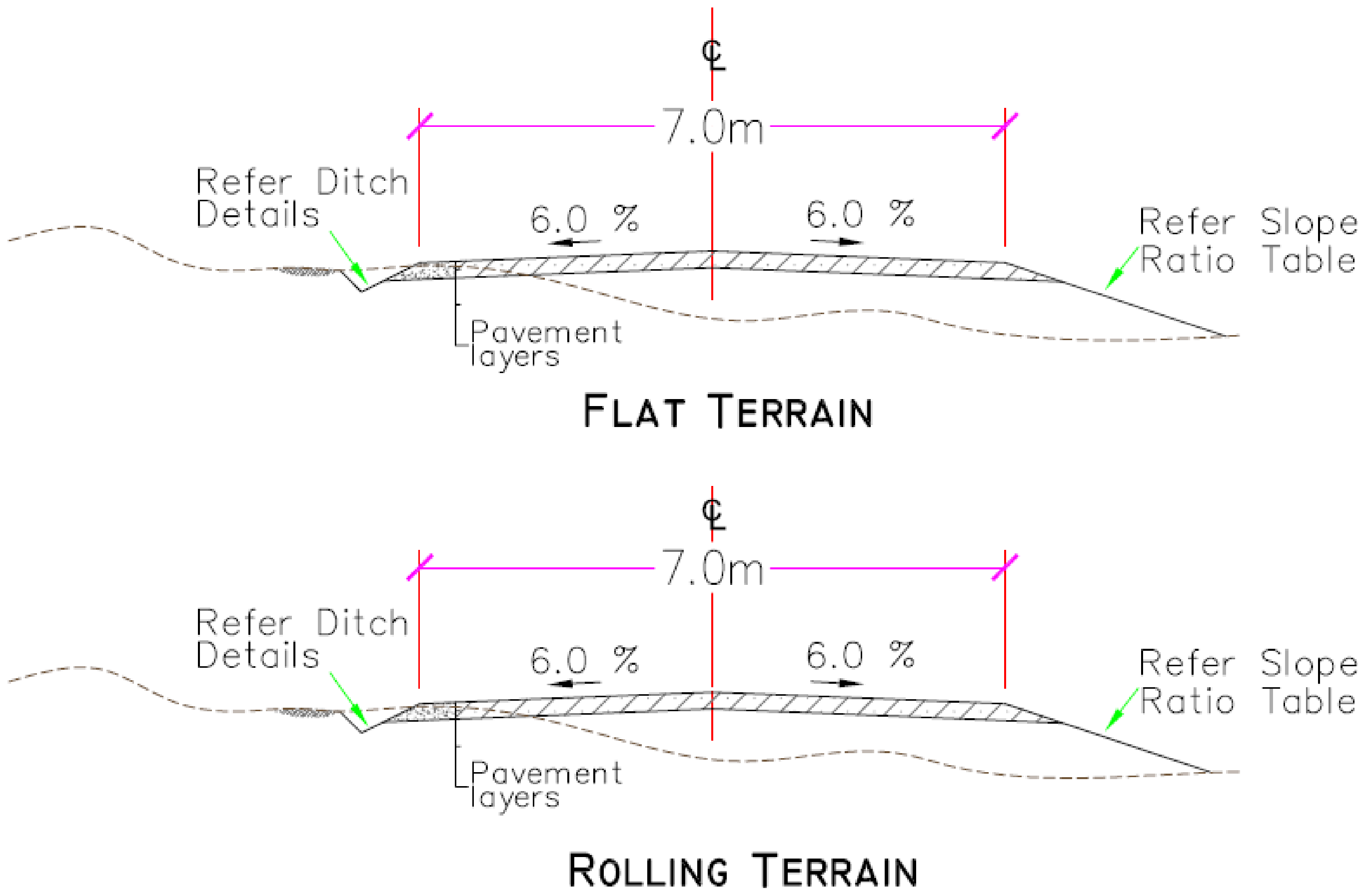
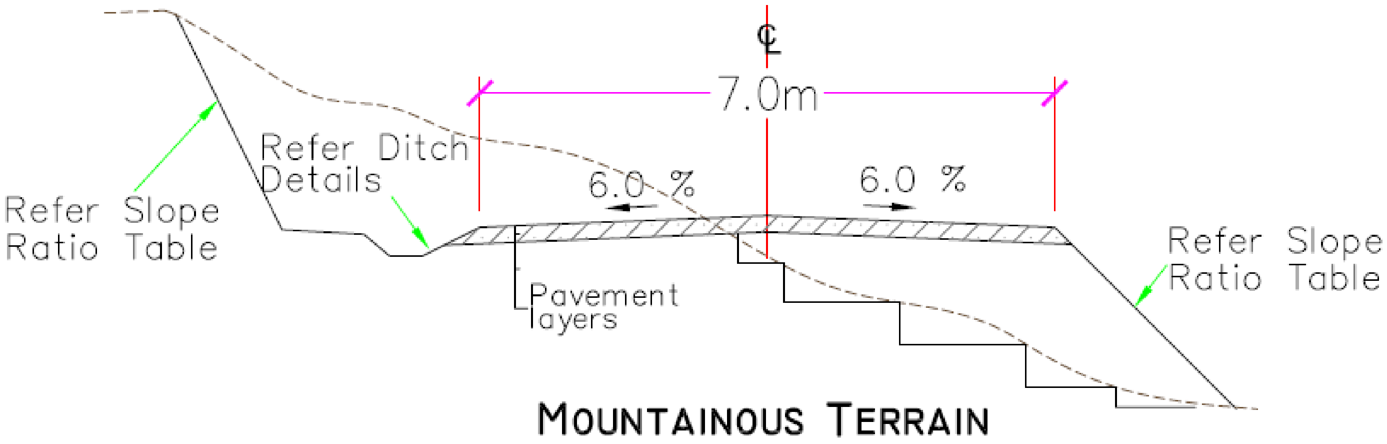


Figure F-4(a): DC4 Unpaved Flat and Rolling Terrain Typical Cross Section



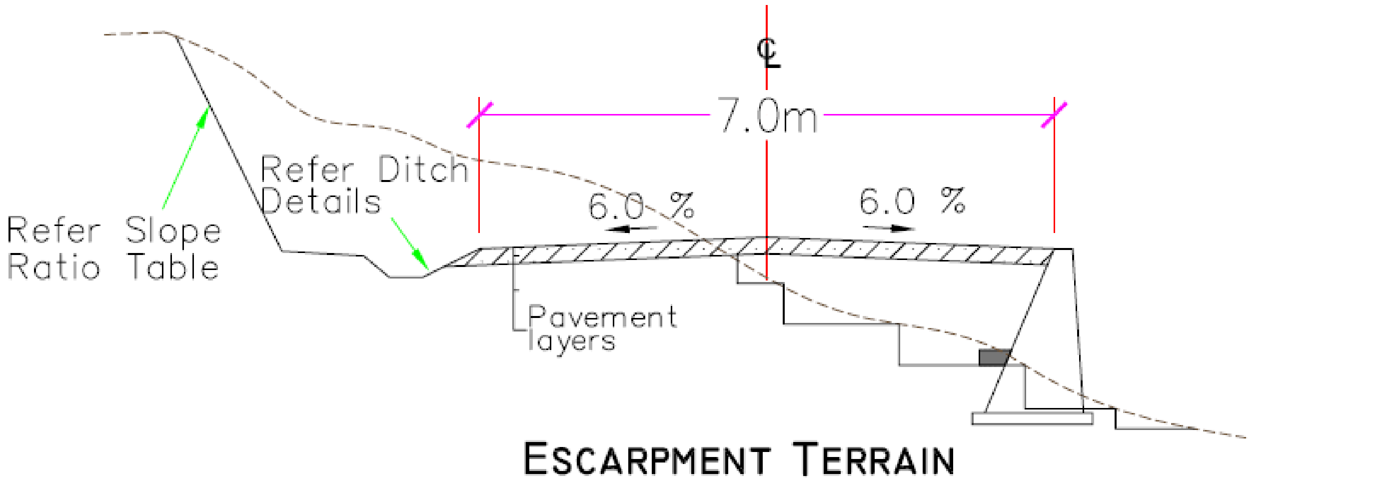
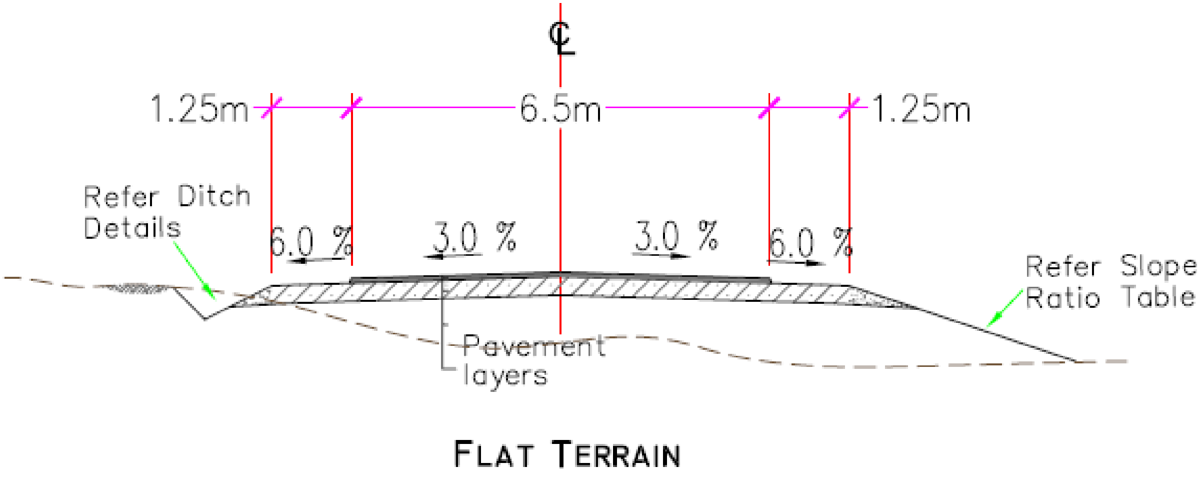


Figure F-4(b): DC4 Unpaved Mountainous and Escarpment Terrain Typical Cross Section



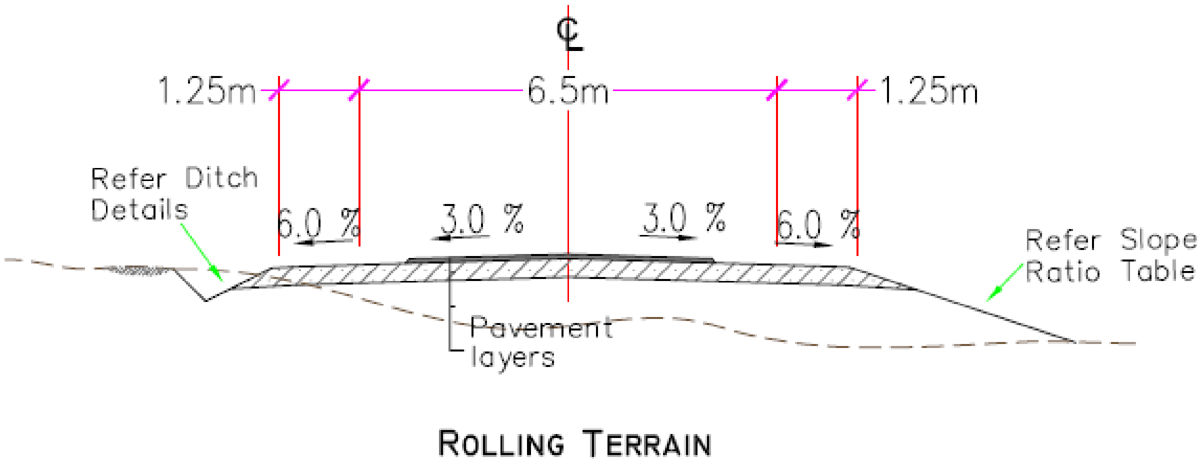
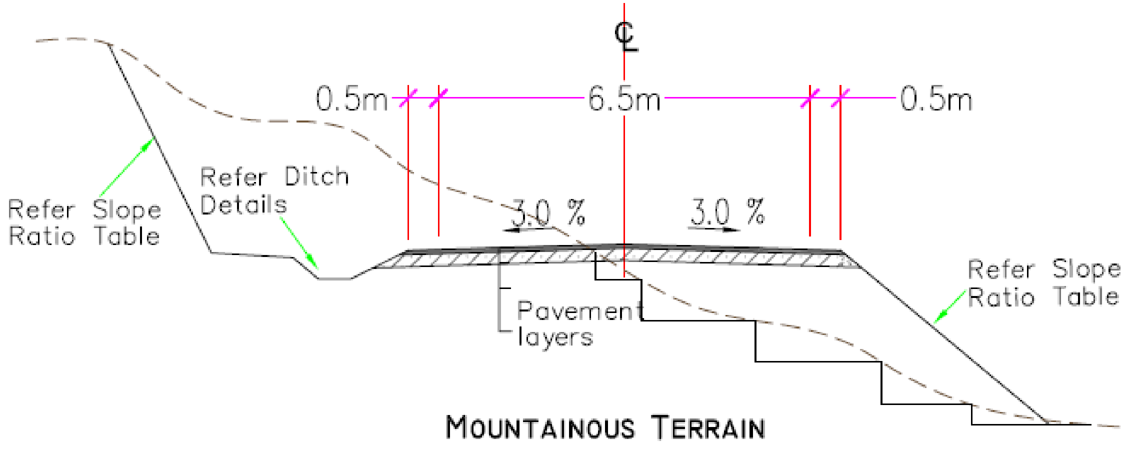


Figure F-4(c): DC4 Paved Flat and Rolling Terrain Typical Cross Section



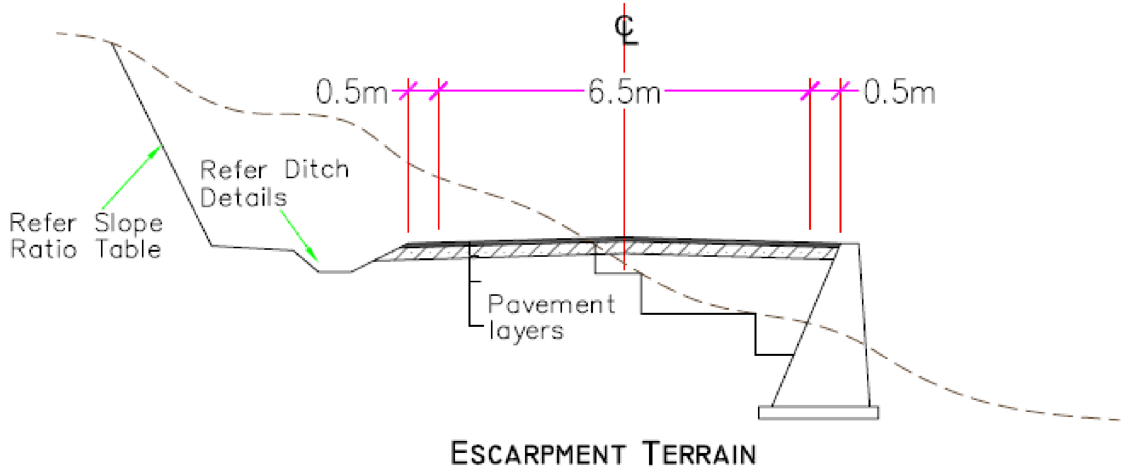


Figure F-4(d): DC4 Paved Mountainous and Escarpment Terrain Typical Cross Section

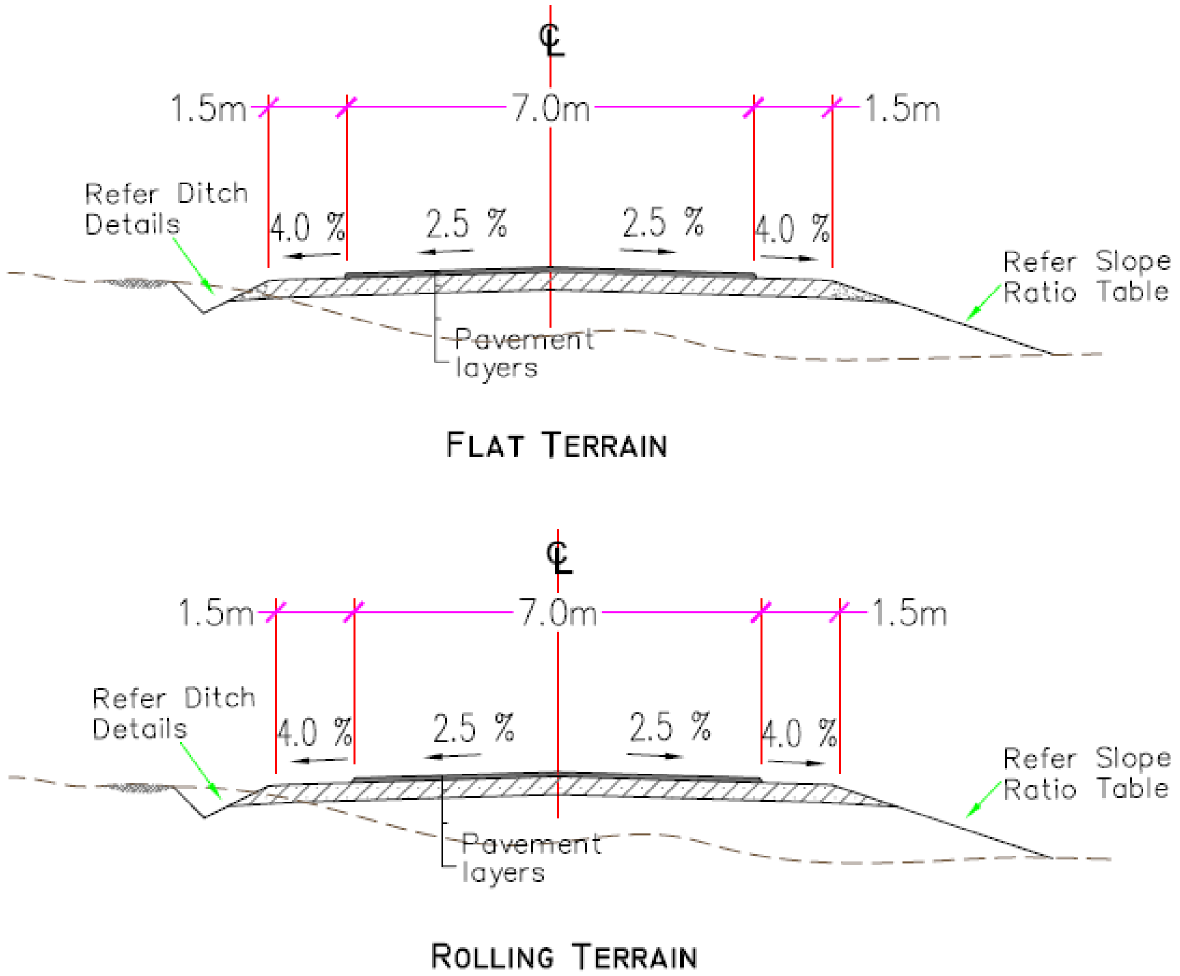
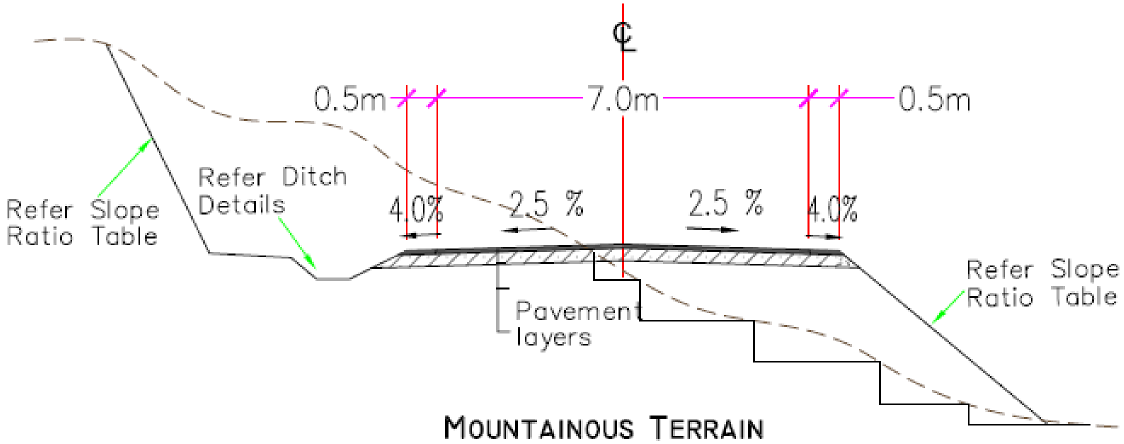


Figure F-5(a): DC5 Paved Flat and Rolling Terrain Typical Cross Section



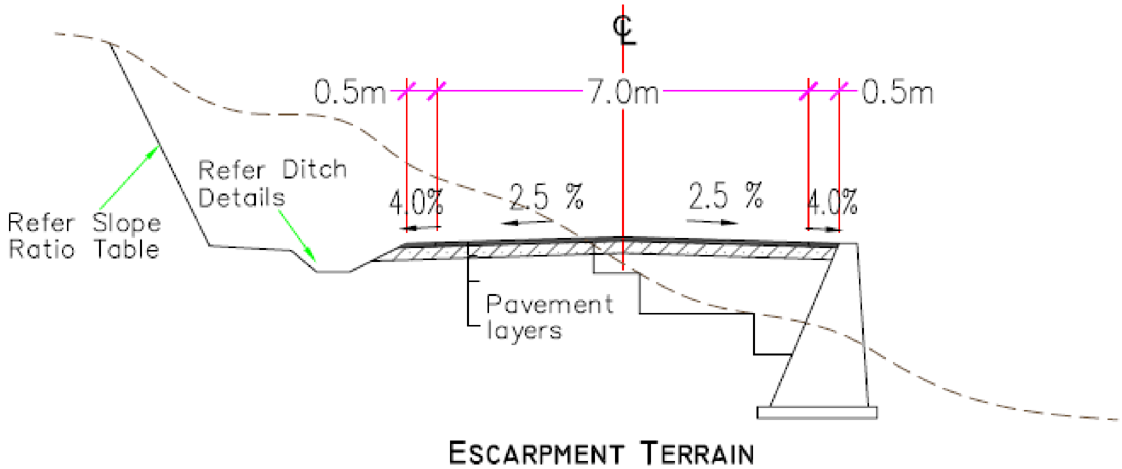
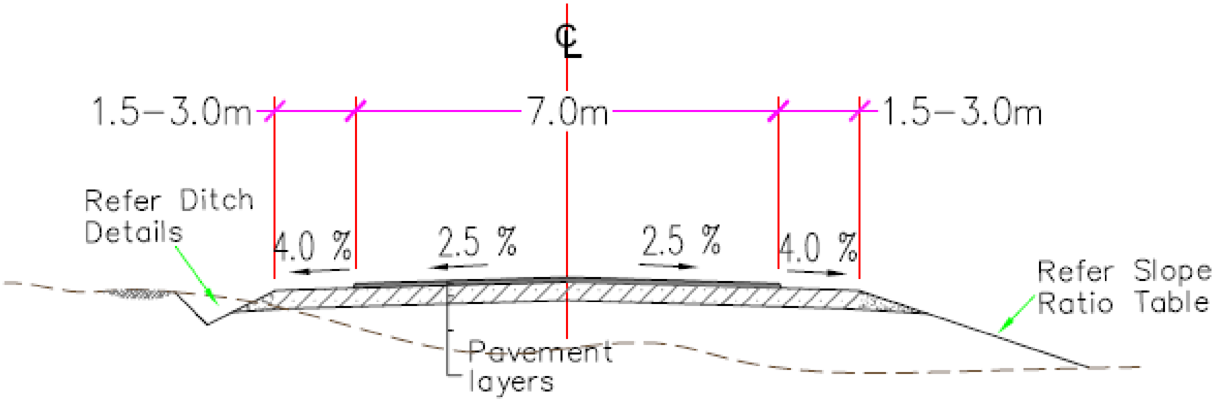
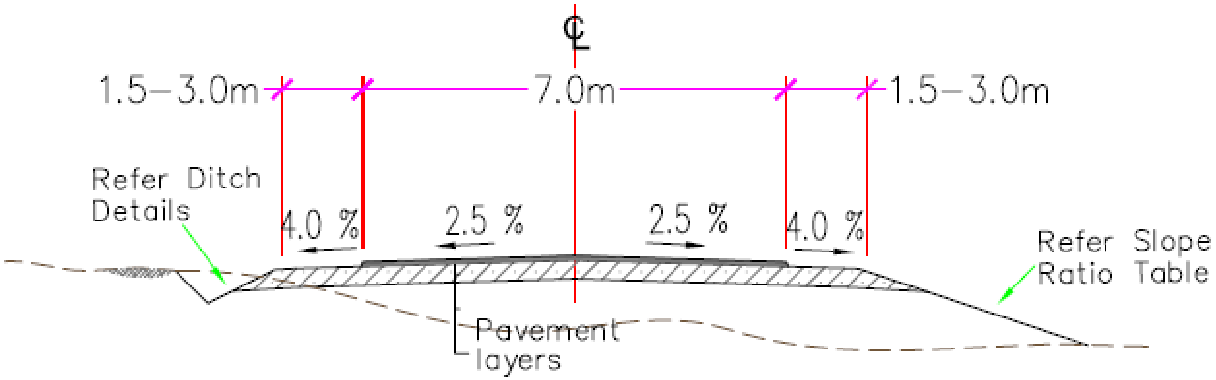


Figure F-5(b): DC5 Paved Mountainous and Escarpment Terrain Typical Cross Section



FLAT TERRAIN



ROLLING TERRAIN

Figure F-6(a): DC6 Paved Flat and Rolling Terrain Typical Cross Section

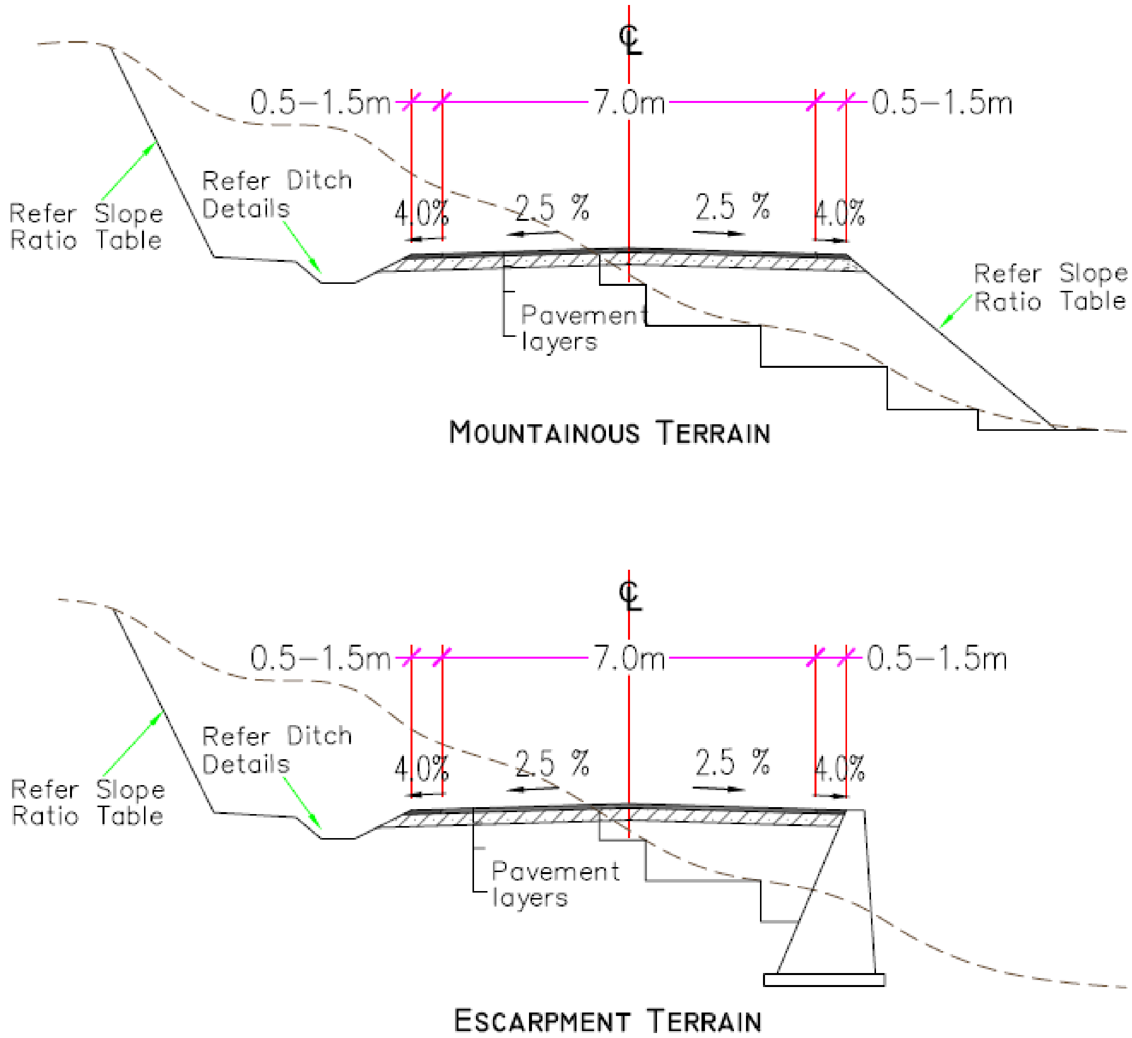
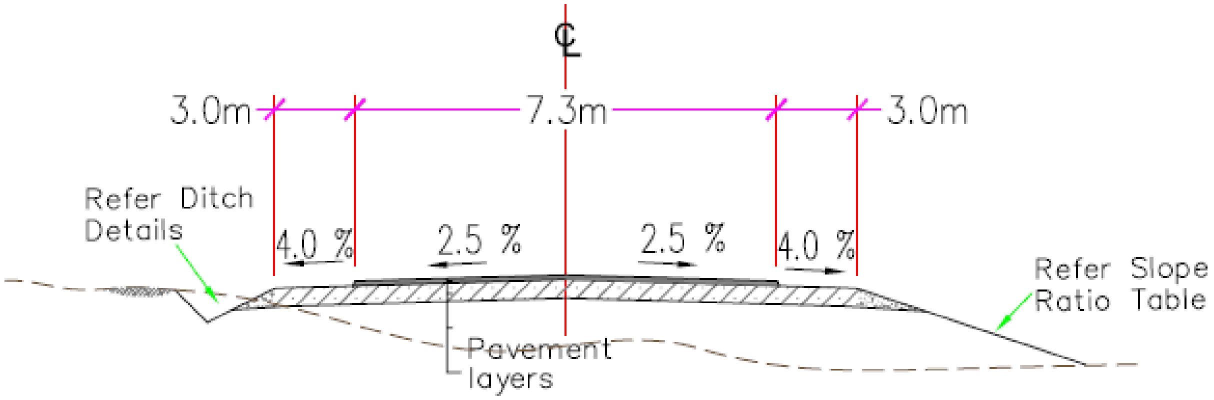
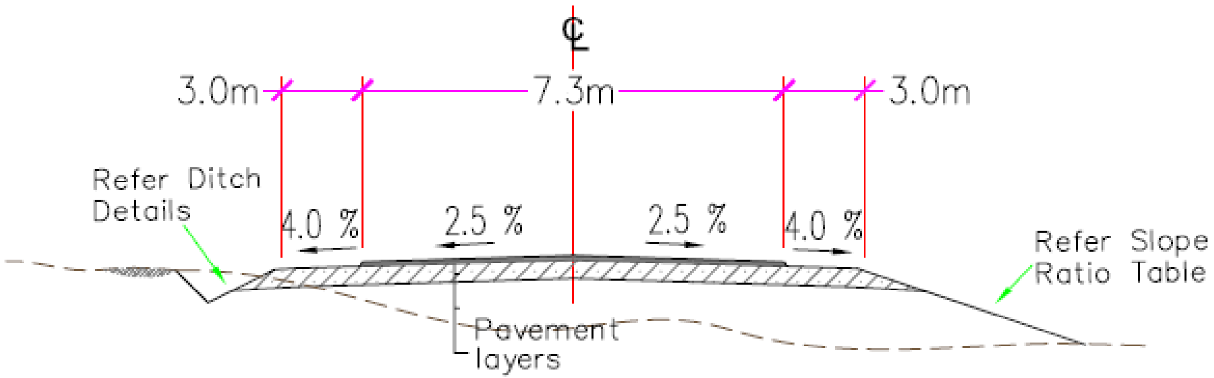


Figure F-6(b): DC6 Paved Mountainous and Escarpment Terrain Typical Cross Section

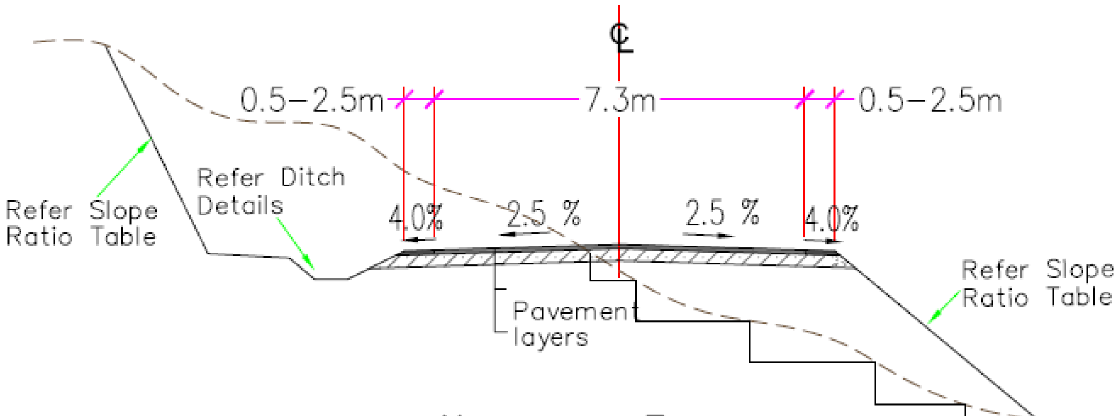


FLAT TERRAIN



ROLLING TERRAIN

Figure F-7(a): DC7 Paved Flat and Rolling Terrain Typical Cross Section



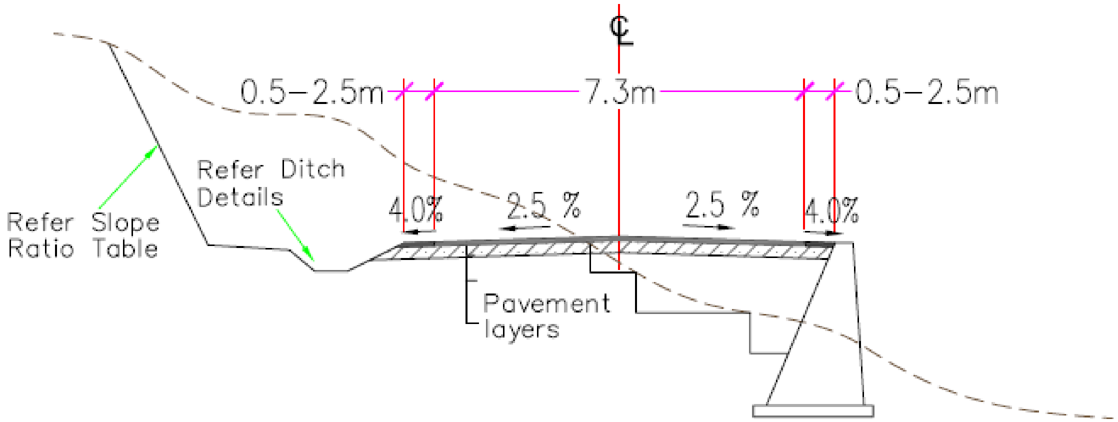
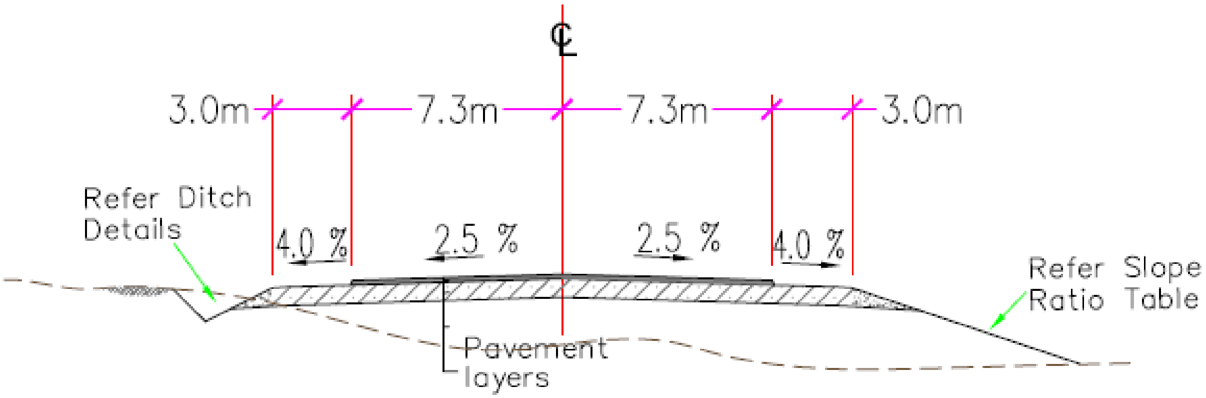
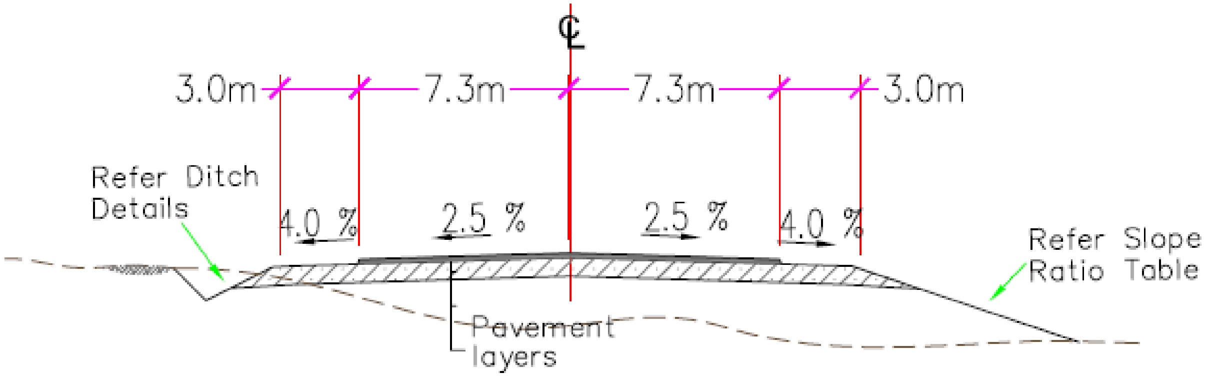


Figure F-7(b): DC7 Paved Mountainous and Escarpment Terrain Typical Cross Section

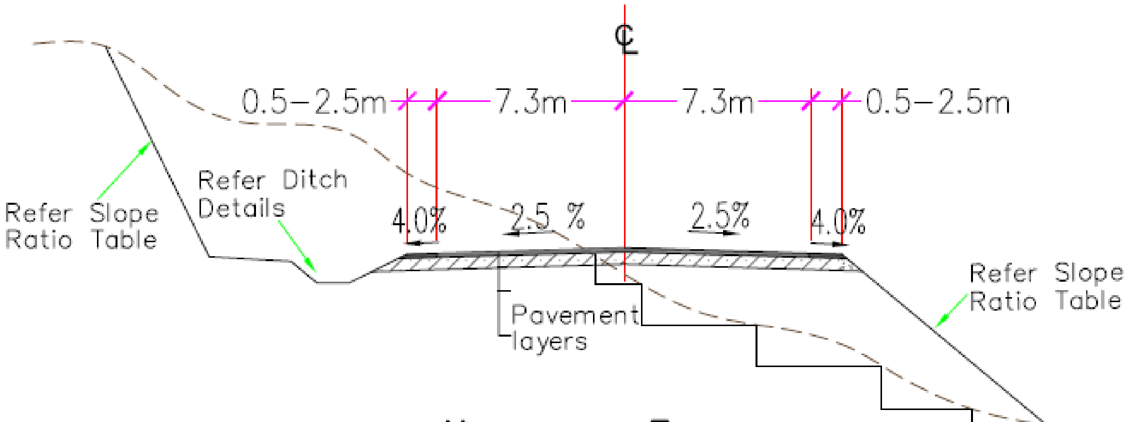


FLAT TERRAIN

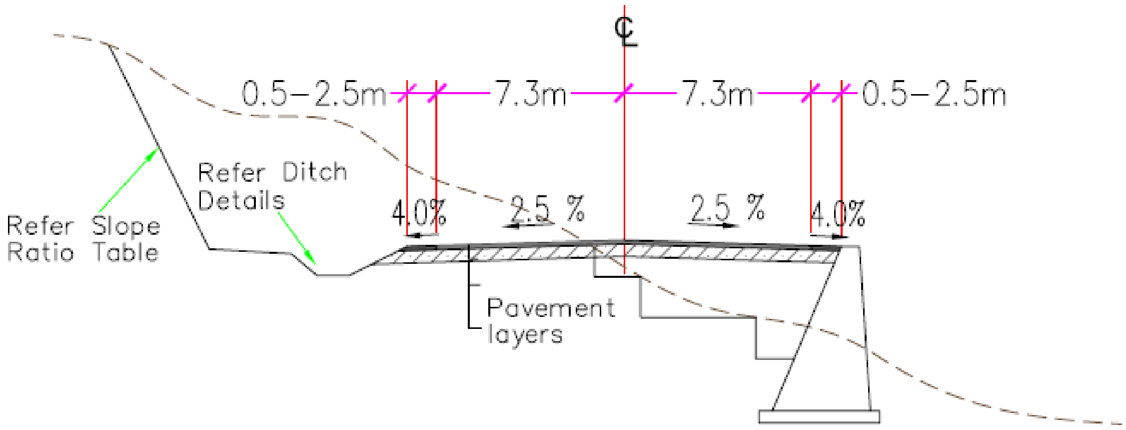


ROLLING TERRAIN

Figure F-8(a): DC8 Paved Flat and Rolling Terrain Typical Cross Section



MOUNTAINOUS TERRAIN



ESCARPMENT TERRAIN

Figure F-8(b): DC8 Paved Mountainous and Escarpment Terrain Typical Cross Section

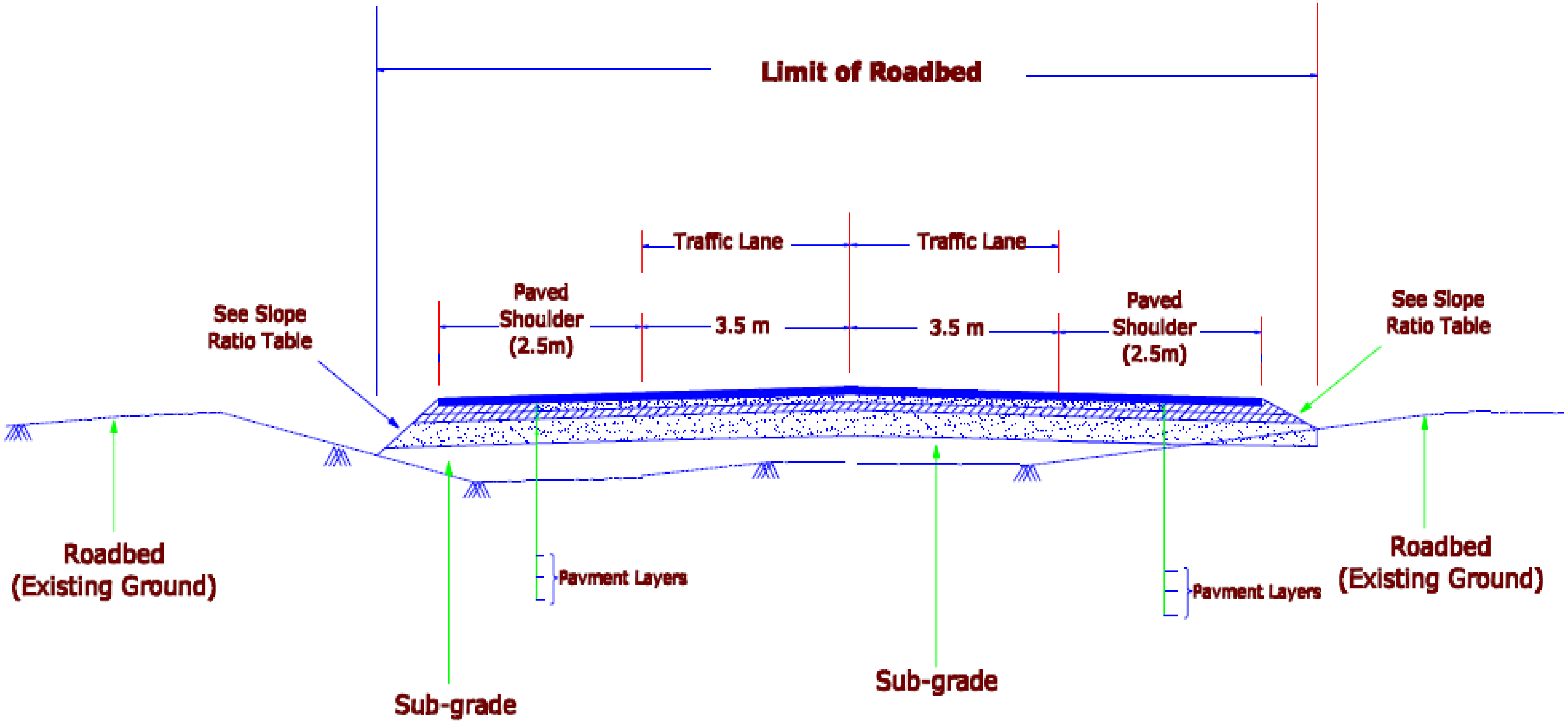


Figure F-9: Divided Lane (Kebele Seat) Typical Town Section

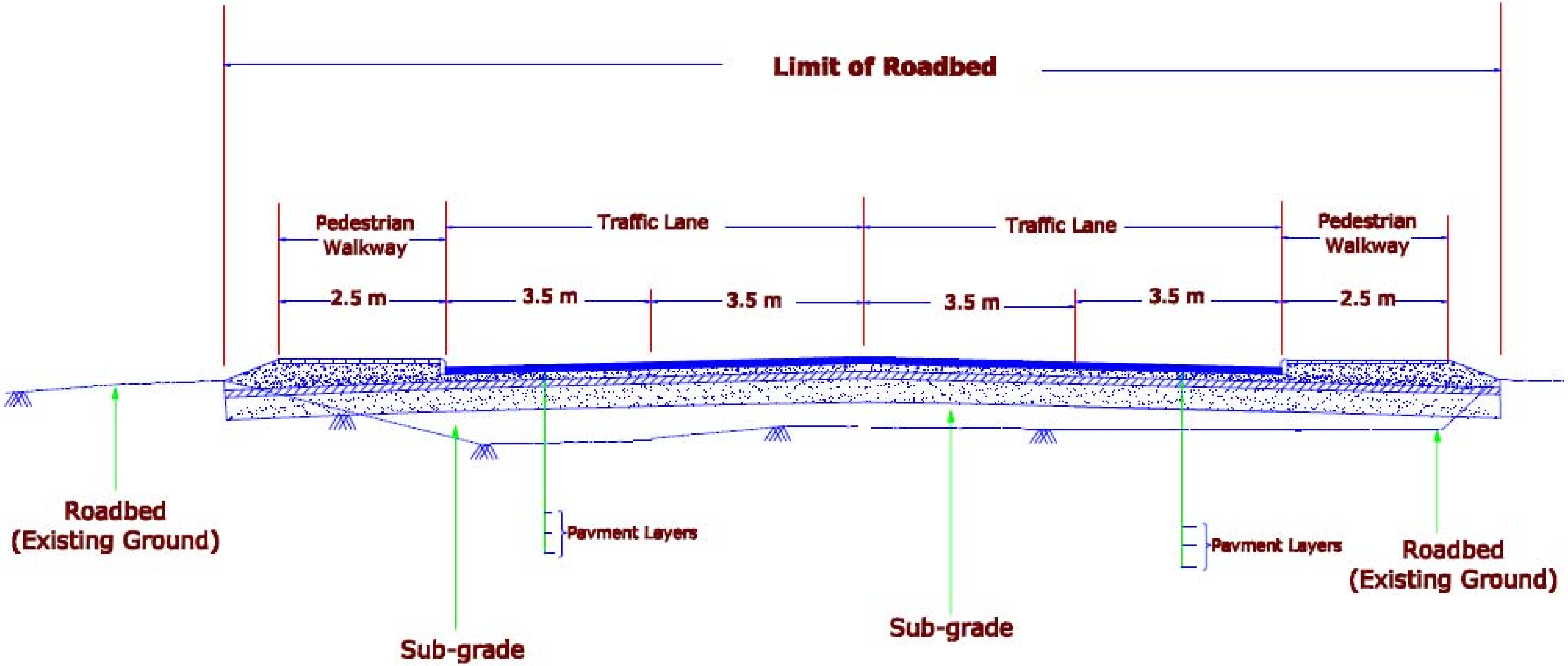


Figure F-10: Divided Two Lane (Wereda Seat) Typical Town Section

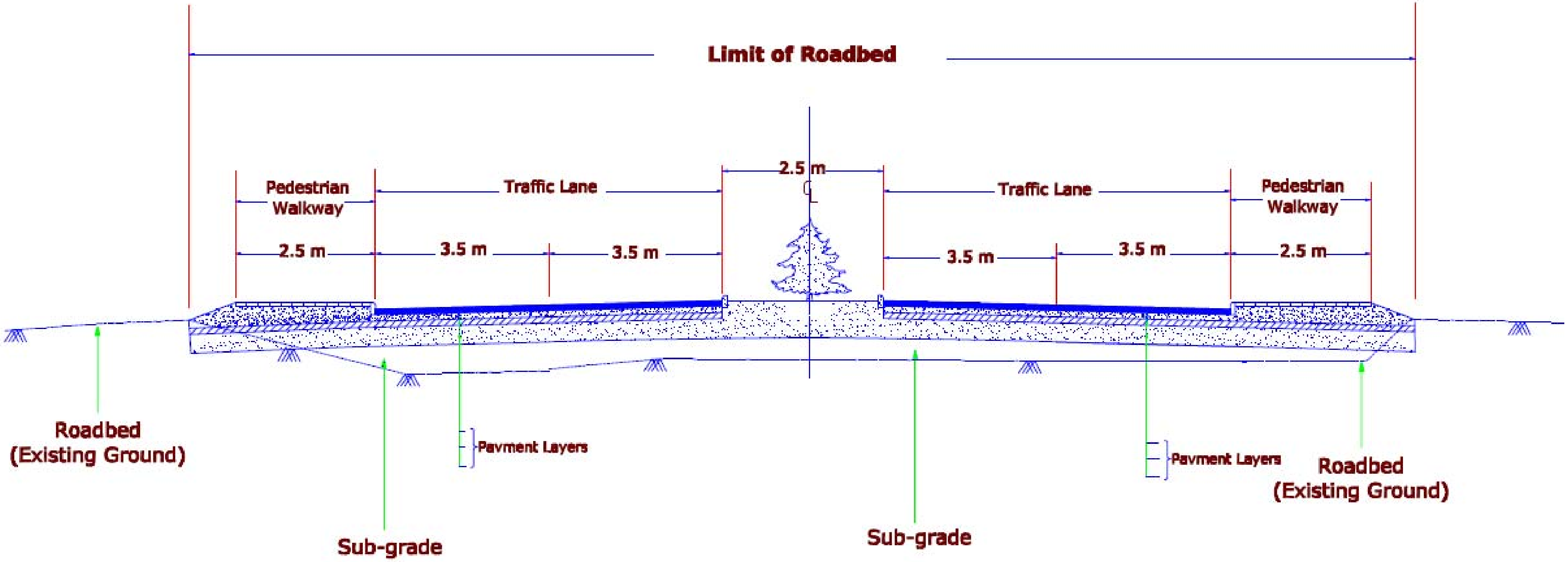


Figure F-11: Divided Two Lane (Zonal Seat) Typical Town Section

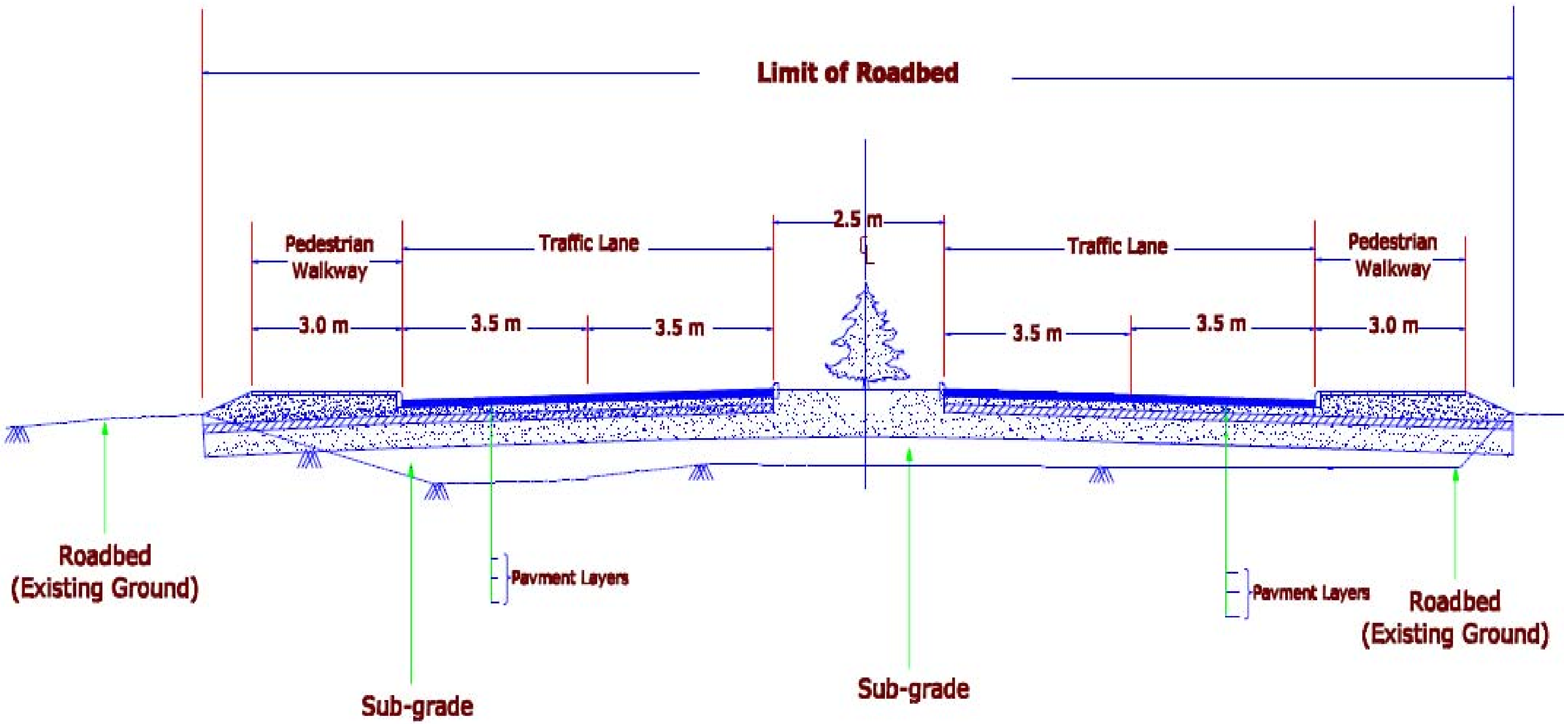


Figure F-12: Divided Two Lane (Regional Seat) Typical Town Section

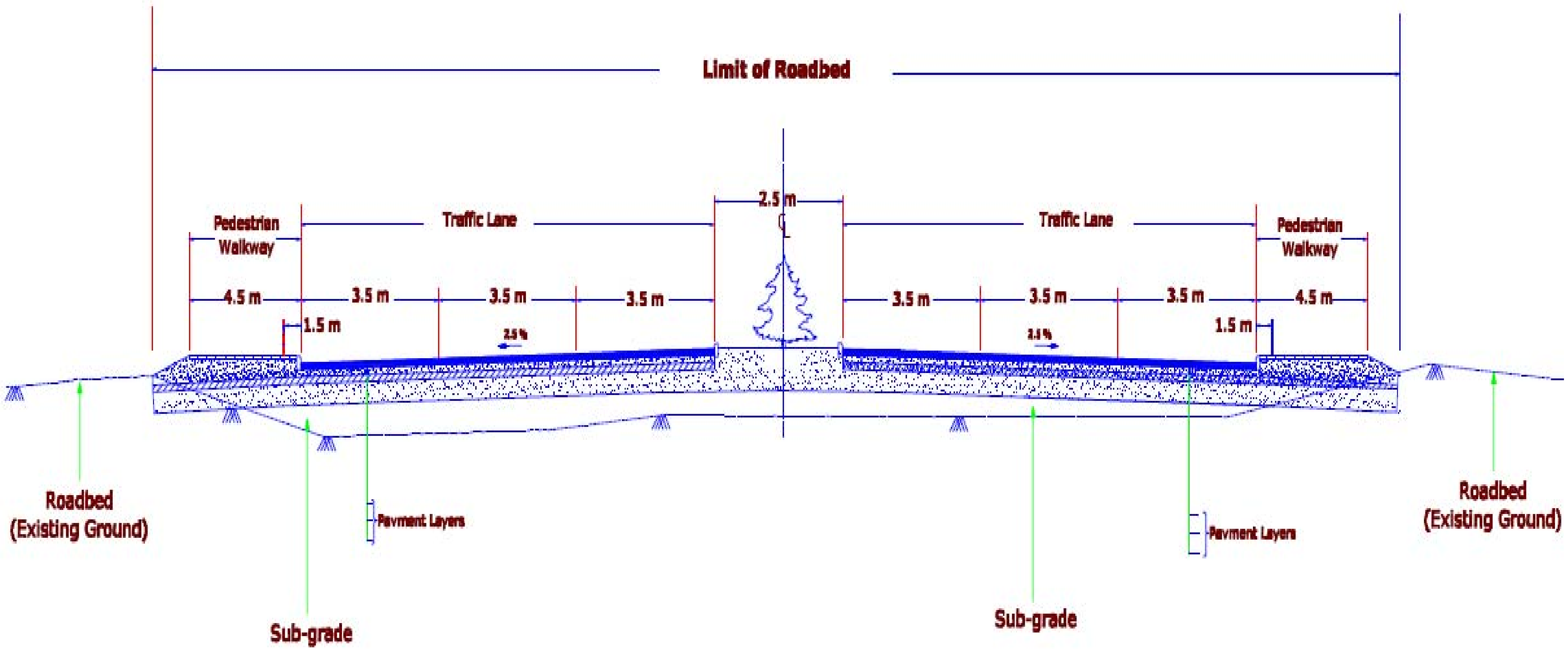


Figure F-13: Divided Three Lane (Addis Ababa) Typical Town Section

Appendix G PROCEDURE FOR DESIGN

G.1 Introduction

This chapter has been prepared as a guide and checklist for personnel engaged in road design. It is not intended as a textbook nor should the contents be considered mandatory in the design of all road projects. It is recognized that whilst the design procedure is generally the same for any project, there are many specific issues, which the designer must consider in proceeding with the design of each individual project.

Reference is made in this chapter to the procedures to be followed in the preparation of data for processing by computer-aided methods. However, whether the designer uses manual or computer methods, he must always be aware that good engineering judgment must be exercised at all times. A complete review of all field data, project requirements, economics and applicable specifications must be considered in order to produce the most technically sound and economic design for each project.

The designer must also be aware of any relevant safety and environmental regulations and incorporate these features in the design. The chapter ends with the presentation of a checklist for road link design.

G.2 Review of Field Data

***Horizontal Alignment*.**

In many cases the designer of the final alignment has little control over the location of the horizontal alignment due to right-of-way restrictions, previous commitments with local community groups and other factors. The following items should be considered:

1. Review topographical data along with horizontal alignment to ensure that steep gradients are avoided where possible.
2. Review topography to ensure that alignment does not run parallel to major drainages, which may result in channel changes, extra culvert lengths due to skews, etc.
3. Review curve data to ensure that the degree of curvature meets the allowable maximum for the required design speed. Also avoid broken curves in alignment or short tangents on switch back curves that make it impossible to design adequate super- elevation transitions.

Projects to be designed by computer-aided methods should have horizontal alignment data submitted as follows:

1. If the project has been located on photogrammetric manuscripts, the designer need only submit beginning station and degree of curvature and coordinates of every horizontal P.I.
2. If the project has been located by ground surveys, the designer should submit horizontal alignment data in notebook form.

The designer will furnish ERA with copies of horizontal alignment listing (HAL), which will list all stations and coordinates of P.C., P.I. and P.T. of each curve, including curve data.

Terrain Cross Sections

The terrain cross sections (original cross sections) should be reviewed for accuracy keeping the following points in mind:

1. Check to see that notes are complete and that cross sections extend sufficient distances from the centre line to accommodate the template and will permit shifting of the alignment, if necessary, without re-cross sectioning.
2. Check all benchmarks, turning points, H.I.'s etc., to ensure that cross section elevations will be reduced correctly.

Check to see that stream profiles have been surveyed and noted in the field notes to distinguish them from cross sections.

For projects where terrain data are taken directly from aerial photography, the designer must convert the data for processing. The level of detail from aerial photographs depends on the flying height, cloud cover and ground vegetation.

Topography Data

Topography data obtained by field ground surveys will generally be collected electronically for downloading to appropriate computer-aided design software.

Topography data obtained by photogrammetry can be more complete and accurate in that it will include all planimetric features such as fences, walls, utility poles, and existing improvements, in addition to elevation contours and spot elevations.

The topography data should be checked for completeness and accuracy against other maps available to the designer.

Topography data will be of great use to the designer in determining right-of-way restrictions, swamp or rock locations, and the need for relocating utility lines or other physical features that affect the design.

G.3 Design criteria

Prior to commencing the design, the designer should tabulate all design criteria to be used on the particular project so that other personnel working on the project will be aware of it. This data will be obtained through discussions with the ERA Engineer, commitments to local governing bodies, or through his own analysis of design requirements.

The following items should be tabulated and included in the design file:

1. Design speed.
2. ADT present and future. The ADT should be noted as actual count, listing the date of count or estimate.
3. A typical section sketch should be prepared showing the proposed geometric section, such as finished and subgrade widths, ditch dimensions, crown and super-elevation, cut and fill slope sections, etc.
4. The files should show design criteria used in determining drainage requirements, such as 25-year flood frequency, hydrologic chart, etc.

5. Right-of-way widths should be noted including agreements for construction easement.
6. If a soils profile has been taken, it should be part of the file with cover thickness tabulated. In some areas, a chemical analysis at culvert locations may be required to determine need for protective coating of pipes.
7. The designer should note whether the profile grade should roll with the terrain, avoiding high fills and cuts, or whether grades should be as smooth as possible.
8. All control profile elevations should be noted such as, existing pavements to be matched, minimum pipe cover requirements, railroad elevations crossings, bridge elevations to be matched, as well as any other elevation requirements, which may have been agreed to with abutting property owners.
9. Locations of possible borrow and aggregate pits should be noted.
10. The designer should note the requirements for earthwork balancing such as maximum balance distance, whether tight balance or borrow and waste designs should be followed and whether ditches can be widened or slopes flattened to achieve better balanced design.
11. Shrinkage and swell factors should be noted, whether calculated or estimated.
12. Location of rock outcrops, swamps, springs should be indicated.
13. The need for fencing, gates, cattle guards, cattle passes, curbing, guard rail, etc., should be noted and criteria used for determining these needs.
14. The need for road approaches, and acceleration, deceleration and passing lanes should be noted.
15. All available information on base course and surfacing should be noted to determine thickness, asphalt requirements, need for bituminous seal coats, etc.
16. All other items that may have a bearing on the design should be noted such that all personnel involved in the design will be aware of criteria and design requirements. The designer should obtain copies of all correspondence on the project and make it part of the design file.

G.4 Site Trip

It is imperative that the designer makes a field trip to the site and walks the entire alignment reviewing the topography before start of actual design. At this time he can note special problems and design considerations, discussing these items with the ERA Engineer or his staff. The designer should take photographs of special problem areas to enable him to best solve design problems.

After this field trip and discussions, the designer has a much better overall picture of the project and its requirements and can best determine the extent of construction details to go on the plans. Projects to be designed for contract work will require more details and explanation on pay quantities than force account projects.

G.5 Preliminary Design Drawings

During the early stages of design, preferably prior to the field trip, the following preliminary design drawings or sketches should be prepared:

1. Typical section of roadway showing geometric (sketch is adequate).

2. Drainage plan map showing all drainage areas and stationing were they cross the proposed road. This drawing need not be to scale.
3. Profile plot of existing centre-line ground line. This can be plotted at a scale of usually 1:1000 vertical and 1:10,000 horizontal.
4. Plan plot of alignment. This shows all planimetric features of the terrain - (scale is usually 1: 10,000). Right-of-way lines should be superimposed on this drawing to determine if there will be any restrictions.
5. Soils profile plot and material pit drawings showing type and depth of material.
6. Stream profiles extending a sufficient distance upstream and downstream to determine pipe grade and skew angles.

G.6 Preliminary Design Procedure

Up to this time the designer has been primarily engaged in reviewing and assembling field data, setting design criteria and establishing project requirements with very little design work accomplished. The following guideline will assist the designer in proceeding with preliminary design:

1. If the Designer is at liberty to change the horizontal alignment he should make the decision whether the proposed alignment is the most feasible or whether offsetting the alignment could result in a more economical design. The horizontal alignment should be coordinated with the profile by studying a continuous stretch of the plan and profile, visualizing the road in three dimensions to determine if changes are required.

2. Calculate drainage areas and waterway openings for all pipe culverts, box culverts and bridges. After determining the waterway openings the pipe sizes should be calculated to determine control points on the grade line. The Designer need not make the final decision as to exact pipe size or type of structure (CMP, RCP, or concrete box culvert) at this time, and an approximation will suffice.

   No effort is made in this manual to describe the methods used to determine drainage structure requirements since these methods are given in the ERA Drainage and Bridge Design Manuals. It is recommended that the Engineer obtain copies of these manuals for drainage structure calculations.

3. Spot culvert locations on the profile plot noting flow line elevations and sizes of pipes. Note control elevations that must be met such as minimum pipe cover, existing pavement, and bridge elevations.

4. The first trial grade line can now be drawn on the profile plot. The grade can either be the finished pavement or the subgrade profile.

The following items should be considered in laying the trial grade:

1. Projects starting at a T-intersection should begin with a minimal grade for a minimum of 20 metres from the intersection before entering steeper grades in order to provide better sight distance and to allow stopping and starting of vehicles on near-level grades.
2. Vertical P.I.'s should be placed at even stations or plus 50's for ease in calculating grades and staking. Where possible, V.P.I.'s should be spaced 300 - 500 metres

apart and curve lengths should be determined to provide proper site distance for the design speed. Where possible, minimum curve lengths of 100 metres should be maintained and extremely long crest vertical curves should be avoided since many drivers refuse to pass on vertical curves.

3. Avoid placing V.P.I.'s where intersecting side roads will be on crest vertical curves as this reduces sight distances and creates safety hazards.

G.7 Checklist for Final Road Link Design

A flowchart for the design process is presented in Figure 5-1. Figure G-1 serves as a checklist for ensuring that all critical elements of the final design have been addressed.

Figure G-1: Checklist for Final Road Link Design

General

- ❑ 1. Has the design standard been selected?
- ❑ 2. Does the selected standard fit into the environment and road hierarchy?
- ❑ 3. Has the design speed been selected?
- ❑ 4. Does the standard selected provide for the design vehicle(s)?
- ❑ 5. Does the standard selected provide for the traffic volume?
- ❑ 6. Does sufficient right-of-way exist, or can it be obtained?

Cross Sections

- ❑ 7. Are the cross-section widths adequate for the levels of traffic flow predicted?
- ❑ 8. Have cross-sections been widened at sharp horizontal curves?
- ❑ 9. Does the design include adequate shoulders?
- ❑ 10. Has the full carriageway cross-section been continued across culverts and minor bridges up to 30m span?
- ❑ 11. Are side slopes and back slopes adequate?
- ❑ 12. Does the cross section provide for drainage?
- ❑ 13. Have the clear zone requirements been met?

Alignment

- ❑ 14. Are stopping sight distances along the road above the minimum values required for the design speeds for both horizontal and vertical alignments?
- ❑ 15. Are the radii of horizontal curves, with superelevation where required, above the minimum values identified for the design speed?
- ❑ 16. Have reverse curves, broken-back curves, and compound curves been avoided?
- ❑ 17. Are isolated curves of sufficient radius?
- ❑ 18. Is superelevation according to standards?
- ❑ 19. Are the vertical curves adequate for the design speeds?
- ❑ 20. Are the gradients below the maximum permissible values?
- ❑ 21. Are the gradients as indicated on cross-sections and longitudinal sections sufficient to avoid standing water?
- ❑ 22. Does the alignment allow regular overtaking opportunities, including for single lane standards?

- ❑ 23. Have climbing lanes been introduced where necessary to provide adequate and safe overtaking opportunities?
- ❑ 24. Is road access too open or too restricted?
- ❑ 25. Do the combined geometric design elements produce a consistent and safe alignment?
- ❑ 26. Will the road allow safe driving in darkness?
- ❑ 27. Will the design lead to reduced severity in the event of an accident?

Junctions

- ❑ 28. Does the junction design minimize potential conflicts to the extent possible/practical?
- ❑ 29. Is the type of junction selected appropriate for the conditions present?
- ❑ 30. Do the junction approaches provide adequate visibility?
- ❑ 31. Does the junction cater to the needs of pedestrians and non-motorized vehicles as required?
- ❑ 32. Have turning lanes been considered?
- ❑ 33. Has lighting been considered?

Provision for Pedestrians

- ❑ 34. Has pedestrian usage been estimated, and have appropriate facilities been provided for pedestrian and non-motorized safety both along and across the road?

Miscellaneous

- ❑ 35. Has provision been made for the placement of utilities?
- ❑ 36. Have traffic signs, signals, road markings and guideposts been provided?
- ❑ 37. Have centerline and edge-line markings been designed which give adequate guidance/control for drivers?
- ❑ 38. Has adequate provision been made for the provision of bus lay-byes, and for parked and stopped vehicles?
- ❑ 39. Are railway crossings adequately designed?
- ❑ 40. Have guardrails been considered, especially at bridge approaches?
- ❑ 41. Is there any scope for the installation of emergency escape ramps?
- ❑ 42. Have speed limit zones, safety barriers, and emergency escape ramps been considered?

Departures from Standards

- ❑ 43. Are departures from standards necessary? Have these been documented and has proper action been taken?

Date: .... Designer Date: .... Responsible Engineer

....

Appendix H PLANS AND DRAFTING

H.1 General

The final plans are the graphic portrayal of the complete highway design. These plans, together with the specifications, enable the Engineer and the contractor to locate and construct on the ground the highway facility as visualized by the designer. Plans should be kept as simple as possible yet be complete enough so that the need not be compelled to try to guess what the designer intended.

Unique drawing numbers must be included on the drawings. The drawings should not contain any estimated quantities. The minimum size of lettering chosen must be legible after plan reduction.

H.2 Completeness of Plans

The completeness of plans and profiles and level of detail adopted for a given project shall be consistent with the type of road, and the nature of the work to be undertaken.

Plans for tender and construction purposes shall be produced at 1:2000 scale on A1 size paper, which may be reduced to A3 size for tender purposes. The plans shall include a title sheet, project location plan, plan and profile drawings, typical cross sections, structural details, material pits and road furniture details.

1. Title sheet: The title sheet is a standard form and shall show the road functional classification number, the project number, and project length and location, with arrows designating the beginning and ending stations on the route map. Design criteria such as design speed, maximum curvature, gradient, terrain class, etc., will be tabulated. An index of all drawings will be shown and a title block for signatures by the Consultant will appear in the lower right hand corner.

2. Typical cross section sheet: This sheet will show the road section or sections with all dimensions, such as lane, carriageway, and shoulder widths and other details.

3. Plan and Profile sheet: These sheets are standard with the upper half for alignment and the lower half for profile.

   i) The alignment in rural areas is normally plotted at a scale of 1:4000 (A3) and will show centerline in heavy lines and right-of-way lines in lighter lines. Thick marks will be shown on the centerline every 100 meters, with a heavier tick mark every 5 stations. The alignment will be clearly defined with stationing, bearings, curve data, and north arrow. Alternatively, the setting-out data can be presented in a computer file.

   ii) The location of all land lines, forest boundaries, city limits, railroads, present roads, existing and proposed fences pole lines, channels, ditch structures, bridges, culverts, utility lines, large trees, improvements within or adjacent to right-of-way, approach roads, right-of-way markers, and detour roads shall be properly shown to scale.

iii) Notes will accompany the notations clearly stating the work to be accomplished, such as: to be removed, to remain in place, work by other, construct, etc.

iv) Description, location and elevation of all benchmarks are generally noted along the profile portion of the plan and profile sheet.

v) The profile is normally plotted at a scale of 1:200 vertical and 1:2000 horizontal (A1) on the lower half of the plan and profile sheet. The horizontal scale must be the same as the scale used in the alignment, or plan view.

vi) The elevation of the ground and proposed grade line should be plotted accurately, noting P.V.I. station, elevation, length of curve, middle ordinate, beginning and ending of curve and percent gradient. The grade line should be continuous along parabolic curves and should be the profile grade (top of finished surface or subgrade) as noted on the typical section.

vii) All cross drainage structures and bridges shall be noted, including graded ditches, ditch blocks, grade of special ditches and cross section of ditches and dikes. Existing culverts must be shown with dashed lines, and new structures must be shown with solid lines.

4. Structure detail sheets. These sheets include bridge details, curb and gutter details, concrete box culverts, headwalls, drains, underpasses, cattle guards and other special structures. These drawings should be complete and include location, elevations, dimensions, estimate of quantities, and applicable specifications, if not covered in the specifications.

5. Material Pits. This drawing will show the location, shape and size of material pits for borrow, subbase, base course, mineral aggregate for bituminous material and chips, and concrete aggregates. In addition to the sketch of the pit, the location of test holes and the results of sample tests should be tabulated. Existing roads to be used or haul roads (length and location) should be noted.

6. Detail Drawings: These can include junction and driveway details, bus lay-by details, climbing lane details, guard rail details, striping details, destination sign details, and fencing details, as appropriate. Standard details are given in the ERA Standard Detail Drawings.

7. Mass Haul Diagram: Earthwork quantities are represented graphically and are tabulated on this sheet.

H.3 Size of Plans

All drawings should be the standard A1 size and printing should be bold and large to permit legibility when reduced to A3 size.

H.4 Uniformity of Plans

The designer should strive to maintain uniformity in preparation of road plans rather than giving them "his personal touch". Plans prepared using uniform standards are easier to understand by both the engineer and the contractor and will generally result in fewer disputes or claims.

H.5 Abbreviated Plans

Certain types of construction projects use less intense design than other types. Projects such as rehabilitation projects usually do not consist of new alignments and may have only spot geometric improvements. Such projects may require only minor engineering control, and therefore the plans may be abbreviated. For instance, in the above example, no plan and profile sheets need be included except in areas requiring relocations or substantial grade or curve changes. Plans will be similar to complete plans except plan and profile sheets will be limited to sections affected by such improvements.

However, sufficient information concerning horizontal and vertical alignment must be shown to enable the contractor to stake the project.

Appendix I PERSPECTIVES IN GEOMETRIC DESIGN

To illustrate the advantages of visualising the alignment in three dimensions and to guide the design towards good practice, a number of alignment combinations are shown in Figures I.1 to I.16.

Figure I.1 shows the advantage of maintaining a constant, uniform grade for as long as possible. Local dips to minimise earthworks that result in a disjointed alignment will be there for the life of the road.

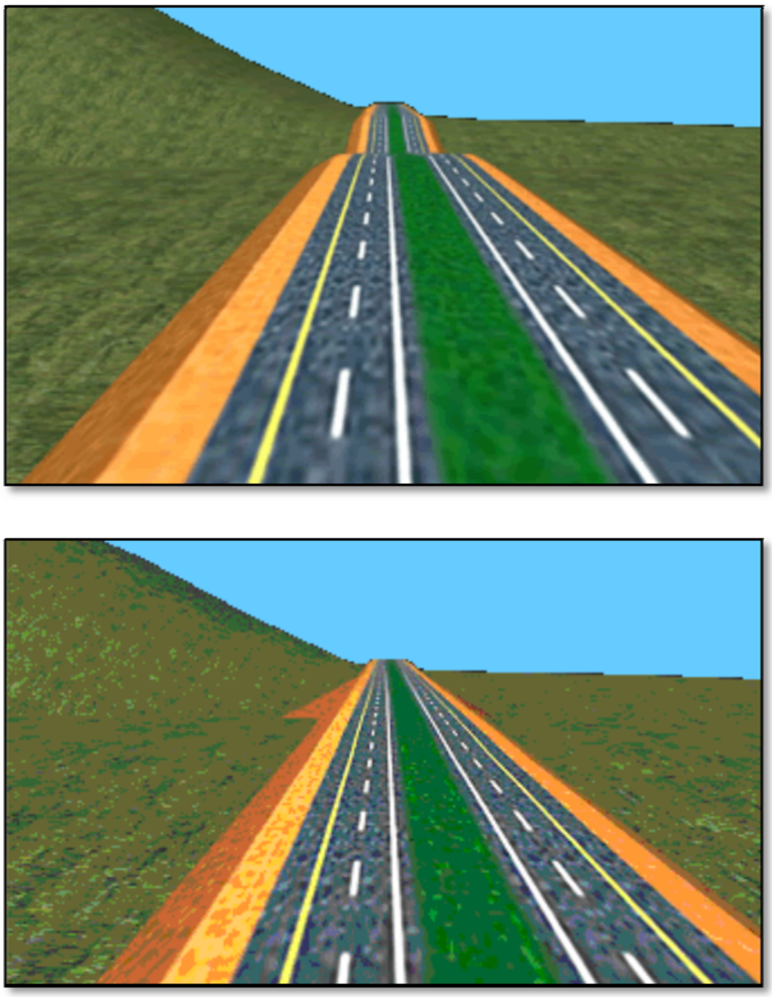


Figure I.1: Effect of eliminating a dip on a long grade

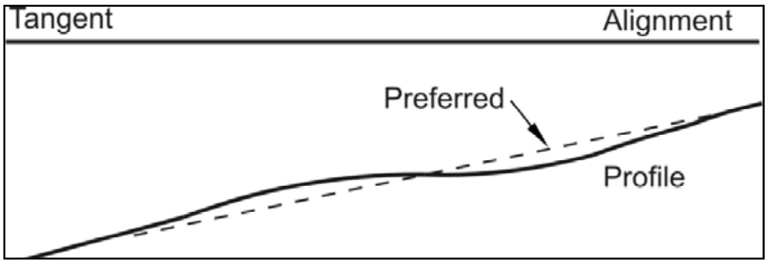


Figure I.2: Short sag curve on a long tangent

Short crests and sags should also be avoided on horizontal curves, as shown in Figure I.3. Maintaining a constant grade is the preferred option.

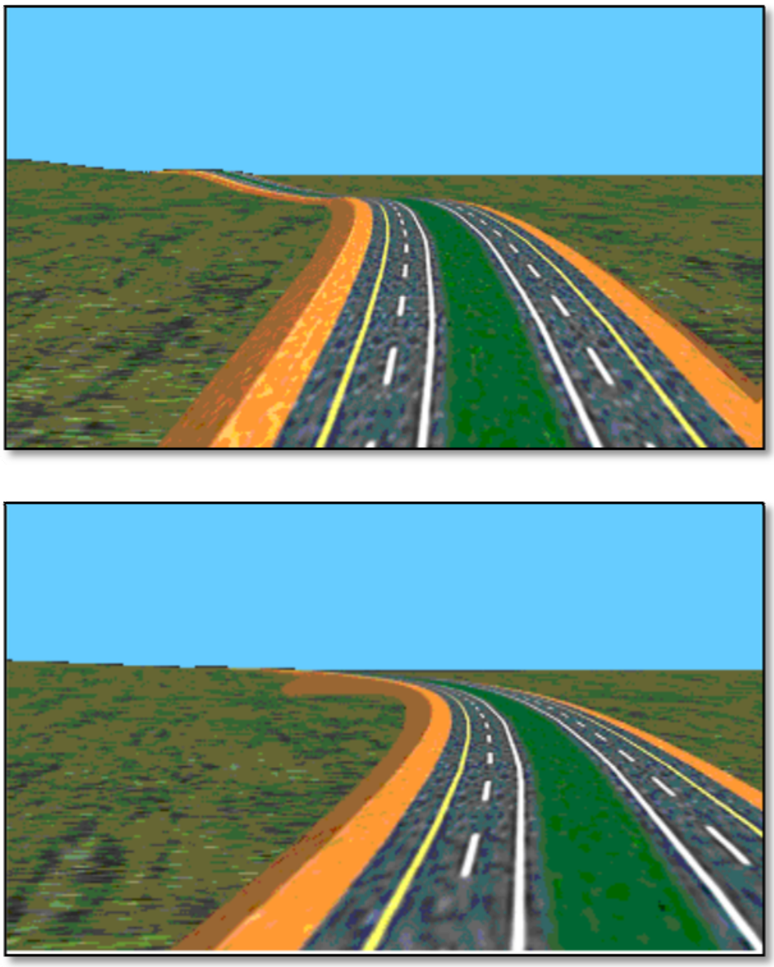


Figure I.3: Removal of humps on a horizontal curve

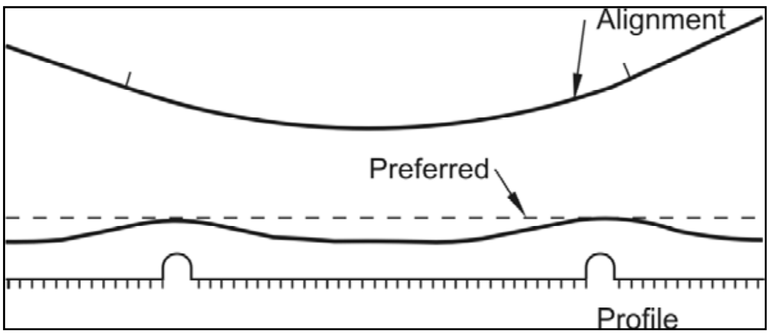


Figure I.4: Short humps on a long horizontal curve

A short discontinuity or dip in the alignment preceding a horizontal curve creates a particularly discordant view. Eliminating the crest curves in advance and following the sag curve improves the appearance, as shown in Figure I.5 and I.6.

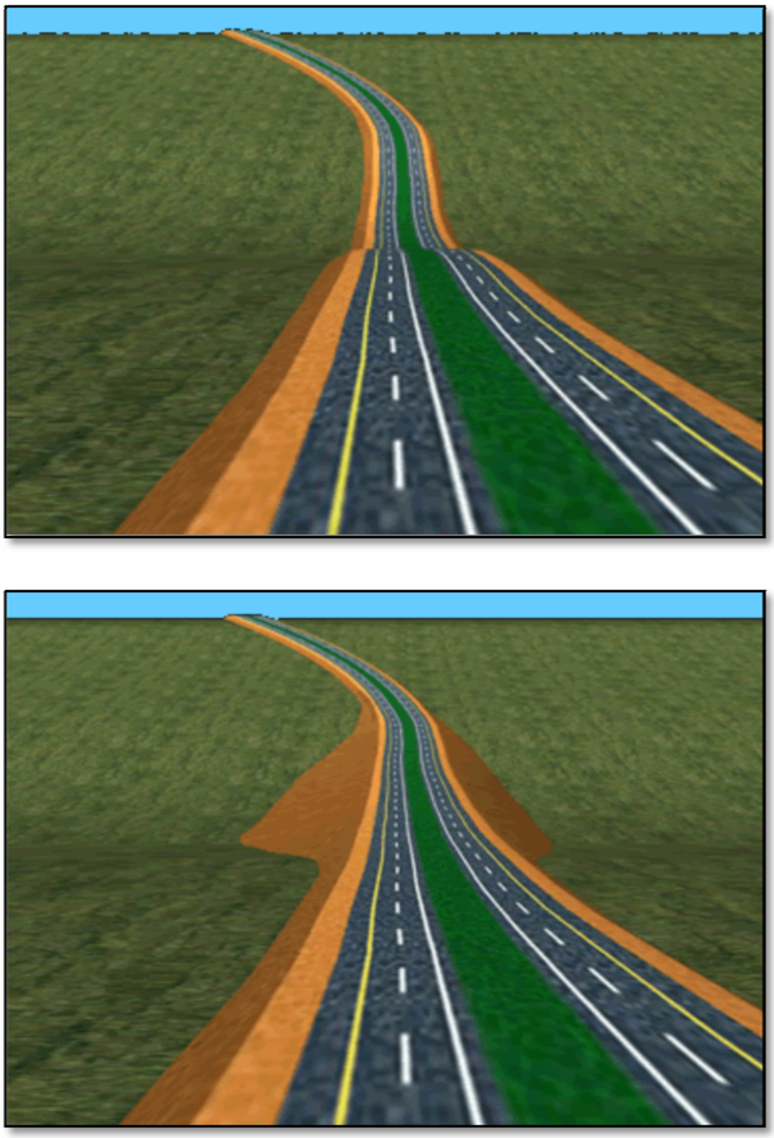


Figure I.5: Short hump and dip preceding horizontal curve replaced by long sag curve linking into horizontal curve.

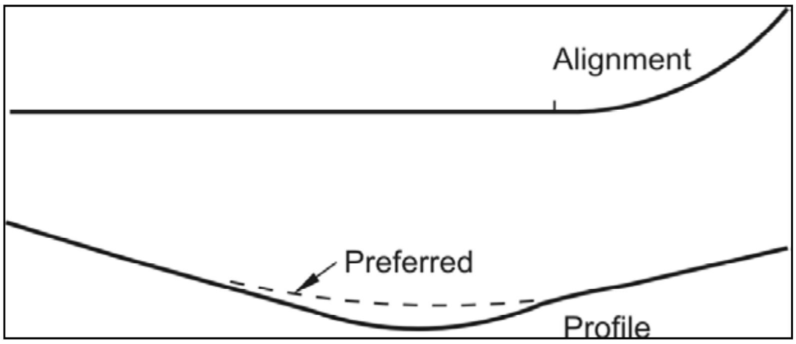


Figure I.6: Short vertical curves preceding a long horizontal curve.

A common fault in road alignment is illustrated in Figure I.7 and I.8. The roadway is often unnaturally curved to cross a small stream or grade separation at right angles. The advantages in the alignment aesthetics of a skew crossing often far outweigh the savings deriving from a square crossing.

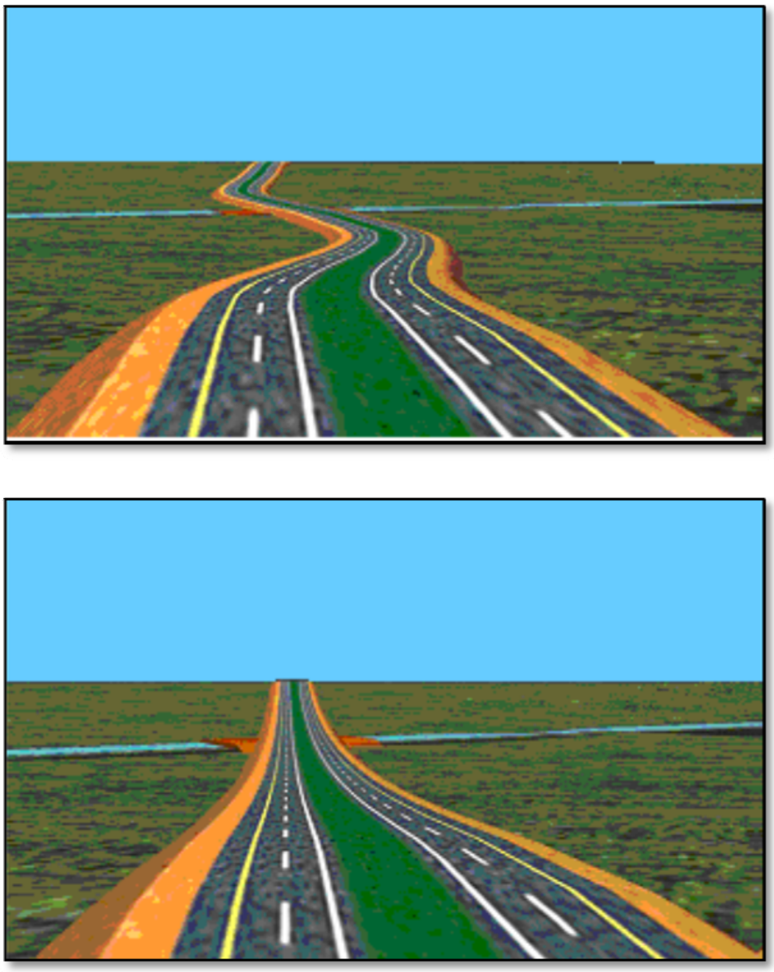


Figure I.7: Skew crossing improves horizontal alignment

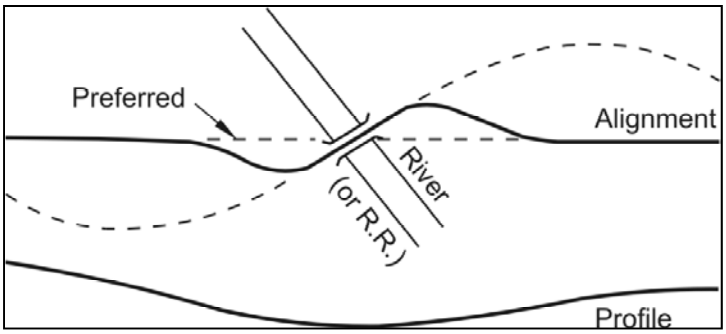


Figure I.8: Distorted alignment at bridge crossing.

Figure I.9(a) illustrates the broken-back horizontal curve, or two curves in the same direction separated by a short tangent. The sag curve on the separating tangent intensifies the broken-back effect. The advantages of using a single radius curve throughout are illustrated in Figure I.9(b).

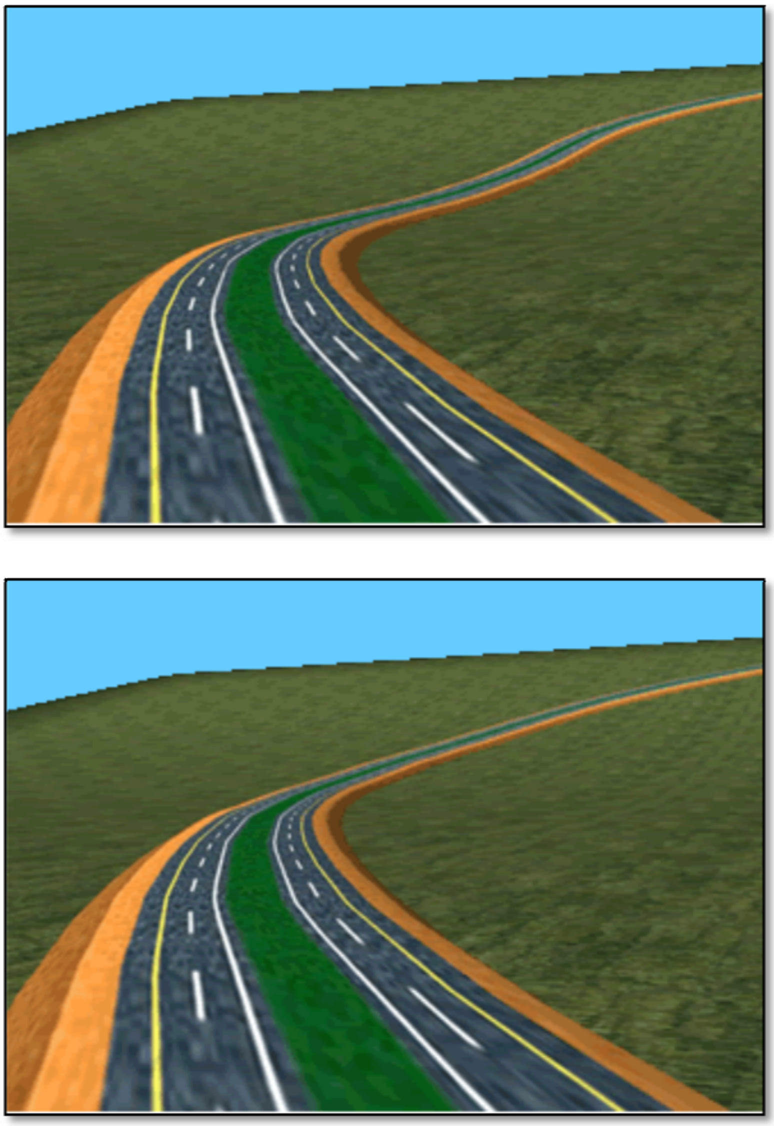


Figure I.9(a) and I.9(b): Replacement of broken-back curve by single radius long curve

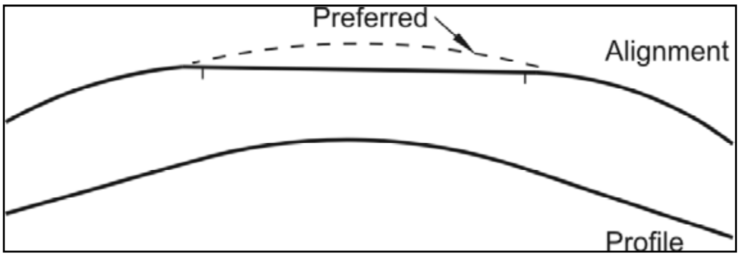


Figure I.10: Broken-back curve

Minor changes in grade or rolling of the vertical alignment as shown in Figure I.11 should be avoided on long horizontal curves.

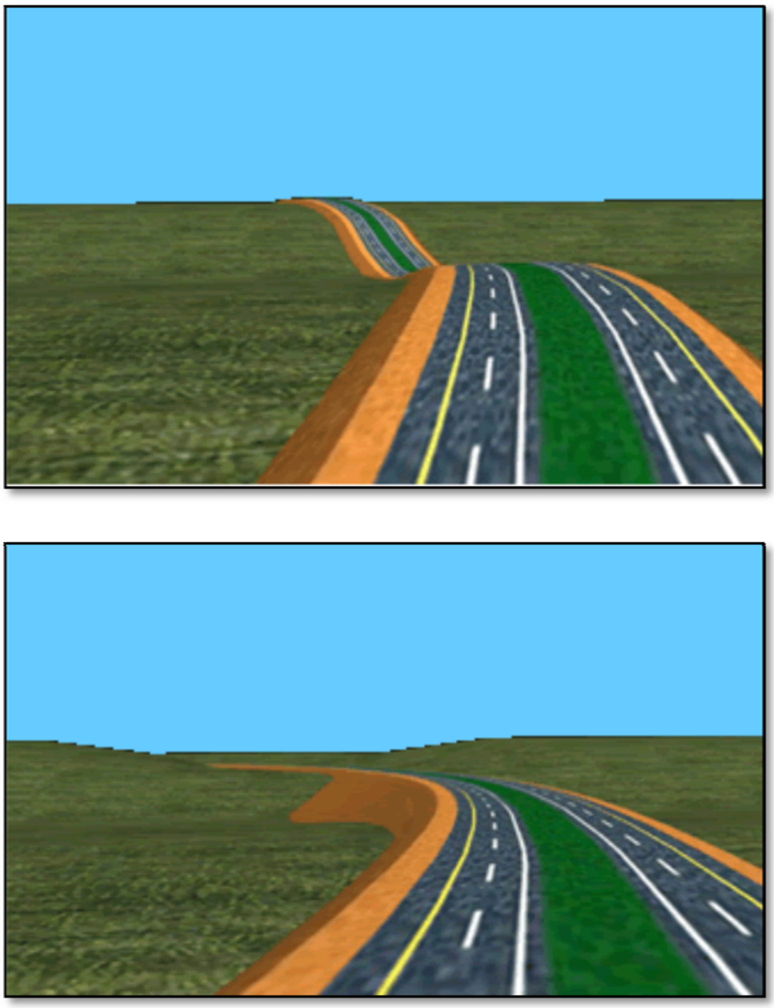


Figure I.11: Rolling grade line and its elimination

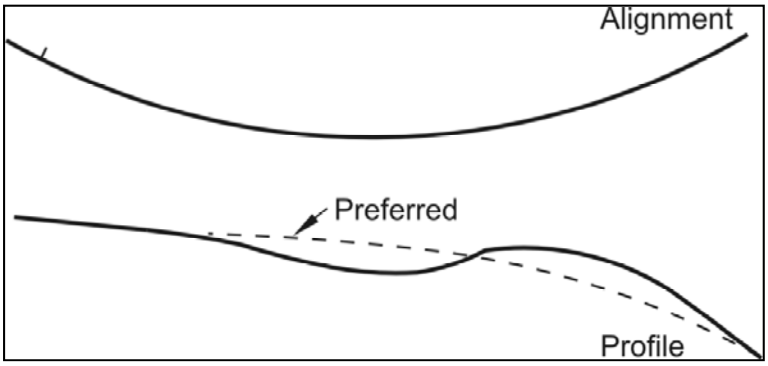
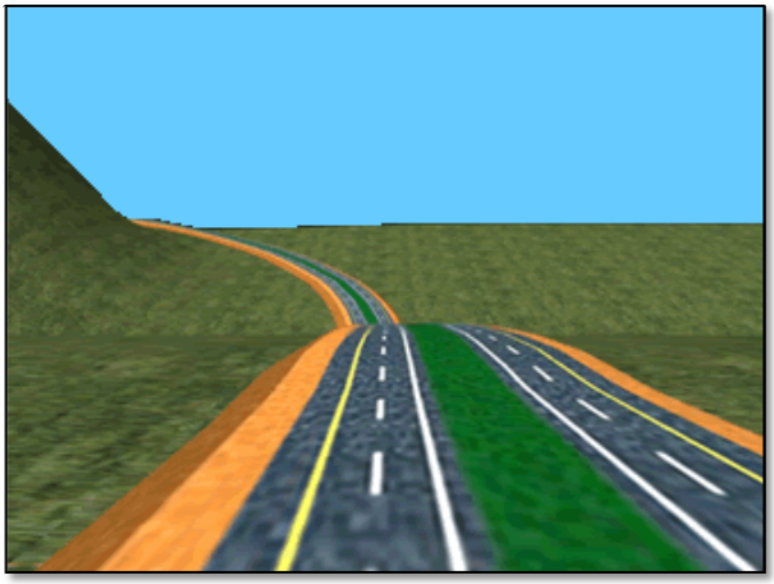


Figure I.12: The advantages of co-ordinating the horizontal and vertical alignments

Figure I.13 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.



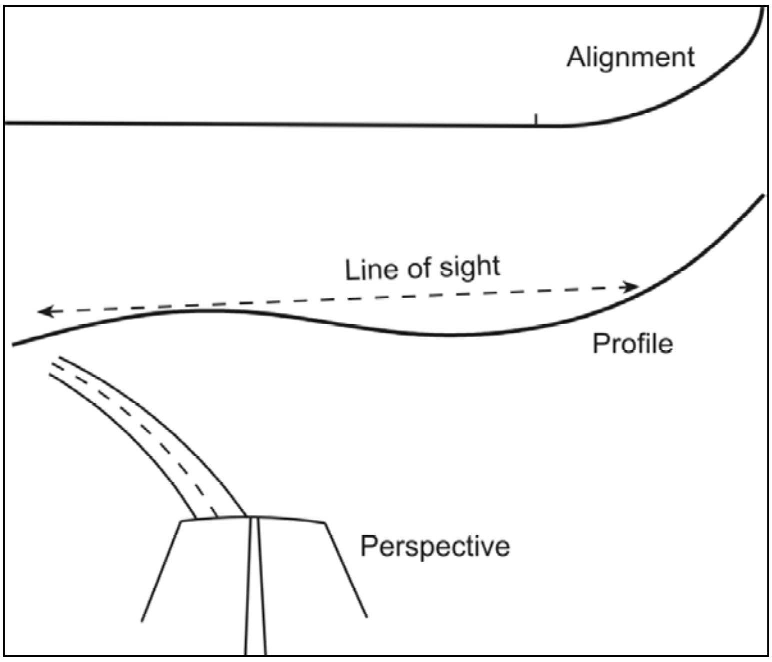


Figure I.13: Apparent break in horizontal alignment when start of horizontal curve is hidden by a crest

A sag curve at the start of a horizontal curve has the effect of enhancing the sharp angle appearance as shown in Figure I.14, 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 I.14: Out-of-phase vertical and horizontal alignments.

Figures I.15 and Figure I.16 illustrate the advantages of co-ordinating the horizontal and vertical alignment. In each case the vertical curve is contained within the horizon. Figure I.15 shows a well-coordinated crest curve and horizontal curve and Figure I.16 shows a well-coordinated sag and horizontal curve.

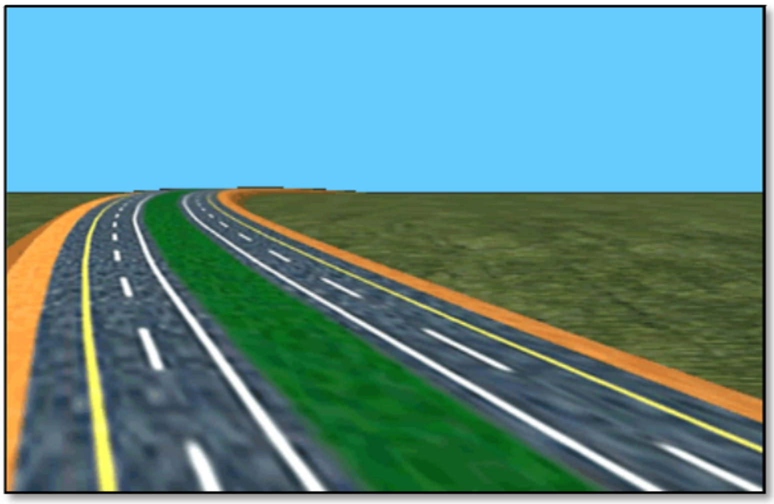


Figure I.15: Well-coordinated crest and horizontal curve.

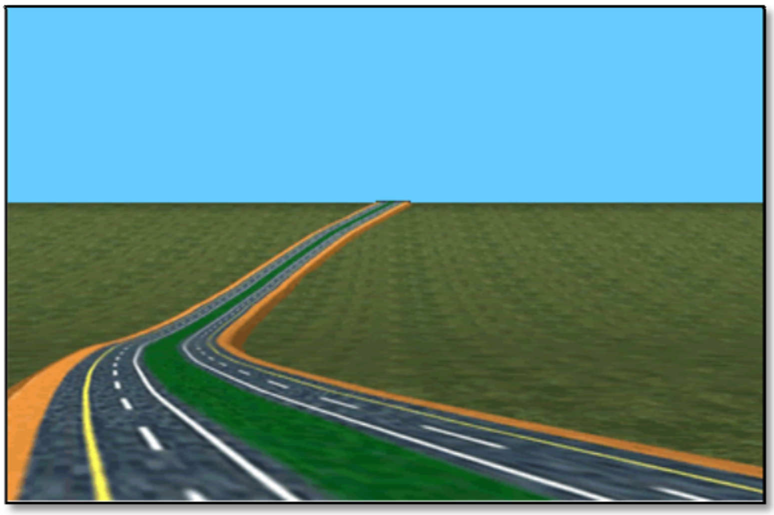


Figure I.16: Well-coordinated sag and horizontal curves