



Federal Republic of Nigeria

Federal Ministry of Works

Highway Manual Part 1: Design

Volume I:

Geometric Design

2013

GEOMETRIC DESIGN

FOREWORD

The vision statement of the Federal Ministry of Works is to elevate Nigerian roads to a standard where they become National economic and socio-political assets, contributing to the Nation's rapid growth and development, and to make Federal roads functional, safe, pleasurable, and an avenue for redeeming Nigerians' trust and confidence in Government. This vision statement is in tune with the Transformation Agenda of the President of the Federal Republic of Nigeria, His Excellency, Dr Goodluck Ebele Jonathan, GCFR. Based on the foregoing, our mission is to use the intellectual, management and material resources available to the Ministry to make Nigerian roads functional all the time. The principal goal of the Ministry is to drive the transformation agenda by improving road transport infrastructure for the overall socio-economic derivable benefits and development of our great country, Nigeria.

In exercising this mission and in discharging its responsibilities, the Ministry identified the need for updated and locally relevant standards for the planning, design, construction, maintenance and operation of our roads, in a sustainable manner. One of the main reference documents for this purpose is the Highway Manual, which previously included Part 1: Design and Part 2: Maintenance. Both current parts of the Highway Manual were first published in 1973 and 1980 respectively and have been subjected to partial updating at various times since then. The passage of time, development in technology, and a need to capture locally relevant experience and information, in the context of global best practices, means that a comprehensive update is now warranted.

The purpose of the Highway Manual is to establish the policy of the Government of the Federal Republic of Nigeria with regard to the development and operation of roads, at the Federal, State and Local Government levels, respectively. In line with this objective, the Manual aims to guide members of staff of the Ministry and engineering practitioners, with regard to standards and procedures that the Government deem acceptable; to direct practitioners to other reference documents of established practice where the scope of the Manual is exceeded; to provide a nationally recognized standard reference document; and to provide a ready source of good practice for the development and operation of roads in a cost effective and environmentally sustainable manner.

The major benefits to be gained in applying the content of the Highway Manual include harmonization of professional practice and ensuring uniform application of appropriate levels of safety, health, economy and sustainability, with due consideration to the objective conditions and needs of our country.

The Manual has been expanded to include an overarching Code of Procedure and a series of Volumes within each Part that cover the various aspects of development and operation of highways. By their very nature, the Manual will require periodic updating from time to time, arising from the dynamic nature of technological development and changes in the field of Highway Engineering.

The Ministry therefore welcomes comments and suggestions from concerned bodies, groups or individuals, on all aspects of the document during the course of its implementation and use. All feedback received will be carefully reviewed by professional experts with a view to possible incorporation of amendments in future editions.



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1 General Information

1.1 Description of the Highway Manual

1.1.1 Introduction to the Manual

The Highway Manual aims to guide members of staff of the Ministry and engineering practitioners, with regard to standards and procedures that the Government deems acceptable for the planning, design, construction, maintenance, operation and management of roads. The Manual directs practitioners to other reference documents of established practice where the scope of the Manual is exceeded; provides a nationally recognized standard reference document; and provides a ready source of good practice for the development and operation of roads in a cost effective and environmentally sustainable manner.

1.1.2 Arrangement of the Manual

The Highway Manual comprises a Code of Procedure and two parts, each of which has been divided into separate volumes, in the manner shown in Figure 1.

1.2 Overview of Volume I: Geometric Design

1.2.1 General

Volume I of the Highway Manual Part 1: Design deals with the Geometric Design of Highways.

1.2.2 Purpose

The purpose of this volume is to give guidance and recommendations to the engineers responsible for the geometric design of roads in Nigeria.

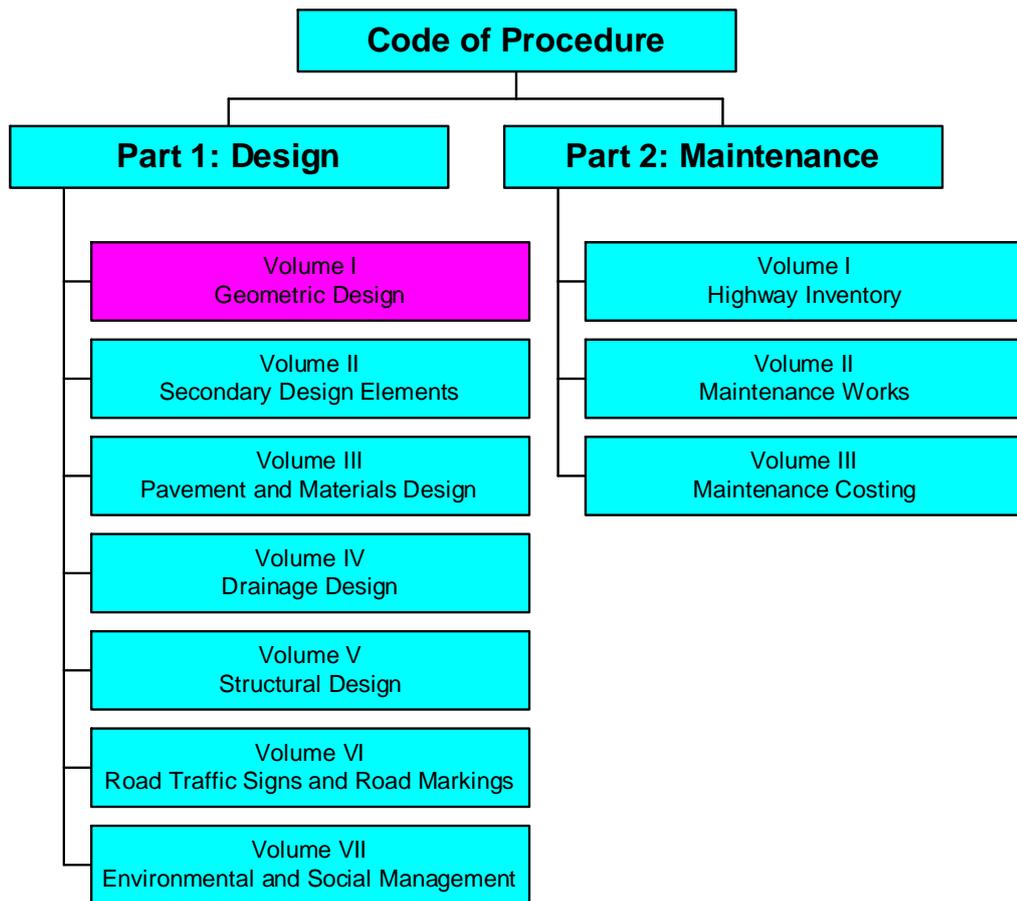


Figure 1: Arrangement of the Highway Manual showing context of Volume I

1.2.3 Scope of this Volume

This volume has been developed by the Federal Ministry of Works with the intent to reflect policy and establish uniform procedures and standards for planning and designing highways. The procedures presented in this volume are applicable to all classes of federal roads defined in this Geometric Design volume.

The contents of the volume are partly guidelines and recommendations and partly standards which should be adhered to as a general rule. The information, guidance and references contained in this volume are not intended as a substitute for sound engineering judgment. It should be recognized that situations may be encountered during the design of highways that are beyond the scope of this volume. The references at the end of this volume list numerous sources of comprehensive information that should be used to supplement the information contained in this

volume. In some instances special conditions may require the use of other references and/or standards and the use of these standards can only be sanctioned by the Director Highway Design.

1.2.4 Contents of the Volume

This volume is divided into 5 chapters as follows:

Chapter 1 deals with general items including an introduction, some terminology, and a philosophy of the geometric design of roads.

Chapter 2 covers the road system, and discusses the road categorization and classification of roads. It deals with division of roads into classes and design classes and gives summarized geometric design parameters for the different design classes of roads. The chapter also discusses the design vehicle, traffic data collection and estimation, traffic characteristics and capacity.

Chapter 3 covers elements of design and discusses the various sight distances and horizontal and vertical alignments. Also dealt with are the cross section elements, including lane widths, shoulders, medians, clear zones, right of way, cut and fill slopes.

Chapter 4 discusses at-grade junctions, including design requirements, selection of junction types, t-junctions, cross junctions and roundabouts; sight distances; and junction elements including turning lanes and traffic islands.

Chapter 5 covers interchanges or grade-separated junctions. It discusses warrants, location and spacing of interchanges as well as lane balance, auxiliary lanes and ramp design.

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

1.3 Road Design Philosophy

1.3.1 Overall Philosophy of Road Design

Current philosophy is based on the assumption that any design that complies with established geometric design guidelines is safe and that those that do not are unsafe. This is accepted by designers.

Despite decades of international research, the complex relationship between vehicle, roadway, driver, and operational safety is not fully understood. Many researchers have investigated the relationship between accident rates and geometric design elements and while many of the more important factors in providing safe roads have been established and are included in this Manual, specific on site combinations of peripheral items can lead to unsafe situations even on the most carefully designed roads.

The design philosophy that should be adopted by road designers encompasses two levels:

- In the first instance the focus should be on geometric planning. These issues dictate how user friendly the ultimate design will be. This level includes a holistic approach to ensuring the road is located in the optimal position, that the alignment considers social and environmental and safety considerations, and that the standards to be used are the most appropriate for the situations pertaining along and around the route alignment.
- Geometric design then focuses on the specific measures that provide for efficient and appropriate operation of the road, as well as provide for all the specific detail that make roads safe and compatible with the social and environmental circumstances surrounding the road.

This Volume of the Manual touches on the first level of this philosophy but is primarily focussed on guiding the design engineer to achieve the second level in the philosophy.

This philosophy emphasises that safety should be the prime consideration. Sacrificing safety in the interests of efficiency is not acceptable practise. A comprehensive approach should be founded on the concept of reducing the

probability of failure to the lowest possible level and to minimise the consequences of failures that could occur. To achieve this, a design must start with a clear understanding of purpose and functionality. From this basis design elements should be selected to integrate into the landform and current and future land use.

Professionalism in road design is the ability to foresee and optimise conflicting objectives inherent in any design.

1.3.2 Road Safety Audits

With the above emphasis on safety, Road Safety Audits form an integral and essential part of road geometric design. The Manual therefore provides for a six stage Road Safety Audit process that is outlined in the Code of Procedure and detailed in Volume II: Secondary Design Elements. These audits ensure that, at each stage of the planning, design, implementation and operation of a road project, as many of the road safety issues as possible are addressed.

1.3.3 Environmental Management Issues

Environmental management issues form an integral and essential part of road geometric design and the process for environmental management is covered in the Procedure Manual.

1.4 Terminologies

The following figures illustrate the meaning of certain key terms used in this volume.

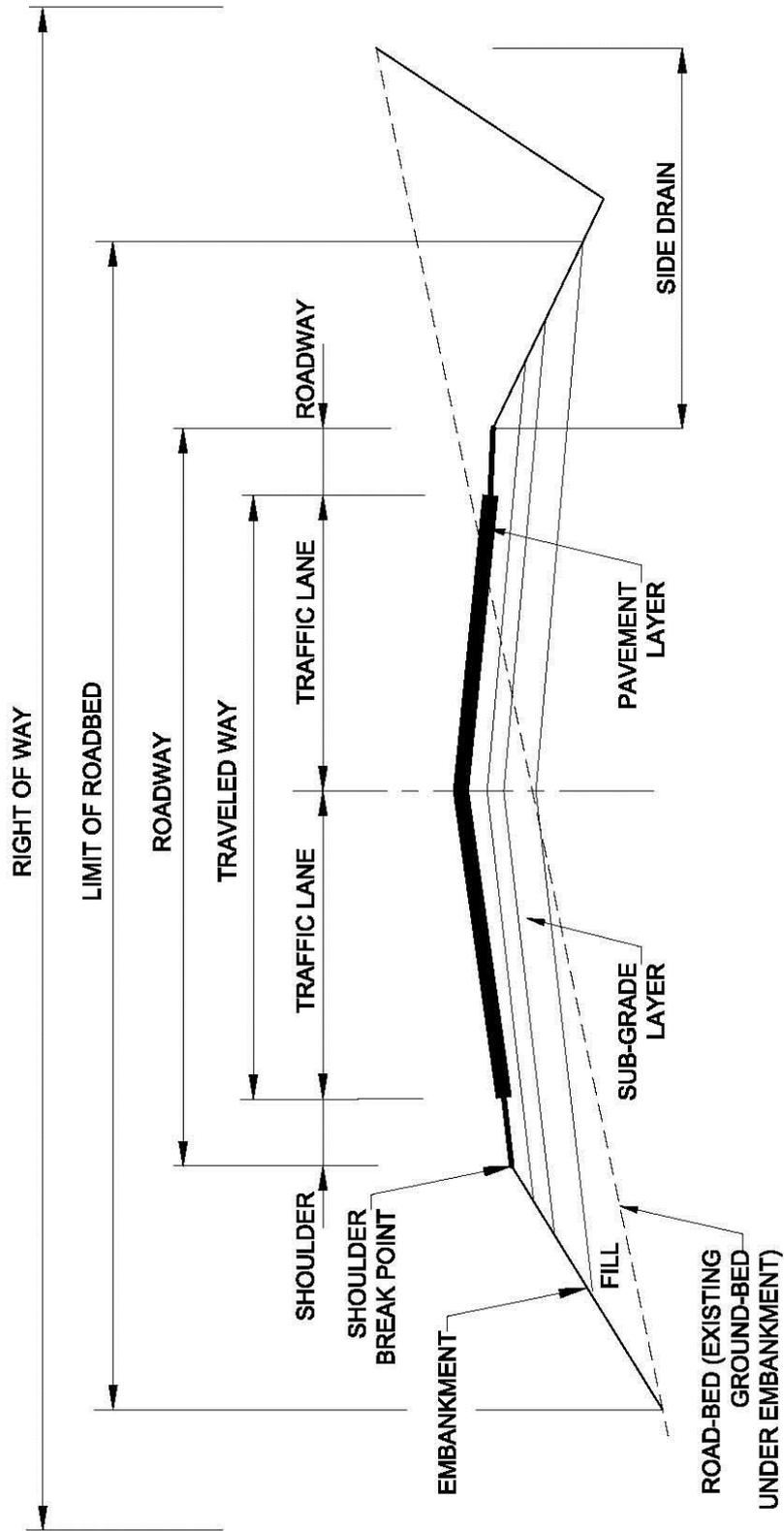


Figure 2: Road Cross Section Elements

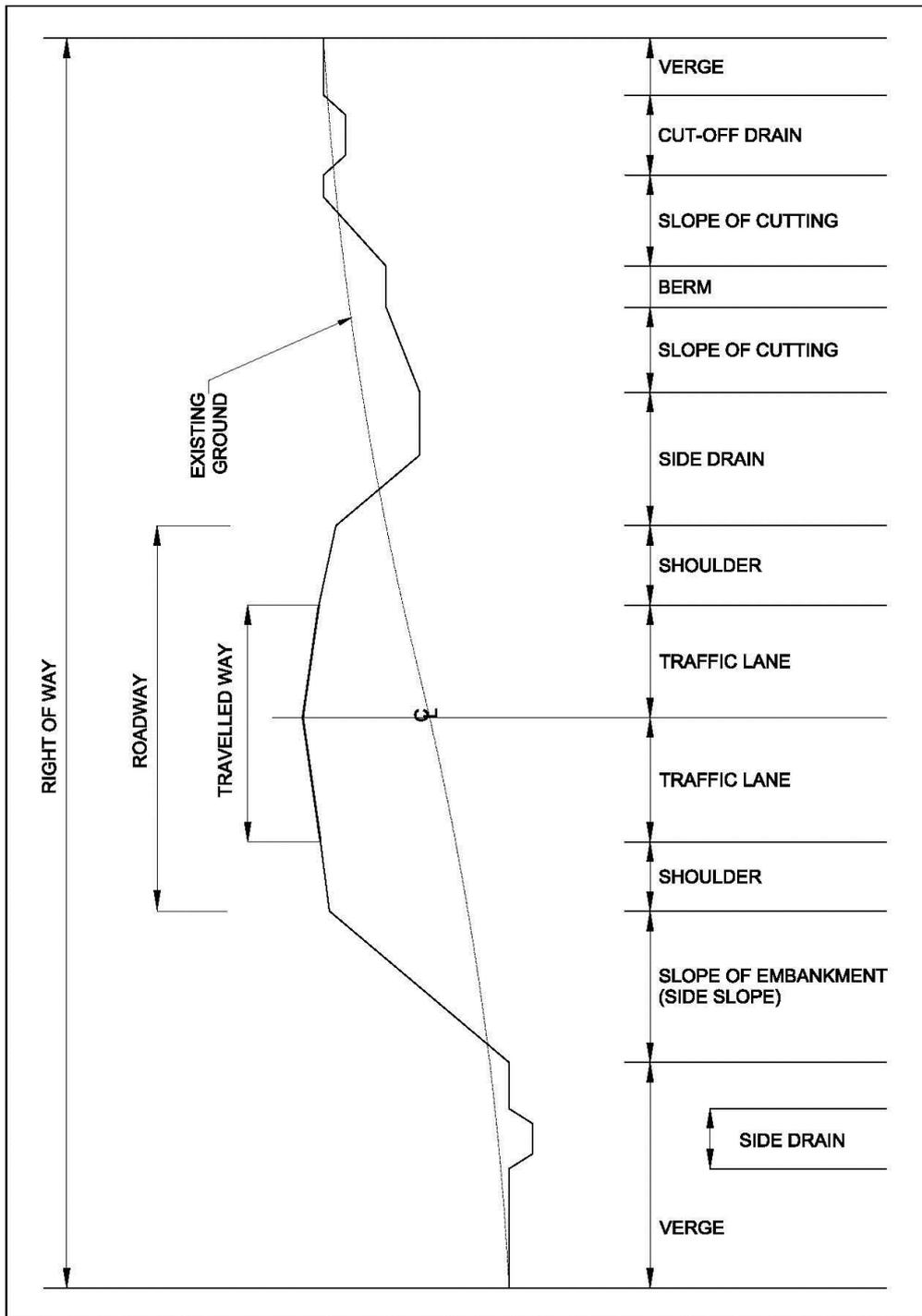


Figure 3: Cross Section Terminology



2 The Road System

2.1 Road Classification

2.1.1 General

Roads can be categorised in several ways but the two main category systems used are by “ownership” and by “function”. Roads have previously been classified as “Federal Highways”, “State Roads”, “Local Roads” and “Rural Roads”. This classification relates to the ownership of the road and to which organisation has responsibility for such road. From a design point of view, however, the “function” of the road is more important to determine, as this dictates the standard of the road to be constructed. Functional classification is therefore fundamental to the design of roads and is elaborated on in the following sub-sections.

2.1.2 Functions of Roads

Roads have two functions: to provide mobility and to provide land access. However, from a design standpoint, these functions are incompatible.

For mobility, high or continued speeds are desirable and variable or low speeds undesirable; for land access, low speeds are desirable and high speeds undesirable. For example, freeways provide a high degree of mobility, with access provided only at spaced interchanges to preserve the high-speed, high-volume characteristics of the facility. The opposite is true for local, low-speed roads that primarily provide local access. The general relationship of functionally classified systems in serving mobility and land access is illustrated in Figure 4

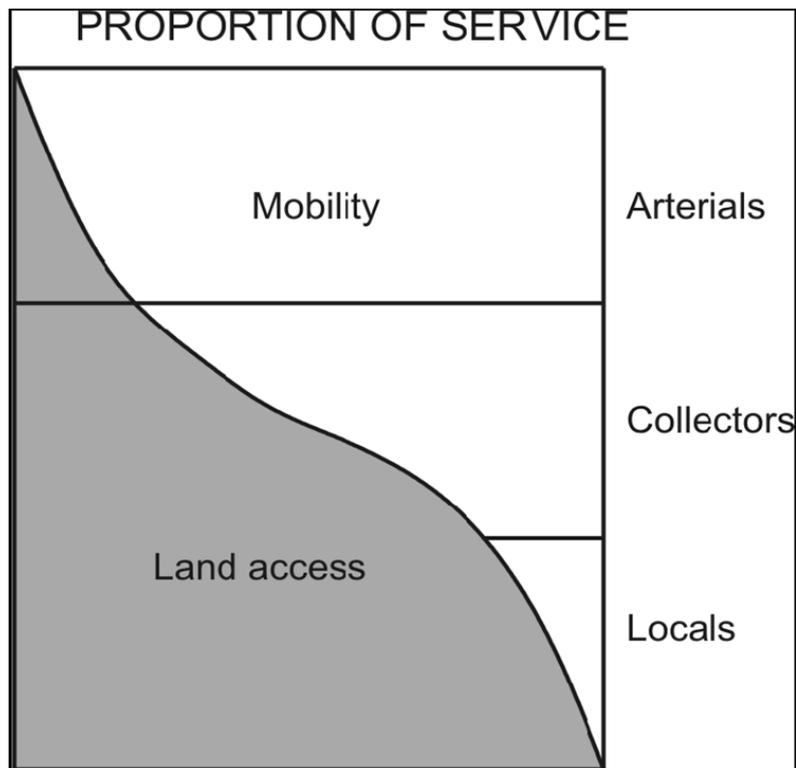


Figure 4: Proportion of Service

Once the function of a road is established, then appropriate design criteria can be applied to encourage the use of the road as intended. Design features that can convey the level of functional classification to the driver include width of roadway, continuity of alignment, spacing of intersections, frequency of access points, building setbacks, alignment and grade standards, and traffic controls.

2.1.3 Concept of Classification

The classification of roads into different operational systems, functional classes or geometric types is necessary for communication between engineers, administrators and the public. Classification is the tool by which a complex network of roads can be subdivided into groups having similar characteristics. In any classification system the division between classes is often arbitrary and, consequently, opinions differ on the best definition of any class. There are various schemes for classifying roads and the class definitions generally vary depending on the purpose of classification. The principal purposes of road classification are to:-

- Establish logical integrated systems that, because of their particular service, should be administered by the same jurisdiction;
- Relate geometric design control and other design standards to the roads in each class, and
- Establish a basis for developing long range programmes, improvement priorities and financial plans.

Classification of roads by design types based on the major geometric features (e.g. freeways) is the most helpful one for design purposes. Classification by route numbering is the most helpful for road traffic operational purposes, whilst administrative classification is used to denote the level of government responsible for, and the method of, financing road facilities. Functional classification, the grouping of roads by the character of service they provide, was developed for transportation planning purposes.

2.1.4 Selection of Functional Class

The Federal roads in Nigeria can be divided into the following classes according to their major function in the road network:

Class A: National Trunk Roads

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

Class B: Primary Roads

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

Class C: Secondary Roads

Roads linking locally important centres to each other, to a more important centre, or to a higher class road (rural/market centres) and linkage between locally important traffic generators and their rural hinterland. Function is to provide both mobility and access.

Class D: Minor Roads

Any roads linking to minor centres (market/local centre) and all other trafficable roads. Major function is to provide access to land adjacent to the secondary road system.

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

a. Control of Access

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

Although control of access is one of the most important means for preserving the efficiency and road safety of major roads, roads without access control are equally essential as land service facilities. The following three levels of access control are applicable:

- Full access control: means that the authority to control access is exercised to give preference to through traffic by providing access connections with selected public roads only and by prohibiting direct access connections.
- Partial access control: means that the authority to control access is exercised to give preference to through traffic to a degree in that, in addition to access connections with selected public roads, there may be (some) private access connections.
- Unrestricted access: means that preference is given to local traffic, with the road serving the adjoining areas through direct access connections. However, the detailed location and layout of the accesses should be subject to approval by the Road Authority in order to ensure adequate standards of visibility, surfacing, drainage, etc.

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

Table 1: Access Control by Functional Class of Roads

Functional Class	Classification	Level of Access Control	
		Desirable	Reduced
A	National Roads	Full	
B	Primary Roads	Partial	Partial or Unrestricted
C	Secondary Roads	Partial	Unrestricted
D	Minor Roads	Partial or Unrestricted	Unrestricted

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

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

2.1.5 Selection of Design Class

The most widely accepted design type criteria are those developed by AASHTO which classify a road system into:

- Freeways;
- Arterial roads other than freeways, and
- Collector roads.

For each classification, specific design standards and criteria and access and other policies have been developed and are applied. For use in the present document the following service or design classifications are proposed, related to design traffic volumes given in ADT terms. Each classification groups roads with similar functions. The factors that influence the classification of a roadway to a certain group include;

- Trip purpose (for the majority of users);
- Trip length;
- Size and type of population centre served;
- Traffic characteristics, and
- Network and system requirements

Design designations of these specific roads are as follows:

Class A National Trunk Roads

Class A1 Freeways in rural areas

- Illustrative threshold ADT (with more than 12% heavy vehicles) = 15 000 veh/d.
- Average travel distances on links indicated by at least 45 per cent of the trips being more than 2 hours in duration;
- May be provided for network continuity purposes, and
- Minimum design speed 100 km/h.

Class A2 Freeways in metropolitan areas

- Illustrative threshold ADT = 20 000 veh/d
- Average travel distance on links indicated by majority of trips being less than 2 hours in duration
- Form integral element of Metropolitan Road Network
- Generally extension of Rural Freeways (Class A1 roads)
- Minimum design speed 100 km/h

Class B Primary Roads

Class B1 Primary Rural Arterials

- Can be provided as 2 lane single carriageway road or a 4 lane single carriageway road if freeway is not financially affordable

- Illustrative threshold ADT : 8 000 – 10 000 veh/d, with bottom end of scale applicable where percentage of truck traffic exceeds 15 per cent
- Minimum design speed 80 km/h

Class B2 Primary Metropolitan Arterial

- Design in context in which it operates.

Class C Secondary Roads

Provided to address inter-regional travel demands, or providing access to tourists or National resource areas

- Two lane, single carriageway roads

Class C1

- Design ADT greater than 4 000 veh/d
- Minimum design speed 80 km/h

Class C2

- Design ADT 500 - 4 000 veh/d
- Minimum design speed 80 km/h

Class C3

- Design ADT less than 500 veh/d
- Minimum design speed 60 km/h.

Class D Minor Roads

- Minimum design speed 40 km/h.

The division into functional classes is governed by the design speed and design traffic. There are many factors that affect the capacity of a road. These include design speed, width, lateral clearance, grade, alignment, weaving sections, ramp terminals, traffic composition, type of surface, and level of service. It is therefore not possible to stipulate precise design volumes for each class of road. The values given in Table 2 should be used as a guide. Each road must be assessed individually during the design stage.

Table 2: Typical Design Speeds

Road Type	Design speed (km/h)
Limited Access Roads	
Freeways in urban areas	100
Freeways and expressways in rural areas	80 – 100
Expressways in urban areas	80 – 110
Conventional Roads	
Rural	
Flat terrain	80 – 100
Rolling terrain	80 – 100
Mountainous terrain	60 – 80
Urban	
Arterial streets	60 – 80
Arterial streets with extensive development	40 - 60

2.2 Design Vehicles

2.2.1 General

Three general classes of vehicles have been selected for this design manual: passenger cars, trucks and buses. The passenger car class includes compacts and subcompacts, recreational utility vehicles and all light vehicles and light delivery trucks (vans and pickups). The truck class includes single-unit trucks, truck tractor-semi trailer combinations, and trucks or truck tractors with semi-trailers in combination with full trailers. Buses include single unit buses, articulated buses and intercity buses. In establishing the design dimensions for the various vehicle classes, this manual focuses on vehicles in regular operation only.

2.2.2 Vehicle Characteristics

a. *Dimensions*

The dimensions adopted for the various design vehicles are given in Table 3. The WB15 vehicle has an overall length of 17 m, whereas the regulations allow for an articulated vehicle semi-trailer to have an overall length of 18.0 m. The multiple vehicle combination, being an articulated vehicle plus trailer and typically in an interlink configuration, is also limited to a maximum overall length of 18 m. Examples of these vehicles are illustrated in Figure 4.

If these vehicles are expected to use a route with any frequency, the designer will have to carefully plan the layout of the intersections to ensure that they can be accommodated. As described below, accommodation does not necessarily require lane widths sufficient to complete a turning movement within the lane. A degree of encroachment on adjacent lanes is permissible depending on the frequency of occurrence.

Table 3: Dimensions of Design Vehicles

Vehicle	Wheel Base	Front Overhang	Rear Overhang	Width
Passenger car (P)	3.1	0.7	1.0	1.8
Single Unit (SU)	6.1	1.2	1.8	2.5
Single Unit + Trailer (SU+T)	6.7+3, 4*+6.1	1.2	1.8	2.5
Single Unit Bus (BUS)	7.6	2.1	2.6	2.6
Semi-Trailer (WB – 15)	6.1+9.4	0.9	0.6	2.5

b. Turning Radii

In constricted situations where the templates are not appropriate, the capabilities of the design vehicle become critical. Minimum turning radii for the outer side of the vehicle are given in Table 4. It is stressed that these radii are appropriate only to crawl speeds.

Table 4: Minimum Turning Radii

Vehicle	Radius (m)
Passenger car (P)	6.8
Single Unit (SU)	10.0
Bus (B)	11.5
Semi-Trailer (WB – 15)	11.0

c. Vehicle height

Traffic legislation limits the overall height of any other vehicle to 3.353 metres. The 15th percentile height of a passenger car has been established to be 1.3 m. This has been selected for design purposes as the passenger car is also an object that has to be seen by the driver in the cases of passing and intersection sight distance.

d. Driver Eye Height

The passenger car is taken as the critical vehicle for driver eye height and a figure of 1.05 metres is recommended. For buses and single unit vehicles a typical value is 1.8 metres and for semi-trailer combinations the height of the eye can vary between 1.9 metres and 2.4 metres.

e. Vehicle Turning Paths

The designer should allow for the swept path of the selected design vehicle as it turns. The swept path is established by the outer trace of the front overhang and the path of the inner rear wheel. This turn assumes that the outer front wheel follows the circular arc defining the minimum turning radius as determined by the vehicle steering mechanism.

It is assumed that the turning movements critical to the design of roadway facilities are done at low speeds. At these speeds, the turning behaviour of vehicles is mainly determined by their physical characteristics. The effects of friction and dynamics can safely be ignored. It is also assumed that groups of evenly spaced axles mounted on a rigid bogie act in the turn as a single axle placed at the centre of the group for the purpose of measuring critical turning dimensions.

Commercially available templates and computer software define the turning envelope of vehicles in forward motion and also support plotting of the turning envelope of reversing non-articulated vehicles. Prediction of the reversing behaviour of articulated vehicles is, however, very complex, mainly because this behaviour is inherently unstable, and additional turning controls come into play.

2.2.3 Selecting a Design Vehicle

In general, buses and heavy vehicles should be used as the design vehicle for cross section elements, with the car as the design vehicle for the horizontal and vertical alignment. For most major intersections along arterial roads or within commercial areas, it is common practice to accommodate the semi-trailer. The occasional larger vehicle may encroach on adjacent lanes while turning but not on the sidewalk. Many authorities designate and signpost specific truck routes. The intersections of two truck routes or intersections where trucks must turn to remain on a truck route should be designed to accommodate the largest semi-trailer combination expected to be prevalent in the turning traffic stream. Where local residential roads intersect truck routes or arterials, the intersections should be specifically designed not to accommodate trucks easily, in order to discourage them from travelling through the residential area.

On major haulage routes, large tractor-trailer combination trucks are prevalent and these routes should be designed to accommodate them. Raised channelizing islands are typically omitted in recognition of low pedestrian volumes and other constraints such as right of way and construction costs. The absence of raised islands also allows more manoeuvring area for large trucks.

2.3 Traffic

2.3.1 General

Factual information on expected traffic volumes is an essential input to design. This indicates the need for new roads or improvements to existing roads and directly affects the geometric features and design. Traffic flows vary both seasonally and during the day, and change over time, usually in relation to the change of economic activity in the areas of influence of the road. The designer should be familiar with the extent of these fluctuations to be able to assess the flow patterns. The directional distribution of the traffic and the manner in which its composition varies are also important parameters. A thorough understanding of the manner in which all of these parameters and characteristics interact is a matter that is a basic requirement of any realistic design.

2.3.2 Traffic Flow Information

Traffic flow information is represented in different situations by different statistics.

These can include:

- Hourly Traffic (HT) – the total number of vehicles passing a particular point on the road in one hour.
- Daily Traffic (DT) – the total number of vehicles passing a point on the road in 24 hours.
- Average Daily Traffic (ADT) – an average of Daily Traffic passing a point on the road over the duration of at least seven days.
- Monthly Average Daily Traffic MADT – an average of the number of vehicles passing a point on the road each day for a month.
- Annual Average Daily Traffic (AADT) – This is the average of the Daily Traffic passing a point on the road over 365 days.

Due to variations in traffic flow that occur through the day, through the week, and through different months and seasons, the longer the period that can be counted and included in the averaging process, the more reliable the statistic obtained is in indicating the true usage of the road.

The AADT represents the most reliable indicator of the overall use of a road, which will take account of daily monthly and seasonal variations of traffic flow, and provide the average over the year.

The MADT provides an indication of the use of the road that will take out fluctuations of traffic within the monthly and weekly cycles, but will not account for any fluctuations in the monthly volumes (for example peaks over holiday periods that occur only in some months, season variations due to climate etc.).

The ADT is an indication of the traffic usage of a road that has eliminated the variations that take place over a weekly cycle of working days and weekends, but will not have taken account of the variations in ADT that may occur in different weeks of the month (often traffic increases in the last or first week of the month),

while also not taking account of the monthly and seasonal variations that may occur over longer periods.

DT and HT are very short term indicators and need to be determined from longer term counts to be used with any certainty.

These statistics are illustrated for a typical rural road in Figure 5 which indicates that a short term count may be significantly from true AADT or ADT values.

The progression from daily count to the annual average daily traffic (AADT) is

$$DT_d > ADT > MADT_m > AADT$$

DT_d is traffic count for day d of the week. ADT is obtained by averaging DT_d for seven days. ADT for any month, m , of the year is $MADT_m$.

The operating equations are;

$$AADT = MADT_m / MTR_m \qquad \text{Equation 2-1}$$

and

$$AADT = MADT_m \times MTF_m \qquad \text{Equation 2-2}$$

MTR_m is the monthly traffic ratio for month m – which is the ratio between the $MADT$ of month m and the AADT.

MTF_m is the monthly traffic factor for month m – which is the factor by which $MADT_m$ is multiplied to get the AADT

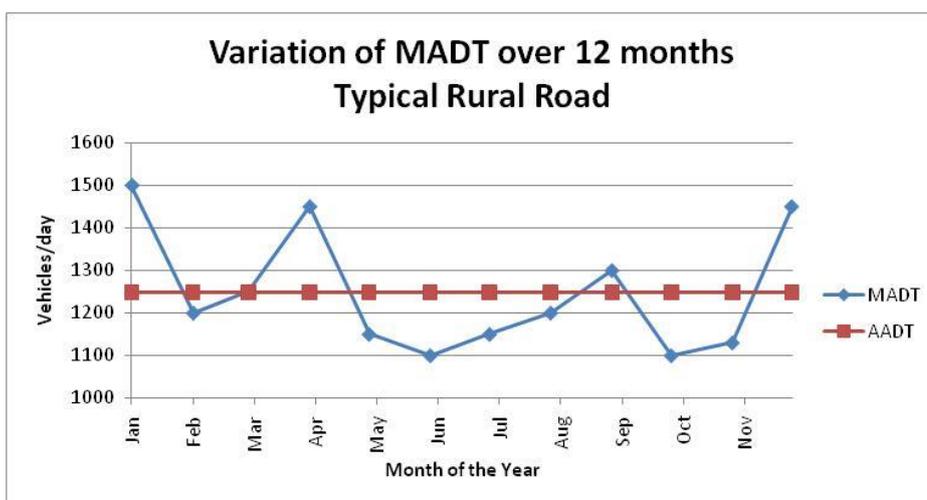
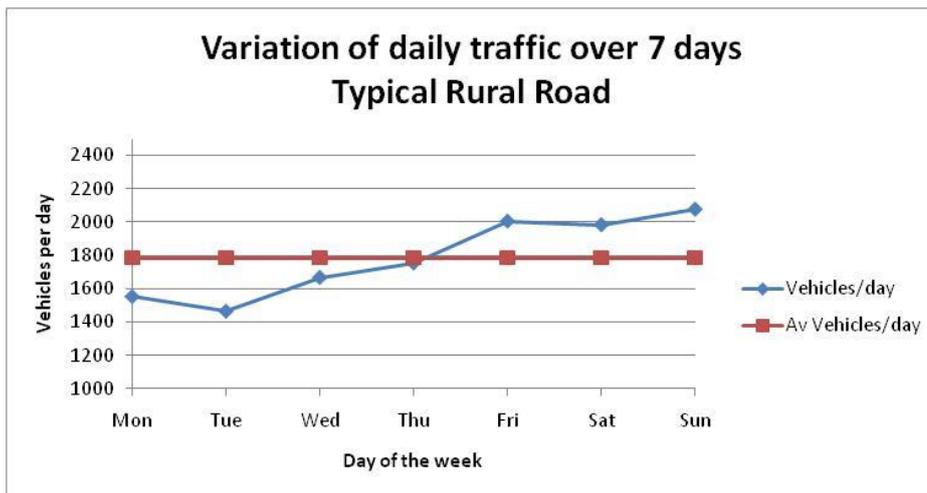
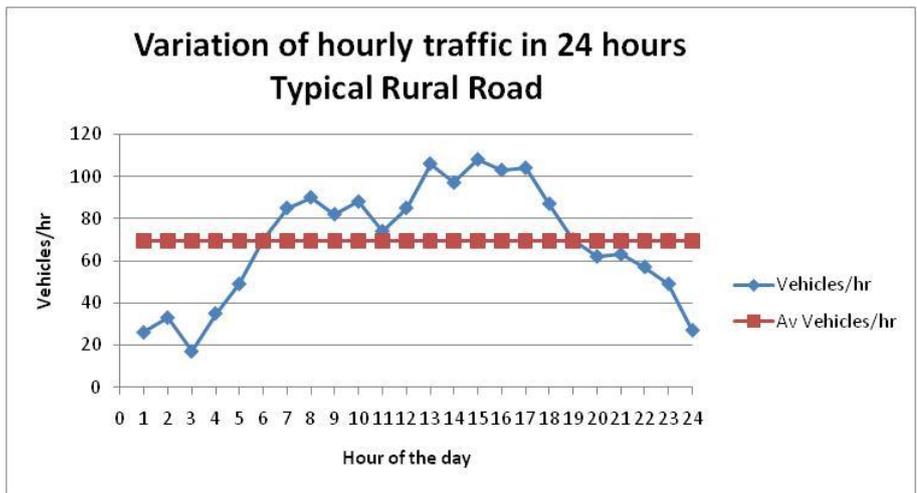


Figure 5: Illustration of Traffic Statistics counted over various periods

It must be noted that these factors and ratios vary with vehicle types, across different environments (urban, peri-urban and rural) and spatially across the country. Research will be needed to investigate the variations, after which the developmental effort will be to establish what the variations are for each vehicle type for use in Nigeria, i.e., incorporation in the Highway Manual.

2.3.3 Traffic, Vehicle Load and Origin Destination Surveys

To obtain estimates of HT, DT, ADT and AADT traffic needs to be counted. Counts can be done manually, with automatic traffic counters or with more sophisticated methods such as video monitoring. Sophisticated methods require specialised equipment and excessive analysis of video data etc., are not discussed here. Traffic counting for road planning, design and management is generally undertaken by a combination of automatic traffic counters with some manual traffic counts. For design purposes it is normal to require at least 7 day 24 hour continuous automatic counts to be carried out along the road at strategic points, where traffic volumes change. These counts should classify the vehicle stream, if possible. Manual counts, often carried out at intersections to obtain estimates of turning volumes as well as classified traffic on the main road, are then usually taken for 12 to 14 hours (depending on when traffic peaks occur) to provide detailed traffic movement information. These can then be associated with the 7 day 24 hour counts to obtain design traffic as discussed in paragraph 2.3.5.

Where available, ADT (7 day counts) should be associated with permanent count stations that provide annual counts that show the monthly and weekly variations in the traffic stream. This allows an estimate of the MTR ratio or MTF factor to be determined for the time the count is taken.

Usually, to assist in road pavement design, heavy vehicle axle weight surveys are also carried out to obtain axle and vehicle weights to determine the average number of Equivalent Standard Axles per class of vehicle.

Where road network planning and traffic diversion onto or off the road being designed is possible, Origin – Destination Surveys (O-D Surveys) will also be required to indicate where traffic is coming from and going to, to assess the propensity to change to or from the road project under consideration.

2.3.4 Vehicle Classes

In determining traffic flows it is essential that the vehicle stream is classified to allow road capacities and road pavement strengths to be accurately determined. The basic minimum classification is to establish the percentage of heavy vehicles in the ADT. However with inductive loop traffic counters, it is also possible to determine the speed of the vehicle, the length of the vehicle as well as various other factors. Manual counts can provide the full classification of vehicles by visual identification. The different classes of vehicle can vary depending on the purpose of the traffic counts but should include the classes included in Table 5 as a minimum.

Table 5: Classes of Vehicle for Traffic Counts

Class No.	Vehicle Class/subclass	Description
1	Motor cycle	
2a	Light vehicle	Cars, vans, pick-up<3.5 tonne GVM
2b	Light vehicle	Minibus<20 seats
3	Bus (Heavy vehicle)	Bus>20 seats
4	Heavy Vehicle	2 axles
5	Heavy Vehicle	3-4 axles
6	Heavy Vehicle	5+ axles

2.3.5 Estimating Design Traffic Volumes

Traffic counts as described will provide hourly flows, as well as ADTs over the 7 day counts. Flows should be analysed to provide design parameters such as the "hourly observed traffic volume" or the "thirtieth-highest hour" or "hundredth highest hour", which are commonly used for design purposes. Very short duration flows, such as for a five-minute period, are typically applied to studies of signalised intersections. If hourly flows are ordered from highest to lowest, it is customary, in rural areas, to design for the thirtieth highest hourly flow, i.e. that flow which is exceeded in only 29

hours of the year. This is because rural roads can have very high seasonal peaks and it is not economical to have a road congestion-free every hour throughout the year. In urban areas, seasonal peaks are less pronounced and the 100th highest hourly flow is considered a realistic flow level for design purposes.

To predict hourly flows, it is necessary to know the ADT and the peaking factor, β . The parameter, β , is a descriptor of the traffic flow on a given road and depends on factors such as the percentage and incidence of seasonal traffic, the relative sizes of the daily peaks, etc. The peaking factor can fluctuate between -0.1 and -0.4. A value of -0.1 indicates minimal seasonal peaking. This value of β should be used in urban designs. A value of -0.4 suggests very high seasonal peaks. As a general rule, a value of -0.2 could be used as being a typical value. The equation below can be used to estimate flows between the highest and 1030th highest hour.

$$Q_N = 0.072ADT(N/1030)^\beta \quad \text{Equation 2-3}$$

Where: Q_N = Two-directional flow in the N-th hour of year (veh/h)

ADT = Average Daily Traffic (veh/day)

N = Hour of year

β = Peaking factor.

It is interesting to note that the peak hour factor, K , quoted in the Highway Capacity Manual is often assumed for design purposes to be 0.15. Reference is commonly made to the 30th highest hour of the year as being the design hour. Applying a value of -0.2 to β , and assuming N to be 30, Q_N according to the equation above is 0.146 x ADT for the thirtieth highest hour.

Designers need to estimate future traffic flows for a road section. It is recommended that a design period of 20 years be used for forward planning. The 30th or 100th highest flow used in the design is that occurring in the design year. Stage construction or widening of roads over the design period can mean that a 10 year design period will result in an economical design.

The capacity of rural road sections is influenced by the following key characteristics:

- Road configuration - e.g. two-lane two way, multi-lane divided or undivided;
- Operating speed;
- Terrain;
- Lane and shoulder width;
- Traffic composition, and
- Gradients.

In the case of two-lane two- way roads, the following additional factors are important:

- Directional distribution of traffic flow; and
- Passing opportunities – sight distance, overtaking lanes, climbing lanes or slow vehicle turnout lanes.

2.3.6 Directional Distribution

Directional distribution of traffic is an indication of the tidal flow during the day. In urban areas, the morning peak traffic is typically inbound towards the central business district (with relatively low outward-bound flows), whereas the afternoon peak is in the reverse direction. It is important to realize that the design flow is actually a composite and not a single value. A road must be able to accommodate the major flow in both directions.

The actual distribution to be used for design purposes should be measured in the field. If an existing road is to be reconstructed, the field studies can be carried out on it beforehand. For new facilities, measurements should be made on adjacent roads from which it is expected the traffic will be diverted and modelling techniques applied. Directional distribution is relatively stable and does not change materially

from year to year. Relationships established from current traffic movements are normally also applicable to future movements.

2.3.7 Traffic Composition

Vehicles of different sizes and mass have different operating characteristics. Trucks have a higher mass/power ratio and occupy more roadway space than passenger cars. Consequently, they constitute greater impedance to traffic flow than passenger vehicles, with the overall effect that one truck is equivalent to several passenger cars. For design purposes, the percentage of truck traffic during the peak hours has to be estimated.

For design of a particular highway, data on the composition of traffic should be determined from traffic surveys. Truck traffic is normally expressed as a percentage of total traffic during the design hour in the case of a two-lane road; and as a percentage of total traffic in the predominant direction of travel in the case of a multi-lane road. It is not practical to design for a heterogeneous traffic stream and, for this reason trucks are converted to equivalent Passenger Car Units (PCUs). The number of PCU's associated with a single truck is a measure of the impedance that it offers to the passenger cars in the traffic stream. This topic is exhaustively addressed in the Highway Capacity Manual and is not discussed further here.

Passenger car unit equivalents have, in general, been derived from observations as illustrated in Table 6. The values offered serve only as an indication and the designer should refer to the Highway Capacity Manual for detailed calculation of PCU's appropriate to the different environments and circumstances.

Table 6: Equivalent Passenger Car Units

Vehicle Type	Passenger Car Units			
	Rural Roads	Urban Streets	Roundabouts	Traffic Signals
Cars and light vans	1.0	1.0	1.0	1.0
Heavy vehicles	3.0	1.75	2.8	1.75
Buses and coaches	3.0	3.0	2.8	2.25
Motorcycles	1.0	0.75	0.75	0.33
Pedal cycles	0.5	0.33	0.5	0.2

2.3.8 Traffic Growth

The design of new roads or of improvements to existing roads should be based on the future traffic expected to use the facilities. It is difficult to define the life of a "road" because major segments may have different lengths of physical life. Each segment is subject to variations in estimated life expectancy because of influences not readily subject to analysis such as obsolescence and unexpected changes in land use, resulting in changes in traffic volumes, pattern and load. Regardless of the anticipated physical life of the various elements of the road, it is customary to use a single value as the "design life". In essence, the road is expected to provide an acceptable level of service for this period. Whether or not any of its various components have a longer physical life expectancy than this design life is irrelevant. A typical value of design life is twenty years

2.4 Highway Capacity

2.4.1 General

The term capacity is used here to define the ability of a road to accommodate traffic under given circumstances. Factors to be taken into account are the physical features of the road itself and the prevailing traffic conditions.

a. Prevailing road conditions

Capacity figures for uninterrupted flow on highways have to be modified if certain minimum physical design features are not adhered to. Poor physical features that tend to cause a reduction in capacity are:

- Narrow traffic lanes. Lane widths of 3.65 m are accepted as being the minimum necessary for heavy volumes of mixed traffic, i.e. before capacity of the lane is reduced.
- Inadequate shoulders. The narrowness of shoulders or the lack thereof alongside a road causes vehicles to travel closer to the centre of the carriageway, thereby increasing the medial traffic friction. In addition, vehicles making emergency stops must, of necessity, park on the carriageway. This causes a substantial reduction in the effective width of the road, thereby reducing capacity.
- Lateral obstructions. Vertical obstructions such as poles, bridge abutments, retaining walls or parked cars that are located within about 1.5 m of the edge of the carriageway contribute towards a reduction in the effective width of the outside traffic lane.
- Imperfect horizontal or vertical curvature. Long and/or steep hills and sharp bends result in restricted sight distance. As drivers then have reduced opportunities to pass, the capacity of the facility will be reduced.

In addition to the above, the capacities of certain rural roads and the great majority of urban roads are controlled by the layouts of intersections. Physical features having considerable influence are the type of intersection, i.e. whether plain, channelized, roundabout or signalised, the number of intersecting traffic lanes and the adequacy of speed-change lanes. Unlike the physical features of the highway, which are literally fixed in position and have definite measurable effects on traffic flows, the prevailing traffic conditions are not fixed but vary from hour to hour throughout the day. Hence, the flows at any particular time are a function of the speeds of vehicles, the composition of the traffic streams and the

manner in which the vehicles interact with each other, as well as of the physical features of the roadway itself.

b. Capacity

The term "capacity" was introduced in the USA in the Highway Capacity Manual, in which it is defined as "the maximum number of vehicles that can pass a given point on a roadway or in a designated lane during one hour without the traffic density being so great as to cause unreasonable delay, hazard, or restriction to the drivers' freedom to manoeuvre under the prevailing roadway and traffic conditions". This definition gives a reasonable method of approach but, in practice, it is necessary to choose one or more arbitrary criteria of what constitutes restriction of traffic movement, or "congestion". The Highway Capacity Manual procedure must however be used for specific road capacity designs.

For typical conditions and to balance financial, safety and operational considerations it is recommended that the capacity of a two-lane rural road be taken on average as being between 10 000 and 12 000 vehicles per day while, for freeways, consideration could be given to changing from a four-lane to a six-lane freeway when the traffic flow is of the order of 35 000 to 40 000 vehicles per day.

2.5 Design Controls imposed by Human Behaviour

2.5.1 Drivers

For effective road design and operation an appreciation of driver performance is essential. Design should be compatible with human capabilities (drivers and other road users), to limit opportunities for errors and accidents. Consequently human performance is a vital input into the design task.

Road user behaviour is not consistent and there is a wide range of response. For roadway design the following principles should apply:

- A roadway should conform with drivers' expectations from previous experience.
- Drivers should be presented with clear indications of what is expected of them

a. Driver Workload and Expectations

Driver workload comprises

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

These tasks require the driver to:

- Receive and process inputs
- Consider outcome of alternative actions
- Decide on the most appropriate action and observe its effects through the reception and processing of new information

Problems can occur in the sequence of tasks, arising from capability of drivers and the interface between the driver and other components of the traffic system. These include insufficient input for the task at hand (e.g. complex intersection layouts, insufficient sight distance or night time visibility). When drivers become overloaded they shed part of the input to deal with tasks considered most important. Drivers are not perfect and may make the incorrect decision.

The designer must provide all the information needed by the driver to make the correct decision, at the same time ensuring that the information is provided in such a manner that it does not exceed the driver's ability to absorb it. The following quote is from the American Association of State Highway and Transportation Officials:

“A common characteristic of many high-accident locations is that they place large or unusual demands on the information processing capabilities of drivers. Inefficient operation and accidents usually occur where the chance for information-handling errors is high. At locations where the design is deficient, the possibility of error and inappropriate driver performance increases”

Past experience develops into a set of experiences that allows for anticipation and forward planning. These allow the driver to respond to common situations predictably and successfully. If expectations are violated, problems can occur, resulting from wrong decisions or long reaction time. There are three types of driver expectancy:

- Continuation Expectancy

This is the expectation that events of the immediate past will continue. For example, it results in short headways, as drivers expect that the leading vehicle will not suddenly change speed.

- Event Expectancy

This is the expectation that events that have not occurred will not occur. For example it results in disregard for railway (at-grade) crossings or minor intersections, because drivers expect that no hazard will occur where none has been seen before.

- Temporal Expectancy

This is the expectation that where events are cyclic (e.g. traffic signals), the longer a given state prevails, the greater the likelihood that changes will occur. This is a reasonable expectation but can result in inconsistent responses. For example some drivers may accelerate towards a green signal, as it is increasingly likely it will change, whereas others may decelerate. To respond to this the traffic system should be consistent to encourage consistent driver behaviour.

The combined effect of these expectancies is that:

- Drivers tend to anticipate upcoming situations and events that are common to the road they are travelling.
- The more predictable roadway features the less likely is the chance for errors.
- Drivers experience problems when surprised.
- In the absence of evidence to the contrary, drivers assume that they will only have to react to standard situations.
- The roadway and its environment upstream create an expectation of downstream conditions. Drivers experience problems in transition areas and locations with inconsistent design or operation.
- Expectancies are associated with all levels of driving performance and all aspects of the driving situation. These expectancies include speed path, direction, the roadway, the environment, road geometrics, traffic operation and traffic control devices.

b. Driver Reaction

It takes time to process information. After a person's eyes detect a given situation, a period of time elapses before muscle reaction occurs. Reaction time is appreciable and differs between persons. It also varies in an individual, increasing by fatigue, drinking or other causes. The AASHTO brake reaction time for stopping is set at 2.5 seconds.

Drivers can face complex situations which require more than a simple response such as steering adjustment or application of brakes. Complex decisions are time consuming and highway design should allow for one decision at a time. Anything up to 10 seconds of reaction time may be required in complex situations.

c. Driver Response

Designs should strive to satisfy the following criteria:

- Driver expectation is recognised. Unexpected, unusual or inconsistent design or operational situations should be avoided.
- Predictable behaviour should be encouraged through familiarity. The range of intersection designs should be limited.
- Consistency of design and driver behaviour is maintained from element to element.
- Information provided should decrease driver uncertainty.
- Clear sight lines and adequate sight distance are provided.

2.5.2 Other Road Users

Pedestrians

The interaction of pedestrians and vehicles should be carefully considered in road design. It should be noted that 50% of road fatalities are pedestrians.

Pedestrian actions are less predictable than those of motorists. Pedestrians tend to select the shortest distance between two points. They have a resistance to changes in elevation when crossing roadways and tend to avoid a crossing structure over or under a road which is inconvenient.

Walking speeds vary from 1.2m/s to 1.8m/s.

The age of the pedestrian is a factor that may result in behaviour leading to collisions. It is recommended that older pedestrians be accommodated by using straight forward designs that minimize crossing width and assume lower walking speed.

Pedestrian safety is enhanced by the provision of

- Median refuge islands of sufficient width at wide intersections.

- Lighting at locations that demand multiple information gathering and processing.

Cyclists

Bicycle use is considerable and should be allowed in the road design process. Accommodation of bicycle traffic can be enhanced by the following improvements:

- Paved shoulders
- Wider outer traffic lanes if shoulders are not provided
- Cycle safe drainage grids
- Adjusting manhole covers to grade
- Maintaining a smooth, clean surface

At some locations it may be appropriate to supplement the road system by providing specifically designated cycle paths.

3 Design Elements

3.1 Geometric Plan Elements

3.1.1 Sight Distance

The ability to see ahead is of the utmost importance in the safe and efficient operation of vehicles on a road. Sight distances of sufficient length must be provided so that drivers can control the operation of their vehicles to avoid striking an unexpected object on the travelled way.

Two sight distances are generally considered in design: Stopping Sight Distance [SSD] and Passing Sight Distance [PSD]. SSD is the minimum sight distance to be provided at all points on multilane and 2-lane roads. SSD is also to be provided for all elements of intersections at grade and interchanges, including private road connections.

PSD should be provided on 2-lane highways at frequent intervals and over a substantial portion of their length.

A third value, Decision Sight Distance [DSD] is also taken into account in the design process particularly for freeway and interchange design. It 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. As 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, its values are substantially greater than stopping sight distances.

3.1.2 Stopping Sight Distance

Stopping sight distance is the minimum sight distance required on a road to enable a vehicle travelling at the design speed on a wet pavement to stop before reaching an object in its path.

Stopping sight distance is the sum of two distances:

- the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied and
- the distance required to stop the vehicle from the instant brakes are applied.

These are referred to as brake reaction distance and stopping distance respectively.

Stopping sight distance is measured from the driver's eyes, which are assumed to be 1.05 metres above the pavement surface to all objects 600mm high on the road

3.1.3 Passing Sight Distance

Passing Sight Distance is the minimum sight distance 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 the oncoming vehicle come into view after the overtaking manoeuvre is started. The sight distance available for passing at any place is the longest distance at which a driver, whose eyes are 1.05 metres above the pavement surface, can see the top of an object 1.30 metres high on the road. This height will allow a driver to see the roof of an oncoming car.

Passing sight distance is considered only on two lane roads. At critical locations, additional lanes with stopping sight distance shall be constructed when the provision of passing sight distance for two lane roads is less economical. Lengthy sections of highway with no passing opportunities are hazardous. Conditions for at least one passing opportunity per kilometre shall be provided. The effect of sight restrictions or capacity for the whole length of highway shall also be used as a criterion for providing adequate passing distance

Appreciably grades increase the sight distance required for safe passing. Passing is easier for the vehicle travelling downgrade because the overtaking vehicle can accelerate more rapidly than on the level and thus reduce the time of passing, but the overtaken vehicle may also accelerate easily so that a dangerous situation akin to a race may result.

The sight distances required to permit vehicles travelling upgrade to pass with safety are greater than those required on level roads due to reduced acceleration of the passing vehicle, which increases the time of passing, and due to the likelihood of opposing traffic speeding up, increasing the distance travelled by it. Compensating for this somewhat are the factors that the passed vehicle frequently is a truck that usually loses some speed upon appreciable upgrades, and that many drivers are aware of the greater distances needed for passing compared to passing on level conditions.

If overtaking is to be performed safely on upgrades, the passing sight distance should be greater than the derived minimum. Specific adjustments for design use are unavailable, but the designer should recognize the desirability of increasing the minimums shown in Table 7.

Passing distance criteria do not apply to multilane divided highways where opposing traffic streams are segregated.

Table 7 shows minimum standards for sight distances related to design speed.

Table 7: Sight Distance Standards

Design Speed Km/h	Minimum Sight Distance	
	Stopping (meter)	Passing (meter)
50	65	345
60	85	410
80	130	540
100	185	670
110	220	730
120	250	775
130	285	815

Stopping sight distance is affected by vertical grade and obstacles along the side of the roadway on horizontal alignment. These are to be used in determining stopping sight distance criteria.

Table 8: Stopping Sight Distance on Grades

Design Speed (Km/h)	Stopping sight distance (m)					
	Downgrades			Upgrades		
	3%	6%	9%	3%	6%	9%
50	66	70	74	61	59	58
60	87	92	97	80	77	75
80	136	144	154	123	118	114
100	194	207	223	174	167	160
110	227	243	262	203	194	186
120	263	281	304	234	223	214
130	302	323	350	267	254	243

The minimum stopping sight distance for cycle tracks shall be 18 metres.

3.1.4 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:

- Drivers must 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 stopping. Even with an appropriate complement of standard traffic control devices, stopping sight distances may not provide sufficient visibility for drivers to corroborate advance warning 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 apt 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.

Table 9: Decision Sight Distance

Design Speed (km/h)	Situations					
	Interchanges: distance to (metres)		Sight nose to nose	Lane closure, drop, merges. Sight distance to area (metres)	Lane shift Sight distance to beginning of shift (metres)	Intersections Sight distance to turn lane
	Right Exit	Left Exit				
50	N/A	N/A	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	

The minimum decision sight distances that should be provided for specific situations are shown in Table 9. 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 offered for the left side exit, the designer should bear in mind that exiting from the left is in total conflict with driver expectancy and is highly undesirable. The only reason for providing this value is to allow for the remote eventuality that a left side exit has to be employed.

In measuring decision sight distances, the 1.05m eye height and 0.0m object height have been adopted.

3.1.5 Horizontal Alignment

a. General

One of the most important considerations in highway design is horizontal alignment. Horizontal and vertical alignments are the two primary design control elements. Therefore, the careful coordination of these two elements with design speed, and curvature, their interrelationship with super elevation, sight distance, aesthetic principles and drainage is of primary importance in the early stages of design. After a highway location is determined and right of way purchased, further modification in the horizontal and vertical alignment is both difficult and expensive.

In setting out the horizontal alignment, the following major factors shall be considered:

- Safety
- Grade Profile
- Type of Highway
- Design Speed
- Topography
- Cost (construction, maintenance, operation)

Of these considerations, safety shall be considered to be of paramount importance. The horizontal alignment provided shall ensure that the stopping sight distance for the design speed shall be adequate at all points on the highway. In rolling or hilly terrain the alignment and grade profile may constitute a major control when designing a two-lane highway. In urban areas right-of-way limitations, especially at interchanges, may influence alignment more than any other single factor. Topography controls both curve radius and design speed to a large extent. The design speed, in turn, controls sight distance. The economics of construction, maintenance and operation must be balanced carefully against the other factors in

order to produce the safest alignment consistent with the design classification of the highway.

The specific design elements of design speed, curvature, super elevation, and sight distance are based on empirical or formula derivation. However, some additional considerations based on experience and/or what is considered good design practice are outlined in the following paragraphs.

b. General Controls for Horizontal Alignment

(i) Locating Curves

Alignment shall be as direct as possible but consistent with topography. A flowing alignment consistent with the contours is aesthetically more pleasing than one with long tangents. Natural slopes and plant growth can thus be preserved. Notwithstanding these aesthetic qualities, passing sight distance necessitates long tangents on two-lane highways with passing sight distance on as great a percentage of the length of highway as feasible. In general, the number of curves shall be kept to a minimum.

Sudden reductions in standards introduce the element of surprise to the driver and should be avoided. Where physical restrictions on curve radius cannot be overcome and it becomes necessary to lower standards, the design speed between successive curves shall not change more than 15 km/h. Under no conditions shall a curve for a design speed lower than the design speed for the project be introduced at the end of a long tangent or at any locations conducive to a high approach speed.

(ii) Curve Length and Central Angle

Winding alignment composed of short curves shall be avoided since it results in erratic operation. In general, the length of curve should be at least 155 metres long for a central angle of 5 degrees. The minimum length shall be increased 30 metres for each 1 degree decrease in the central angle. Sight distance or other safety considerations shall not be sacrificed thereby. 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 use of minimum curves derived from a given design speed shall be avoided wherever possible. Flat curves should generally be used, retaining the minimum for the most critical conditions. In general the central angle of each curve shall be as small as physical conditions permit in order to achieve the shortest possible route. This central angle shall be absorbed in the longest possible curve, but on two-lane highways the requirements of passing sight distance govern.

(iii) Tangents for Straights Affording Passing Opportunities

These tangents shall be used to provide passing opportunities on two lane roads. A 750 meter tangent is considered adequate. Passing tangents shall be provided as frequently as possible in keeping with the terrain opportunities. Shorter radii ensuring greater length of intervening tangent shall be preferred to sweeping curves of large radii which reduce the lengths of intervening tangents and can restrict passing opportunities.

However, sharp curves at the end of passing tangents and especially long tangents shall not be used.

Sudden changes from areas of easy curvature to areas of sharp curvature shall be avoided. Where sharp curvature must be introduced this shall be achieved by successive and gradual reductions in curvature.

(iv) Compound Curves

Compound curves shall in general be avoided. Provided the marginal increase in cost is small a simple curve shall be preferred. On a compound curve, the shorter radius shall be at least 2/3 of the longer radius. The total arc length of a compound curve shall not be less than 150 metres.

Compound curves with large differences in curvature introduce the same problems that arise at a tangent approach to a circular curve.

(v) *Reverse Curves*

Reverse curves without an intervening tangent should not be used. In cases where there are severe physical limitations, necessitating the use of reverse curves, the minimum tolerable length of tangent shall be 120 metres. In this respect, great care must be exercised to produce a safe design between the minimum overtaking and the no overtaking zone limits. Abrupt reversal in alignment makes it difficult for a driver to keep within his own lane.

Using reverse curves also makes it difficult to super elevate both curves adequately and thus hazardous and erratic operation may result. Sufficient length of tangent between the two curves must be provided to allow for an adequate super elevation transition between curves.

(vi) *Flat or Broken-back Curves*

This consists of two curves in the same direction joined by a short tangent less than 500 metres long. Broken-back curves are potential hazards because most drivers do not expect succeeding curves to be in the same direction and the sub-conscious habit can result in motorists leaving the road especially where visibility is impaired. In addition, broken-back curves are not aesthetically desirable. If the length of intervening tangent is less than 60 metres a simple or compound curve shall be used. In general, a compound curve wherein there is some degree of continuous super elevation shall preferably be used instead of the broken-back curves.

(vii) *Curvature on Fills*

Curvature should be avoided on high long fills except if the curves are of large radii. In the absence of cut slopes, shrubs, trees, etc., above the roadway, it is difficult for drivers to perceive the extent of curvature and

adjust their operation to the conditions. Further, any vehicle out of control on a high fill is in an extremely hazardous position. Adequately designed and maintained guardrails are necessary to alleviate this danger.

c. Maximum Degree of Curvature

The maximum degree of curvature, or the minimum radius, is a limiting value for a given design speed determined from the maximum rate of super elevation and the maximum side friction factor. Use of sharper curvature for that design speed would call for super elevation beyond the limit considered practical or for operation with tyre friction beyond safe limits, or both. Thus this curvature is a significant value in alignment design. Also it is a necessary and important value for determination of super elevation rates for flatter curves.

The minimum safe radius can be calculated directly from the standard curve formula:

$$R = \frac{V^2}{127(e + f)} \quad \text{Equation 3-1}$$

Where:

R = minimum safe radius in metres

V = design speed, in km/h

e = super elevation rate, in %

f = coefficient of side friction

Table 10 below is based on the recommended values for the co-efficient of side friction suggested by AASHTO and has been presented for varying rates of super elevation.

The calculated results of the above formula are shown in Table 11: Recommended Super Elevation Rates.

Table 10: Minimum Radius Using Limiting Values of e and f.

Design Speed	Maximum e%	Maximum f	Total $(e/100 + f)$	Calculated Radius (m)	Rounded Radius
50	4.0	0.19	0.23	85.6	86
60	4.0	0.17	0.21	135.0	135
70	4.0	0.15	0.19	203.1	203
80	4.0	0.14	0.18	280.0	280
90	4.0	0.13	0.17	375.2	375
100	4.0	0.12	0.16	492.1	492
50	6.0	0.19	0.25	78.7	79
60	6.0	0.17	0.23	123.2	123
70	6.0	0.15	0.21	183.7	184
80	6.0	0.14	0.20	252.0	252
90	6.0	0.13	0.19	335.7	336
100	6.0	0.12	0.18	437.4	437
110	6.0	0.11	0.17	560.4	560
120	6.0	0.09	0.15	755.9	756
130	6.0	0.08	0.14	950.5	951
50	8.0	0.19	0.27	72.9	73
60	8.0	0.17	0.25	113.4	113
70	8.0	0.15	0.23	167.8	168
80	8.0	0.14	0.22	229.1	229

Design Speed	Maximum e%	Maximum f	Total $(e/100 + f)$	Calculated Radius (m)	Rounded Radius
90	8.0	0.13	0.21	303.7	304
100	8.0	0.12	0.20	393.7	394
110	8.0	0.11	0.19	501.5	501
120	8.0	0.09	0.17	667.0	667
130	8.0	0.08	0.16	831.7	832
50	10.0	0.19	0.29	67.9	68
60	10.0	0.17	0.27	105.0	105
70	10.0	0.15	0.25	154.3	154
80	10.0	0.14	0.24	210.0	210
90	10.0	0.13	0.23	277.3	277
100	10.0	0.12	0.22	357.9	358
110	10.0	0.11	0.21	453.7	454
120	10.0	0.09	0.19	596.8	597
130	10.0	0.08	0.18	739.3	739

d. Maximum Curvature vs Stopping Sight Distance

Where there are sight obstructions – walls, cut slopes, wooded areas, buildings, hedgerows, high farm crops, guardrails under certain conditions, etc., on the inside of curves, Figure 6 should be used to check for adequate stopping sight distance. Graphic solutions are also appropriate.

3.1.6 Super Elevation

a. General

A vehicle travelling on a horizontal curve exerts an outward force called centrifugal force. To resist this force and maintain the desired design speed, highway curves need to be super elevated. Super elevation may be defined as the rotation of the roadway cross section in such a manner as to overcome the centrifugal force that acts on a motor vehicle traversing a curve.

On a super elevated highway, the centrifugal force is resisted by:

- the weight component of the vehicle parallel to the super elevated surface and;
- the side friction between the tyres and the pavement.

It is impossible to balance centrifugal force by super elevation alone, because for any given curve radius, a certain super elevation rate is exactly correct for only one operating speed around the curve. At all other speeds, there will be a side thrust outward, or inward relative to the curve centre, which must be offset by side friction.

If the vehicle is not skidding, the forces are in equilibrium as represented by the equation:

$$e + f = \frac{V^2}{127R} \quad \text{Equation 3-2}$$

Where

e = super elevation rate of slope in %

f = side friction factor or coefficient side friction.

R = radius of curve, in metres (m)

V = design speed, in km/h

The value of “ f ” has been researched at length by AASHTO and the values that are recommended in “A Policy on Geometric Design of Highways and Streets” have been adopted in this section.

Super elevation should not be so excessive as to cause a stationary vehicle to slide down the cross slope, regardless of the nature and condition of the road surface.

Super elevation rate shall not be less than the rate of crown slope, i.e. camber or cross fall.

b. Standards for Super Elevation

Table 10 is used to give standard super elevation rates for various curve radii and design speeds. The maximum value of super elevation shall not exceed 10%.

Super elevation slopes on curves shall extend the full width of shoulders, except that the shoulder slope on the low side shall not be less than the minimum shoulder slope used on tangents.

On rural two lane roads different super elevation slopes for each half of the road shall not be used: super elevation shall remain a plane for the full width of the roadbed except on transitions.

Lower super elevation rates may be necessary in urban areas where restricted speed zones at street intersections are controlling factors. Established street grades, kerbs and drains may prove difficult to alter. Different rates for each half of the roadbed may even be tolerated. In warping street areas, in order to have effective drainage, adverse super elevation and roll-over shall be avoided.

Recommended rates of e_{\max} are shown in Table 11

Table 11: Recommended Super Elevation Rates

Domain	Suggested range for e_{max}
Rural roads	5% to 6%
High speed urban roads	4% to 6%
Minor urban roads	2% to 4%

Table 12 is a guideline for super elevation rates at intersections.

Table 12: Super Elevation Rates for Curves at Intersections

Curve Radius Meter	Range in Super Elevation Rate (%) for Intersection Curves with Design Speed (Km/h)					
	25	30	40	50	55	65
20	2 – 9					
30	2 – 7	2 – 10				
45	2 – 5	2 – 8	4 - 10			
70	2 – 4	2 – 6	3 – 8	6 – 10		
90	2 – 3	2 – 4	3 – 6	5 – 9	8 - 10	
130	2 – 3	2 – 3	3 – 5	4 – 7	6 – 9	9 – 10
180	2	2 – 3	2 – 4	3 – 5	5 – 7	7 – 9
300	2	2 – 3	2 – 3	3 – 4	4 – 5	5 – 6
450	2	2	2	2 – 3	3 – 4	4 – 5
600	2	2	2	2	2 – 3	3 – 4
900	2	2	2	2	2	2 - 3

c. Methods of Attaining Super Elevation

There are three generally recognized methods of rotating the pavement cross section in order to attain super elevation. The pavement may be revolved about:

- the centre line;
- the inside edge (edge nearer the centre of curve); and
- the outside edge.

For undivided highways the axis of rotation for super elevation is usually the centre line of the road. However, in special cases such as desert roads where curves are preceded by relatively long level tangents, the plane of the super elevation may be rotated about the inside edge of the pavement to improve perception of the curve. In level country, drainage pockets caused by super elevation may be avoided by changing the axis of rotation from the centreline to the inside edge of the pavement.

On divided multilane highways, the two sides are rotated individually about the edge of the pavement nearest the median taking into consideration the ultimate pavement width. If rotation occurs about the centre of each pavement, a hazardous and unsightly saw-tooth section may result. This does not apply to a median that is both narrow and traversable by traffic, in which case the axis of rotation usually lies in the centre of the median. Figure 8 shows the various stages in the method of obtaining super elevation according to method of rotation.

In the diagrammatic profiles of Figure 8 the tangents profile control lines result in angular breaks at cross sections A, C and E. For general appearance and safety, these breaks should be rounded in final design to smooth flowing lines by insertion of vertical curves as shown in Figure 9.

In the design of divided highways, the inclusion of a median in the cross section alters somewhat the super elevation runoff treatment. Depending upon the width of median and its cross section, there are three general cases for super elevation runoff design:

Case 1: The whole of the travelled way, including the median is super elevated as a plane section.

Case 2: The median is held in a horizontal plane and the two pavements separately are rotated around the median edges.

Case 3: The two pavements are separately treated for runoff with a resultant variable difference in elevation at the median edges.

Case 1 is limited to narrow medians and moderate super elevation rates to avoid substantial difference in elevation of the extreme pavement edges because of the median tilt. For the most part, the method of rotation about the median centreline is used. Diagrammatic profile controls will be similar to those in Figure 8 except that the two median edges will appear as profiles only slightly removed from the centreline. This case will apply if widening for future lanes is anticipated in the median consideration and the median width is reduced.

Case 2 can apply for any width median, but has greatest application with medians of intermediate width. By holding the median edges level the difference in elevation of the extreme pavement edges is limited to that of the previous super elevation. While this type of cross section may be utilized with wide medians, especially in flat terrain, its general use is limited to medians not wide enough to favour the Case 3 treatment. Runoff design for Case 2 usually has the median edge profile as the control. One pavement is rotated about its lower edge and the other about its higher edge. The diagrammatic profile controls are those shown in Figure 8 with the centreline grade control the same for the two pavements.

The Case 3 design is preferable on other than narrow medians in that the difference in elevation of the extreme pavement edges is minimized by a compensating slope across the median. While this might be done on a median of intermediate width by use of sharply sloped sections, usually a fairly wide median is necessary to develop left shoulder areas and desired gentle slopes between. With medians of about 9 metres or more in width, it is possible to separately design the profiles and super elevation runoff for the two pavements, with a tie-in control at the median edges. Accordingly, the pavement rotation can be made by the method otherwise considered appropriate, i.e. any of the methods in Figure 8.

Table 13: Length Required for Super Elevation Runoff – 2 Lane Pavements

Super Elevation Rate %	L = Length of Runoff in Metres for Design Speed, Km/h of:					
	50	60	80	100	105	110
2 4 6 8 10	3.65 Meter Lanes					
	15	15	15	20	20	20
	25	25	30	35	40	40
	35	40	45	50	55	60
	45	50	60	70	75	80
55	65	75	85	90	100	
Minimum Design Length Regardless of Super Elevation (m)	35	40	45	55	60	70

d. Super Elevation Transition.

Super elevation transition is the general term denoting the length of highway needed to accomplish the change in cross slope from a normal crown section to the fully super elevated section or vice versa. To meet the requirements of comfort and safety, super elevation runoff should be affected uniformly over a length adequate for the likely travel speeds.

The super elevation transition can be divided into two sections defined as follows:

- *Tangent Runoff or Run-out.* Beginning at the section of the road at which the normal crown section begins to change (tangent point), this is the distance in which the level of the edge of pavement is raised to a horizontal plane with the centreline grade through the axis of rotation. Super elevation commences from this section.

- *Super Elevation Runoff*: This is the distance in which the super elevation is achieved from its zero value to the maximum design value. This maximum value is then retained as a cross slope along the designated curve section after which the reverse process commences. There is a large degree of arbitrariness attaching to determination of the length of super elevation runoff. The designer can thus vary the relative gradient to accommodate other elements of the design, such as the distance between successive curves or the distance to the following intersection. It is, however, suggested that the relative gradients offered in Table 14 should provide a pleasing appearance and the designer should at least attempt to achieve relative gradients of a similar magnitude.

Table 14: Maximum Relative Gradients

Design Speed (km/h)	Maximum Slope (%)	Equivalent Maximum Relative Slope (1:)
50	0.65	154
60	0.60	167
80	0.50	200
100	0.44	227
110	0.41	244

Length of runoff on this basis is proportional to the total super elevation, which is the product of the lane width and the super elevation rate.

The minimum design super elevation runoff lengths for pavements wider than two lanes should be as follows:

1.5 lanes rotated: 1.25 times length for one lane rotated

<i>2 lanes rotated:</i>	1.5 times length for one lane rotated
<i>3 lanes rotated:</i>	2.0 times length for one lane rotated

For other values of lanes (n) rotated, use the equation: $\text{Factor} = [1+0.5(n-1)]/n$

The super elevation runoff length within the curve shall be 1/3rd of the super elevation transition length.

Super elevation transitions shall be designed as shown in Figure 9. This figure covers the following cases:

- Undivided highways where the super elevation is rotated about the highway centreline as on two lane roads and divided highways with a narrow traversable median.
- Both divided and undivided highways where rotation occurs about one edge of the travelled way.

After a super elevation transition is computed, profiles of the pavement edge should be plotted and irregularities removed by introducing smooth curves at break points. For wide pavements it is often advantageous to plot intermediate profiles. On curved interchange roadways a pronounced and unsightly sag may be developed on the low side of the super elevation. This is corrected by adjusting the grades on the two edges of pavement through the curve.

Super elevation transitions for compound curves shall be accomplished as shown in

.
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Exhibit 3.32 of this publication shows the table for super elevation runoff L_r (1) for two lane highways and L_r (2) for four lane highways etc., in the metric units for varying speeds, from 20 km/h to 100 km/h.

3.1.7 Transition Curves

a. General

Any vehicle entering a circular curve does so by following a spiral path. For most curves, this transition can be accommodated within the limits of normal lane width. At minimum radii for the design speed, longer transition paths are followed and, if these occur on narrow lanes, the shift in lateral position may even lead to encroachment on adjacent lanes. Under these circumstances, it may be convenient to shape the horizontal alignment such that it more accurately reflects the path actually followed by a vehicle entering the circular curve. Various curves can be used to provide a transition from the tangent to the circular curve. Whatever form is used, it should satisfy the conditions that:

- It is tangential to the straight;
- Its curvature should be zero (i.e. infinite radius) on the straight;
- The curvature should increase (i.e. radius decrease) along the transition;
- Its length should be such that, at its junction with the circular curve, the full super elevation has been attained;
- It should join the circular arc tangentially, and
- The radius at the end of the transition should be the same as that of the circular curve.

Candidate curves are the lemniscate, the cubic parabola (also known as Froude's spiral), and the clothoid (also known as Euler's spiral). The cubic parabola achieves a maximum value and then flattens out again and is thus not a true spiral and the lemniscate requires an unacceptable length of arc to achieve the desired radius. The clothoid, which has the relationship whereby the radius, R , at any point on the spiral varies with the reciprocal of the distance, L , from the start of the spiral, is thus the preferred option. See Figure 10 for elements of a clothoid transition. Expressed mathematically, this relationship is

$$R = A^2/L \quad \text{Equation 3-3}$$

Where:

A is a constant called the spiral parameter and has units of length.

The length of a transition curve may be based on one of three criteria, these are as follows:

- Rate of change of centripetal acceleration, essentially a comfort factor, varying between 0.4 m/s³ and 1.3 m/s³;
- Relative slope as proposed in Table 14; or
- Aesthetics.

Since relative slope is applied to curves where transitions are not provided, its use also for transitioned curves would be a sensible point of departure. For the various design speeds, a radius corresponding to a specified centripetal acceleration can be calculated. These radii are listed in Table 15 for an acceleration of 1.3 m/s². There is little point in applying transition curves to larger radii where the centripetal acceleration would be lower.

b. Setting Out of Transitions

Application of the spiral has the effect that the circular curve has to be offset towards its centre. It is thus located between new tangents that are parallel to the original tangents but shifted from them by an amount, s , known as the shift. The value of the shift is given by

$$s = L^2/2R \quad \text{Equation 3-4}$$

where

L = selected length of transition curve (m)

R = radius of circular curve (m)

The starting point of the spiral is located at a distance, T , from the Point of Intersection (PI) of the original tangents with

$$T = (R + s) \tan \theta/2 + L/2 \quad \text{Equation 3-5}$$

Where θ = deviation angle of the circular curve

The most convenient way to set out the spiral is by means of deflection angles and chords and the deflection angle for any chord length, l , is given as

$$a = l^2/6RL \times 57,246 \quad \text{Equation 3-6}$$

Table 15: Maximum Radii for use in Spiral Transition Curves

Design Speed (km/h)	Maximum Radius (m)
40	100
50	150
60	210
70	290
80	380
90	480
100	590
110	720
120	850
130	1000

3.1.8 Pavement Widening

Pavements on curves sometimes are widened to make operating conditions on curves comparable to those on tangents. Pavement widening is needed on certain open highway curves because:

- the vehicle or truck occupies a greater width, since the rear wheels generally track inside front wheels in rounding curves, and
- the drivers have some difficulty in steering their vehicles to hold to the centre of the lane.

Widening should be attained gradually on the approaches to the curve to ensure a reasonably smooth alignment on the edge of pavement and to fit the paths of vehicles entering, or leaving the curve. The following are the principal points of concern in design. They apply to both ends of highway curves.

- On simple curves, widening should be applied on the inside edge of pavement only. The final marked centreline and desirably any central longitudinal joint should be placed midway between the edges of the widened pavement.
- Curve widening should be attained gradually over a length sufficient to make the whole of the pavement fully usable. While a long transition is desirable for traffic operation, it may result in narrow pavement slivers that are difficult and expensive in construction. Preferably, widening should be attained over the super elevation runoff length, but shorter lengths are sometimes used. Changes in width normally should be effected in a distance of 30 to 60 metres.
- From the standpoint of utility and appearance, the pavement edge through the widening transition should be a smooth, graceful curve. A tangent transition edge should be avoided; the transition ends should be rounded to avoid an angular break at the pavement edge.
- Pavement widening areas can be fully detailed on construction plans or else general controls cited on construction or standard plans and final details left to the field engineer.

Pavement widening should be designed in accordance with Table 16.

Table 16: Travelledway Widening Standards

Radius of Curve (m)	Travelledway width = 7.2m					Travelledway width = 6.6m					Travelledway width = 6.0m							
	Design Speed (Km/h)					Design Speed (Km/h)					Design Speed (Km/h)							
	50	60	70	80	90	100	50	60	70	80	90	100	50	60	70	80	90	100
3000	0	0	0	0	0	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.5	0.5	0.6	0.6	0.6	0.6
2500	0	0	0	0	0	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.5	0.6	0.6	0.6	0.6	0.6
200	0	0	0	0	0.1	0.3	0.3	0.3	0.3	0.3	0.4	0.6	0.6	0.6	0.6	0.6	0.6	0.7
1500	0	0	0.1	0.1	0.1	0.3	0.3	0.4	0.4	0.4	0.4	0.6	0.6	0.6	0.7	0.7	0.7	0.7
1000	0.1	0.1	0.1	0.2	0.2	0.4	0.4	0.4	0.5	0.5	0.5	0.7	0.7	0.7	0.7	0.8	0.8	0.8
900	0.1	0.1	0.2	0.2	0.2	0.4	0.4	0.5	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.9
800	0.1	0.2	0.2	0.2	0.3	0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.8	0.8	0.8	0.8	0.9	0.9
700	0.2	0.2	0.2	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.8	0.8	0.8	0.9	0.9	1	1
600	0.2	0.3	0.3	0.3	0.4	0.5	0.6	0.6	0.6	0.7	0.7	0.8	0.9	0.9	0.9	1	1	1
500	0.3	0.3	0.4	0.4	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1	1	1.1	1.1	1.1
400	0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1	1	1	1.1	1.1	1.2	1.2
300	0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1	1.1	1.1	1.1	1.1	1.2	1.2	1.3	1.4	1.4
250	0.6	0.7	0.8	0.8	0.9	0.9	1	1.1	1.1	1.2	1.2	1.2	1.3	1.3	1.4	1.4	1.5	
200	0.8	0.9	1	1		1.1	1.2	1.3	1.3			1.4	1.5	1.6	1.6	1.6	1.6	
150	1.1	1.2	1.3	1.3		1.4	1.5	1.6	1.6			1.7	1.8	1.9	1.9	1.9		
140	1.2	1.3				1.5	1.6					1.8	1.9					
130	1.3	1.4				1.6	1.7					1.9	2					
120	1.4	1.5				1.7	1.8					2	2.1					
110	1.5	1.6				1.8	1.9					2.1	2.2					
100	1.6	1.7				1.9	2					2.2	2.3					
90	1.8					2.1						2.4						
80	2					2.3						2.6						
70	2.3					2.6						2.9						

Notes:

Values shown are for WB-15 design vehicle and represent widening in metres.

Values less than 0.6m may be disregarded.

For 3 lane roadways, multiply above values by 1.5

For 4 lane roadways, multiply above values by 2

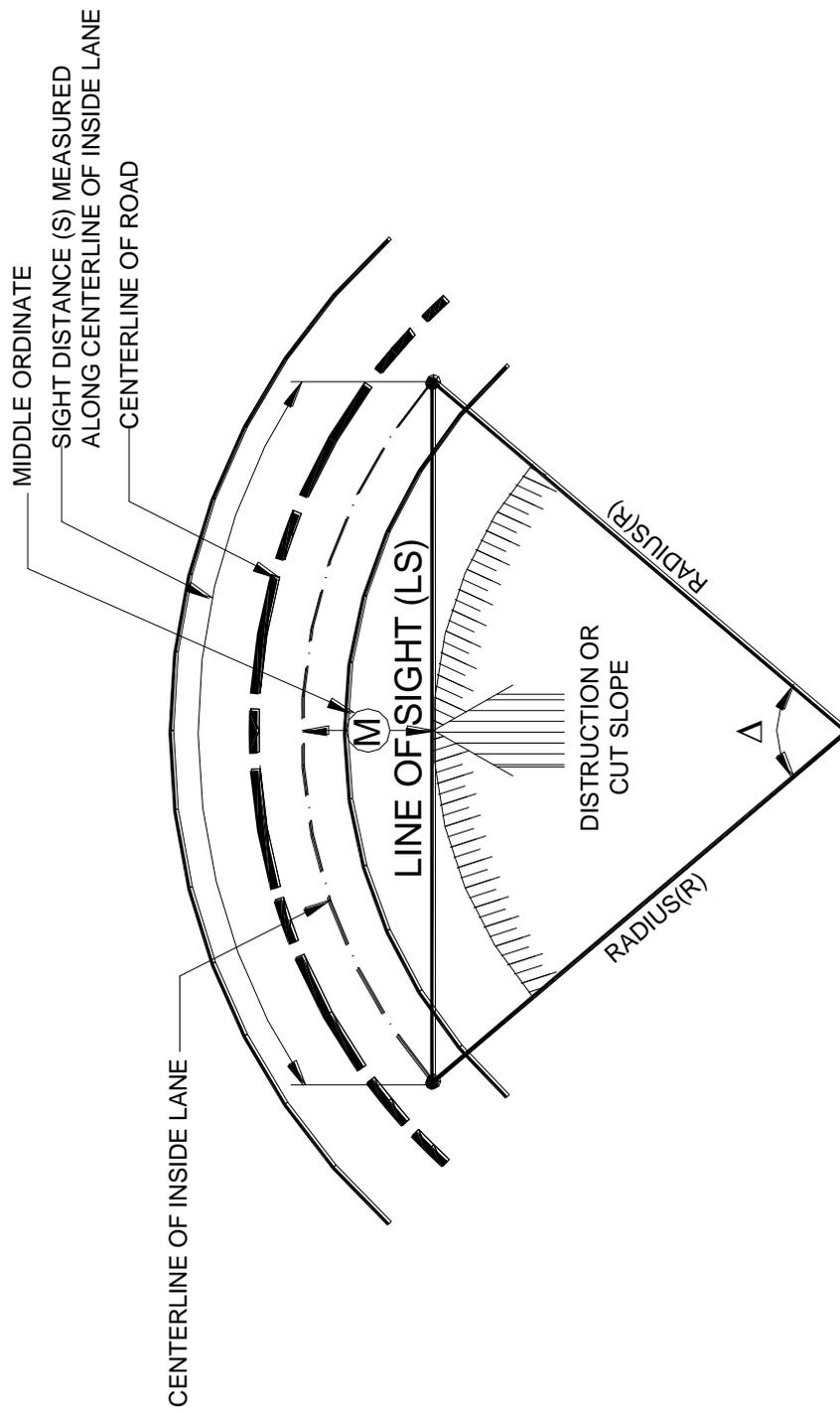


Figure 6: Sight Distance on Horizontal Curve

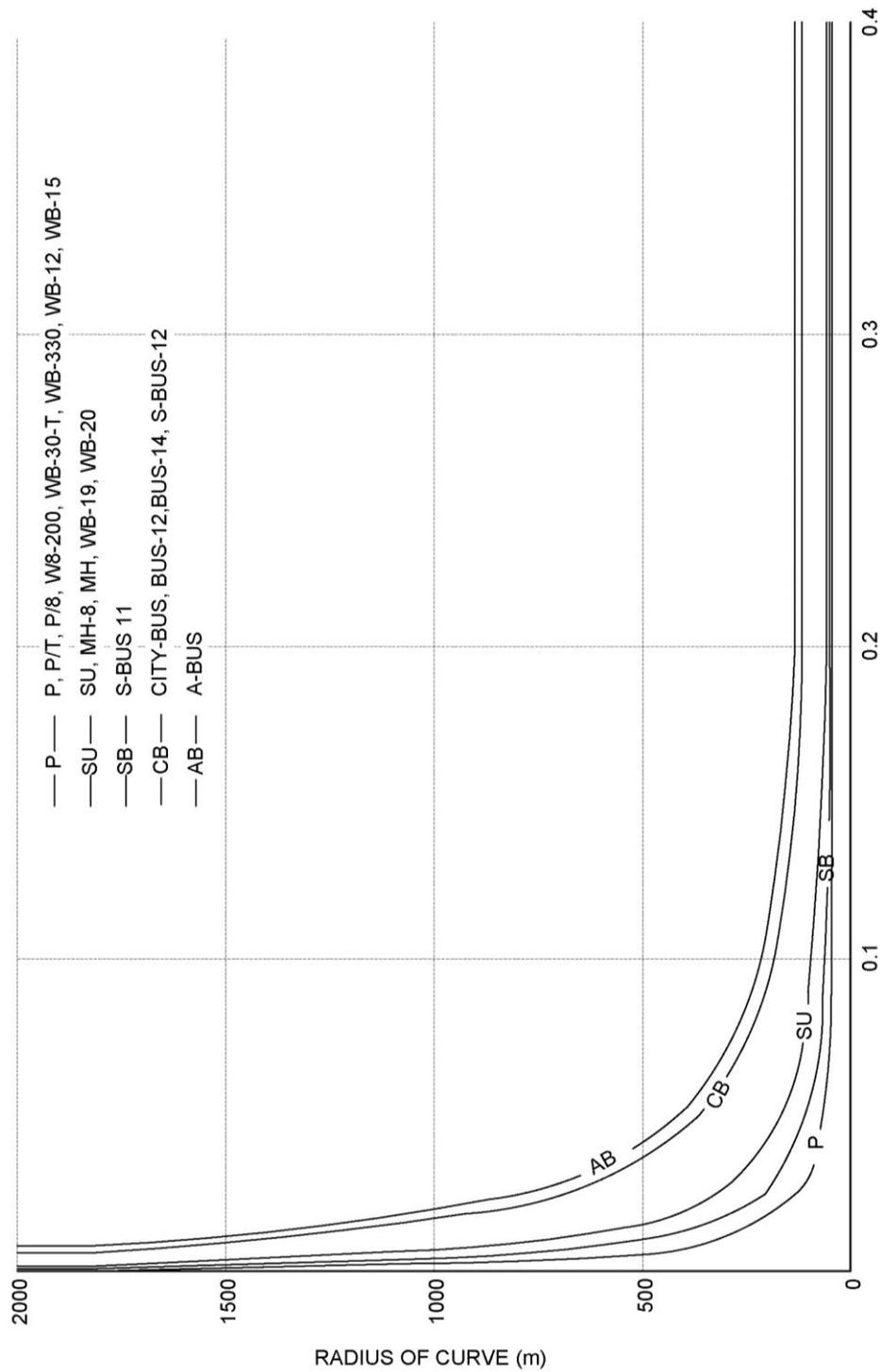
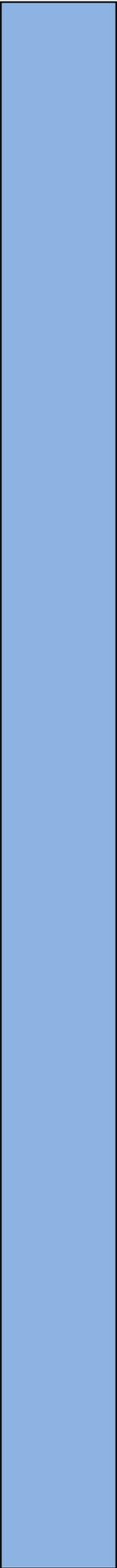


Figure 7: Diagrammatic Profiles Showing Methods of Attaining Super Elevation



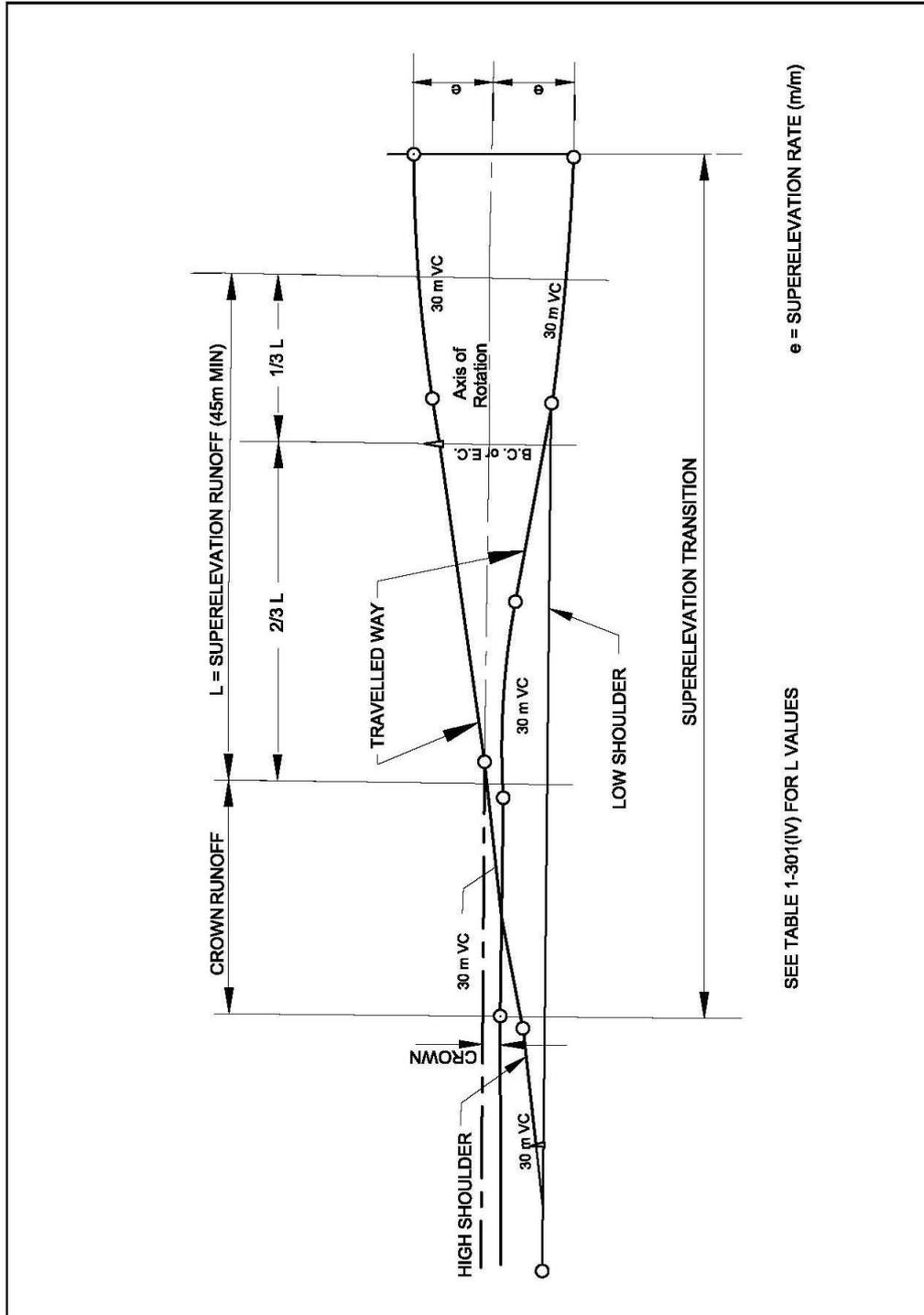


Figure 8: Super Elevation Transition

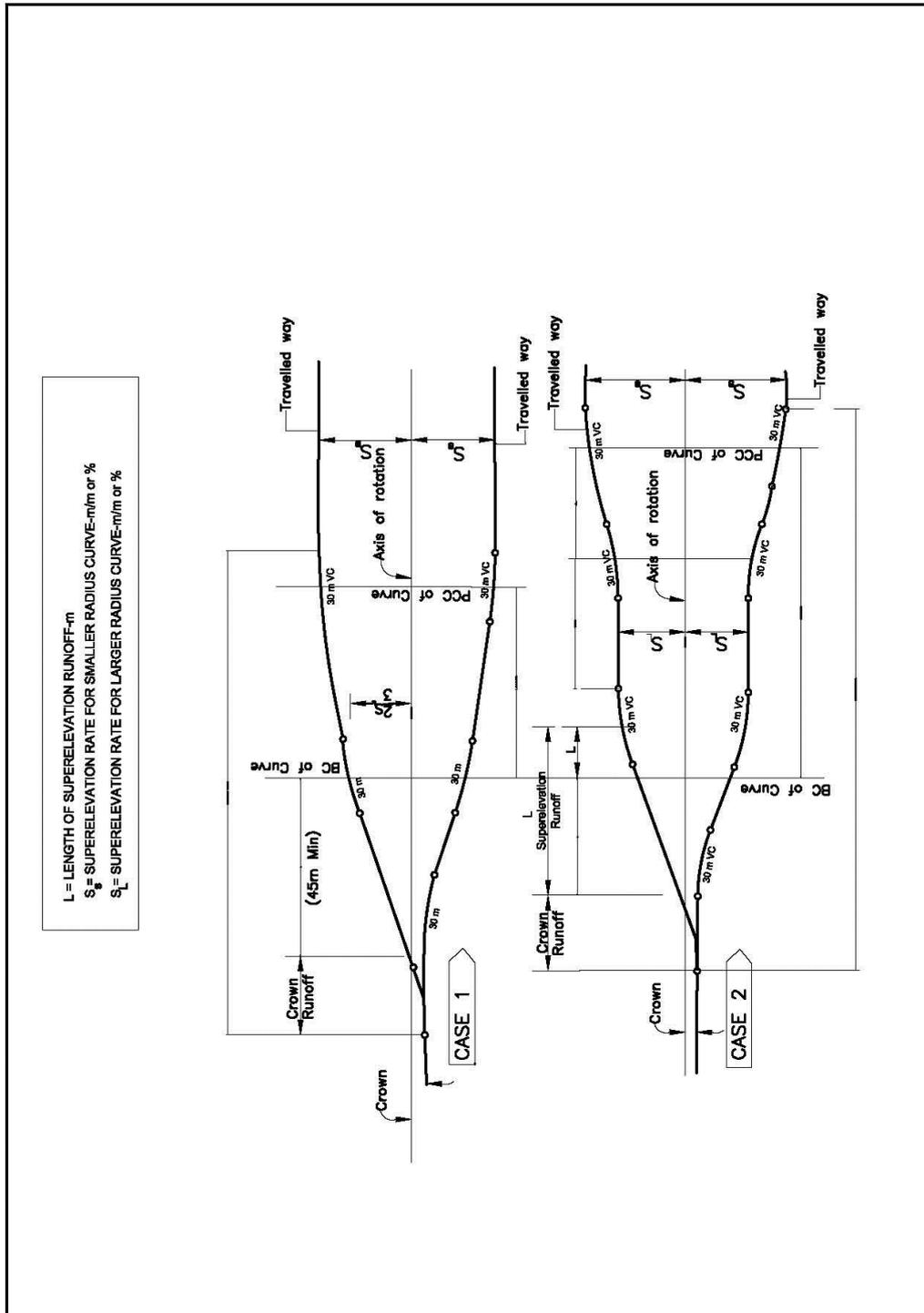
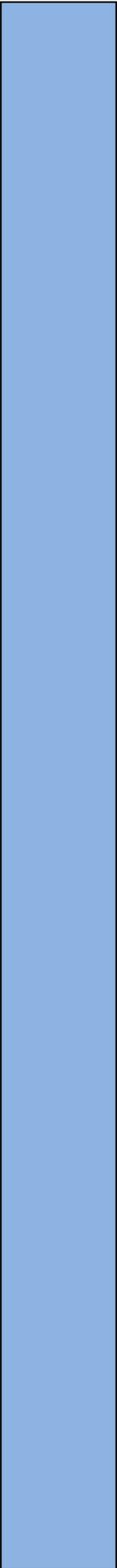


Figure 9: Super Elevation of Compound Curves



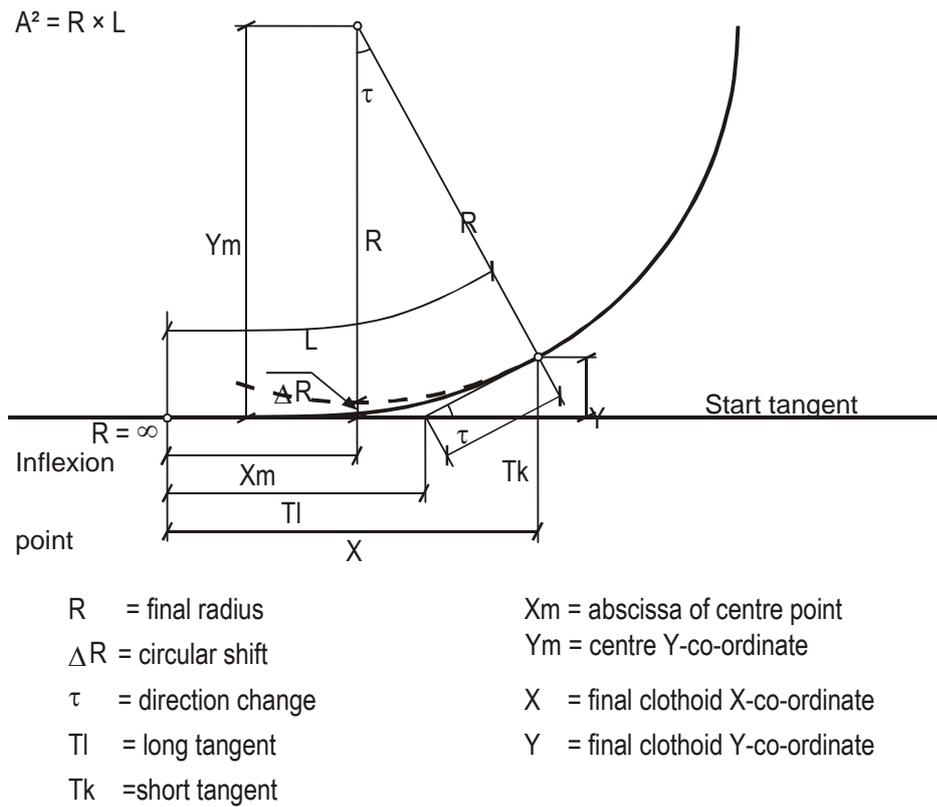


Figure 10: Transition [Clothoid] Elements

3.2 Geometric Profile Elements

3.2.1 Vertical Alignment

a. General

The vertical alignment (roadway profile) is the longitudinal section of the road showing the gradients and vertical curves. General controls for selecting a vertical alignment are as follows:

- The grade line is a reference line by which the elevation of the pavement and other features of the highway are established. Though controlled mainly by topography, other facts such as horizontal alignment, safety, sight distance, speed, constructions costs and the performance of heavy vehicles on a grade should be considered.

- All portions of the grade line shall meet sight distance requirements for the design speed classification of the road.
- In level terrain the elevation of the grade line is often controlled by drainage considerations. In rolling terrain a reasonably undulating grade line is desirable from the standpoint of operation and construction economy.
- A smooth grade line with gradual changes consistent with the class of highway and terrain should be adopted rather than a line with numerous breaks and short lengths of grades.
- The “hidden-dip” type of profile shall be avoided. Such profiles occur on relatively straight horizontal alignments where the roadway profile follows closely rolling natural ground. Apart from their aesthetically unpleasant appearance they are extremely hazardous contributing in a large measure to overtaking manoeuvre accidents on two lane roads and direct head on collisions on narrow roads. The “hidden-dip” profile can be avoided by introducing horizontal curvature or by more generous cuts and fills to obtain gradual grades.
- Undulating grades involving substantial lengths of momentum grades should be appraised for effect on traffic operation and construction economy.
- Two vertical curves in the same direction separated by a short section of tangent grade shall in general be avoided, particularly in valley curves.
- On long grades it is desirable to place the steepest grades at the bottom and ease the grades near the top of the ascent, or to break the sustained grade by short intervals of easier grade instead of a uniform sustained grade even though such a grade is below the allowable maximum.
- Where intersections at grade occur on highway sections with moderate to steep grades, it is desirable to reduce the gradient through the intersection. Turns are negotiated with reduced mechanical wear and fuel consumption and increased safety.
- Where the critical lengths of grade for commercial vehicles must be exceeded for physical or economic reasons then an added climbing lane must be provided if trends show substantial truck traffic.

- Alternative grade profiles shall be considered and the most efficient from economic considerations shall be adopted. By weighting the cost of construction to reduce grades as against the reduction in vehicle operation costs, the present worth of savings shows that a 3% maximum grade is the most economical, having regard to the cost of maintenance of slopes and drainage.

b. Standards for Grades

(i) Minimum Grades

Flat and level grades on uncurbed pavements will function satisfactorily when the pavement is adequately crowned to drain the surface laterally. When flat or level grades are used, independent profile gradients are required for the roadway median and roadside ditches to ensure efficient drainage.

With curbed pavements, longitudinal grades should be provided to facilitate surface drainage. A minimum grade for the usual case is 0.5% but a grade of 0.3% may be used where there is an adequate pavement, accurately crowned and supported on a firm sub grade.

Particular attention should be given to the design of storm water inlets and their spacing to keep the spread of water on the travel way within tolerable limits on kerbed roadway sections.

(ii) Maximum Grades

○ *Vehicles*

Maximum grades shall be as shown in Table 17

Table 17: Relation of Maximum Grades to Design Speed – Main Highways

Type of Topography	Design Speed Km/h						
	50	60	80	100	110	120	130
Flat	6	5	4	3	3	3	3
Rolling	7	6	5	4	4	4	4
Mountainous	9	8	7	6	5		

The maximum design grade should be used infrequently rather than as a value to be used in most cases. At the other extreme, for short grades less than 150 metres and for one-way downgrades, the maximum gradient may be about 1% steeper than shown in Table 17. For low volume rural highways grades may be 2% steeper.

- *Pedal Cycles*

The maximum grade shall not exceed 5% for ramps. It is desirable not to exceed 2% generally.

- *Pedestrians*

The maximum ramp grade shall not exceed 10%, with a desirable maximum of 5%. Where grades on ramps exceed 3%, handrails shall be provided.

- *Critical Length of Grade*

Where feasible, upgrades should not be of a length which causes loaded trucks to reduce speed unduly. The critical grade length is determined for a selected truck as that which will cause a 15 km/h reduction in speed below the average running speed on the approach to the upgrade. On this basis, critical lengths of upgrades when approached by level or nearly level sections of road are as shown in Figure 17.

Where critical lengths of upgrade are substantially exceeded, consideration should be given to providing climbing lanes, particularly where truck volume is high.

- *Vertical Curves*

The length of a vertical curve to satisfy the requirements of minimum stopping sight distance comfort and appearance should not be shorter than $L=KA$, where L is the length of the vertical curve in metres, A is the algebraic difference of grades in percent. The values of K for varying design speeds are given in the table below:

Table 18: Minimum K Values for Vertical Curves

Design Speed Km/h	50	60	80	100
Minimum K Value for:				
Crest Vertical Curves	7	11	26	52
Sag Vertical Curves	13	18	30	45

Much longer and sometimes impracticable lengths of crest vertical curves are required to provide passing sight distance. Ordinarily, passing sight distance can be provided only on sections of road without crest vertical curves, or those having very small algebraic difference in grades.

Figure 11 to Figure 14 give the length of vertical curve required to satisfy the passing and stopping sight criteria. Figure 15 shows the mathematical relationships for vertical curves.

3.2.2 Coordination of Horizontal and Vertical Alignment

a. General

Horizontal and vertical alignment should not be designed independently. They complement each other and poorly designed combinations can spoil the good points and aggravate the deficiencies of each.

Horizontal alignment and profile are among the more important of the permanent design elements of the highway, for which thorough study is warranted.

Proper combination of horizontal alignment and profile is obtained by engineering study and consideration of the following general controls:

- (i) Curvature and grades should be in proper balance. Tangent alignment or flat curvature at the expense of steep or long grades, and excessive curvature with flat grades, are both poor designs. A logical design is a compromise between the two, which offers the most in safety, capacity, ease and uniformity of operation and pleasing appearance within the practical limits of terrain and area traversed.
- (ii) Vertical curvature superimposed upon horizontal curvature, or vice versa generally results in a more pleasing facility but it should be analyzed for effect upon traffic. Successive changes in profile not in combination with horizontal curvature may result in a series of humps visible to the driver for some distance; a hazardous condition as previously discussed.
- (iii) Horizontal curvature and profile should be made as flat as feasible at highway intersections where sight distance along both highways is important and vehicles may have to slow down or stop.
- (iv) On divided highways, variation in the width of median and the use of separate profiles and horizontal alignments should be considered to derive design and operational advantages of one-way roadways. Where traffic justifies provision of 4 lanes, a superior design without additional cost generally can result from the concept and logical design basis of one-way roadways.

The use of horizontal and vertical alignments in combination however, may also result in certain hazardous arrangements.

- (i) Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This condition is hazardous in that the driver cannot perceive the horizontal change in alignment, especially at night when the headlight beams go straight ahead into space. The hazard of this arrangement is avoided if the horizontal curvature leads the vertical curvature, i.e. the horizontal curve is made longer than the vertical curve. Also suitable design can be made by using design values well above the minimums for the design speed.
- (ii) Sharp horizontal curvature should not be introduced at or near the low point of a pronounced sag vertical curve. Because the road ahead is foreshortened, any but flat horizontal curvature assumes an undesirable distorted appearance. Further vehicular speeds, particularly of trucks, often are high at the bottom of grades and erratic operation may result, especially at night.
- (iii) On two lane highways the need for safe passing sections at frequent intervals and for an appreciable percentage of the length of the highway often supersedes the general desirability for combination of horizontal and vertical alignment. In these cases it is necessary to work toward long tangent sections to secure sufficient passing sight distance in design.

3.2.3 Climbing Lanes

a. General

There are four warrants for climbing lanes:

- Reduction of truck speed through a given amount or to a specified speed
- Reduction of truck speed in association with a given volume of traffic
- Reduction in Level of Service (LOS) through one or more levels
- Economic analysis

The first three focus on the performance of trucks and their impact on passenger vehicles. Freedom of operation on two lane highways, besides being governed by the extent and frequency of passing sections, is adversely affected by truck traffic operating on grades of sufficient length to result in speeds which could impede following vehicles. It is desirable to provide a climbing lane, an extra lane on the upgrade edge of a two lane highway, where the critical length of grade is exceeded (Chapter 3.2 1 (4)).

The fourth directly quantifies the effects on slow moving vehicles (not necessarily exclusively trucks) on traffic streams in terms of delay over the design life of the climbing lane and compares the benefit of the removal of delay with the cost of providing and maintaining the climbing lanes.

b. Design

If the climbing lane is justified it should be designed in accordance with Figure 16. In no instance will the reduction result in a shoulder width of less than 1.2 metres. Adequate signing and delineation is essential so that the presence of an auxiliary lane is immediately recognized. Consideration should be given to extending the lane over crests whenever possible so that vehicles may accelerate as quickly as possible when leaving the lane.

3.2.4 Airport – Highway Clearances

When locating highways near airport runways, care should be taken that the roadway, vehicles on the roadway, or roadway appurtenances such as lighting standards, roadway signing, etc., do not violate the approach surfaces. Contact with local airport authorities and the Federal Ministry of Transport is essential to ensure all requirements are met.

The requirements for clearance are set out in the latest edition of the International Civil Aviation Organisation (ICAO) document “Volume 1: Aerodrome Design and Operations; Annexure 14 Aerodromes”.

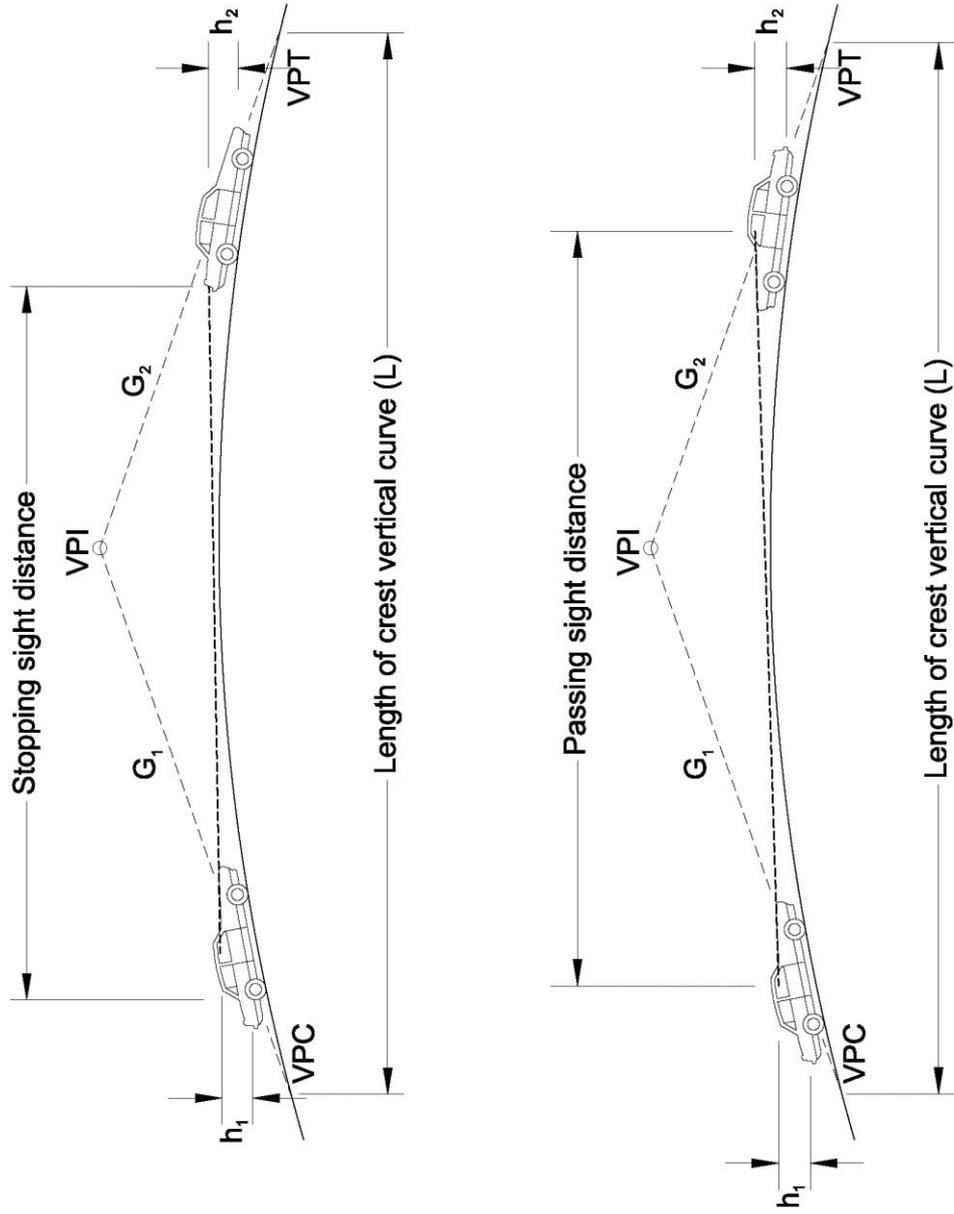


Figure 11: Parameters Considered in Determining Length of Crest Vertical Curve to Provide Sight Distance

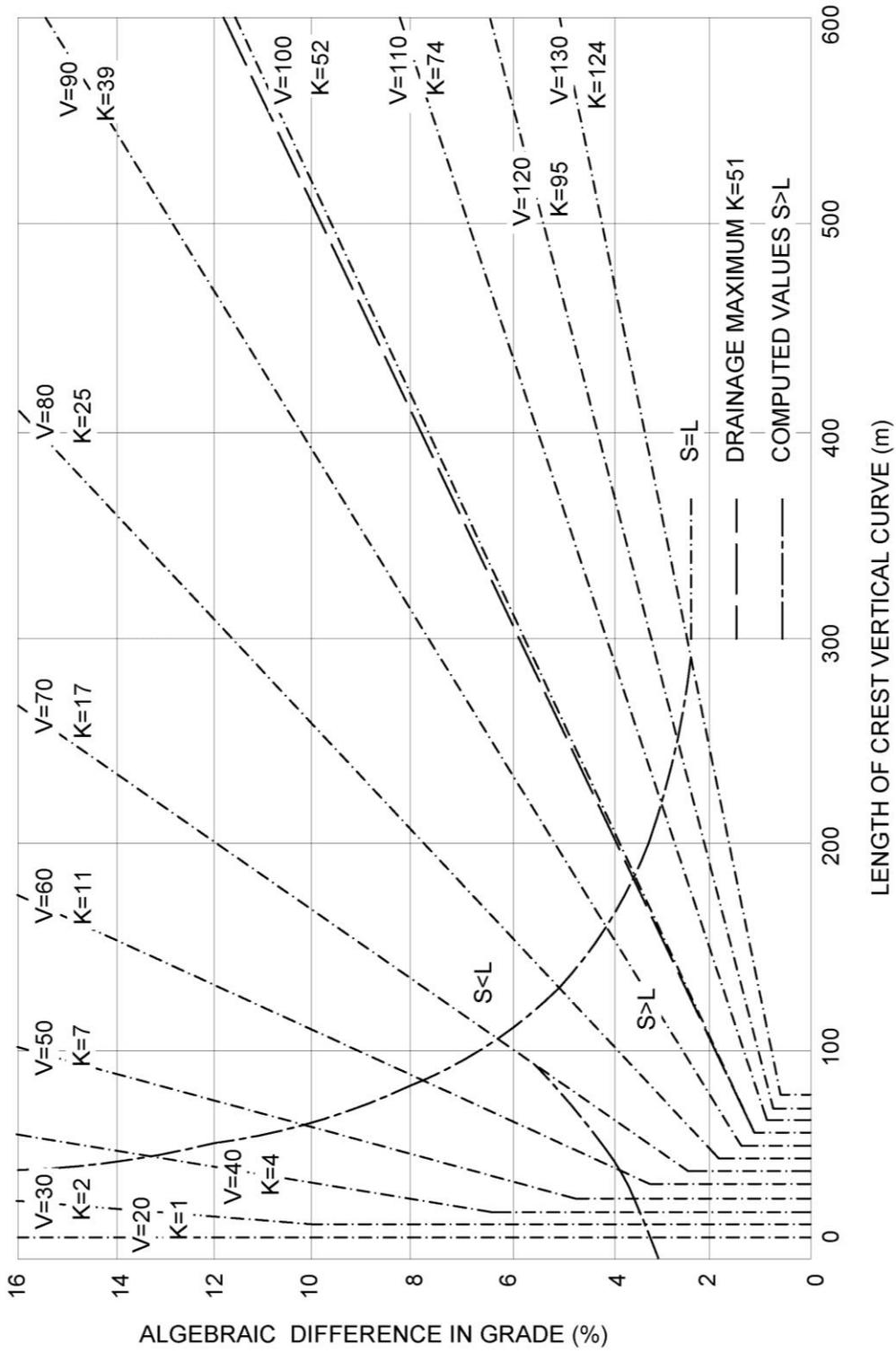


Figure 12: Design Controls for Crest Vertical Curves



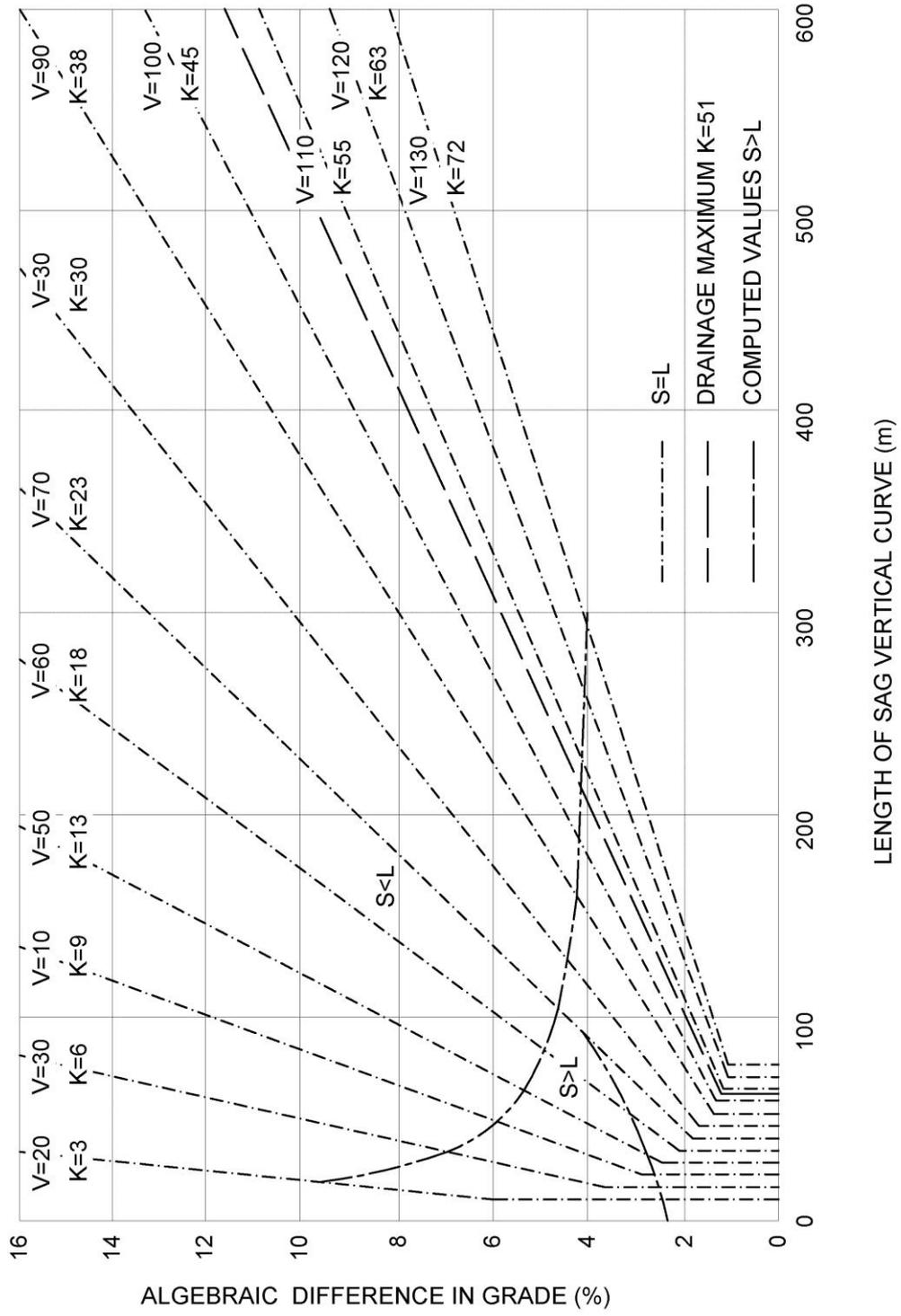


Figure 13: Design Controls for Sag Vertical Curves

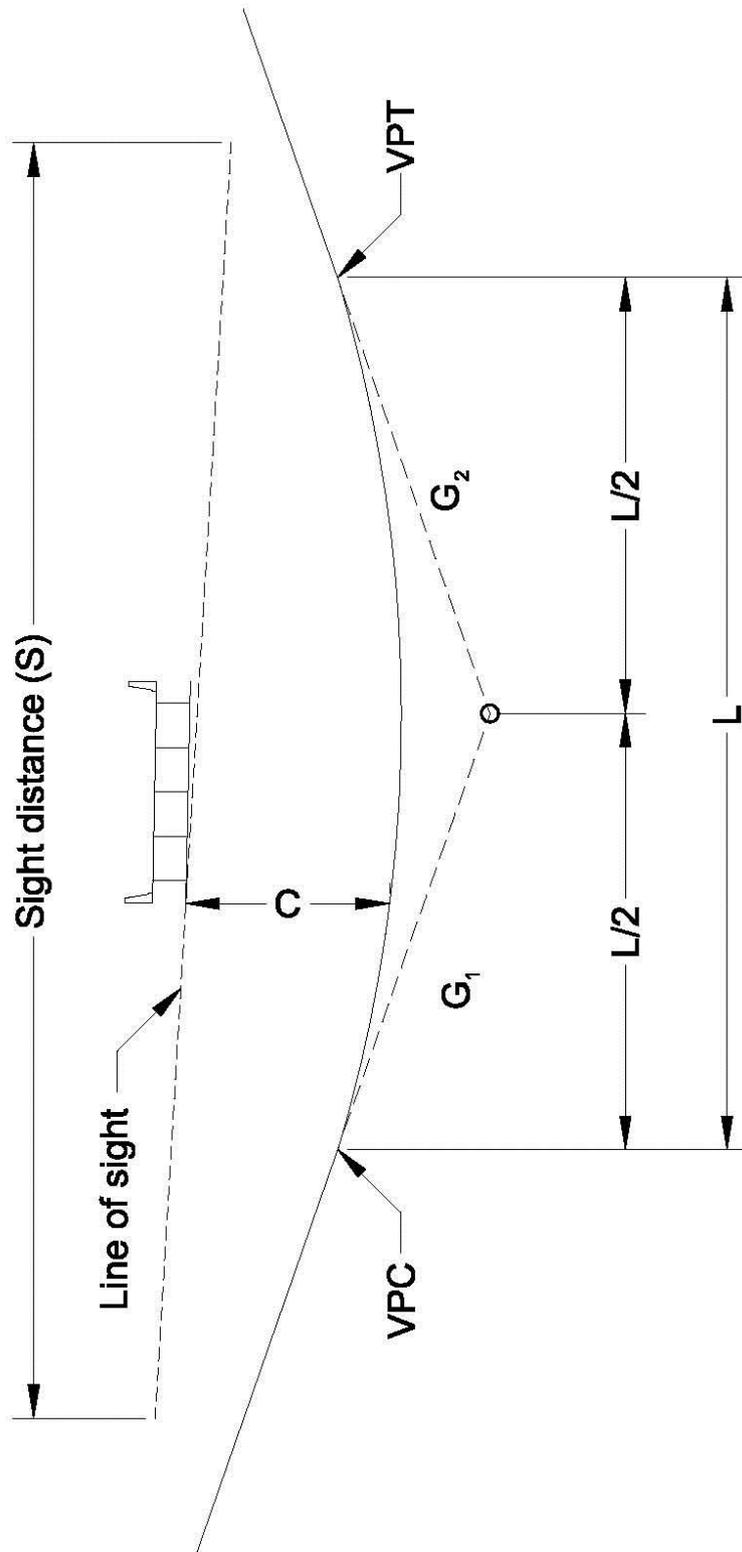
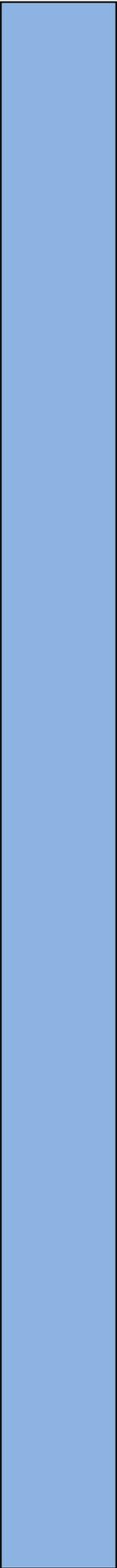


Figure 14: Sight Distance at Underpasses



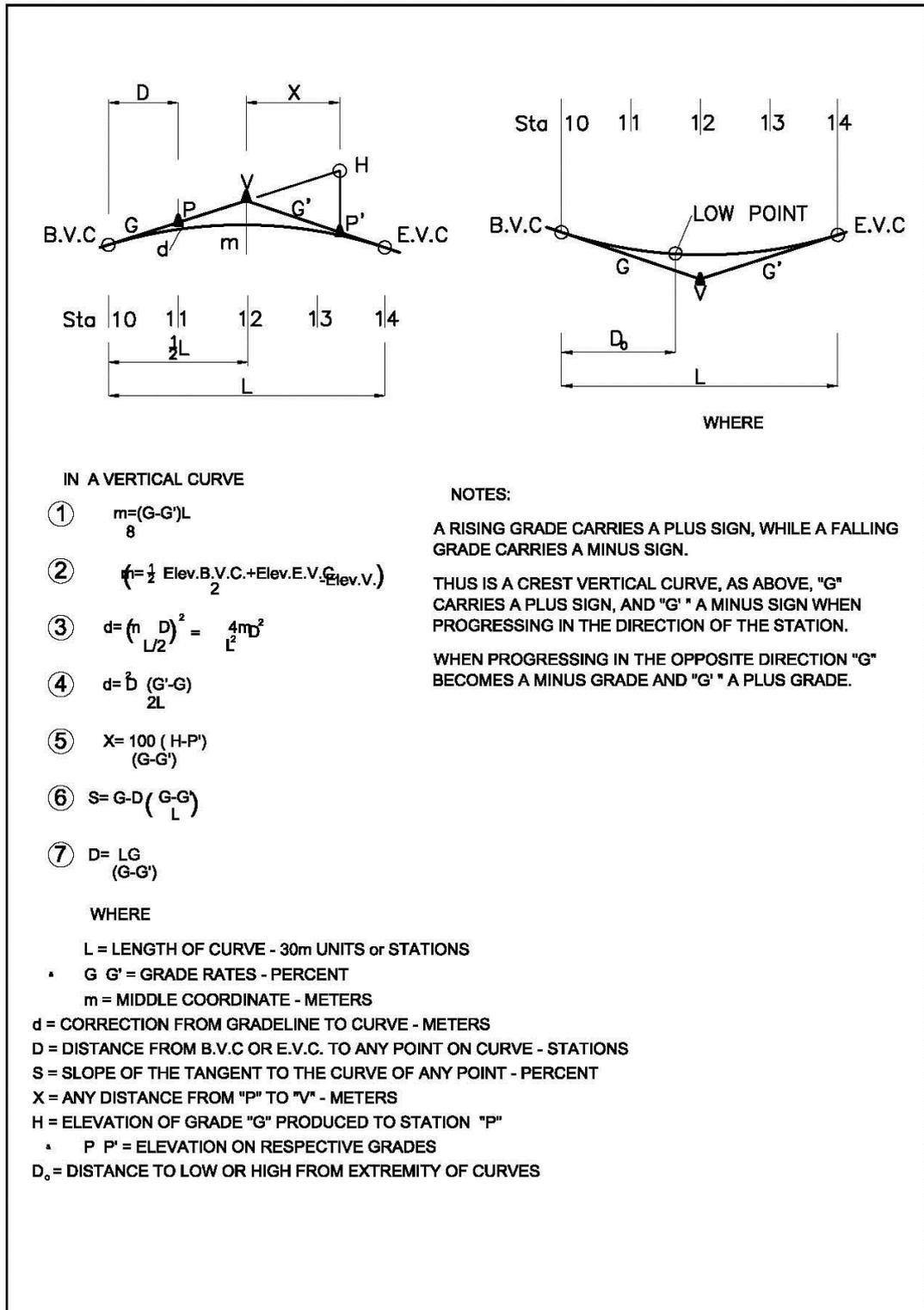


Figure 15: Vertical Curves

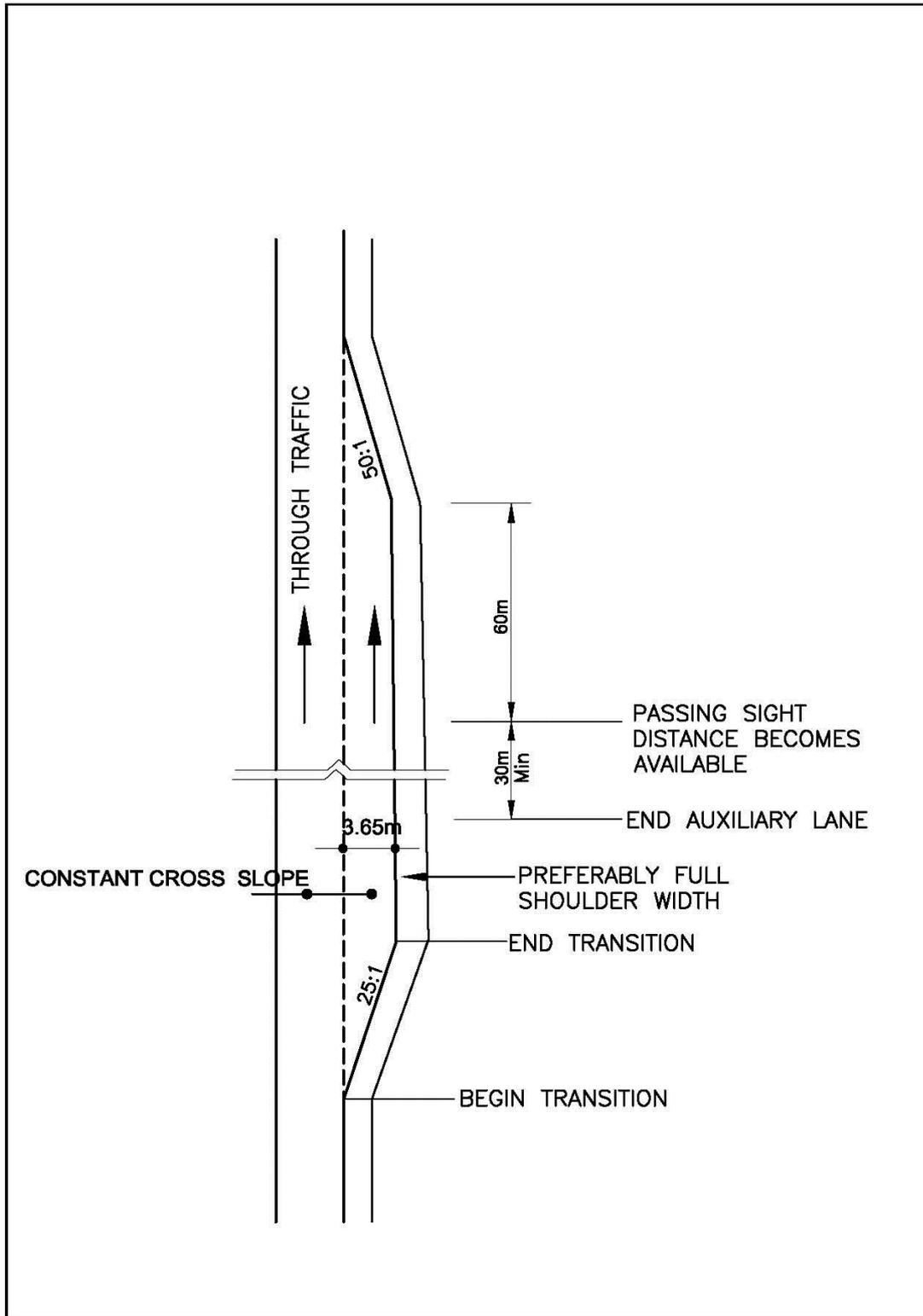


Figure 16: Auxiliary Climbing Lane Geometric Design



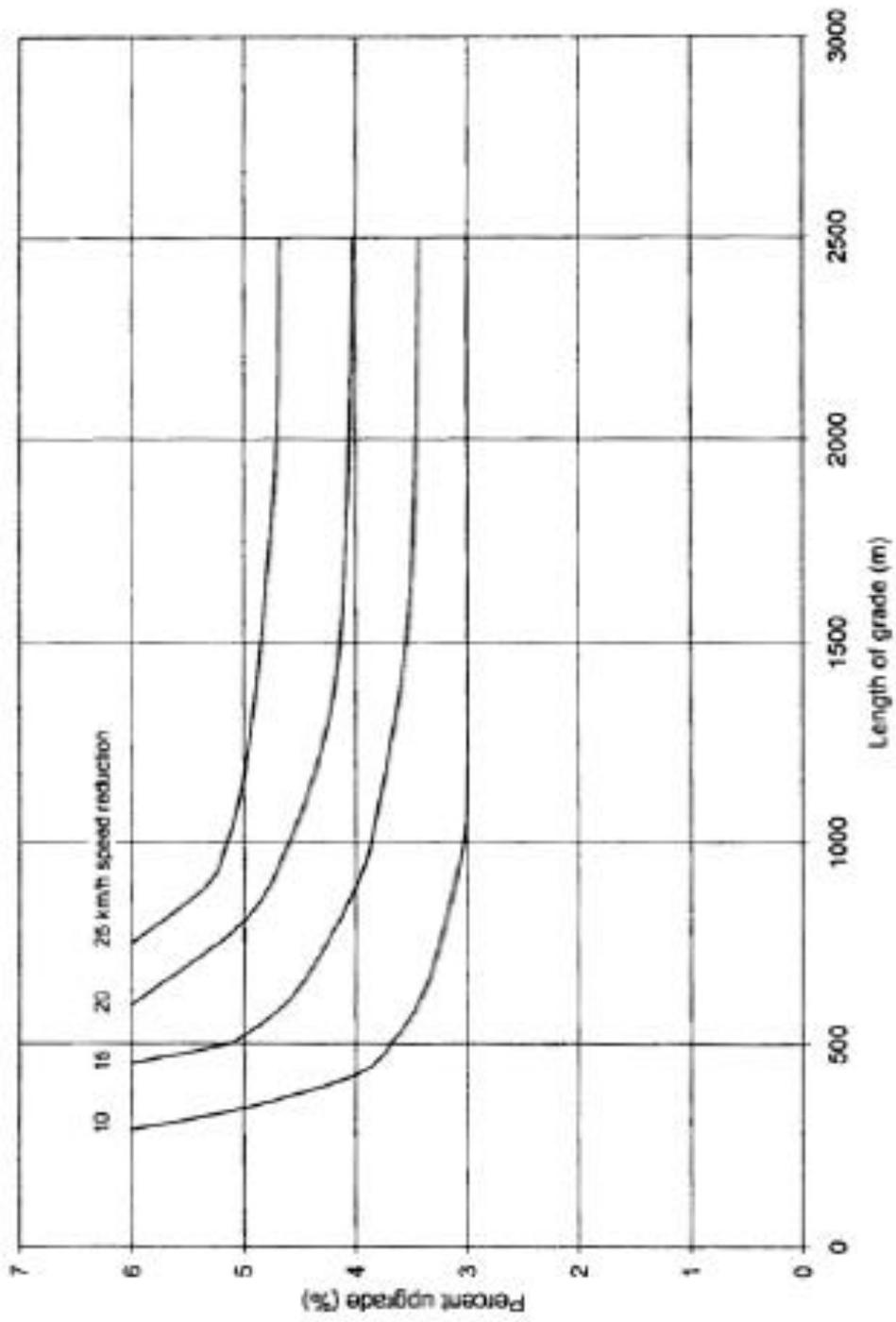


Figure 17: Critical Lengths of Grade for Design

3.3 Geometric Cross Section Elements

3.3.1 General

In developing the geometric cross section of a highway most of the basic design elements are used. The cross section is determined from the number and width of roadway lanes, shoulder width, median width, super elevation, pavement type, side slopes, drainage, sidewalks and cycle tracks and the right-of-way width.

3.3.2 Travelled Way Section

a. Number of Lanes

The number of roadway lanes required is determined by the traffic volumes for the design year used.

b. Lane Widths

In the interest of safety, efficiency and ease of operation lane widths of 3.35 to 4.00 metres are desirable and should be used whenever possible. Table 19 shows the minimum widths of surfacing for two lane roadways.

Table 19: Minimum Widths of Surfacing for Two Lane Highways

Minimum Widths of Surfacing in Metres for Design Volumes of:					
Design Speed Km/h	Current ADT	Current ADT	Current ADT		
	50 - 230	250 - 400	400 – 750	DHV 100 - 200	DHV 200 - 400
50	6.0	6.0	6.0	6.0	7.3
60	6.0	6.0	6.7	6.7	7.3
80	6.0	6.0	6.7	7.3	7.3
100	6.0	6.7	6.7	7.3	7.3

c. Normal Cross Slope

Normal cross slopes to be used on the travelled way are determined by the type of pavement used. Table 21 shows standard values for different pavement types.

On undivided highways with two or more lanes on a normal tangent section, the high point of the crown shall be on the centre line of the pavement and the pavement sloped toward the edges at a uniform grade.

On four and six lane divided highways on a tangent, each pavement should have a uniform cross slope with the high point at the edge nearest the median. Eight lane divided highways are special cases and should be treated the same as six lane highways unless potential roadway drainage problems are anticipated. In these cases, particular attention must be given to the roadway drainage and possible scenarios analysed to determine the best solution.

At intersections, interchange ramps or in unusual situations, the crown position may vary depending upon drainage or other controls.

Table 20: Standards for Pavement Cross Slopes

Type of Pavement	Cross Slope (%)
Portland Cement Concrete	2.5
Bituminous Mix Pavements	2.0 – 3.0
Penetration Treated Earth or Gravel	2.5 – 3.0
Un-surfaced Graded Section	2.5 – 3.0

3.3.3 Shoulder Sections

a. Widths

The shoulder width shall be the minimum continuous width of shoulder which provides an all-weather surface. Usable shoulder widths of 3 metres are desirable on all highways but narrower widths are acceptable on low volume highways. Table 21 shows widths of shoulders on rural two lane highways.

Table 21: Widths of Shoulders for Two Lane Rural Highways

Design Volumes		Usable Shoulder Width - Metres	
Current ADT	Design Hourly Volume	Minimum	Desirable
50 – 250	-	1.2	1.8
250 – 400	-	1.2	2.5
400 – 750	100 – 200	1.8	3.0
-	200 – 400	2.5	3.0
-	400 and over	3.0	3.65

Shoulder widths for divided highways shall be as shown in Table 22.

Table 22: Shoulder Widths for Divided Highways

Type of Roadway	Shoulder Width – Metres	
	Left of Traffic	Right of Traffic
4 Lane Freeway	1.20	3.00
6 Lane Freeway	1.85	3.00
8 Lane Freeway	2.50	3.00
Separated Roadways	2.50	3.00
Ramps and Auxiliary Lanes	2.00	2.50

b. Normal Cross Slopes

Shoulders to the right of the direction of travel of traffic shall normally be sloped at the rate of 3% and shoulders to the left at the rate of 2.5% towards the median. In super elevated sections the cross slope for the shoulder shall be the same as on the roadway when the super elevation slope exceeds the normal shoulder cross slope.

3.3.4 Cycle Tracks and Sidewalks

a. Widths

Minimum width of sidewalks shall be 1.5 metres and for cycle tracks 3.0 metres. Criteria for establishing sidewalks are in Volume II: Secondary Design Elements.

b. Cross Slope

Normal cross slope shall be 2%. The major consideration in establishing the cross slope is drainage.

c. Location

Whenever possible cycle tracks and sidewalks should be separated from the roadway traffic. Kerbs, guardrail or planting strips can be used as separators

3.3.5 Median Sections

a. General

The prime function of a median is to separate opposing traffic lanes. Medians also provide space for:

- Drainage facilities

- Vehicle storage space for crossing and/or left turn movements at intersections
- Aesthetic treatment, including planted or natural foliage to reduce headlight glare from opposing traffic
- Safety refuge areas for out of control or disabled vehicles
- Bridge piers for overpass structures
- Increased safety, comfort and ease of operation.

b. Widths

Minimum median width shall be 1.2 metres. The 1.2 meter median is appropriate only for special circumstances where a wider median would not be justified. Typical situations include stage construction, locations where the terrain is such that heavy excavations are required or where frequent sight distance restrictions occur on a two lane road.

Desirable minimum median widths for divided highways are show in Figure 25.

c. Cross Slopes

Wide un-surfaced medians shall be sloped downward from the adjoining shoulders to form a shallow valley in the centre. Slopes of 10:1 or flatter are preferred with slopes of 6:1 considered to be the maximum.

d. Contrast with Pavement

Pavement and median areas should contrast effectively in colour, texture or both under either wet or dry conditions. Un-surfaced medians usually give adequate contrast whether planted or not. Surface types that contrast with the travelled way shall be used for surfaced medians.

3.3.6 Slopes

a. General

Side slopes should be designed for functional effectiveness, ease of maintenance and pleasing appearance.

b. Standard Slopes

Standard slopes shall be as shown in Table 23 through Table 26.

Table 23: Slope Selection Data – Divided Multilane Highway

Height of Cut	Slope	Height of Fill
0 to 1.5 metres	6:1	0 to 3.0 metres
1.5 to 6.0 metres	3:1	3.0 to 9.0 metres
Over 6.0 metres	2:1	Over 9.0 metres

Table 24: Slope Selection Data – Undivided Multilane Highway

Height of Cut	Slope	Height of Fill
0 to 1.5 metres	4:1	0 to 3.0 metres
	3:1	3.0 to 6.0 metres
Over 1.5 metres	2:1	Over 6.0 metres

Table 25: Slope Selection Data – Two Lane Highways

Height of Cut	Slope		Height of Fill
	Desirable	Minimum	
0 to 1.5 metres	6:1	4:1	0 to 3.0 metres
1.5 to 6.0 metres	3:1	2:1	3.0 to 9.0 metres
Over 6.0 metres	2:1	2:1	Over 9.0 metres

Table 26: Slope Selection Data – Freeway Ramps

Height of Cut	Slope	Height of Fill
0 to 1.5 metres	6:1	0 to 3.0 metres
1.5 to 6.0 metres	3:1	3.0 to 9.0 metres
Over 6.0 metres	2:1	Over 9.0 metres

c. Clearance from Slope to Right of Way Line

The minimum clearance from the right-of-way line to the catch point of a cut or fill slope should be 3.0 metres for all types of cross sections. When feasible at least 4.5 metres should be provided.

For cuts higher than 9.0 metres the following minimum clearances are recommended:

- 6 metres for cuts from 9 metres to 15 metres
- 8 metres for cuts from 15 metres to 23 metres
- One third the height of cut for cuts above 23 metres but not to exceed 15 metres.

d. Benched Slopes

The necessity for benches, their width and vertical spacing shall be established only after an adequate materials investigation. Since greater traffic benefits are realized from widening a cut than from benching the slope, benches should be used sparingly and only where they are justified by sound engineering reasons including the following:

- In unstable material, where it is more economical to bench than to flatten the slope.
- To intercept and store loose material resulting from minor slides and erosion.

For ease of maintenance a 6 meter bench width is satisfactory. Benches should be sloped to form a valley at least 300 mm deep with the low point a minimum of 1.5 metres from the toe of the upper slope. Access for maintenance equipment should be provided to the lowest bench and if feasible to the higher benches.

e. Slope Rounding

The tops of all cut slopes shall be rounded where the material is other than solid rock. A layer of earth overlying a rock cut shall also be rounded.

3.3.7 Drainage Channels

Roadside drainage channels are used to remove surface water from the highway right of way to prevent damage to the roadway and avoid creating a hazard to traffic.

Drainage channels include roadside channels in cut sections to remove water from the highway cross section; toe-of-slope channels to convey the water from the cut section and from adjacent slopes to a natural water course, intercepting channels placed behind the top of cut slopes to intercept surface water and chutes to carry collected water down steep cut or fill slopes.

Median drainage channels are generally shallow depressed areas or swales at or near the centre of the median formed by the flat side slopes of the divided roadways. The swale is sloped longitudinally for drainage and at intervals the water is intercepted by inlets or transverse channels and discharged from the roadway in storm drains or culverts.

Refer to Volume IV Drainage Design of this part of the Highway Manual for further information on drainage design.

3.3.8 Right-of-Way Widths

a. General

The right-of-way width is determined by the space required for the pavement, shoulders, side ditches and side slopes. On divided highways, additional width for the median must be included.

b. Minimum Widths

Minimum right-of-way widths for rural areas shall be 30 metres for two lane highways and 12 metres beyond the edge of pavement for multilane highways. In urban areas the minimum right-of-way should not be less than the width required for the necessary cross section elements.

3.3.9 Separated Roadways

Separated roadways may be particularly fitting to certain topographic conditions. Separate horizontal and/or vertical alignments may be used on multilane roadways to provide a superior facility at less cost. Separate grade lines are not considered appropriate when the median width is less than 11 metres wide. Figure 25 shows differences in elevations between roadways for different widths of medians.

3.3.10 Typical Roadway Sections

Figure 20 through Figure 25 represent the basic geometric configurations of the above mentioned roadway elements. They are not intended to represent all design problems that can be encountered in highway design but are to be used as a guide in design.

3.4 Roadside Safety Design

3.4.1 General

Road crashes, to varying degrees, are caused by defects attributable to the vehicle, the driver or the road or by combinations of these defects. In addition, a significant percentage of deaths on roads occur as a result of pedestrians being on the road. The cost of road crashes to society can exceed the annual expenditure on roads, thus the expenditure of considerable sums of money can be justified in reducing the crash rates on roads through improved design and by catering for the presence of pedestrians on roads. Crashes resulting from simply leaving the roadway regardless of the underlying cause represent a substantial portion of the total road crash problem i.e. "run-off-the-road" (ROR) accidents can be 25 percent of all road crashes. They occur on both straight and curved sections of road and generally involve either rollover of the vehicle or collision with fixed objects, such as trees, roadside structures etc.

Broadly, there are three courses of action possible for mitigating road crashes and their consequences:

- By reducing the possibility that run-off-the-road will occur;
- If a run-off does occur, by providing opportunities for the driver of the vehicle to recover and return to the road without incident, and
- If a crash does occur, by providing design elements to reduce the severity of the collision.

The first of these courses of action is dealt with by incorporating features in the overall design of the road which will reduce the possibility that run-off will occur. These features are influenced to a large extent by the selected design speed.

This section deals with the latter two courses of action. It provides the designer with guidance on the design of the roadside environment that includes elements to allow for recovery on the part of the driver, as well as features which are intended to reduce the severity of such accidents that do occur.

3.4.2 The "Forgiving Roadside"

A forgiving roadside is one free of obstacles which could cause serious injuries to occupants of an errant vehicle. To the extent possible, a relatively flat, unobstructed roadside recovery area is desirable, and when these conditions cannot be provided, hazardous features in the recovery area should be made breakaway or shielded with an appropriate barrier.

When a vehicle leaves the traffic lane, the path of the vehicle and any object in or near that path become contributing factors to the degree of severity of the crash. By designing a forgiving roadside the severity of crashes can be reduced. The concept of designing a forgiving roadside should not be regarded as a by-product of the application of safety criteria to each design element, but as an integral part of the total engineering for the road.

3.4.3 Roadside Clear Zone

Figure 27 illustrates the clear zone concept in the context of the roadside recovery zone.

Recoverable slopes are those on which a driver may, to a greater or lesser extent, retain or regain control of a vehicle. A non-recoverable slope may be traversable, but a vehicle will continue to the bottom. A clear run-out area is located at the toe of a non-recoverable slope, and is available for safe use by an errant vehicle. There is also provision for a smooth transition between slopes to allow for the safe passage of vehicles.

3.4.4 Width of Clear Zone

Table 27 provides an indication of the appropriate width of a clear zone on a straight section of road, measured in metres from the edge of the lane, according to design speed, traffic volumes and cut or fill slope values. The values in Table 27 are taken from the 1996 AASHTO Roadside Design Guide, and suggest only the approximate centre of a range to be considered and not a precise distance, since, in making their choice, designers should also consider specific site conditions.

Where side slopes are steeper than 1:4 (i.e. non-trafficable) designers should give consideration to the provision of a protection barrier.

For sections of road with horizontal curvature, these distances should be increased on the outside of curves by a factor that depends on the operating speed and the radius of the curve. Figure 29 provides guidelines on adjustment factors for clear zones on the outside of curves.

Table 27: Roadside Clear Zone Distances (metres)

Design Speed (km/h)	Design ADT (veh/day)	Fill Slopes		Cut Slopes	
		1:4 to 1:5	1:6 or flatter	1:4 to 1:5	1:6 or flatter
<60	<750	2.0 – 3.0	2.0 – 3.0	2.0 – 3.0	2.0 – 3.0
	750 – 1500	3.5 – 4.5	3.0 – 3.5	3.0 – 3.5	3.0 – 3.5
	1500 – 6000	4.5 – 5.0	3.5 – 4.5	3.5 – 4.5	3.5 – 4.5
	>6000	5.0 – 5.5	4.5 – 5.0	4.5 – 5.0	4.5 – 5.0
70 – 80	<750	3.5 – 4.5	3.0 – 3.5	2.5 – 3.0	3.0 – 3.5
	750 – 1500	5.0 – 6.0	4.5 – 5.0	3.5 – 4.5	4.5 – 5.0
	1500 – 6000	6.0 – 8.0	5.0 – 5.5	4.5 – 5.0	5.0 – 5.5
	>6000	7.5 – 8.5	6.0 – 6.5	5.5 – 6.0	6.0 – 6.5
90	<750	4.5 – 5.5	3.5 – 4.5	3.0 – 3.5	3.0 – 3.5
	750 – 1500	6.0 – 7.5	5.0 – 5.5	4.5 – 5.0	5.0 – 5.5
	1500 – 6000	7.5 – 9.0	6.0 – 6.5	5.0 – 5.5	6.0 – 6.5
	>6000	8.0 – 10.0	6.5 – 7.5	6.0 – 6.5	6.5 – 7.5
100	<750	6.0 – 7.5	5.0 – 5.5	3.5 – 4.5	4.5 – 5.0
	750 – 1500	8.0 – 10.0	6.0 – 7.5	5.0 – 5.5	6.0 – 6.5
	1500 – 6000	10.0 – 12.0	8.0 – 9.0	5.5 – 6.5	7.5 – 8.0
	>6000	11.0 – 13.5	9.0 – 10.0	7.5 – 8.0	8.0 – 8.5
	<750	6.0 – 8.0	5.5 – 6.0	4.5 – 5.0	4.5 – 4.9
>110	750 – 1500	8.5 – 11.0	7.5 – 8.0	5.5 – 6.0	6.0 – 6.5
	1500 – 4000	10.5 – 13.0	8.5 – 10.0	6.5 – 7.5	8.0 – 8.5
	>6000	11.5 – 14.0	9.0 – 10.5	8.0 – 9.0	8.5 – 9.0

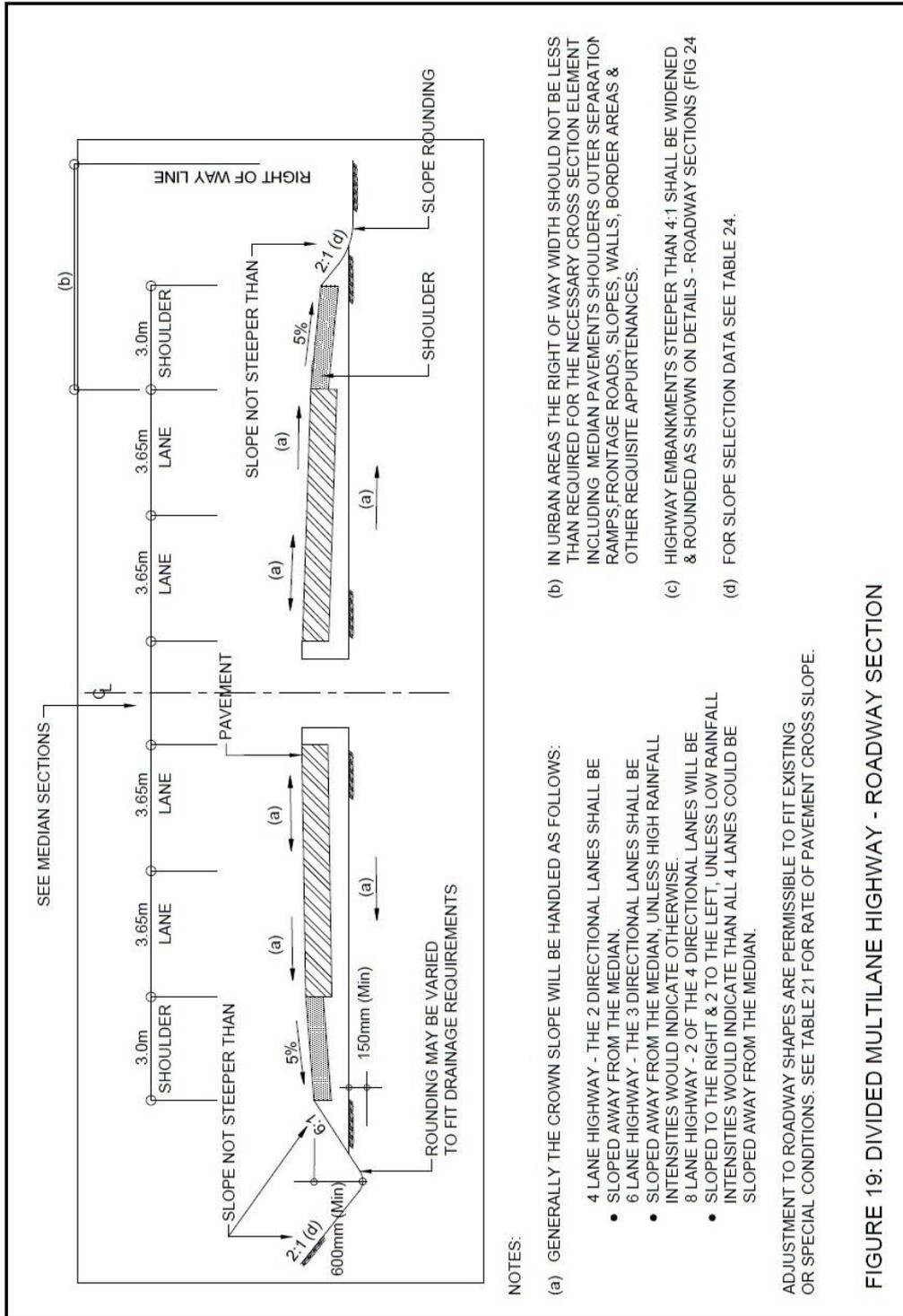
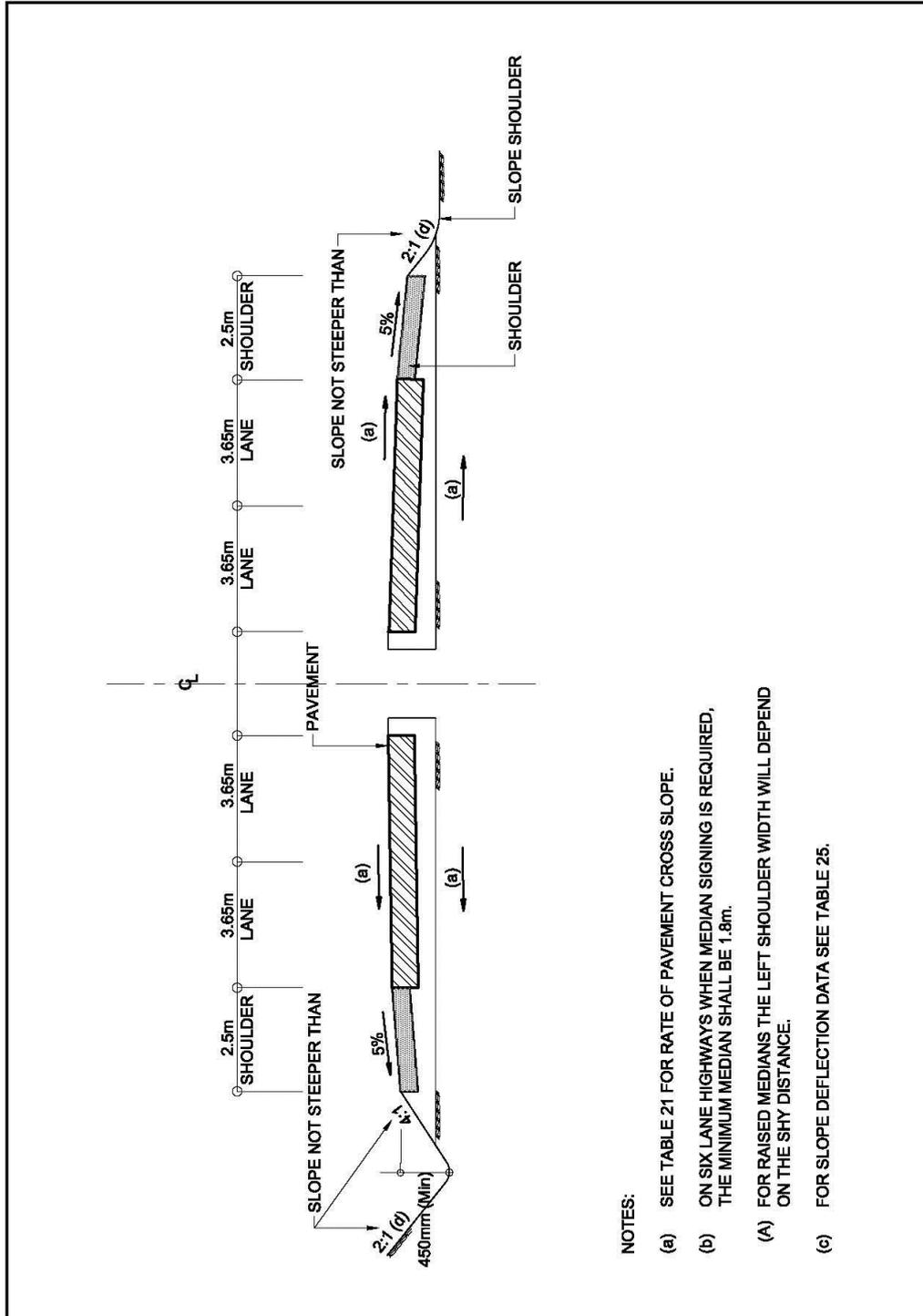


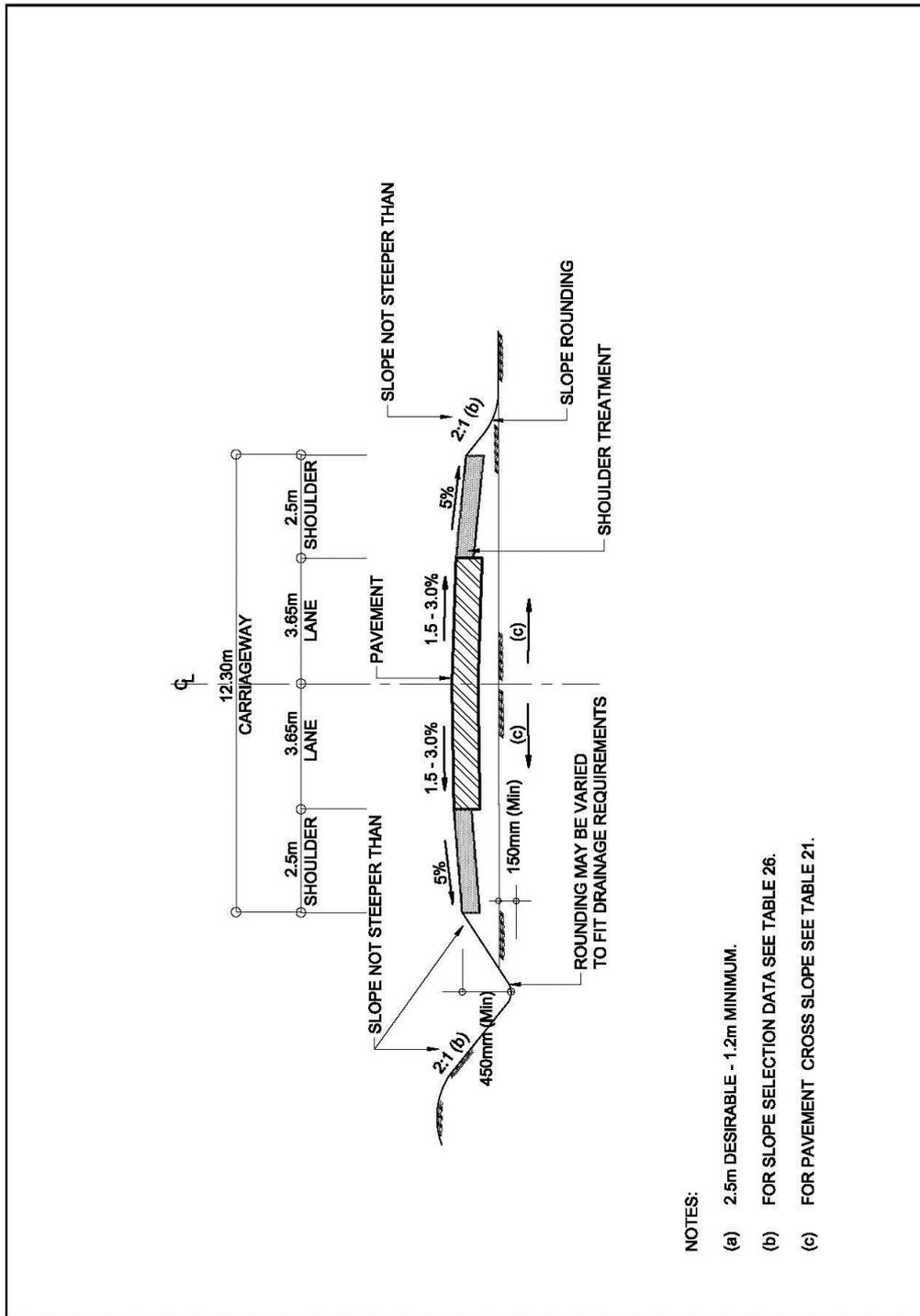
Figure 18: Divided Multilane Highway Roadway Section



NOTES:

- (a) SEE TABLE 21 FOR RATE OF PAVEMENT CROSS SLOPE.
- (b) ON SIX LANE HIGHWAYS WHEN MEDIAN SIGNING IS REQUIRED, THE MINIMUM MEDIAN SHALL BE 1.8m.
- (A) FOR RAISED MEDIANS THE LEFT SHOULDER WIDTH WILL DEPEND ON THE SHY DISTANCE.
- (c) FOR SLOPE DEFLECTION DATA SEE TABLE 25.

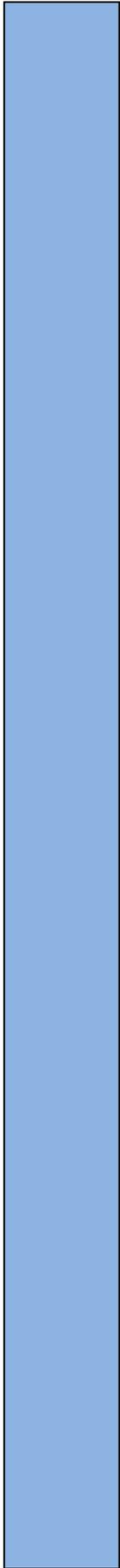
Figure 19: Undivided Multilane Highway Roadway Section



NOTES:

- (a) 2.5m DESIRABLE - 1.2m MINIMUM.
- (b) FOR SLOPE SELECTION DATA SEE TABLE 26.
- (c) FOR PAVEMENT CROSS SLOPE SEE TABLE 21.

Figure 20: Two Lane Highway Roadway Section



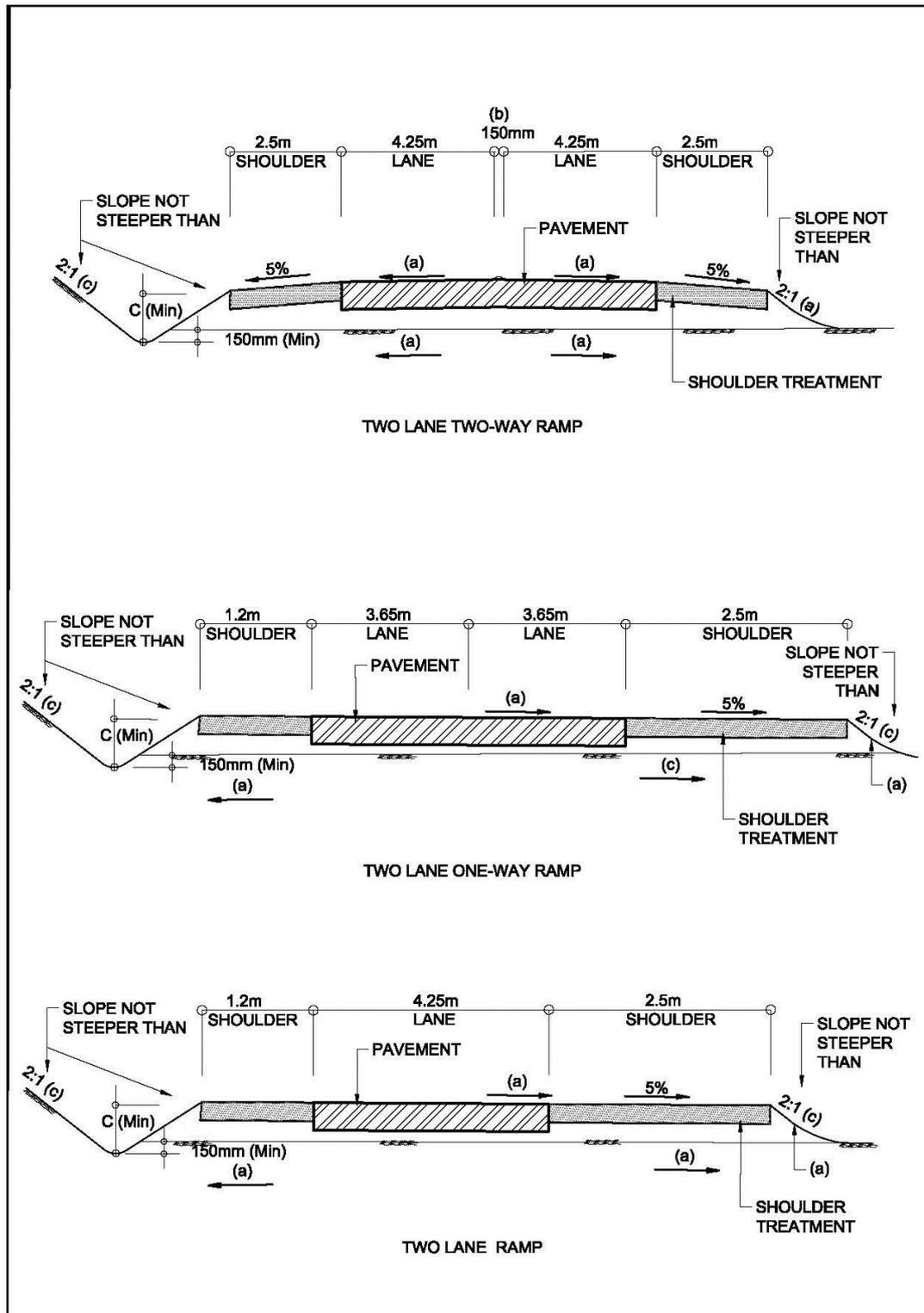


Figure 21: Ramp Roadway Sections

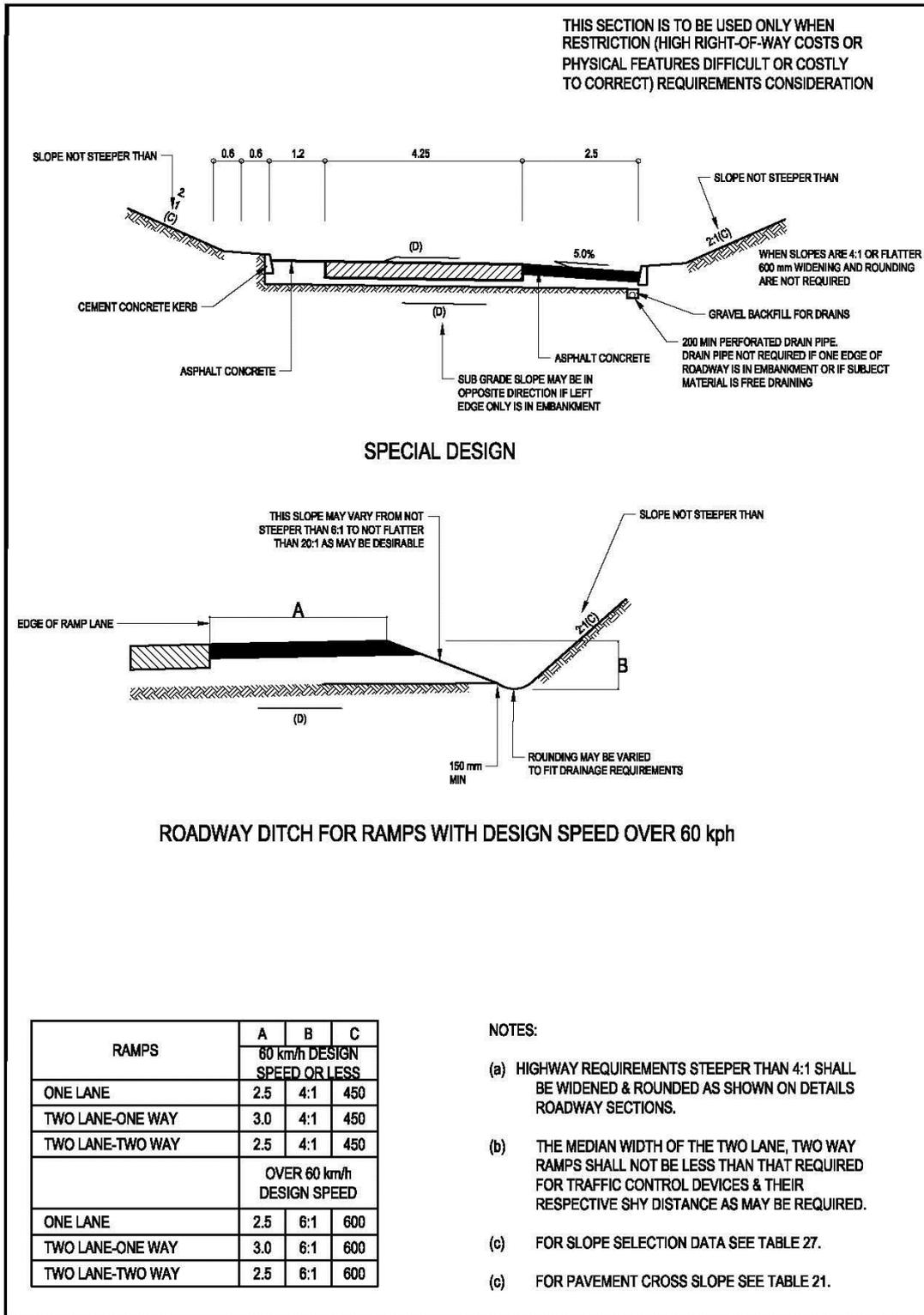
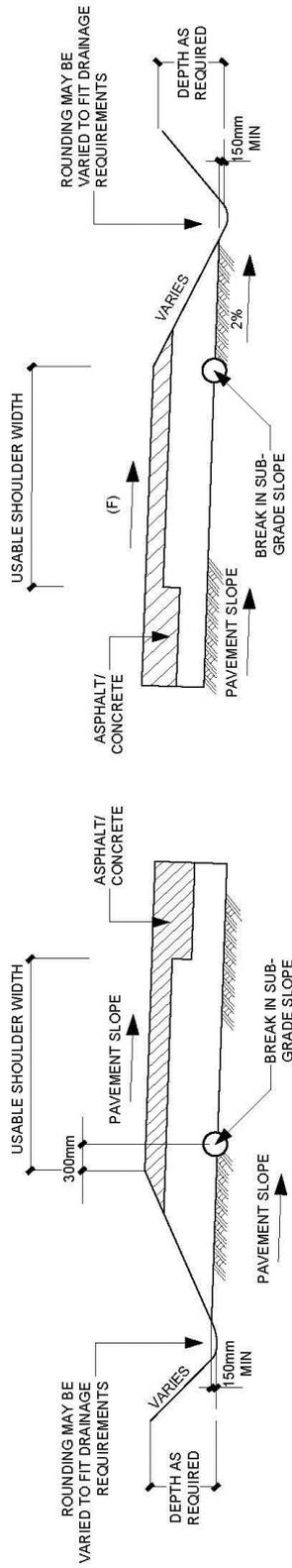
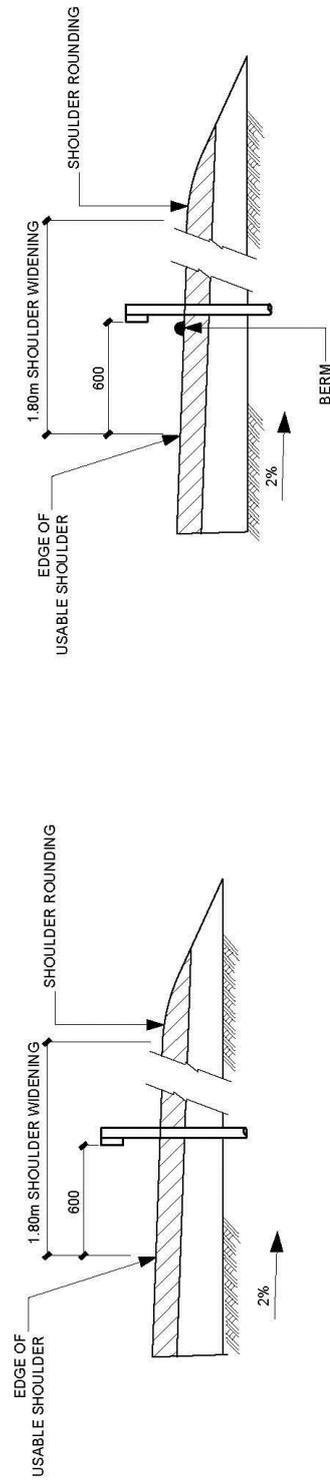


Figure 22: Ramp Roadway Sections

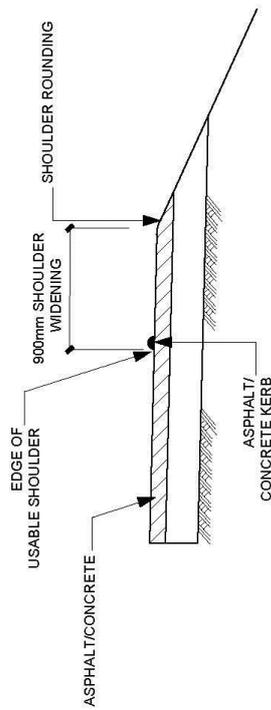


SHOULDER DESIGN ON CURVES



GUARDRAIL REPLACEMENT

Figure 23: Details – Roadway Sections

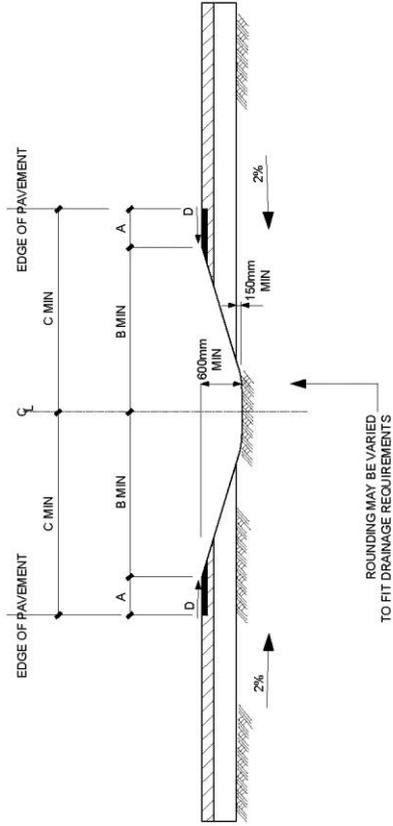


ASPHALT CONCRETE KERB PLACEMENT

NOTES:

- (a) ASPHALT CONCRETE SHOULDERS ARE REQUIRED WHERE EVER ASPHALT CONCRETE KERB IS PLACED. ASPHALT CONCRETE KERB IS TO BE USED ONLY WHERE IT IS NECESSARY TO CONTROL DRAINAGE FROM ROAD WAY RUNOFF THAT MAY CAUSE A SLOPE EROSION PROBLEM.
- (b) IN SPECIAL CASES WHERE A SHALLOWER DEPTH OF ASPHALT CONCRETE IS APPROVED, IT SHALL BE PLACED IN A UNIFORM LAYER.
- (c) IF WIDENING IS REQUIRED FOR THE KERB, GUARDRAIL OR OTHER PURPOSES, ADDITIONAL EMBANKMENT WIDENING IS NOT REQUIRED AND THE ROUNDING MAY BE DONE WITHIN THE AVAILABLE AREA.
- (d) WHEN ROUNDING IS REQUIRED, IT SHALL BE USED UNIFORMLY ON ALL RAMP AND CROSS ROADS, AS WELL AS THE MAIN ROADWAY. ROUNDING ON THE CROSSROAD SHOULD ORDINARILY END JUST BEYOND THE RAMP TERMINANTS AND AT A SIMILAR LOCATION WHERE ONLY A GRADE SEPARATION IS INVOLVED.
- (e) SHOULDERS 1.8m OR LESS IN WIDTH SHALL SLOPE AT A RATE OF 2% AND SHOULDERS OVER 1.8m WIDE SHALL SLOPE AT A RATE OF 5%. ON DIVIDED MULTILANE HIGHWAYS, WITH ASPHALT CONCRETE PAVEMENT'S, 1.2m AND 1.8m SHOULDERS SHALL SLOPE IN THE DIRECTION OF THE ADJACENT LANE WHERE CEMENT CONCRETE PAVEMENT IS USED, THE SHOULDER MAY SLOPE TOWARDS THE MEDIAN.
- (f) ON SUPER ELEVATED SECTIONS THE INSIDE OF CURVE SHOULDER SLOPE SHOULD EQUAL THE PAVEMENT SLOPE BUT NO FLATTER THAN THE STANDARD SHOULDER SLOPES, AS IN NOTE (e).

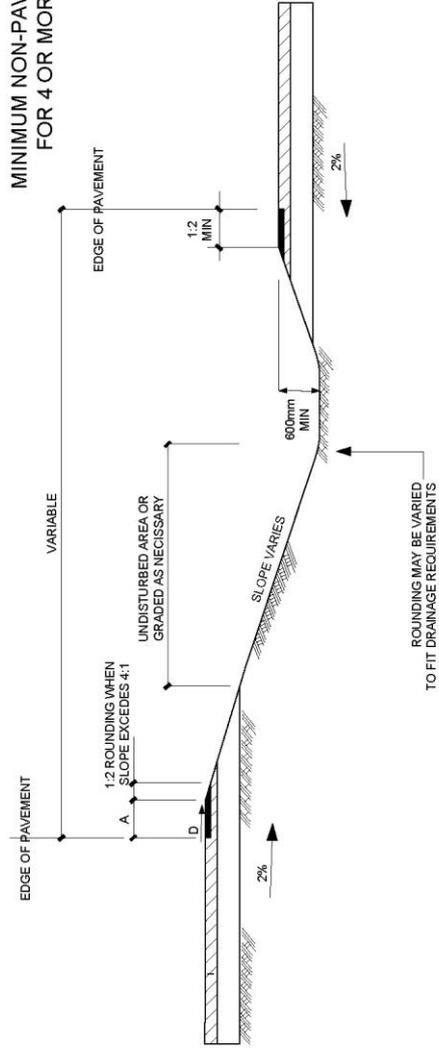
Figure 24: Details – Roadway Sections



DIMENSIONS IN METERS

	A	B	C	D
4 LANES	1.2	5.0	6.0	2%
6 LANES	1.8	5.5	7.3	2%
8 LANES	3.0	4.2	7.3	5%

MINIMUM NON-PAVED MEDIAN FOR 4 OR MORE LANES



MINIMUM FOR 4 OR MORE LANES WITH INDEPENDANT ALIGNMENT

Figure 25: Divided Highway Median Sections

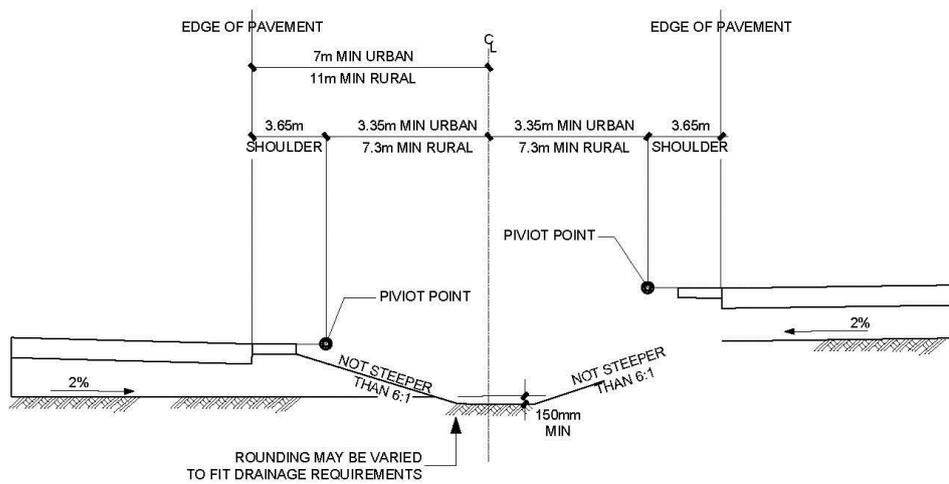
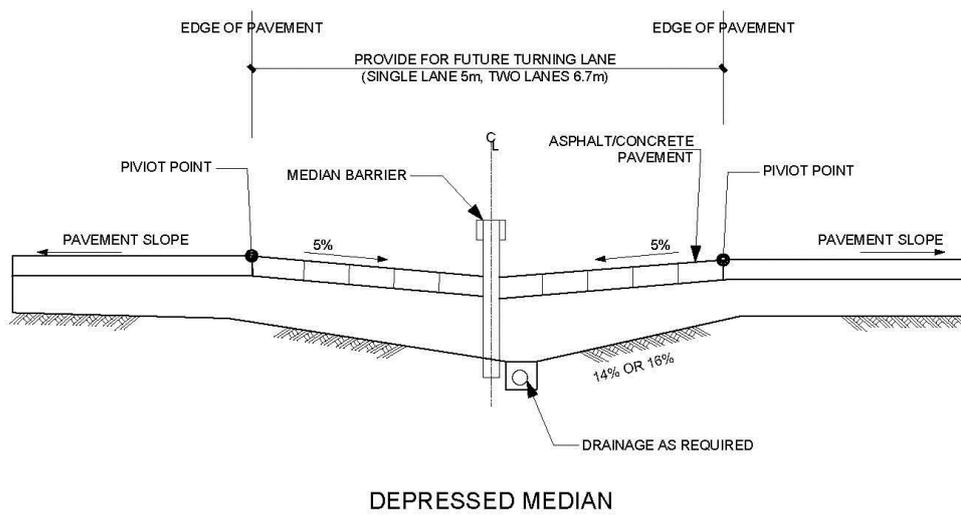
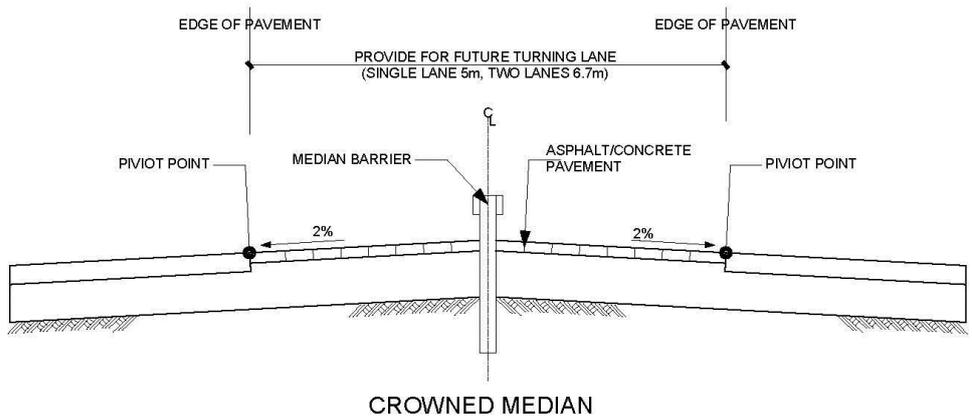
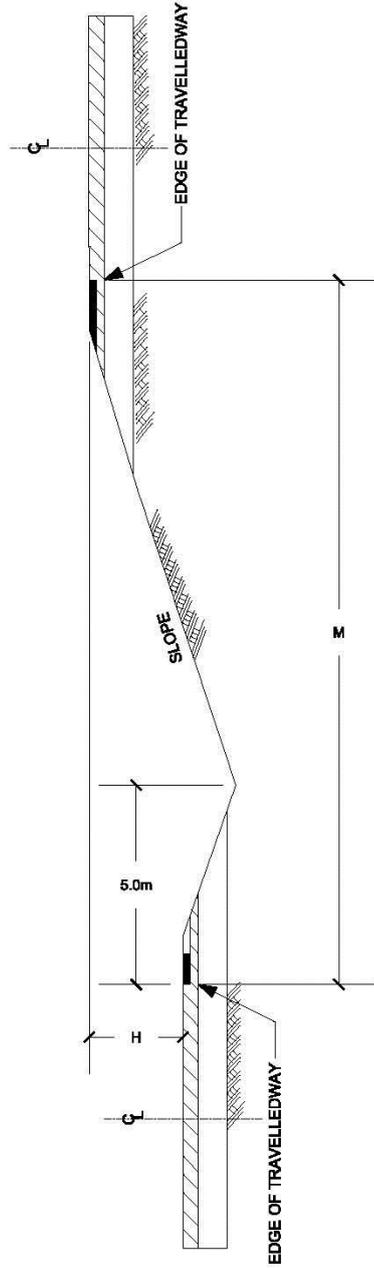


Figure 26: Divided Highway Median Sections



M MEDIAN WIDTH (metre)	H DIFFERENCE IN ELEVATION OF PAVEMENTS IN METRE FOR SLOPE OF			
	6 : 1	4 : 1	3 : 1*	2 : 1*
12	0.9	1.2	1.5	2.1
12	1.2	1.8	2.4	3.6
12	1.8	2.7	3.6	5.1
12	2.1	3.3	4.5	6.6
12	2.7	4.2	5.4	8.1
12	3.6	5.7	7.5	--

* MEDIAN SLOPES STEEPER THAN 4:1 NOT RECOMMENDED

Figure 27: Relation of Difference in Elevation Median

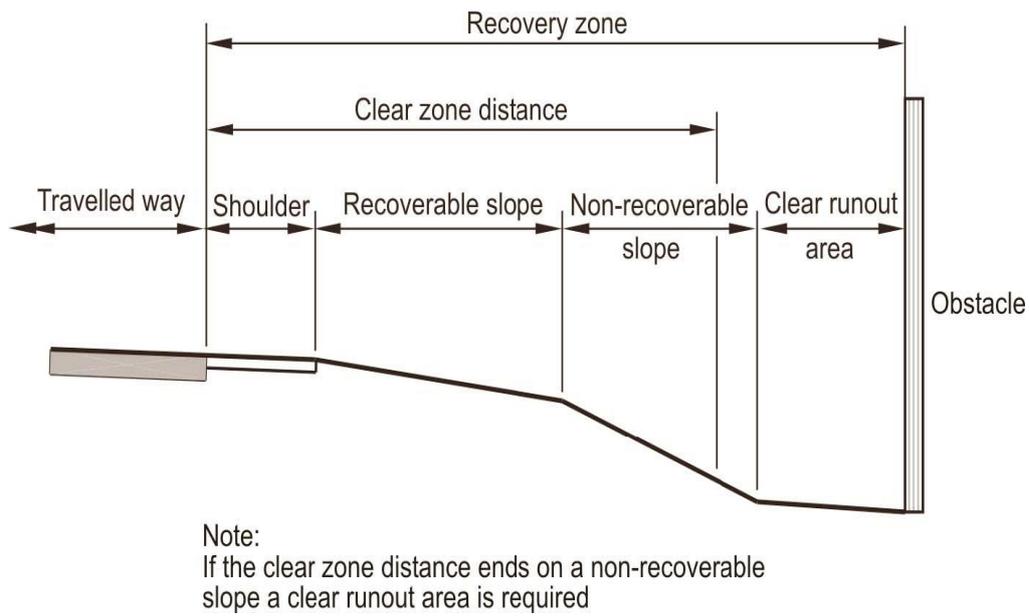


Figure 28: Roadside recovery zone

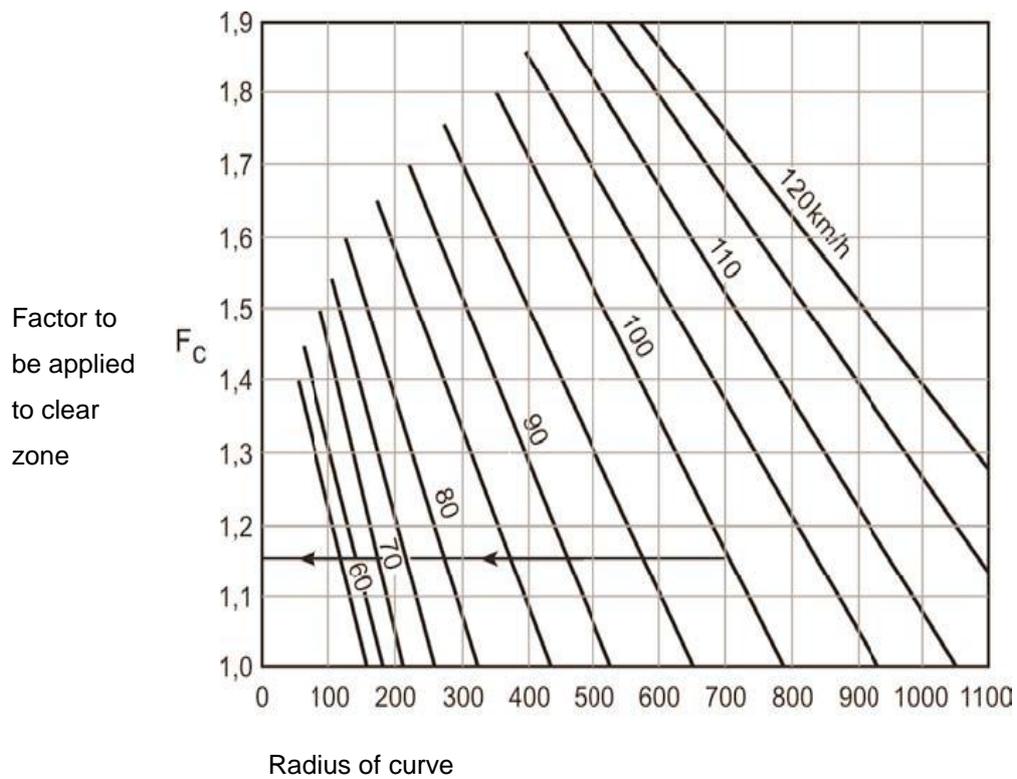


Figure 29: Adjustment for clear zones on curves

4 Intersections at Grade

4.1 General

Most highways other than freeways with limited access have intersections at grade. An at grade intersection is the area where two or more roads join, or cross, within which are included the carriageway(s) and roadside facilities needed for traffic movement. They are the most critical element in the road network because they involve vehicles, pedestrians and bicycles travelling in many directions, which share a common area, often at the same time. Their capacity is limited and there is an inherent risk of collision. The operation of an intersection is influenced by its geometric layout and the type of traffic control which affects its accident potential and capacity. Queue lengths and delays at an intersection are a function of inadequate capacity at an intersection, volume of traffic and vehicle operating characteristics. Mitigation of the resulting conflicts is a major objective of intersection design. This conflict resolution is, in turn, influenced by construction and maintenance costs, environmental factors and the ease of implementation.

Good intersection design should allow transition from one route to another or from one through movement on the main route to an intersecting route, with minimum delay and maximum safety. The need for careful design of intersections, to limit inherent hazards and to maintain acceptable traffic capacity is of great importance. To accomplish this, the layout and operation of the intersection should be clear to the driver, and should provide good visibility between conflicting movements. Furthermore, the number of intersections should be kept to a minimum consistent with traffic demands and spacing should be consistent with the class of road.

Intersections should be located where they can be clearly seen and are easily understood by drivers on all the approach roads and where desirable, as opposed to minimum, safety standards attainable at reasonable cost.

Intersections should not be located where it is difficult or expensive to provide adequate sight distance or driving comfort. Locations which should be avoided are where earthworks are extensive, near bridges, on small radius curves, on the outside of super elevated curves, on high embankments, on steep grades (>3%) or on crests of vertical curves. Careful location as well as landscaping of the

surrounding terrain and planting can be used to improve the visual guidance and visibility at intersections. This can be of great importance for the perception and comprehension of the intersection.

The design of an at grade intersection is based on traffic volume and turning movements, the time and duration of peak traffic volumes, traffic distribution, and the composition of vehicular traffic during peak hours. Design is further influenced by the extent of control required as determined by factors such as vehicle speeds, design capacities, pedestrian movements, and accident data.

An intersection is considered to be safe when it is perceptible, comprehensible and manoeuvrable. These three requirements can generally be met by complying with the following guidelines.

a. Perception

- The intersection should be sited so that the major road approaches are readily visible from all approaches.
- Early widening of the intersection approaches.
- The use of traffic islands on the minor road to emphasize a “yield” or “stop” requirement.
- The use of early and highly visible traffic signs.
- Optical guidance by the use of road landscaping and furniture, especially where an intersection must be located on a crest curve.
- The provision of splays which ensure unobstructed sight lines along the major road.
- The angle of intersection of the major and minor roads should be between 60 and 120 degrees. (Refer to Figure 30)
- The use of single lane approaches on the minor road is preferred in order to avoid mutual sight obstruction from two vehicles waiting next to each at the stop or yield line.

b. Comprehension

- Vehicle priority should be obvious and logical from the intersection layout.
- The types of intersection used throughout the whole road network should be standardized as far as possible.
- Optical guidance should be provided by the use of clearly visible kerbs, traffic islands, road markings, road signs and other road furniture.

c. Manoeuvrability

- All traffic lanes widths should be adequate to accommodate the appropriate vehicle turning characteristics. To accommodate truck traffic, turning radii shall be 15 metres minimum.
- The edges of traffic lanes should be clearly indicated by road markings.
- Traffic islands and kerbs should not conflict with the natural vehicle paths.

The operation of a priority junction depends principally upon the frequency of gaps that occur between vehicles on the main road. These gaps should be of sufficient duration to permit vehicles from the minor road to merge with, or cross, the major road traffic flow. As a consequence junctions are limited in capacity, but this capacity may be optimized by judicious use of canalization or the separation of traffic movements.

4.1.1 Intersection Types

There are four basic types or classes of intersection: three-legged T and Y intersections; four legged intersections; multi-legged intersections; and roundabouts. The last-mentioned type of intersection includes all three of the previous types. The various intersection types are shown schematically in Figure 30. The designer's selection of basic intersection type is normally based on the design context, as intersections can vary greatly in scope, shape, degree of

channelization and traffic control measures. Important factors to be considered in the selection of an intersection type are:

- Cost of construction;
- Type of area;
- Land use and land availability;
- Functional classes of the intersecting roads;
- Approach speeds on intersecting roads;
- Traffic volumes to be accommodated; and
- Proportion of traffic on each approach.

Careful consideration of these factors, and application of the warrants for selecting appropriate traffic control devices, will lead to an appropriate choice of intersection or to a limited number of alternatives from which to make the final selection. The critical factors are cost and capacity. A worldwide review of intersection design practice reported that "typically the cheapest intersection type providing the required level of service should be selected". This cost is usually the sum of the design, construction and right-of-way costs.

Safety at intersections is of key importance. The following summary of the interrelationship between intersections and accidents can act as a guide to the selection of an appropriate intersection type and layout:

- (i) The U.S. National Safety Council estimated that 56 per cent of all urban accidents and 32 per cent of all rural accidents occur at intersections.
- (ii) The number of accidents is proportional to the volume and distribution of traffic between the major and minor roads.
- (iii) Roundabouts have considerable safety advantages over other types of at grade intersection. Inadequate sight distance leads to significantly higher injury and total accident rates. However, at roundabouts the number of accidents may increase with increasing sight distance.

- (iv) Medians should be as wide as practical at rural, unsignalised intersections to provide refuge in the median for turning vehicles. Medians at signalized intersections should be as narrow as practicable to reduce traffic signal clearance times.
- (v) Channelization is usually beneficial but kerbed islands on the major road may be hazardous in rural areas.
- (vi) The hazard of an intersection increases as the approach speed increases.

4.1.2 Data Required for Design

a. Traffic Analysis

The detailed procedures used for traffic capacity analysis of roadways and for intersections are provided in the “Highway Capacity Manual”. Refer also to Chapter 2.4 of this manual.

A traffic flow diagram showing average daily traffic and design hourly volumes, according to time of day (am or pm), is used to determine the conflicting traffic movements. The flow diagram shall indicate the design year for which the indicated volumes are anticipated and the expansion factor used, or to be used, to derive the design traffic volume.

The type of traffic control, if any, to be installed must be determined in advance of intersection design, since it frequently affects canalization. If traffic signals are required, the phasing of the signals is required for the design.

When high pedestrian movements are anticipated detailed pedestrian studies must be undertaken.

b. Site Topography

Site topography details should be obtained from aerial photographs, ground surveys or topographic maps. A scale of 1:500 or larger is required. Site topography details include all improvements and physical controls within the

area that may be affected by the design. Access requirements must be determined based on existing improvements. Profiles of the intersecting roadways are required.

c. Accident Data

For existing intersections, accident data and accident analysis can assist in identifying existing problems to be addressed in the re-design of the intersection.

4.1.3 Traffic Control for Minor Intersections

Intersections with low traffic volumes will usually require few controls in addition to those imposed by the geometrics. The design shown in Figure 31 should be used whenever possible. In more complex situations where major traffic volumes and large turning conflicts occur, plans must be developed based on the channelization required to establish control for the specific situation and to give preference to the major traffic demand.

Intersections with more than four legs, roads intersecting at acute angles, Y intersections and staggered junctions should be avoided wherever possible unless economics dictates their use or retention. Un-signalised intersections should be designed to achieve cross traffic movements at right angles (or as near as possible) even if this means reconstructing sections of the cross road.

The geometric design requirements shown in Figure 31 are to be used for the design of intersections where design hourly volumes DHV do not exceed the following:

<u>Highway Design Hourly Volume</u>	<u>Crossroad Design Hourly Volume</u>
2 000	25
1 500	50
1 000	100

Note: Crossroad DHV should not be confused with crossroad traffic volume.

The minimum angle of intersection for this design is 75 degrees. This standard may be modified where improvement of an existing intersection is restricted by right of way, topography, or other factors.

Adequate intersection sight distance must be provided at all intersections for the design vehicle (refer to Figure 32) for intersections with STOP control on the minor road). For intersections with YIELD control on the minor road, the sight distance to be provided is indicated in Figure 33, Table 32 and Table 33. Required sight distances need to be adjusted for grade on the minor or intersecting road (refer to Table 33).

Vertical sight distance should equal or exceed the values shown for required horizontal sight distance. This can be checked by graphical means assuming an object height of 1.30 metres and an eye height of 1.05 metres.

a. Channelization

The intersection design should provide for channelization as specified in Figure 33 and section 4.5 (which can be easily installed in the future). Alternatively, turning paths can be provided with painted striping if the pavement type used does not clearly define the limits of the turning area.

Channelization should be considered for minor intersections on two lane highways whenever:

- the volume of left turning vehicles exceeds 25 during the design hour;
- where grade or sight distance requires the protection of turning vehicles;
- or if accident experience otherwise indicates that special safeguards are necessary.

b. Highway Crossroad Profile

Crossroad grade must exceed plus or minus 2% for at least 15 metres beyond the edge of the through traffic lane. Crossroad vertical curves on the approach to the intersection are to be designed based on safe stopping sight distance.

Highway shoulders should be warped to fit crossroad shoulders at a rate that is aesthetically acceptable.

4.1.4 Traffic Control at Major Intersections

Many of the principles covered in section 4.4 apply equally to major intersections. Additional design points applicable to major intersections follow.

a. Channelization

The purpose of channelization is to achieve safe and efficient traffic operation by managing the conflicts that are inherent to intersections. NCHRP Report 279 reports that the objectives of good intersection design are:

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

It is necessary to ensure that the channelization used will accomplish the desired objectives under peak traffic loads and adverse operating conditions. Storage lanes must be long enough to keep through lanes clear and to accommodate turning movements.

The specific type of additional control devices (described in subparagraph 4 below) required at an intersection should be taken into account early in the development of geometric design to permit incorporation in the detail design.

b. Traffic Islands

Where the inner edges of pavement for right turns at intersections are designed to accommodate WB-12 and WB-15 heavy vehicles, or where the design permits passenger vehicles to turn at speeds of 25 km/h or more the pavement area at the intersection may become excessively large for proper control of traffic. To avoid this, a corner island should be provided to form a separate turning roadway.

Channelizing islands should define the traffic channels to drivers and pedestrians alike in the clearest manner possible. To prevent confusion, the number of islands should be kept to the minimum commensurate with good design and the use of small islands must be avoided. The approach end of an island should be designed to give the driver adequate warning of its presence, and should never require a sudden change in vehicle direction or speed. Transverse lane shifts should begin far enough in advance of the intersection to allow gradual transitions. The design should avoid the placing of traffic islands on a horizontal or vertical curve. If the use of islands on curves cannot be avoided, adequate sight distance and illumination must be provided.

The design must provide minimum turning radii for all types of vehicles using the intersection and conform to the angles of intersection given in Figure 31 shows the configuration of a standard island and kerb details for a typical channelized intersection.

(i) Shying Distance.

Table 28 gives the required shying distance for mainline roadways where either mountable or barrier-type kerbs are used. When mountable kerbs are used on ramps, no shying distance is needed.

Table 28: Required Shying Distance

Lane Width (W)	On Left		On Right
	Rural (S_1) mm	Urban (S_1) mm	Rural and Urban (S_1) mm
3.65 m	300	300	900 minimum
3.35 m	600	300	
3.00 m	900	600	

(ii) Flaring

A 13.8 meter flare should be used on the left and a 23 meter flare on the right wherever island length permits. Flares of 9 or 4.5 metres on the left and 15 or 7.5 metres on the right may be used for shorter islands. Islands less than 4.5 metres on the left and 7.5 metres on the right need not be flared.

(iii) Kerbing

Pre-cast raised traffic kerbing shall generally be used for small islands unless the use of other types or special design is warranted.

(iv) Size

Triangular islands must be at least 5.0 square metres in area with legs no less than 2.5 metres long excluding the rounding of corners.

(v) Delineation

Islands may be delineated by means of paint striping, traffic buttons, kerbs, or well defined or contrasting pavement edges. The method used in each

case depends on the island’s size, location, and function. (See Manual on Uniform Traffic Control Devices.)

c. Basic Channelization Design

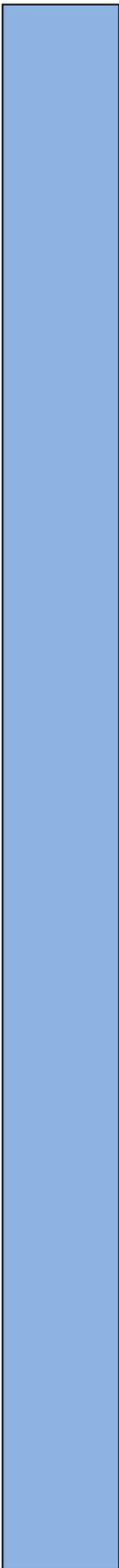
For highways with high approach speeds, channelization should normally be accomplished using painted pavement markings supplemented with plastic lane markers or traffic buttons. Under urban conditions kerbed channelization may be necessary for traffic control.

Under normal conditions the length of the storage lane for turning movements is as follows:

Design Hourly Volume	25 – 180	180 - 210	210 – 240	240 - 270	270 – 300
Length of Storage Lane (m)	25	45	60	70	80

The design year for which the channelization is designed shall be indicated on the plan. In instances where a 20 year design life is not used, the design hour volume should be based on traffic counts projected to the design year. Traffic data will consist of classified counts surveyed over a minimum period of 2 hours each in the morning and afternoon peak traffic periods.

Traffic buttons may be used to delineate channelization if adequate turning radii and clearance are provided. The traffic buttons on the left of the storage lane shall provide an 11 meter turning radius for passenger vehicles and a 15 meter radius for trucks. The length of the traffic buttons may be considered to be part of the required storage lane length.



(vi) *Additional Control Devices*

Appropriate additional controls may be installed whenever their use will increase capacity or safety or where it is anticipated that they may be required at a future date.

(vii) *Beacons*

The use of beacons mounted above intersections that flash yellow on the main street and red on the side street may be effective in reducing angle or turning movement collisions and may be considered.

(viii) *Signals*

Many channelized grade intersections will require traffic signal control as traffic volumes increase. Warrants for the installation of signal control by pre-timed or traffic actuated devices are given in the Manual on Uniform Traffic Control Devices (MUTCD).

(ix) *Signage*

Directional signage is important to control traffic at intersections by clarifying alternative route choices. Directional signs should be designed and located so that they can be understood by motorists that are not familiar with the area.

4.1.5 Minimum Designs for Sharp Turns

a. *General*

Where it is necessary to provide for turning vehicles within minimum space, the minimum turning paths of design vehicles should be applied.

Choice of design vehicle depends on expected turning traffic, but may be based on the following:

P design – used at minor intersections on parkways, at local road intersections on major highways where turns are only occasionally made by SU or WB-15 vehicles or at intersections with minor highways carrying low traffic volumes.

SU design – used at most rural highway intersections. Preferably use larger radii or speed change lanes or both for important turning movements on major highways, particularly with large percentage of trucks.

WB-12 and WB-15 design – applicable where truck combinations turn regularly.

b. Left Turns

An important factor in designing median openings or intersections is the path of each design vehicle making minimum radius left turns at low speed.

The sharpest paths of the four design vehicles making left turns at low speeds without reversing or swinging wide are shown in Table 29. At median openings, simple arcs to fit these paths, tangential to the median edge and the centreline of the crossroad determines the minimum dimensions of median openings.

Table 29: Minimum Left Turn Radii

Control Radius (m)	12	15	25
Design Vehicles Accommodated			
Predominant	P	SU	WB-12
Occasional	SU	WB-12	WB-15

c. Right Turns

For minor intersections the recommended minimum radius for right turns is 17, which will accommodate an SU design vehicle. In all cases the minimum radius will be 10 metres as required by a P design vehicle. WB-12 and WB-15 design

vehicles can turn on a 17 meter radius if they are permitted to encroach beyond the turning lane pavement.

For the WB-12 and WB-15 design vehicles it is not practical to fit a simple circular arc to their minimum design paths. However, such vehicles can turn, where traffic lanes are 3.65 metres wide, without encroachment into adjacent lanes when the radius of the simple curve on the inner edge of pavement is approximately 23 metres for the WB-12 unit and 30 metres for the WB-15 unit.

4.1.6 Turning Roadway Widths

Roadway pavements need to be widest at intersections and on horizontal interchange ramp curves to provide sufficient widths for turning vehicles.

The widths of turning roadways should be designed in accordance with Table 30 and the following:

- Pavement widths should be based on anticipated design vehicle use.
- Provide turning roadway pavement widths throughout the length of the curve. Make pavement transitions on tangent symmetrical about the centreline, and at a ratio of not less than 1:25.
- Increase the tabulated widths by 300 mm where a barrier kerb is constructed at one pavement edge, and 600 mm where barrier kerbs are adjacent to both pavement edges.
- Design turning roadways, with continuous traffic flow, for curve radii of 30 metres or more, except where minor traffic volumes are involved or where the cost is inconsistent with the benefits.
- To permit emergency passing of stalled vehicles on one lane, one way pavements, provide shoulders of 1.2 metres left and 2.4 metres right for all radii.

Radius on Inner Edge of Pavement (meter)	Case 1				Case 2				Case 3			
	One Lane, One-Way Operation				One Lane One-Way Operation With Provision for Passing a Stalled Vehicle by Another of the Same Type				Two Lane Operation			
	No Provision for Passing				Pavement Width				Either One or Two Way (Same type of vehicle in both lanes)			
	P	SU	WB-12	WB-15	P	SU	WB-12	WB-15	P	SU	WB-12	WB-15
15	13	3.5	6.7	7.9	6.1	8.8	11	13.4	7.9	10.7	12.8	15.2
22.5	13	5.2	5.8	6.7	5.8	8.2	9.5	36	7.6	10.1	11.2	12.8
30	13	4.9	5.5	6.4	5.8	7.6	8.8	34	7.6	9.4	10.7	12.2
45	12	4.9	5.2	5.8	5.4	7.3	8.2	29	7.3	9.1	10.1	10.7
60	12	4.9	4.9	5.2	5.4	7	7.5	27	7.3	8.8	9.4	10.1
90	12	4.6	4.9	5.2	5.4	6.7	7.3	25	7.3	8.5	9.1	9.4
120	12	4.6	4.9	4.9	5.2	6.7	7	24	7	8.5	8.8	9.1
150	12	4.6	4.6	4.9	5.2	6.7	7	24	7	8.5	8.8	9.1
Tangent	12	4.6	4.6	4.6	5.2	6.4	6.4	21	7	8.2	8.2	8.2

Note:

P = Passenger Vehicles

SU = Single-Unit Trucks

WB-12 = Semi-Trailer Combinations

WB-15 = Semi-Trailer Combinations

4.1.7 Roundabout Design

Modern roundabouts differ from the older style traffic circles in their uniform characteristics and operation. Internationally, roundabouts operate on the "Yield on entry" rule so that, where vehicles drive on the right, vehicles yield to the left and vice versa. A number of geometric elements are incorporated in the design of roundabouts and all elements appear in all roundabouts. These elements are illustrated in Figure 36.

a. Operation of Roundabouts

Roundabouts operate by deflecting the vehicle path so as to slow traffic and promote yielding. The roadway entry is usually flared to increase capacity. Delays at roundabouts are usually less than at conventional intersections and capacity is higher, provided that the combined intersection flow from all approaches should not exceed 3 500 veh/h. Reduced delays reduce vehicle operating costs.

Roundabouts have fewer potential conflict points than conventional intersections. The four-legged intersection has 32 conflict points whereas the modern roundabout has only 8, as illustrated in Figure 37.

For both roundabouts and conventional intersections, the diverge is regarded as a conflict point. Roundabouts generally offer superior safety performance to that of conventional intersections and the reduced number of conflict points at roundabouts results in a significant reduction in accident rates.

Despite their advantages, roundabouts are not appropriate for every situation.

They may be inappropriate:

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

- Where traffic flows are unbalanced, with high flows on one or more approaches;
- At intersections of major roads with minor roads, where roundabouts would cause serious delays to traffic on the major roads;
- Where there are substantial pedestrian flows;
- As an isolated intersection in a network of linked signalised intersections;
- In the presence of reversible lanes;
- 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 adequate widths for turning movements; and
- Where downstream signalised traffic control could cause a queue to back-up through the roundabout.

Roundabouts can be considered when:

- Intersection volumes do not exceed 3 000 veh/h at three-legged or 4 000 veh/h at four-legged intersections;
- The proportional split between the volumes on the major and minor road does not exceed 70/30 where;
 - the intersection flow is less than 1 500 veh/h (three-legged intersections) or,
 - the intersection flow is less than 2 000 veh/h (four-legged intersections);
- The proportional split between the volumes on the major and minor road does not exceed 60/40 where;
 - the intersection flow is greater than 1 500 veh/h (three-legged intersections) or;

- the intersection flow is greater than 2 000 veh/h (four-legged intersections);
- One major flow has a predominant through movement that is:
 - Between 50 and 80 per cent of the approach volume; or
 - Between 25 and 40 per cent of the intersection volume; and
 - High volumes of right-turning movements, i.e. more than 25 per cent of the approach volume occurs which experience long delays (e.g. 15 seconds or more per vehicle) and a high incidence of right-angled accidents.

b. Design Speed

The design speed within the roundabout should ideally range between 40 to 50 km/h. This requires a radius of between 60 and 80 metres and an overall diameter of the roundabout of 150 metres. Very often, the space for this size of intersection will not be available and a reduced design speed will have to be accepted.

Where the design speeds on the approaches are high, e.g. more than 15 km/h faster than the design speed within the facility, it may be necessary to force a reduction in vehicle speed. This could be achieved by introducing reverse horizontal curvature. The ratio between the radii of successive curves should be of the order of 1.5:1.

Speed humps should not be used 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 must be long and the height low enough to ensure that the vertical acceleration caused by the speed hump does not cause the driver to lose control. Bus passengers, particularly those sitting in the rear overhang of the vehicle, would find traversing a speed hump distinctly uncomfortable. Speed humps should only be used at roundabouts in residential areas or where the intention is to apply traffic calming measures.

c. Sight Distance

Sight visibility is important in the design of roundabouts. Specifically, approaching drivers must have a clear view of the nose of the splitter or separator island. At the yield line and while traversing the roundabout, they must have an uninterrupted view of the opposing legs of the intersection at all times. The elaborate landscaping schemes sometimes placed on the central islands of roundabouts are totally inappropriate for the intended function of the layout. Decision sight distance for intersections should be provided on each approach to the roundabout to ensure that drivers can see the nose of the splitter island. Roundabouts should not be located on crest curves with restricted sight distance.

d. Components

The various components of a roundabout are illustrated in Figure 36.

(i) Deflection

A very important component in the design of roundabouts is the deflection forced on vehicles on the approaches to the roundabout. The objective is to reduce the speed of vehicles so that, within limits, the greater the deflection the better. The limit is that the minimum acceptable angle of skew at an intersection is 60° , as discussed previously in this chapter. 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.

(ii) Entries and exits

The single lane approaches to roundabouts are typically 3.4 to 3.7 metres wide. The entry width is one of the most important factors in increasing the capacity of the roundabout and this width can be increased 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 could add a full lane to the

entry to increase capacity. The width of a two-lane entry should be approximately 8 metres. A variation on the two-lane entry is to have a single-lane circulatory roadway with an auxiliary lane provided for vehicles turning left 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 per cent or more of the entry flow or more than 300 veh/h during peak hours.

(iii) 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 1.2 times 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 thus be not less than 1.6 metres with 2.0 metres preferred. To ensure that vehicles do not travel faster than the design speed, the maximum radius on the vehicle path should not exceed 100 metres. As a general guideline, the circulatory roadway should be sufficiently wide to allow a stalled vehicle to be passed.

The proposed width of a circulatory roadway is:

- *6.5 metres – single lane operation*
- *8 metres – two lane operation*
- *Increase by 3 metres – significant truck proportion for both cases*
- *13 metres for single lane – significant semi-trailer proportion*
- *16 metres for two lane - significant semi-trailer proportion*

A circulatory road width of 13 metres would make it possible for passenger cars to traverse the roundabout on relatively large radius curves 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.

(iv) Central Island

The central island consists of a raised non-traversable area, except in the case of mini-roundabouts where the central island may simply be a painted dot. The island is often landscaped but it should be ensured that the landscaping does not obscure the sight lines across the island. It is customary to provide circular islands. While for semi-trailers the width of the circulatory road between kerbs would have to be 13 metres in a single-lane configuration, all other vehicles could be served by a road width of 9.5 metres. A mountable area or apron could thus be added to the central island to accommodate this difference. The apron should have cross fall steeper than that of the circulatory road, to discourage passenger vehicles from driving on it. A cross fall of 4 to 5 per cent is recommended.

(v) Splitter islands

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

- Allow drivers to perceive the upcoming roundabout and to reduce entry speed;
- Provide for a comfortable deceleration distance;
- Physically separate entering and exiting traffic;
- Prevent highly dangerous wrong-way driving;
- Control entry and exit deflections;
- Provide a refuge for pedestrians and cyclists; and
- Provide a place to mount traffic signs.

The sizes of splitter islands are governed by the dimensions of the central island and the 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 the splitter islands should be equal to the comfortable deceleration distance to accommodate the transition 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.

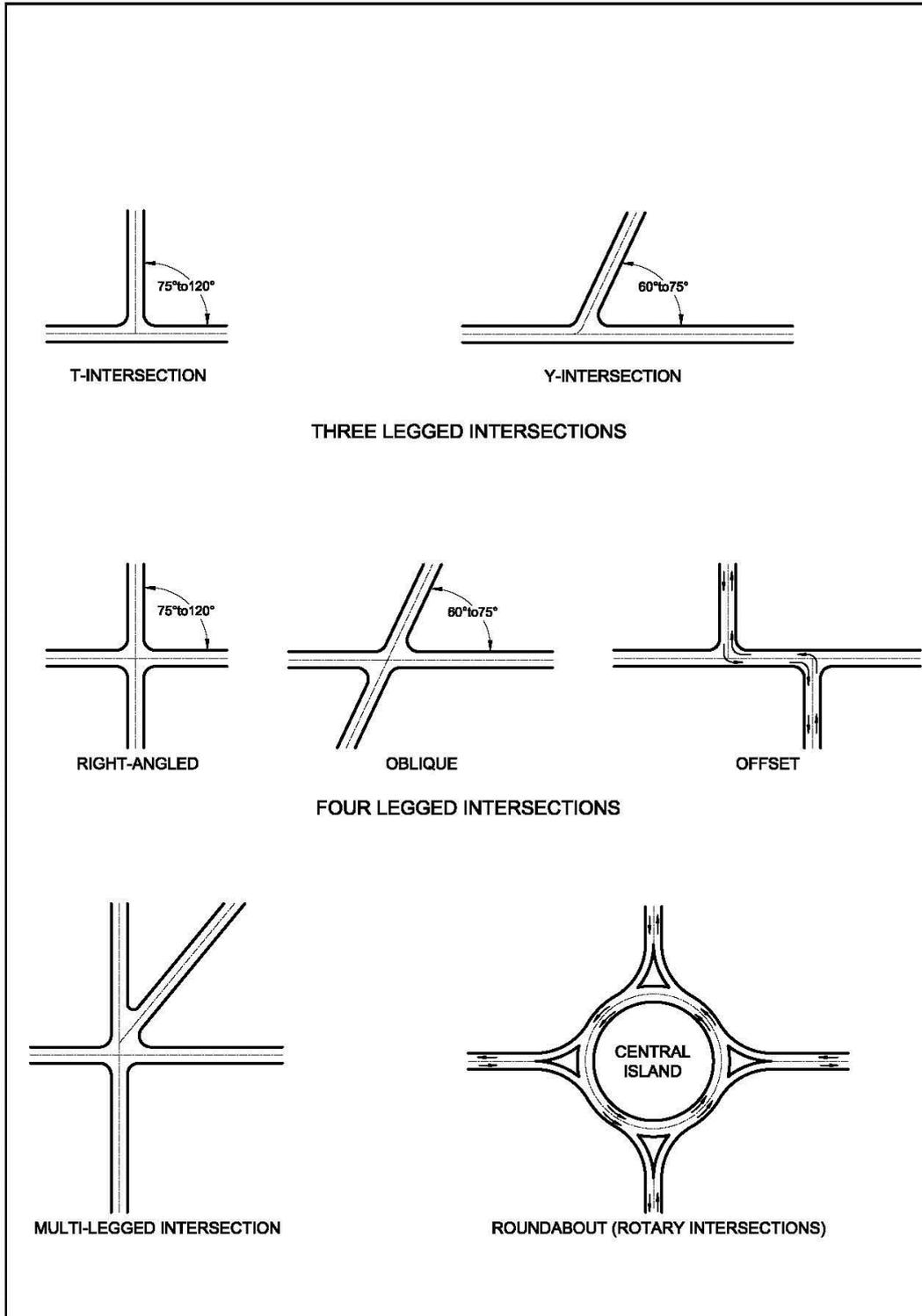


Figure 30: Intersection Types

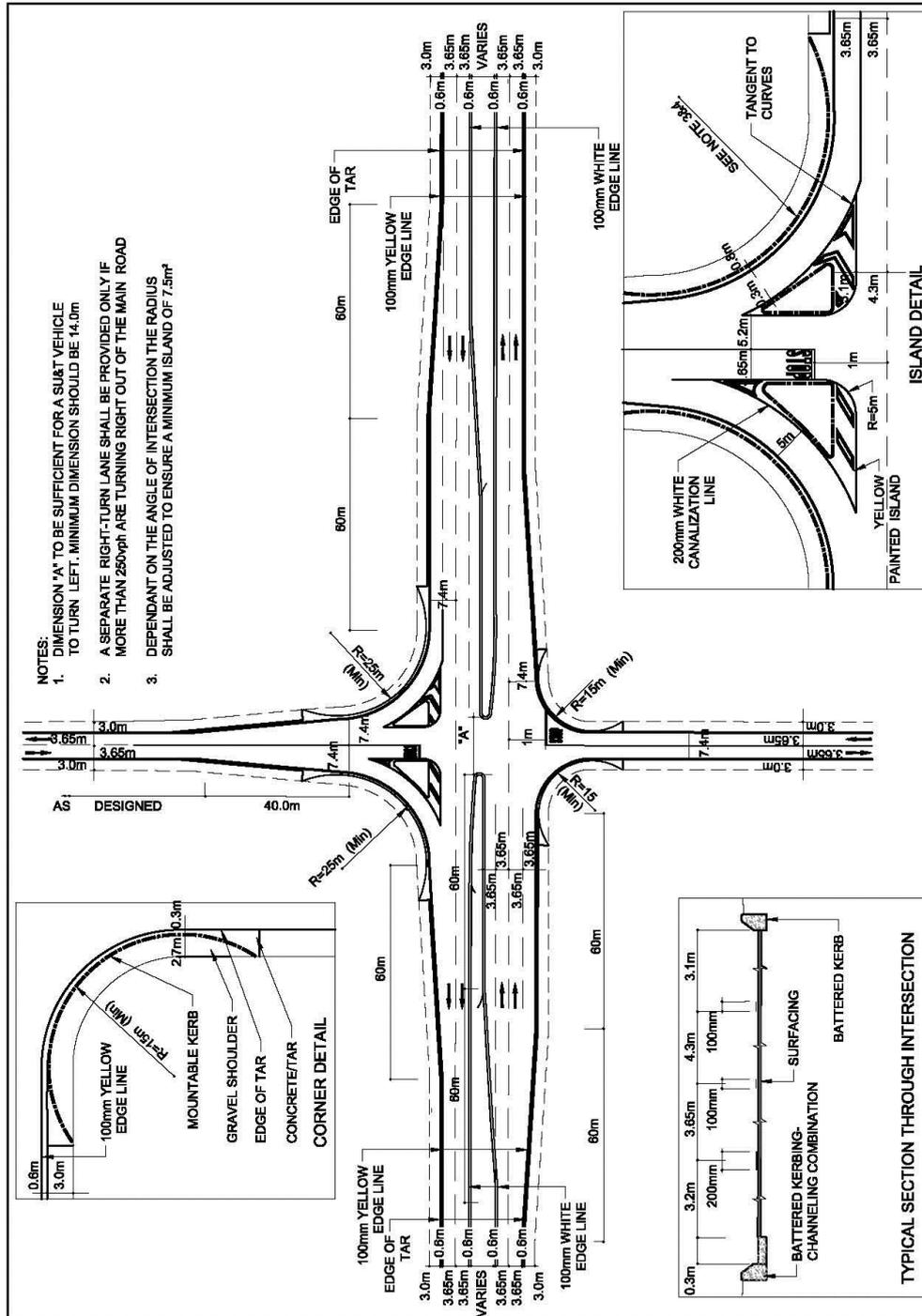


Figure 31: Minor Intersection Control

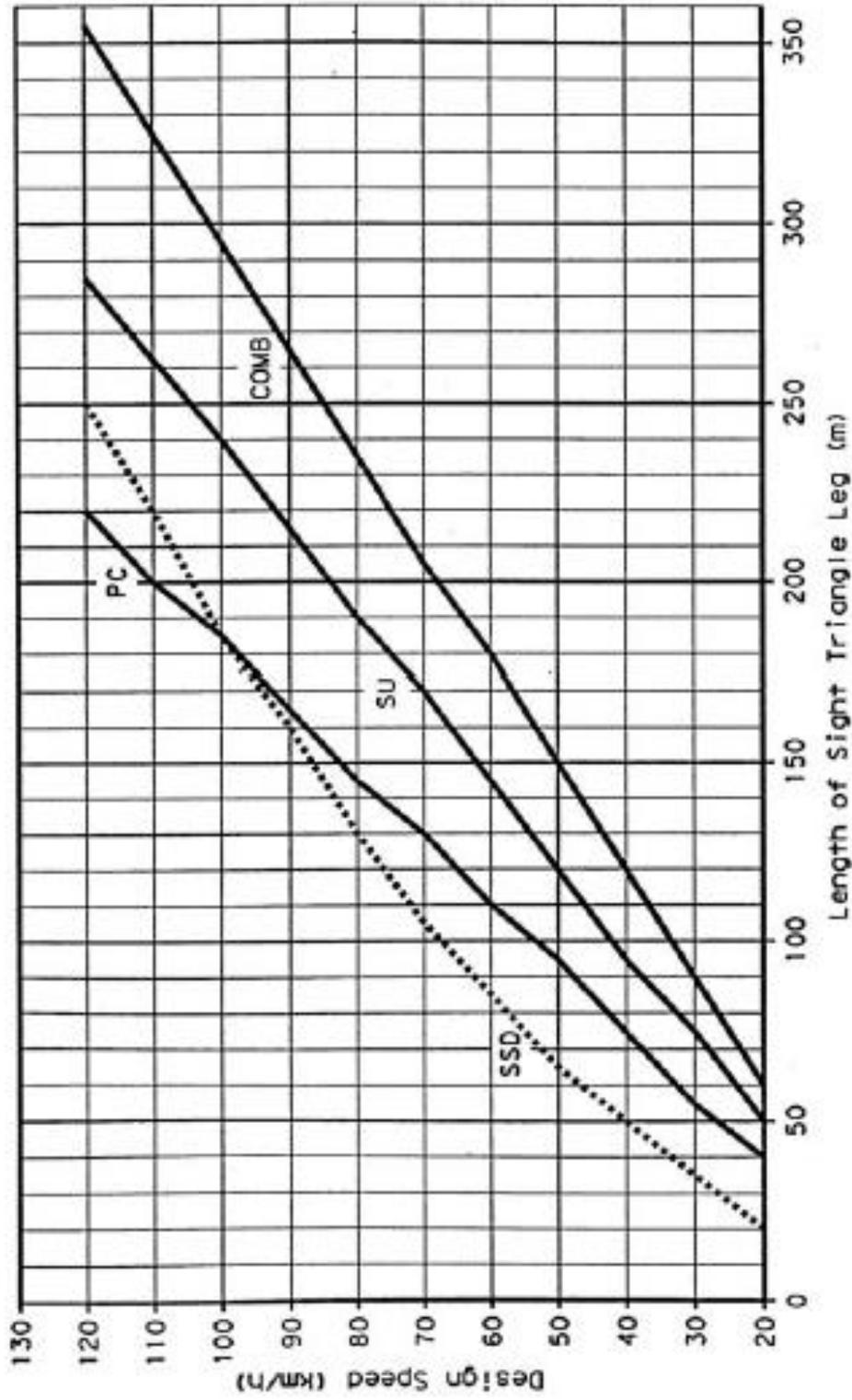


Figure 32: Intersection Sight Distance Right Turn from Stop and Crossing Manoeuvre



Table 31: Crossing Manoeuvres from Yield Controlled Approaches-Length of Minor Road Leg and Travel Times

Design speed (Km/h)	Minor road approach		Travel time (t_g) (seconds)	
	Length of leg ¹ (m)	Travel time $t_a^{1,2}$ (seconds)	Calculated value	Design value ^{3,4}
20	20	3.2	7.1	7.1
30	30	3.6	6.2	6.5
40	40	4.0	6.0	6.5
50	55	4.4	6.0	6.5
60	65	4.8	6.1	6.5
70	80	5.1	6.2	6.5
80	100	5.5	6.5	6.5
90	115	5.9	6.8	6.8
100	135	6.3	7.1	7.1
110	155	6.7	7.4	7.4
120	180	7.0	7.7	7.7
130	205	7.4	8.0	8.0

Notes:

- For minor road approach grades that exceed 3%, multiply the distance or the time in this table by the appropriate adjustment factor from Table 33.
- Travel time applies to a vehicle that slows before crossing the intersection but does not stop.
- The value of t_g should equal or exceed the appropriate time gap for crossing the major road from a stop controlled approach.
- Values shown are for a passenger car crossing a two lane highway with no median and grades 3% or less.

Table 32: Length of Sight Triangle Leg along Major Road and Crossing Manoeuvre at Yield Controlled Intersections

Major road design speed (Km/h)	Stopping sight distance (m)	Minor road design speed (Km/h)						
		20	30 - 80	90	100	110	120	130
		Design values (m)						
20	20	40	40	40	40	45	45	45
30	35	60	55	60	60	65	65	70
40	50	80	75	80	80	85	90	90
50	65	100	95	95	100	105	110	115
60	85	120	110	115	120	125	130	135
70	105	140	130	135	140	145	150	160
80	130	160	145	155	160	165	175	180
90	160	180	165	175	180	190	195	205
100	185	200	185	190	200	210	215	225
110	220	220	200	210	220	230	240	245
120	250	240	220	230	240	250	260	270
130	285	260	235	250	260	270	280	290

Note:

Values in the table are for passenger cars and are based on the unadjusted distances and times in Table 32.

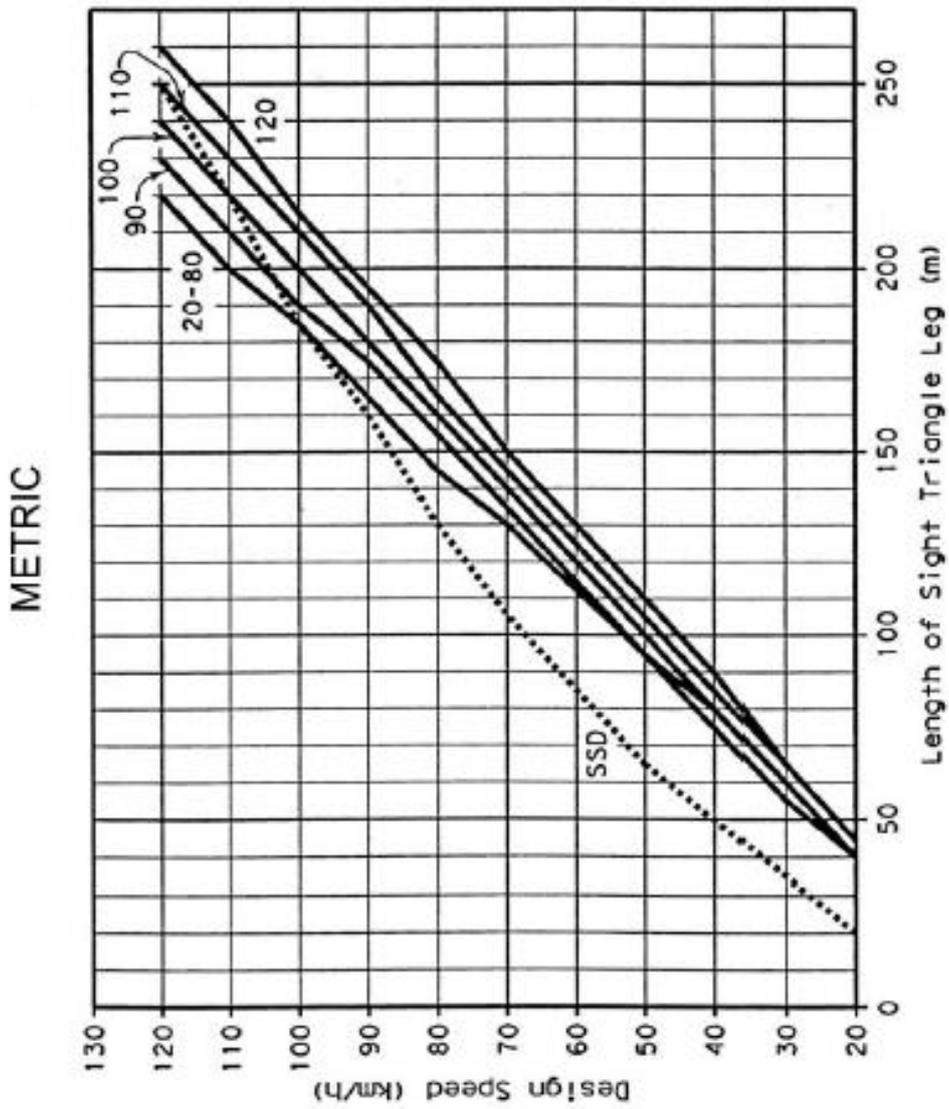
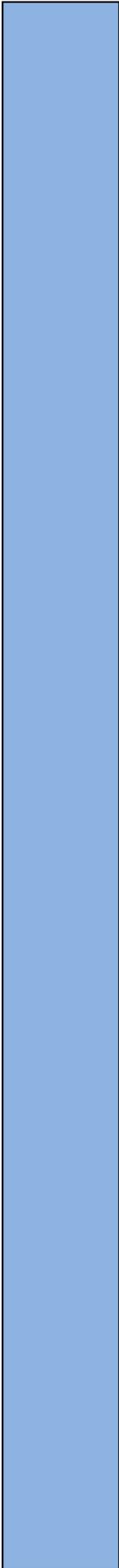


Exhibit 9-62 AASHTO

Figure 33: Length of Sight Triangle Leg along Major Road for Passenger Cars

Table 33: Adjustment Factors for Sight Distance Based on Approach Grade

Approach grade (%)	Design speed (Km/h)											
	20	30	40	50	60	70	80	90	100	110	120	130
-6	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2
-5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
-4	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
-3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9



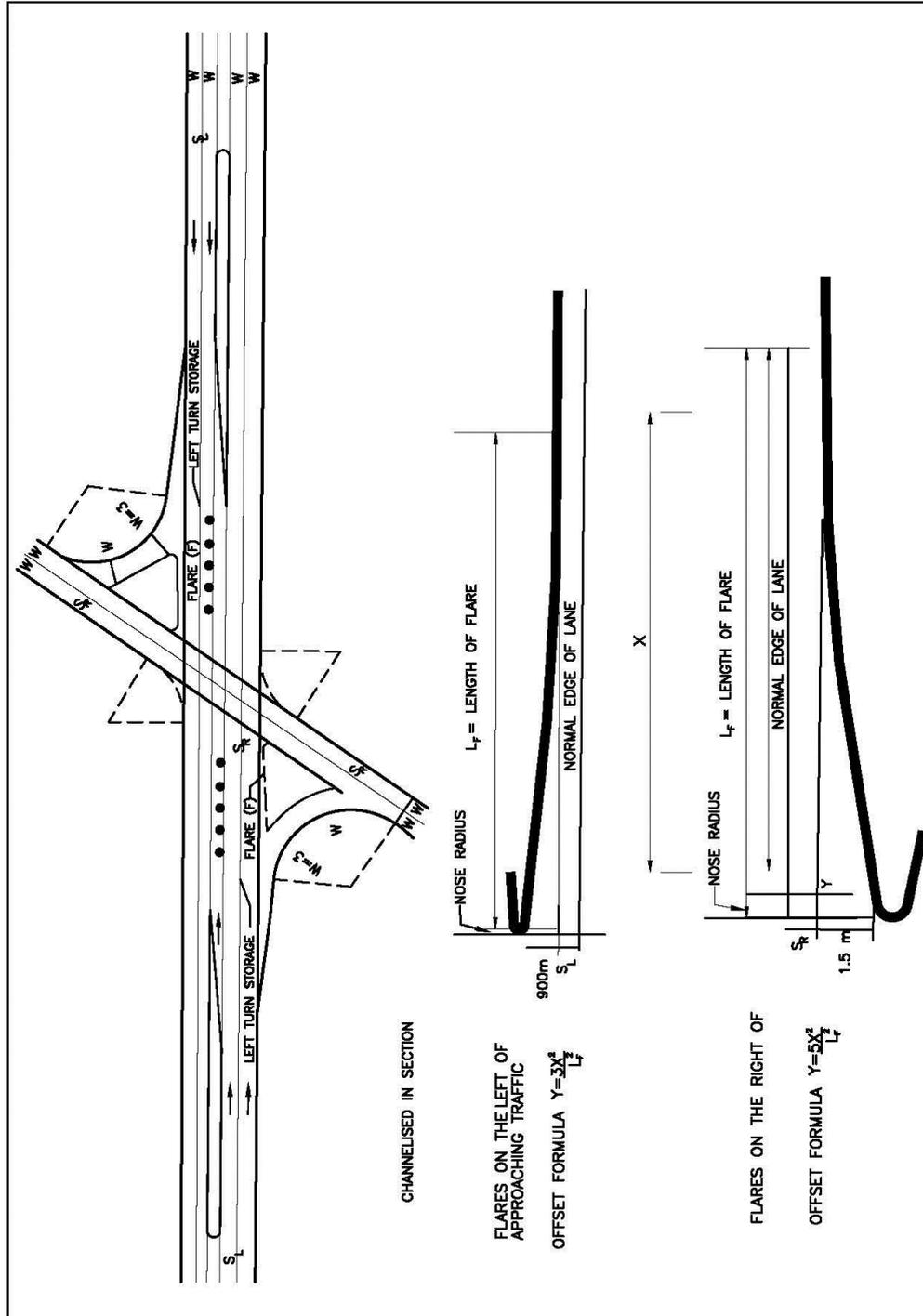


Figure 34: Island and Kerb Details

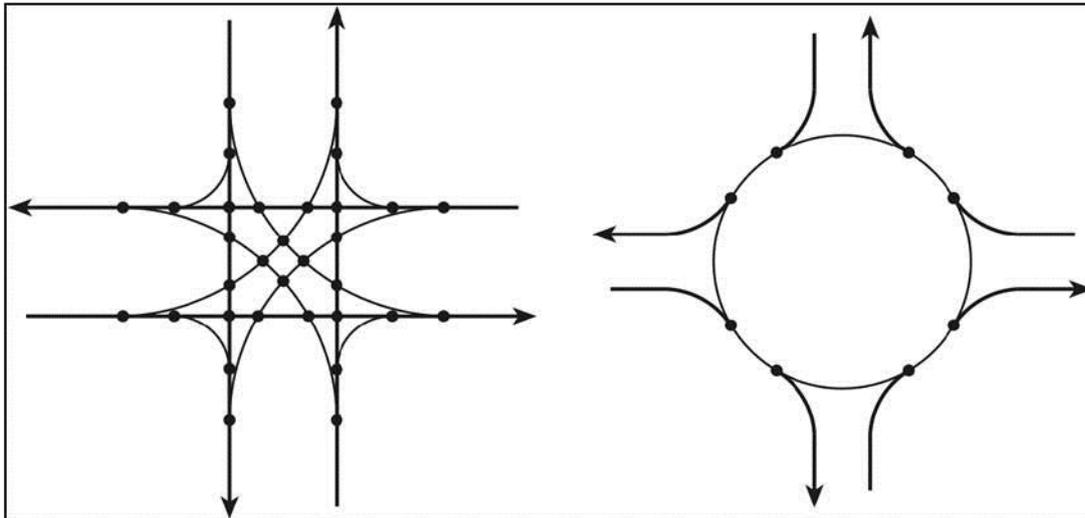


Figure 35: Intersection Conflict Points

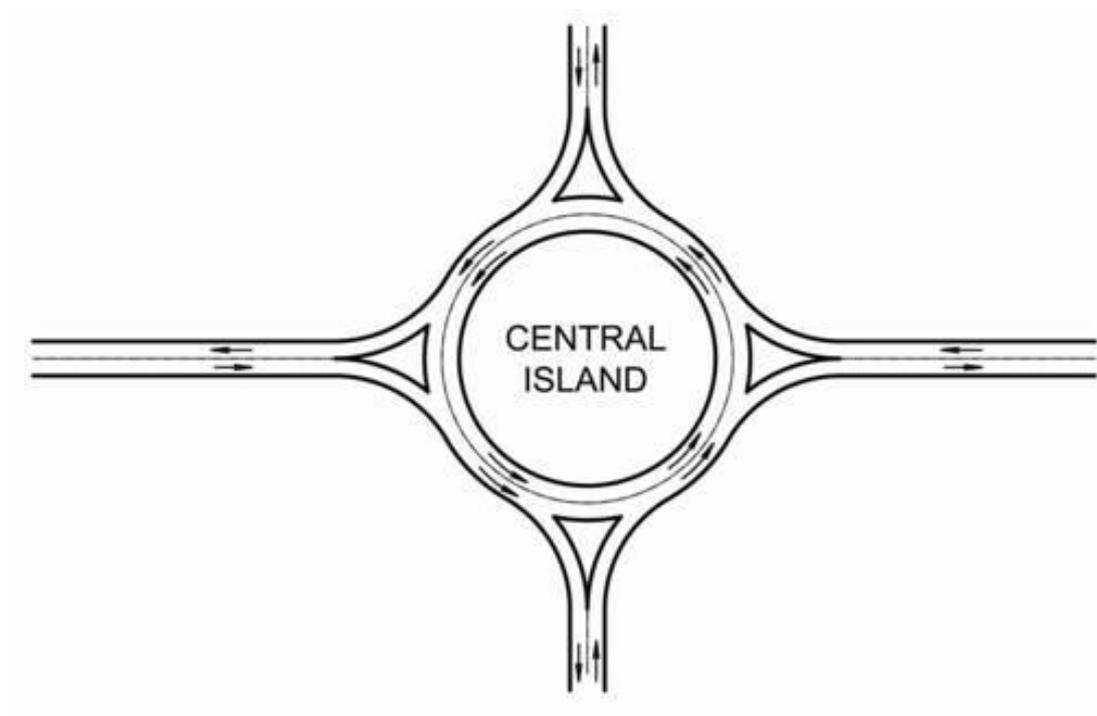


Figure 36: Elements of Roundabouts

5 Interchanges

5.1 General

The principal difference between interchanges and other forms of intersection is that, in interchanges, crossing movements are separated in space whereas, in the latter case, they are separated in time. At-grade intersections accommodate turning movements either within the limitations of the crossing roadway widths or through the application of turning roadways whereas the turning movements at interchanges are accommodated on ramps. The ramps replace the slow turn through an angle of skew that is approximately equal to 90 degrees by high-speed merging and diverging manoeuvres at relatively flat angles.

The first interchange, which was constructed in the U.S.A., provided loops for all left turns and outer connectors for all right turns thus creating the Cloverleaf Interchange. Since then, a variety of interchange forms has been developed. These include the:

- Diamond;
- Par-Clo (from Partial Cloverleaf);
- Directional and the
- T or Trumpet

The various types of interchange configuration are described in paragraph 5.8.3. Each basic form can be divided into sub-types. For example, the Diamond Interchange is represented by the narrow diamond, the wide diamond and the split diamond. The most recent development in the Diamond interchange form is the Single Point Diamond Interchange. This form is also referred to as the Urban Interchange.

Historically, the type of interchange to be applied at a particular site would be selected as an input into the design process. In fact, like the cross-section, the

interchange is the aggregation of various elements. A more sensible approach is thus to select the elements appropriate to a particular site in terms of the topography, local land usage and traffic movements and then to aggregate them into some or other type of interchange.

The interchange approaches the ultimate solution to the intersection at grade as it promotes safety and maximum traffic capacities on the intersecting roadways. An interchange provides at least one connection for traffic between intersecting roadways. Crossing conflicts are completely eliminated by grade separation and turning conflicts are minimized. Interchanges shall be provided on all highways and major routes on which full access control is required and at any other location where traffic cannot be controlled safely and efficiently by intersections at grade.

The chief advantages of interchanges are:

- The capacity of the through travelled ways within the interchange can be made to approach or equal that outside the interchange.
- Increased safety is provided for through and left turning traffic. Right turning movements make the same manoeuvre as on at grade intersections but generally on a much higher type facility, which also results in greater safety.
- Stops and appreciable speed changes are eliminated for through movements. Drivers making turning movements usually slow down, but not excessively on adequately designed facilities.
- The highway grade separation is flexible in design and may be adapted to almost all likely angles and positions of intersection roads.

The chief disadvantages of interchanges stem from cost considerations and the practical aspect of providing desirable designs in areas of limited right-of-way and in difficult terrain.

5.2 Design Principles

Manoeuvres in an interchange area occur at high speeds close to the freeway and over relatively short distances. It is therefore important that drivers should experience no difficulty in recognising their route through the interchange irrespective of whether that route traverses the interchange on the freeway or diverges to depart from the freeway to a destination that may be to the left or the right of the freeway. In following their selected route, drivers should be disturbed as little as possible by other traffic. These requirements can be met through the application of the basic principles of interchange design.

The driver has a number of tasks to execute successfully to avoid being a hazard to other traffic. It is necessary to:

- select a suitable speed and accelerate or decelerate to the selected speed within the available distance;
- select the appropriate lane and carry out the necessary weaving manoeuvres to effect lane changes if necessary; and
- diverge towards an off-ramp or merge from an on-ramp with the through traffic.

To maintain safety in carrying out these tasks, the driver must be able to understand the operation of the interchange and should not be surprised or misled by an unusual design characteristic. Understanding is best promoted by consistency and uniformity in the selection of types and in the design of particular features of the interchange.

Interchange exits and entrances should always be located on the right. Left hand side entrances and exits are counter to driver expectancy and also have the effect of mixing high speed through traffic with lower-speed turning vehicles. The problem of extracting turning vehicles from the median island and providing sufficient vertical clearance either over or under the opposing freeway through lanes is not trivial. The application of left hand entrances and exits should only be considered under

extremely limiting circumstances. Even in the case of a major fork where two freeways are diverging, the lesser movement should, for preference, be on the right.

Route continuity substantially simplifies the navigational aspects of the driving task. For example, if a driver simply wishes to travel on a freeway network through a city from one end to the other it should not be necessary to deviate from one route to another.

Uniformity of signing practice is an important aspect of consistent design and reference should be made to the Highway Manual Part 1 Design: Volume VI Road Traffic Signs and Road Monitoring.

Ideally, an interchange should have only a single exit for each direction of flow with this being located in advance of the interchange structure. The directing of traffic to alternative destinations on either side of the freeway should take place clear of the freeway itself. In this manner, drivers will be required to take two binary decisions, (yes/no) followed by (left/right), as opposed to a single compound decision. This spreads the workload and simplifies the decision process, hence improving the operational efficiency of the entire facility. Closely spaced successive off-ramps could be a source of confusion to the driver leading to erratic responses and manoeuvres.

Single entrances are to be preferred, also in support of operational efficiency of the interchange. Merging manoeuvres by entering vehicles are an interruption of the free flow of traffic in the right lane of the freeway. Closely spaced entrances exacerbate the problem and the resulting turbulence could influence the adjacent lanes as well.

From the standpoint of convenience and safety in particular prevention of wrong-way movements, interchanges should provide ramps to serve all turning movements. If for any reason this is not possible or desirable, it is nevertheless to be preferred that, for any travel movement from one road to another within an interchange, the return movement should also be provided.

Provision of a spatial separation between two crossing streams of traffic raises the problem of which to take over the top. The choice of whether the crossing road should be taken over or under the freeway depends on a number of factors, not the

least of which is the matter of terrain and construction costs. There are, however, a number of advantages in carrying the crossing road over the freeway. These are:

- Exit ramps on up-grades assist deceleration and entrance ramps on downgrades assist acceleration and have a beneficial effect on truck noise.
- Rising exit ramps are highly visible to drivers who may wish to exit from the freeway.
- The structure has target value, i.e. it provides advance warning of the possibility of an interchange ahead necessitating a decision from the driver whether to stay on the freeway or perhaps to change lanes with a view to the impending departure from the freeway.
- Dropping the freeway into cut reduces noise levels to surrounding communities and also reduces visual intrusion.
- For the long-distance driver on a rural freeway, a crossing road on a structure may represent an interesting change of view.
- The crossing road ramp terminals may include right and left turn lanes, traffic signals and other traffic control devices. Not being obstructed by bridge piers and the like, these would be rendered more visible by taking the crossing road over the freeway.

The other design principles, being continuity of basic lanes, lane balance and lane drops are discussed in paragraph 3.5.9 as matters of detailed design.

5.3 Warrants for Interchanges

5.3.1 Traffic Volumes

With increasing traffic volumes, a point will be reached where all the options of temporal separation of conflicting movements at an at-grade intersection have been exhausted. One of the possible solutions to the problem is to provide an interchange.

The elimination of bottlenecks by means of interchanges can be applied to any intersection at which demand exceeds capacity and is not necessarily limited to

arterials. Under these circumstances, it is necessary to weigh up the economic benefits of increased safety, reduced delay and reduced operating and maintenance cost of vehicles against the cost of provision of the interchange. The latter includes the cost of land acquisition, which could be high, and the cost of construction

5.3.2 Freeways

The outstanding feature of freeways is the limitation of access that is brought to bear on their operation. Access is permitted only at designated points and only to vehicles travelling at or near freeway speeds. As such, access by means of intersections is precluded and the only permitted access is by way of interchanges. Interchanges are normally provided with crossing roads that are high in the functional road hierarchy, e.g. arterials, although, if these are very widely spaced, it may be necessary to provide an interchange serving a lower order road, for example a collector. It follows that the connection between two freeways would also be by means of an interchange, in which case reference is to a systems interchange as opposed to an access interchange.

5.3.3 Safety

Some at-grade intersections exhibit high accident rates that cannot be lowered by improvements to the geometry of the intersections or through the application of control devices. Such situations are often found at heavily trafficked urban intersections. Accident rates also tend to be high at the intersections on heavily trafficked rural arterials where there is a proliferation of ribbon development. A third area of high accident rates is at intersections on lightly trafficked, low volume rural locations where speeds tend to be high. In these cases, low-cost interchanges such as the Jug-handle layout may be an adequate solution to the problem.

5.3.4 Topography

The topography may force a vertical separation between crossing roads at the logical intersection location. As an illustration, the through road may be on a crest curve in cut with the crossing road at or above ground level. If it is not possible to

relocate the intersection, a simple Jug-handle type of interchange as illustrated in Figure 49 may be an adequate solution to the problem.

As can be deduced from the above, warrants are necessarily general and the provision of an interchange must, to a large extent, be based on engineering judgment. The provision of an interchange is warranted as an integral part of a freeway since it provides the only means of access to and from the freeway.

5.4 Weaving

The Highway Capacity Manual (2000) defines weaving as the crossing of two or more traffic streams travelling in the same general direction without the aid of traffic control devices but then goes on to address the merge-diverge operation as a separate issue. However, the merge-diverge operation, associated with successive single-lane on-and off-ramps where there is no auxiliary lane, does have two streams that, in fact, are crossing. Reference to weaving should thus include the merge-diverge.

Three types of weave are illustrated Figure 37, Figure 38 and Figure 39. Type A weave requires all weaving vehicles to execute one lane change. Type B weaving occurs when one of the weaving streams does not have to change lanes but the other has to undertake at most one lane change. Type C weaving allows one stream to weave without making a lane change, whereas the other stream has to undertake two or more lane changes.

The Type B weave is, in essence, a Type A weave but with the auxiliary lane extending either up or downstream of the weaving area and with an additional lane being provided either to the on or to the off-ramp. It follows that a Type A weaving section can be easily converted into a Type B weave. At any site at which a Type A weave appears, it would thus be prudent to check the operation at the site for both types of weave.

5.5 Location and Spacing of Interchanges

The location of interchanges is based primarily on service to adjacent land. On rural freeways bypassing small communities, the provision of a single interchange may be adequate, with larger communities requiring more. The precise location of interchanges would depend on the particular needs of the community but, as a general guide, would be on roads recognised as being major components of the local system.

Rural interchanges are typically spaced at distances of eight kilometres apart or more. This distance is measured from centre line to centre line of crossing roads.

The generous spacing applied to rural interchanges would not be able to serve intensively developed urban areas adequately. As an illustration of context sensitive design, trip lengths are shorter and speeds lower on urban freeways than on rural freeways. As drivers are accustomed to taking a variety of alternative actions in rapid succession a spacing of closer than eight kilometres can be considered.

At spacings appropriate to the urban environment, reference to a centre line-to-centre line distance is too coarse to be practical. The point at issue is that weaving takes place between interchanges and the available distance is a function of the layout of successive interchanges. For a common centre line-to-centre line spacing, the weaving length available between two diamond interchanges is significantly different from that between a Par-Clo-A followed by a Par-Clo-B. Weaving distance is defined in the Highway Capacity Manual and other sources as the distance between the point at which the separation between the ramp and the adjacent lane is 0.5 metres to the point at the following off ramp at which the distance between ramp and lane is 3.7 m as illustrated in Figure 40.

If this definition is adopted, the weaving length becomes a function of the rates of taper applied to the on and off ramps. Reference to the Yellow Line Break Point (YLBP) distance is totally unambiguous and is the preferred option.

Three criteria for the spacing of interchanges can be considered. In the first instance, the distance required for adequate signage should ideally dictate spacing of successive interchanges. If it is not possible to achieve these distances, consideration can be given to a relaxation based on achieving Level Of Service

(LOS) D conditions on the freeway. The third criterion is that of turbulence, which is applied to the merge-diverge situation.

- a. The distance required to provide adequate sign posting which, in turn, influences the safe operation of the freeway, is used to define the minimum distance between ramps. The minimum distances to be used for detailed design purposes, as measured between Yellow Line Break Points, for different areas and interchange types should be not less than the values stated in Table 34.

Table 34: Interchange Spacing in terms of Signage Requirements

Configuration	Urban Areas	Rural Areas
Access to access	1 300 m	2 200 m
Access to systems	2 100 m	3 300 m
Systems to access	1 400 m	2 200 m
Systems to systems	2 500 m	3 800 m

- b. In exceptional cases, where it is not possible to meet the above requirements, relaxation of these may be considered. It is, however, necessary to ensure that densities in the freeway left hand lane are not so high that the flow of traffic breaks down. Densities associated with LOS E would make it difficult, if not actually impossible, for drivers to be able to change lanes. Formulae according to which densities can be estimated are provided in the Highway Capacity Manual. Drivers need time to locate a gap and then to position themselves correctly in relation to the gap while simultaneously adjusting their speed to that required for the lane change. The actual process of changing lanes also requires time.

- c. In the case of the merge-diverge manoeuvre, turbulence caused on the left lane of the freeway by a close succession of entering and exiting vehicles becomes an issue. According to Roess and Ulerio this turbulence manifests itself over a distance of roughly 450 metres upstream of an off-ramp and downstream of an on-ramp. A spacing of 900 metres would suggest that the entire length of freeway between interchanges would be subject to turbulent flow. The likelihood of breakdown in the traffic flow would thus be high and the designer should ensure that space is available for one area of turbulence to subside before onset of the next.

- d. In off-peak periods, vehicles would be moving between interchanges at the design speed or higher. The geometry of the on and off ramps should be such that they can accommodate manoeuvres at these speeds. Increasing the taper rates or reducing the length of the speed change lanes purely to achieve some or other hypothetically acceptable Yellow Line Break Point distance does not constitute good design.

- e. The spacing between successive interchanges will have an impact on traffic operations on the crossing roads and vice versa. If the crossing road can deliver vehicles to the freeway faster than they can carry out the merge, stacking of vehicles will occur on the on-ramp with the queue possibly backing up on to the crossing road itself. Stacking can also occur on an off ramp if the crossing road ramp terminal cannot accommodate the rate of flow arriving from the freeway. The queue could conceivably back up onto the freeway, which would create an extremely hazardous situation.

It should be realised that relaxations below the distances recommended under (a) above will result in an increase in the driver workload. Failure to accommodate acceptable levels of driver workload in relation to reaction times can be expected to result in higher than average accident rates. Twomey et al demonstrate that, at spacings between noses of greater than 2 500 metres, the accident rate is fairly constant, i.e. the presence of the following interchange is not a factor in the accident

rate. At spacings of less than 2500 m between noses, the accident rate increases until, at about 500 m between noses, the accident rate is nearly double that of the 2500 m spacing. This is illustrated in Figure 41 below

The question that must be addressed is the benefit that the community can expect to derive in exchange for the cost of the higher accident rate. By virtue of the fact that freeway speeds tend to be high, there is a high probability that many of the accidents would be fatal. It is therefore suggested that the decision to reduce the interchange spacing below those listed Table 34 should not be taken lightly.

It would be necessary to undertake a full-scale engineering analysis of the situation that would include:

- estimation of future traffic volumes at a ten to twenty year time horizon, comprising weaving and through volumes in the design year;
- calculation of traffic densities;
- assessment of the local geometry in terms of sight distances, and horizontal and vertical alignment;
- development of a sign sequence; and
- a form of benefit/cost analysis relating community benefits to the decrease in traffic safety.

Density offers some indication of the level of exposure to risk and, for want of any better measure, it is suggested that a density higher than 22 vehicles/kilometre/lane, corresponding to LOS D, would not result in acceptable design. It would be necessary to pay attention to remedial actions to prevent interchange constraints, such as inadequate ramp capacity, signalling or crossroad volumes, causing back up onto the freeway.

In summary: spacings of interchanges in terms of their YLBP distances should desirably be in accordance with Table 34. If these spacings are not achievable and an interchange is absolutely vital for service to the community and adjacent land uses, relaxations may be considered but then, to minimise the risk of accidents, the

density calculated according to the above-mentioned formulae should not exceed 22 vehicles/kilometre/lane, which corresponds to LOS D.

5.6 Basic Lanes and Lane Balance

Basic lanes are those that are maintained over an extended length of a route, irrespective of local changes in traffic volumes and requirements for lane balance. Alternatively stated, the basic number of lanes is a constant number of lanes assigned to a route, exclusive of auxiliary lanes.

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

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

At merges, the number of lanes downstream of the merge should be one less than the number of lanes upstream of the merge. This is typified by a one-lane ramp merging with a two-lane carriageway that, after the merge, continues as a two-lane carriageway as is the case on a typical Diamond Interchange layout. This rule precludes a two-lane ramp immediately merging with the carriageway without the addition of an auxiliary lane.

At diverges, the number of lanes downstream of the divergence should be one more than the number upstream of the divergence. 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 divergence is reduced by one.

This can be used to drop a basic lane to match anticipated flows beyond the divergence. 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.

The application of lane balance and coordination with basic number of lanes is illustrated in Figure 42.

5.7 Auxiliary Lanes

As in the case of the two-lane two-way road cross-section with its climbing and passing lanes, and the intersection with its right and left turning lanes, the auxiliary lane also has its role to play in the freeway cross-section and the interchange. In a sense, the application of the auxiliary lane in the freeway environment is identical to its application elsewhere. It is added to address a local operational issue and, as soon as the need for the auxiliary lane is past, it is dropped. Important features to consider in the application and design of the auxiliary lane are thus:

- The need for an auxiliary lane
- The terminals
- Driver information

5.7.1 The Need for an Auxiliary Lane

Auxiliary lanes are normally required on free-ways either:

- as climbing lanes; or
- to support weaving; or
- to support lane balance.

The climbing lane application is similar to that discussed in Chapter 3.2 in respect of two-lane two-way roads whereas the weaving and lane balance applications are unique to the freeway situation.

a. Climbing Lanes

Ideally, maximum gradients on freeways are in the range of three to four percent ensuring that most vehicles can maintain a high and fairly constant speed. However, in heavily rolling country it is not always possible to achieve this ideal without incurring excessive costs in terms of earthworks construction. Because of the heavy volumes of traffic that necessitate the provision of a freeway, lane changing to overtake a slow-moving vehicle is not always easy and, under peak flow conditions, may actually be impossible. Speed differentials in the traffic stream are thus not only extremely disruptive but may also be potentially dangerous. Both conditions, i.e. disruption and reduction in safety, require consideration.

If a gradient on a freeway is steeper than four percent, an operational analysis should be carried out to establish the impact of the gradient on the Level of Service. A drop through one level, e.g. from LOS B through LOS C to LOS D, would normally suggest a need for a climbing lane.

Accident rates increase exponentially with increasing speed differential. For this reason, international warrants for climbing lanes normally include a speed differential in the range of 15 to 20 km/h. A truck speed reduction of 20 km/h is taken to warrant for climbing lanes. If, on an existing freeway, the measured truck speed reduction in the outermost lane is thus 20 km/h or higher, the provision of a climbing lane should be considered. In the case of a new design, it will be necessary to construct a speed profile of the truck traffic to evaluate the need for a climbing lane.

b. Weaving

In the urban environment, interchanges are fairly closely spaced and local drivers are inclined to use freeways as part of the local circulation system. To ensure that the freeway is not unduly congested because of this practice, an auxiliary lane can be provided between adjacent interchanges resulting in Type A weaving as described in paragraph 5.4.

If a large number of vehicles are entering at the upstream interchange, it may be necessary to provide a two-lane entrance ramp. Some of these vehicles may exit at the following interchange but those wishing to travel farther will have to weave across traffic from still further upstream that intends exiting at the following interchange and then merge with through traffic on the freeway. The auxiliary lane is then extended beyond the downstream interchange to allow a separation between the two manoeuvres. Similarly, a large volume of exiting vehicles may necessitate a two-lane exit, in which case the auxiliary lane should be extended upstream. Type B weaving thus comes into being. The desired length of the extension of the auxiliary lane beyond the two interchanges is normally assessed in terms of the probability of merging vehicles locating an acceptable gap in the opposing traffic flow.

c. Lane balance

As discussed in paragraph 5.6, lane balance requires that:

- In the case of an exit, the number of lanes downstream of the diverge should be one more than the number upstream; and
- In the case of an entrance, the number of lanes downstream of the merge should be one less than the number upstream

This is illustrated in Figure 42.

Single-lane on and off ramps do not require auxiliary lanes to achieve lane balance in terms of the above definition. It should be noted that, unless two lane on and off ramps are provided, the Type A weave is actually a violation of the principles of lane balance.

To achieve lane balance at an exit, three lanes upstream of the diverge should be followed by a two-lane off ramp in combination with two basic lanes on the freeway. The continuity of basic lanes requires that the outermost of the three upstream lanes should be an auxiliary lane.

If all three upstream lanes are basic lanes, it is possible that traffic volumes beyond the off ramp may have reduced to the point where three basic lanes are no longer necessary. Provision of a two-lane exit would thus be a convenient device to achieve a lane drop while simultaneously maintaining lane balance. The alternative would be to provide a single lane off ramp, carrying the basic lanes through the interchange and dropping the outside lane some distance beyond the on ramp terminal. In view of the additional construction costs involved, this approach is not recommended. If a two lane on ramp is joining two basic lanes, lane balance will require that there be three lanes beyond the merge. The outside lane of the three could become a new basic lane if the increase in traffic on the freeway merits it. On the other hand, it is possible that the flow on the ramp is essentially a local point of high density and that two basic lanes are all that are required downstream of the on ramp. In this case, the outside lane could be dropped as soon as convenient.

5.7.2 Auxiliary Lane Terminals

An auxiliary lane is intended to match a particular situation such as, for example, an unacceptably high speed differential in the traffic stream. It follows that the full width of auxiliary lane must be provided over the entire distance in which the situation prevails. The terminals are thus required to be provided outside the area of need and not as part of the length of the auxiliary lane.

Entering and exiting from auxiliary lanes require a reverse curve path to be followed. It is thus suggested that the taper rates discussed in paragraph 5.6 be employed rather than those normally applied to on- and off-ramps. The entrance taper should thus be about 100 metres long and the exit taper about 200 metres long.

5.7.3 Driver Information

The informational needs of drivers relate specifically to needs with regard to the exit from the auxiliary lane and include an indication of:

- the presence of a lane drop;
- the location of the lane drop; and
- the appropriate action to be undertaken.

5.8 Interchange Types

5.8.1 General

There are several basic interchange forms or patterns of ramps for turning movements at a grade separation. Their application at a particular site is determined by the number of intersection legs, the expected volumes of through and turning movements and topography.

The interchange configurations frequently used as the basis for design are given below:

a. Directional

A directional interchange is the most effective design for connecting intersecting freeways. The directional pattern has the advantage of reduced travel distance, increased speed of operation, and higher capacity. These designs often eliminate weaving and have a further advantage over semi-directional designs in avoiding the loss in sense of direction drivers experience in travelling around a loop.

b. Diamond

A diamond interchange has four ramps which are essentially parallel to the major artery. Each ramp provides for one right and one left turn movement.

Because left turns are made at grade across conflicting traffic on the crossroad, maximum sight distance should be provided.

The diamond design is the most generally applicable and serviceable interchange configuration and usually requires less space than any other type. It should be the first to be considered when an interchange is to be provided unless another design is clearly dictated by traffic, topography, or special conditions.

c. Cloverleaf

The full cloverleaf interchange has four quadrants. Loop ramps are used to eliminate the four left turn conflicts. Outer ramps provide for the four right turns. Cloverleaf designs often incorporate a collector-distributor (C-D) road to minimize signing difficulties and to reduce weaving conflicts where speed change lanes of the inner loops overlap.

The principal advantage of this design is the complete elimination of left turn conflicts, and it is especially useful when turning movements are equal in all quadrants. Because all movements are merging movements, it is adaptable to any grade line arrangement. Its main disadvantages are the circuitous travel for left turn movements and the large areas required for loops.

d. Partial Cloverleaf

A partial cloverleaf has semi-directional loop ramps in one, two or three quadrants, which are used to eliminate the major left turn conflicts. These loops may also serve right turns for which facilities may be lacking in undeveloped quadrants. Outer ramps are provided for the remaining turns. Grade lines must be satisfactory to provide sight distance between vehicles approaching these ramps.

e. Trumpet

The trumpet type is suitable for T intersections. Only four turning movements are involved. Preferential alignment should be provided for the two major movements. A loop ramp is usually adequate for the minor left turn. If this minor loop movement is eliminated the separation structure can be used for one-way traffic and the interchange then becomes a simple and direct branch connection suitable for heavy traffic volumes.

In the case of freeway to freeway interchanges, reference is made to systems interchanges. Systems interchanges exclusively serve vehicles that are already on the freeway system.

Access to the freeway system from the surrounding area is via interchanges on roads other than freeways, for which reason these interchanges are known as access interchanges. Service areas, providing opportunities to buy fuel, or food or simply to relax for a while are typically accessed via an interchange. In some instances, the services are duplicated on either side of the freeway, where access is via a right-in/right-out configuration. The requirements in terms of deflection angle, length of ramp and spacing that apply to interchange ramps apply equally to right-in/right-out ramps. In effect, this situation could be described as being an interchange without a crossing road.

The primary difference between systems and access/service interchanges is that the ramps on systems interchanges have free-flowing terminals at both ends, whereas the intersecting road ramp terminals on an access interchange are typically in the form of at-grade intersections.

Interchanges can also be between non-freeway roads, for example between two heavily trafficked arterials. In very rare instances there may even be an application for an interchange between a major and a local road, as suggested above in the case where local topography may force a grade separation between the two roads.

In addition to the classification and nature of the intersecting road, there are a number of controls guiding the selection of the most appropriate interchange form for any particular situation. In the sense of context sensitive design, these include:

- Safety;
- Adjacent land use;
- Design speed of both the freeway and the intersecting road;
- Traffic volumes of the through and turning movements;
- Traffic composition;
- Number of required legs;
- Road reserve and spatial requirements;
- Topography;
- Service to adjacent communities;
- Environmental considerations, and
- Economics.

The relative importance of these controls may vary from interchange to interchange. For any particular site, each of the controls will have to be examined and its relative importance assessed. Only after this process will it be possible to study alternative interchange types and configurations to determine the most suitable in terms of the more important controls.

While the selection of the most appropriate type and configuration of interchange may vary between sites, it is important to provide consistent operating conditions in order to match driver expectations.

5.8.2 Systems Interchanges

Directional interchanges provide high-speed connections to right and to left provided that the ramp exits and entrances are on the right of the through lanes. Where turning volumes are low or space is limited, provision of loops for right turning traffic can be considered. Directional interchanges that include one or more loops are

referred to as being partially-directional. If all left turns are required to take place on loops, the cloverleaf configuration emerges. Various forms of systems interchanges are illustrated below.

a. Four-legged interchanges

The fully directional interchange illustrated in Figure 43(i) provides single exits from all four directions and directional ramps for all eight turning movements. The through roads and ramps are separated vertically on four levels.

Partial directional interchanges allow the number of levels to be reduced. The Single Loop Partially-directional Interchange, illustrated in Figure 43(ii), and the Two Loop arrangement, illustrated in Figure 43(iii) and (iv), require three levels. The difference between Figure 43(iii) and (iv) is that, in the former case the freeways cross and, in the latter route continuity dictates a change in alignment. Loop ramps are normally only used for lighter volumes of left-turning traffic. A three-loop arrangement is, in effect, a cloverleaf configuration, with one of the loops being replaced by a directional ramp and is not likely to occur in practice, largely because of the problem of weaving discussed below.

The principal benefit of the cloverleaf is that it requires only a simple one-level structure, in contrast to the complex and correspondingly costly structures necessary for the directional and partial directional configurations. The major weakness of the cloverleaf is that it requires weaving over very short distances. Provided weaving volumes are not high and sufficient space is available to accommodate the interchange, the cloverleaf can, however, be an option. If weaving is required to take place on the main carriageways, the turbulence so created has a serious effect on the flow of traffic through the interchange area. The cloverleaf also has the characteristic of confronting the driver with two exits from the freeway in quick succession. Both these problems can be resolved by providing collector-distributor roads adjacent to the through carriageways.

b. Three-legged interchanges

Various fully-directional and partially-directional three legged interchanges are illustrated in Figure 44. In Figure 44(i), one single structure providing a three-level separation is required. Figure 44(ii) also requires three levels of roadway but spread across two structures hence reducing the complexity of the structural design.

It is also possible with this layout to slightly reduce the height through which vehicles have to climb. Figure 44(iii) illustrates a fully-directional interchange that requires only two but widely separated structures. If North is assumed as being at the top of the page, vehicles turning from West to South have a slightly longer path imposed on them so that this should ideally be the lesser turning volume.

Figure 44(iv) and (v) show semi-directional interchanges. Their names stem from the loop ramp located within the directional ramp creating the appearance of the bell of a trumpet. The letters "A" and "B" refer to the loop being in Advance of the structure or beyond it. The smaller of the turning movements should ideally be on the loop ramp but the availability of space may not always make this possible.

5.8.3 Access and Service Interchanges

In the case of the systems interchange, all traffic enters the interchange area at freeway speeds. At access and service interchanges, vehicles entering from the crossing road may be doing so from a stopped condition, so that it is necessary to provide acceleration lanes to ensure that they enter the freeway at or near freeway speeds. Similarly, exiting vehicles should be provided with deceleration lanes to accommodate the possibility of a stop at the crossing road.

As previously discussed, there is distinct merit in the crossing road being taken over the freeway as opposed to under it. One of the advantages of the crossing road being over the freeway is that the positive and negative gradients respectively support the required deceleration and acceleration to and from the crossing road. The final decision on the location of the crossing road is, however, also dependent on other controls such as topography and cost.

Access interchanges normally provide for all turning movements. If, for any reason, it is deemed necessary to eliminate some of the turning movements, the return movement for any movement that is provided, should also be provided. Movements excluded from a particular interchange should, desirably, be provided at the next interchange upstream or downstream as, without this provision, the community served loses amenity.

There are only two basic interchange types that are appropriate to access and service interchanges. These are the Diamond and the Par-Clo interchanges. Each has a variety of possible configurations.

Trumpet interchanges used to be considered suitable in cases where access was provided to one side only, for example to a bypass of a town or village. In practice, however, once a bypass has been built it does not take long before development starts taking place on the other side of the bypass. The three-legged interchange then has to be converted into a four-legged interchange. Conversion to a Par-Clo can be achieved at relatively low cost. Other than in the case of the Par-Clo AB, one of the major movements is forced onto a loop ramp. The resulting configuration is thus not appropriate to the circumstances. In practice, the interchange should be planned as a Diamond in the first instance, even though the crossing road, at the time of construction, stops immediately beyond the interchange.

a. Diamond Interchanges

There are three basic forms of Diamond, being:

- The Simple Diamond;
- The Split Diamond, and the
- Single Point Interchange.

The Simple Diamond is easy for the driver to understand and is economical in its use of space. The major problem with this configuration is that the left turn on to the crossing road can cause queuing on the exit ramp. In extreme cases, these queues can extend back onto the freeway, creating a hazardous situation. Where the traffic

on the left turn is very heavy, it may be necessary to consider placing it on a loop ramp. This is the reverse of the situation on systems interchanges where it is the lesser volumes that are located on loop ramps. It has the advantage that the left turn is converted into a right-turn at the crossing road ramp terminal. By the provision of auxiliary lanes, this turn can operate continuously without being impeded by traffic signals.

The Simple Diamond can take one of two configurations: the Narrow Diamond and the Wide Diamond.

The Narrow Diamond is the form customarily applied. In this configuration, the crossing road ramp terminals are very close in plan to the freeway shoulders to the extent that, where space is heavily constricted; retaining walls are located just outside the freeway shoulder breakpoints. Apart from the problem of the left turn referred to above, it can also suffer from a lack of intersection sight distance at the crossing road ramp terminals. This problem arises when the crossing road is taken over the freeway and is on a minimum value crest curve on the structure. In addition, the bridge balustrades can also inhibit sight distance. In the case where the crossing road ramp terminal is signalised, this is less of a problem, although a vehicle accidentally or by intent running the red signal could create a dangerous situation.

The Wide Diamond has the problem of imposing a long travel distance on left-turning vehicles but is not without its advantages. The crossing road ramp terminals are located at the start of the approach fill to the structure. To achieve this condition, the ramps have to be fairly long so that queues backing up onto the freeway are less likely than on the Narrow Diamond. The crossing road ramp terminals are also at ground level, which is a safer alternative than having the intersections on a high fill. Finally, because the ramp terminals are remote from the structure, intersection sight distance is usually not a problem.

The Split Diamond can also take one of two forms: the conventional Split and the transposed Split. This configuration is normally used when the crossing road takes the form of a one-way pair. The problems of sight distance and queues backing up are not normally experienced on Split Diamonds and the most significant drawback is that left-turning vehicles have to traverse three intersections before being clear of the interchange. It is also necessary to construct frontage roads linking the two one-way streets to provide a clear route for left-turning vehicles.

The transposed Split has the ramps between the two structures. This results in a very short distance between the entrance and succeeding exit ramps, with significant problems of weaving on the freeway. Scissor ramps are the extreme example of the transposed Split. These require either signalisation of the crossing of the two ramps or a grade separation. The transposed Split has little to recommend it and is not popular, being discussed here for completeness.

The Single Point Interchange brings the four ramps together at a point over the freeway. This interchange is required where space is at a premium or where the volume of left-turning traffic is very high. The principal operating difference between the Single Point and the Simple Diamond is that, in the former case, the left turns take place outside each other and in the latter they are "hooking" movements. The capacity of the Single Point Interchange is thus higher than that of the Simple Diamond. It does, however, require a three-phase signal plan and also presents pedestrians with wide unprotected crossings.

The various configurations of the Diamond Interchange are illustrated in Figure 45.

b. Par-Clo interchanges

Par-Clo interchanges derive their name as a contraction of PARTial CLOverleaf, mainly because of their appearance, but also because they were frequently a first stage development of a Cloverleaf Interchange. In practice, they could perhaps be considered rather as a distorted form of Simple Diamond Interchange.

Three configurations of Par-Clo Interchange are possible: the Par-Clo A, the Par-Clo B and the Par-Clo AB. As in the case of the Trumpet Interchange, the letters have the significance of the loops being in advance of or beyond the structure. The Par-Clo AB configuration has the loop in advance of the structure for the one direction of travel and beyond the structure for the other. In all cases, the loops are on opposite sides of the freeway. Both the Par-Clo A and the Par-Clo B have alternative configurations: the A2 and A4 and the B2 and B4.

These configurations refer to two quadrants only being occupied or alternatively to all four quadrants having ramps.

The various layouts are illustrated in Figure 43, Figure 44 and Figure 45.

Internationally, the Par-Clo A4 is generally regarded as being the preferred option for an interchange between a freeway and a heavily trafficked arterial. In the first instance, the loops serve vehicles entering the freeway whereas, in the case of the Par-Clo B, the high-speed vehicles exiting the freeway are confronted by the loop. This tends to surprise many drivers and loops carrying exiting traffic have higher accident rates than the alternative layout. Secondly, the right turn from the crossing road is remote from the intersections on the crossing road and the only conflict is between left-turning vehicles exiting from the freeway and through traffic on the crossing road. This makes two-phase signal control possible.

The Par-Clo AB is particularly useful in the situation where there are property or environmental restrictions in two adjacent quadrants on the same side of the crossing road.

The Rotary Interchange illustrated in Figure 48 has the benefit of eliminating intersections on the crossing road, replacing them by short weaving sections. Traffic exiting from the freeway may experience difficulty in adjusting speed and merging with traffic on the rotary.

Rotaries have also been used in the United Kingdom as systems interchanges. In this configuration, a two-level structure is employed.

One freeway is located at ground level and the other freeway on the upper level of the structure with the rotary sandwiched between them.

5.8.4 Interchanges on non-freeway Roads

The application of interchanges where a road other than a freeway is a major route would arise where traffic flows are so heavy that a signalised intersection cannot provide sufficient capacity. In this case, the crossing road terminals would be provided on the road with the lower traffic volume.

An intersection with a particularly poor accident history may also warrant upgrading to an interchange. The accident history would provide some indication of the required type of interchange.

Where the need for the interchange derives purely from topographic restraints, i.e. where traffic volumes are low, a Jug Handle Interchange, illustrated in Figure 49,

would be adequate. This layout, also known as a Quarter Link, provides a two-lane-two-way connection between the intersecting roads located in whatever quadrant entails the minimum construction and property acquisition cost.

Drivers would not expect to find an interchange on a two-lane two-way road and, in terms of driver expectancy, it may therefore be advisable to introduce a short section of dual carriageway at the site of the interchange.

5.9 Ramp Design

5.9.1 General

A ramp is defined as a roadway, usually one-way, connecting two grade-separated through roads. It comprises an entrance terminal, a midsection and an exit terminal.

The general configuration of a ramp is determined prior to the interchange type being selected. The specifics of its configuration, being the horizontal and vertical alignment and cross-section, are influenced by a number of considerations such as traffic volume and composition, the geometric and operational characteristics of the roads which it connects, the local topography, traffic control devices and driver expectations.

A variety of ramp configurations can be used. These include:

- The outer connector, which serves the right turn and has free-flowing terminals at either end;
- The diamond ramp, serving both the right and left turns with a free-flowing terminal on the freeway and a stop-condition terminal on the cross-road;
- The Par-Clo ramp, which serves the left turn and has a free-flowing terminal on the freeway and a stop-condition terminal on the crossing road, with a 180° loop between them;
- The loop ramp, serving the left turn and which has free-flowing terminals at both ends and a 270 degree loop between them;
- The directional ramp also serving the left turn, with a curve only slightly in excess of 90 degree and free-flowing terminals at either end, and

- The collector-distributor road intended to remove the weaving manoeuvre from the freeway.

The express-collector system discussed later is a transfer roadway and is not an interchange ramp.

5.9.2 Design speed

Guideline values for ramp design speeds are given in Table 35. Strictly speaking, the design speed of a ramp could vary along its length from that of the freeway to that of the at-grade intersection, with the design speed at any point along the ramp matching the operating speed of the vehicles accelerating to or decelerating from the design speed of the freeway. The design speeds given in the table apply to the controlling curve on the mid-section of the ramp. The ramp design speed is shown as a design domain because of the wide variety of site conditions, terminal types and ramp shapes.

Table 35: Ramp Design Speed

Roadway Design Speed (km/h)	Ramp Design Speed Domain (km/h)
60	40 – 50
70	40 – 60
80	40 – 70
90	50 – 80
100	50 – 90

In the case of a directional ramp between freeways, vehicles must be able to operate safely at the higher end of the ranges of speeds shown in Table 35.

Factors demanding a reduction in design speed include site limitations, ramp configurations and economic factors. Provided the reduction is not excessive, drivers are prepared to reduce speed in negotiating a ramp so that the lower design speed is not in conflict with driver expectations.

For directional ramps and outer connectors, higher values in the speed domain are appropriate and, in general, ramp designs should be based on the upper limit of the domain. The constraints of the site, traffic mix and form of interchange, however, may force a lower design speed.

A Par-Clo or a loop ramp cannot be designed to a high design speed. A ramp design speed of 70 km/h, being the low end of the domain for a freeway design speed of 130 km/h, would require a radius of between 150 metres and 200 metres, depending on the rate of super elevation selected. It is not likely, particularly in an urban area, that the space required to accommodate a loop with this radius would be available. Even if the space were available, the additional travel distance imposed by the greater radius would nullify the advantages of the higher travel speed. The added length of roadway also adds the penalty of higher construction cost. This penalty also applies to the ramp outside the loop because it has to be longer to contain the larger loop. In general, loop ramps are designed for speeds of between 40 and 50 km/h. Because of the substantial difference between the freeway design speed and that of the loop ramp, it is advisable not to have a loop on an exit ramp if this can be avoided.

Safety problems on ramp curves are most likely for vehicles travelling faster than the design speed. This problem is more critical on curves with lower design speeds because drivers are more likely to exceed these design speeds than the higher ranges. Trucks can overturn when travelling at speeds only marginally higher than the design speed. To minimise the possibility of trucks exceeding the design speed, it is suggested that the lower limits of design speeds shown in Table 35 should not be used for ramps carrying a substantial amount of truck traffic.

If site specific constraints preclude the use of design speeds that more-or-less match anticipated operating speeds, the designer should seek to incorporate effective speed controls, such as advisory speed signing, special pavement treatments, long deceleration lanes and the use of express-collector systems in the design.

5.9.3 Sight Distance on Ramps

It is necessary for the driver to be able to see the road markings defining the start of the taper on exit ramps and the end of the entrance taper. At the crossing road ramp terminal, lanes are often specifically allocated to the turning movements with these lanes being developed in advance of the terminal. The driver has to position the vehicle in the lane appropriate to the desired turn. It is therefore desirable that decision sight distance be provided on the approaches to the ramp as well as across its length.

Appropriate values of decision sight distance are given in Chapter 3.1.

5.9.4 Horizontal Alignment

Minimum radii of horizontal curvature on ramps should comply with that given in Chapter 3.1 for various values of e_{\max} . In general, the higher values of e_{\max} are used in freeway design and the selected value should also be applied to the ramps.

Achieving the step down of radii from higher to lower design speeds on a loop may require the application of compound curves. In general, the ratio between successive radii should be 1.5:1 and, as a further refinement, they could be connected by transition curves. The length of each arc is selected to allow for deceleration to the speed appropriate to the next radius at the entry to that arc. In the case of stepping up through successive radii, the same ratio applies but the design speed for the radius selected should match the desired speed at the terminal end of each arc.

It is recommended that the designer develops a speed profile for the loop and bases the selection of radii and arc lengths on this speed profile. As a rough rule of thumb, the length of each arc should be approximately a third of its radius.

If a crossover crown line, discussed below, is not used, the cross fall on the exit or entrance taper between the Yellow Line Break Point and the nose is controlled by that on the through lanes. Super elevation development can thus only commence at the nose. The distance required to achieve the appropriate super elevation thus determines the earliest possible location of the first curve on the ramp.

Ramps are relatively short and the radii of curves on ramps often approach the minimum for the selected design speed. Furthermore, if there is more than one curve on a ramp, the distance between the successive curves will be short. Under these restrictive conditions, transition curves should be considered.

Ramps are seldom, if ever, cambered and super elevation typically involves rotation around one of the lane edges. Drivers tend to position their vehicles relative to the inside edge of any curve being traversed, i.e. they steer towards the inside of the curve rather than away from the outside. For aesthetic reasons, the inside edge should thus present a smoothly flowing three-dimensional alignment with the outside edge rising and falling to provide the super elevation. Where a ramp has an S- or reverse curve alignment, it follows that first one edge and then the other will be the centre of rotation, with the changeover taking place at the point of zero cross fall.

In view of the restricted distances within which super elevation has to be developed, the crossover crown line is a useful device towards rapid development. A crossover crown is a line at which an instantaneous change of cross fall takes place and which runs diagonally across the lane. The crossover crown could, for example, be located along the yellow line defining the edge of the right lane of the freeway, thus enabling initiation of super elevation for the first curve on the ramp earlier than would otherwise be the case. The crossover crown should, however, be used with caution as it may pose a problem to the driver, particularly to the driver of a vehicle with a high load. This is because the vehicle will sway as it traverses the crossover crown and, in extreme cases, may prove difficult to control. The algebraic difference in slope across the crossover crown should thus not exceed four to five percent.

5.9.5 Vertical Alignment

a. Gradients

The profile of a ramp typically comprises a midsection with an appreciable gradient coupled with terminals where the gradient is controlled by the adjacent road. If the crossing road is over the freeway, the positive gradient on the off-ramps will assist a rapid but comfortable deceleration and the negative gradient on the on-ramp will support acceleration to freeway speeds. In theory, therefore, the greater the gradient, the better. Values of gradient up to eight percent can be

considered but, for preference, gradients should not exceed six percent. Diamond ramps are usually fairly short, possibly having as little as 120 to 360 metres between the nose and the crossing road. The effect of the midsection gradient, while possibly helpful, is thus restricted. However, a steep gradient (8 percent) in conjunction with a high value of super elevation (10 percent) would have a resultant of 12.8 percent at an angle of 53 degrees to the centre line of the ramp. This would not contribute to drivers' sense of safety. In addition, the drivers of slow-moving trucks would have to steer outwards to a marked extent to maintain their path within the limits of the ramp width. This could create some difficulty for them. It is suggested that designers seek a combination of super elevation and gradient such that the gradient of the resultant is less than ten percent. Table 36 provides an indication of the gradients of the resultants of combinations of super elevation and longitudinal gradient.

Table 36: Maximum Resultant Gradients

Super Elevation	Longitudinal Gradient (%)			
	2	4	6	8
4	4.47	5.65	7.21	8.94
6	6.31	7.20	8.49	10.00
8	8.25	8.94	10.00	11.31
10	10.19	10.73	11.63	12.80

The combinations of gradient and super elevation shown shaded in Table 36 should be avoided.

b. Curves

As suggested above, decision sight distance should be available at critical points on ramps. On sag curves, achieving decision sight distance is not a problem so that the K-values for sag vertical curve given in Chapter 3.1 can be applied. If the interchange area is illuminated, as would typically be the case in an urban area, the K-values for comfort which are roughly half of those dictated by stopping sight distance can be used. Unilluminated, i.e. rural, interchanges would require the application of K-values appropriate to headlight sight distance.

5.9.6 Cross-section

Horizontal radii on ramps are typically short and hence are usually in need of curve widening. The width of the ramps normally adopted is thus 4 metres. Because of the inconvenience of changing the lane width of comparatively short sections, this width is applied across the entire length of the ramp.

In the case of a loop ramp, the radius could be as low as fifty metres. At this radius, a semi-trailer would require a lane width of 5.07 metres. It is necessary for the designer to consider the type of vehicle selected for design purposes and to check whether the four metre nominal width is adequate. If semi-trailers are infrequent users of the ramp, encroachment on the shoulders could be considered.

Lane widening is described in detail in Chapter 3.2. As suggested above, regardless of the width required, it should be applied across the entire length of the ramp.

Ramp shoulders have a width of 2 metres for the inner shoulder and 2.5 metres for the outer shoulder. In conjunction with the nominal lane width of 4 metres, the total roadway width is a minimum of 8.7 metres. This width would allow comfortably for a truck to pass a broken-down truck. In addition, it would provide drivers with some sense of security in the cases where the ramp is on a high fill.

5.9.7 Terminals

The crossing road ramp terminals may be free-flowing, in which case their design is as discussed below. Crossing road ramp terminals that are at-grade intersections should be designed according to the recommendations contained in Chapter 3.4. It is worth noting that, from an operational point of view, what appears to be a four-

legged intersection, e.g. the crossing road ramp terminals on a Diamond Interchange, is in fact, two three-legged intersections back-to-back. Ideally, the crossing road ramp terminal should be channelized to reduce the possibility of wrong-way driving.

Vehicles entering or exiting from a freeway should be able to do so at approximately the operating speed of the freeway. Given the fact that the crossing road terminal is invariably a signalised or stop control at-grade intersection, the change in speed across the ramp is substantial. Provision should thus be made for acceleration and deceleration to take place clear of the freeway so as to minimise interference with the through traffic and reduce the potential for crashes. The auxiliary lanes provided to accommodate this are referred to as speed-change lanes or acceleration or deceleration lanes. These terms describe the area adjacent to the travelled way of the freeway, including that portion of the ramp taper where a merging vehicle is still clear of the through lane, and do not imply a definite lane of uniform width.

The speed change lane should have sufficient length to allow the necessary adjustment in speed to be made in a comfortable manner and, in the case of an acceleration lane, there should also be sufficient length for the driver to find and manoeuvre into a gap in the through traffic stream before reaching the end of the acceleration lane. The length of the speed change lane is based on:

- The design speed on the through lane, i.e. the speed at which vehicles enter or exit from the through lanes;
- The control speed of the ramp midsection, i.e. the design speed of the smallest radius curve on the ramp, and
- The tempo of acceleration or deceleration applied on the speed change lane.

Research has shown that deceleration rates applied to off-ramps are a function of the freeway design speed and the ramp control speed. As both speeds increase, so does the deceleration rate, which varies between 1.0 m/s^2 and 2.0 m/s^2 . For convenience, the deceleration rate used to develop Table 37 has been set at 2.0 m/s^2 .

Deceleration should only commence once the exiting vehicle is clear of the through lane. Assuming that a vehicle with a width of 2.5 metres is correctly positioned relative to the ramp edge line to be centrally located within an ultimate ramp width of 4 metres, its tail end will clear the edge of the through lane at a distance, L , from the Yellow Line Break Point where

$$L = 3.2 / T$$

$$T = \text{Taper rate (as listed in Table 9.7)}$$

The maximum length of design vehicles is 18 metres and this should be added to the distance, L , to establish the distance from the Yellow Line Break Point, at which the deceleration can commence so that

$$L_T = 3.2/T + 21 \quad \text{Equation 5-1}$$

Values of L_T , are listed in Table 37. From this table it follows that, in the case of a diamond ramp without curves, the distance from the Yellow Line Break Point to the crossing road ramp terminal should be not less than 436 metres.

The acceleration rate can, according to American literature, be taken as 0.7 m/s^2 . The length of the acceleration lane is thus as shown in Table 38. An important feature of the acceleration lane is the gap acceptance length, which should be a minimum of 100 metres to 150 metres, depending on the nose width. The length of the entering vehicle is not relevant in determination of the taper length.

The lengths of the deceleration and acceleration lanes shown in Table 37 and Table 38 apply to gradients of between - 3 percent and + 3 percent. Acceleration lanes will have to be longer on upgrades and may be made shorter on downgrades, with the reverse applying to deceleration lanes.

The actual entrance or exit points between the freeway and the ramp can take the form of a taper or a parallel lane. The parallel lane has the problem of forcing a reverse curve path, possibly followed after the nose by a further curve to the left, whereas the taper involves only a single change of direction. As such, drivers prefer the taper. In cases where an extended acceleration or deceleration distance is required, for example where the crossing road passes under the freeway resulting in the on-ramp being on an upgrade and the off-ramp on a downgrade, a straight taper would result in an inordinately long ramp. The parallel lane allows the speed change

to take place in an auxiliary lane immediately adjacent to the through lanes thus reducing the spatial demands of the interchange. Furthermore, in the case of the on-ramp, the parallel configuration provides an extended distance to find a gap in the adjacent traffic stream. Typical configurations of on- and off-ramp terminals are illustrated in Figure 46 to Figure 49.

a. Taper

Two different criteria apply to the selection of the taper rate, depending on whether the ramp is an exit from or entrance to the freeway.

Table 37: Length of Deceleration Lanes (m)

Freeway design speed (km/h)	Ramp control speed (km/h)								
	L _T (m)	0	40	50	60	70	80	90	100
60	66	70	60	21					
70	75	95	85	45	25				
80	75	125	115	75	55	30			
90	85	155	150	110	85	60	30		
100	91	190	185	145	125	100	70	35	

Table 38: Length of Acceleration Lanes (m)

Freeway design speed (km/h)	Ramp control speed (km/h)								
	LT (m)	0	40	50	60	70	80	90	100
60	45	200	110	60					
70	54	270	180	130	70				
80	54	350	265	215	155	80			
90	64	450	360	310	250	175	95		
100	70	550	460	415	350	280	200	105	

In the first case, the vehicle is merely required to achieve a change of direction from the freeway to the ramp. The taper rate should thus be sufficiently flat to ensure that the vehicle path can be accommodated within the lane width. In Table 39, the radii of curvature corresponding to a super elevation of 2.0 percent for the various design speeds are listed as are the taper rates corresponding to the condition of the wheel path having an offset of 150 millimetres inside the Yellow Line Break Point.

This rate derives from the application of an "operating speed" previously assumed to be 85 percent of the design speed and the further assumption that the wheel path could be allowed to pass over the Yellow Line Break Point. In practice, a 1: 15 rate requires that drivers accept a higher level of side friction in negotiating the change of direction. Passenger cars can negotiate tapers of this magnitude with ease so that, in the case of existing interchanges, it is not always necessary to incur the expenditure of upgrading to a flatter taper. Trucks do, however, sometimes roll over at the start of the ramp taper.

Table 39: Taper Rates for Exit Ramps

Design Speed (km/h)	Radius (m) for 2% super elevation	Taper rate 1:	Taper length, L_T (m)
60	1000	14	66
70	1500	17	75
80	1500	17	75
90	2000	20	85
100	2500	22	91
112	3000	25	101
120	3500	27	107
130	4000	28	111

If there is a high percentage of truck traffic and if the incidence of roll-overs is unacceptably high, it may be necessary to consider upgrading to the tapers suggested in Table 39.

In the case of the entrance ramp, the driver of the merging vehicle should be afforded sufficient opportunity to locate a gap in the through traffic and position the vehicle correctly to merge into this gap at the speed of the through traffic. Taper rates flatter than those proposed in Table 39 should thus be applied to on-ramps. A taper of 1:50 provides a travel time of about 13 seconds between the nose and the point at which the wheel paths of merging and through vehicles would intersect. This has been found in practice to provide sufficient opportunity to prepare to merge into gaps in the through flow because drivers typically begin to locate usable gaps in the freeway flow prior to passing the nose.

b. Parallel

The parallel lane configuration is also initiated by means of a taper but, in this case, the taper rate does not have to be as flat as suggested above. As in the case of the climbing lane, a taper length of 100 metres would be adequate.

Parallel entrances and exits have a major advantage over straight tapers in respect of the selection of curves on the ramps. In the case of a design speed of 120 km/h, a straight taper provides a distance just short of 300 metres between the Yellow Line Break Point and the nose. A portion of this distance should ideally be traversed at the design speed of the freeway with deceleration commencing only after the vehicle is clear of the outside freeway lane. About 220 metres is available for deceleration and this suggests that a vehicle exiting at 120 km/h is likely still to be travelling at a speed of about 95 km/h when passing the nose. The first curve on the ramp could, allowing for super elevation development to 10 percent, be located 100 metres beyond the nose so that, at the start of the curve, the vehicle speed would be in the range of 80 to 90 km/h. The first curve should thus have a radius of not less than 250 metres. In the case of the parallel ramp, the radius could obviously be significantly shorter, depending on the length of the deceleration lane preceding the nose.

5.10 Collector–Distributor Roads

Collector-distributor roads are typically applied to the situation where weaving manoeuvres would be disruptive if allowed to occur on the freeway. Their most

common application, therefore, is at Cloverleaf interchanges. The exit and entrance tapers are identical to those applied to any other ramps. The major difference between Cloverleaf interchange C-D roads and other ramps is that they involve two exits and two entrances in quick succession. The two exits are, firstly, from the freeway and, secondly, the split between vehicles turning to the right and those intending to turn to the left. The two entrances are, firstly, the merge between the two turning movements towards the freeway and, secondly, the merge with the freeway through traffic.

The distance between the successive exits should be based on signing requirements so as to afford drivers adequate time to establish whether they have to turn to the left or to the right to reach their destination. Nine seconds is generally considered adequate for this purpose and seeing that vehicles may be travelling at the design speed of the freeway as they pass the nose of the first exit, the distance between the noses should desirably be based on this speed.

The distance between successive entrances is based on the length required for the acceleration lane length quoted in Table 38.

5.11 Ramp Metering

Ramp metering consists of traffic signals installed on entrance ramps in advance of the entrance terminal to control the number of vehicles entering the freeway. The traffic signals may be pre-timed or traffic-actuated to release the entering vehicles individually or in platoons. It is applied to restrict the number of vehicles that are allowed to enter a freeway in order to ensure an acceptable level of service on the freeway or to ensure that the capacity of the freeway is not exceeded. The need for ramp metering may arise owing to factors such as:

- Recurring congestion because traffic demand exceeds the provision of road infrastructure in an area;
- Sporadic congestion on isolated sections of a freeway because of short term traffic loads from special events, normally of a recreational nature;

- As part of an incident management system to assist in situations where an accident downstream of the entrance ramp causes a temporary drop in the capacity of the freeway; and
- Optimising traffic flow on freeways

Ramp metering also supports local transportation management objectives such as:

- Priority treatments with higher levels of service for High Occupancy Vehicles; and
- Redistribution of access demand to other on-ramps.

It is important to realise that ramp metering should be considered a last resort rather than as a first option in securing an adequate level of service on the freeway. Prior to its implementation, all alternate means of improving the capacity of the freeway or its operating characteristics or reducing the traffic demand on the freeway should be explored. The application of ramp metering should be preceded by an engineering analysis of the physical and traffic conditions on the freeway facilities likely to be affected. These facilities include the ramps, the ramp terminals and the local streets likely to be affected by metering as well as the freeway section involved.

The stop line should be placed sufficiently in advance of the point at which ramp traffic will enter the freeway to allow vehicles to accelerate to approximately the operating speed of the freeway, as would normally be required for the design of ramps. It will also be necessary to ensure that the ramp has sufficient storage to accommodate the vehicles queuing upstream of the traffic signal.

The above requirement will almost certainly lead to a need for reconstruction of any ramp that is to be metered. The length of on-ramps is typically determined by the distance required to enable a vehicle to accelerate to freeway speeds. Without reconstruction, this could result in the ramp metering actually being installed at the crossing-road ramp terminal.

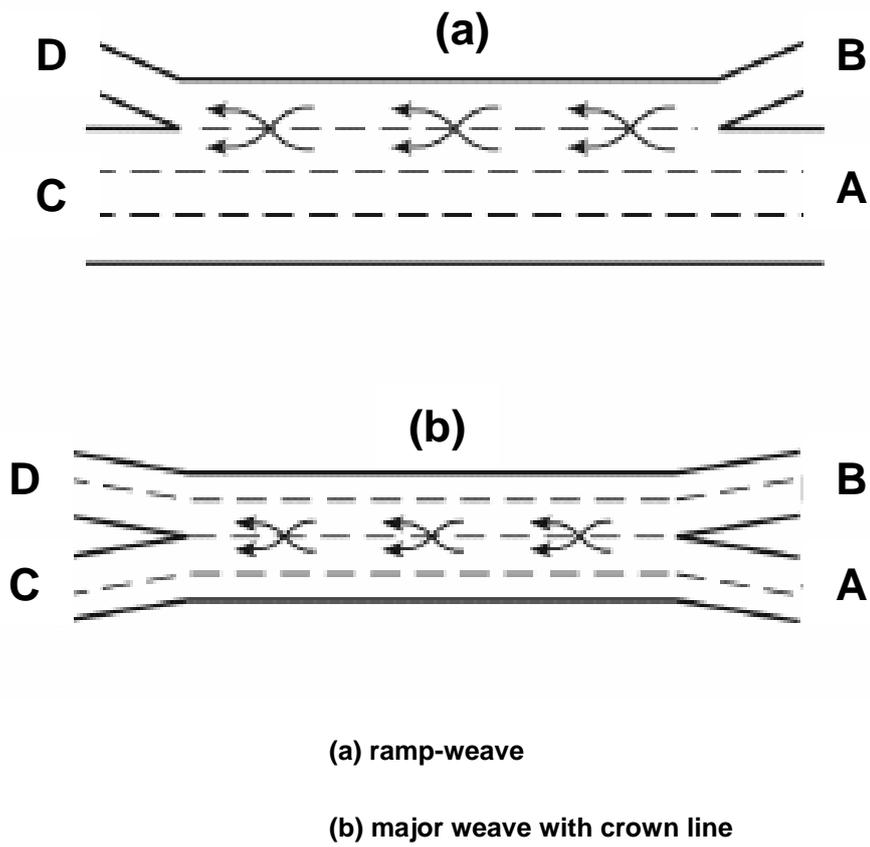
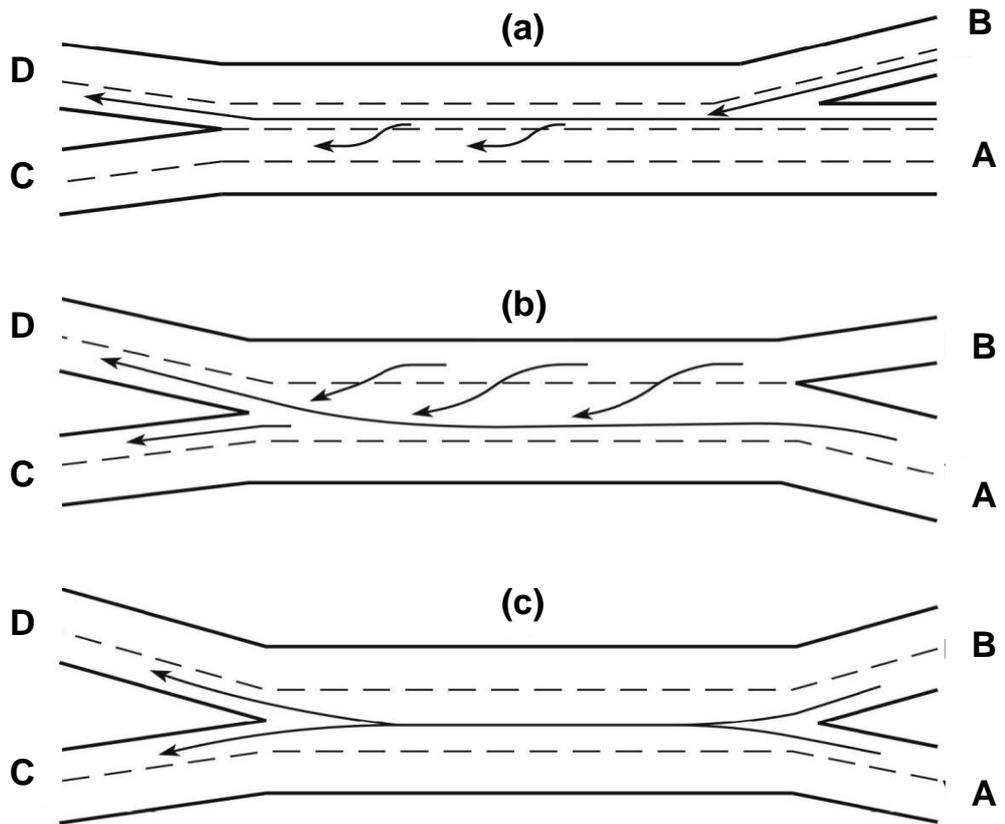


Figure 37: Type A Weaves



- (a) major weave with lane balance at exit
- (b) major weave with merging at entrance
- (c) major weave with merging at entrance and lane balance at exit

Figure 38: Type B Weaves

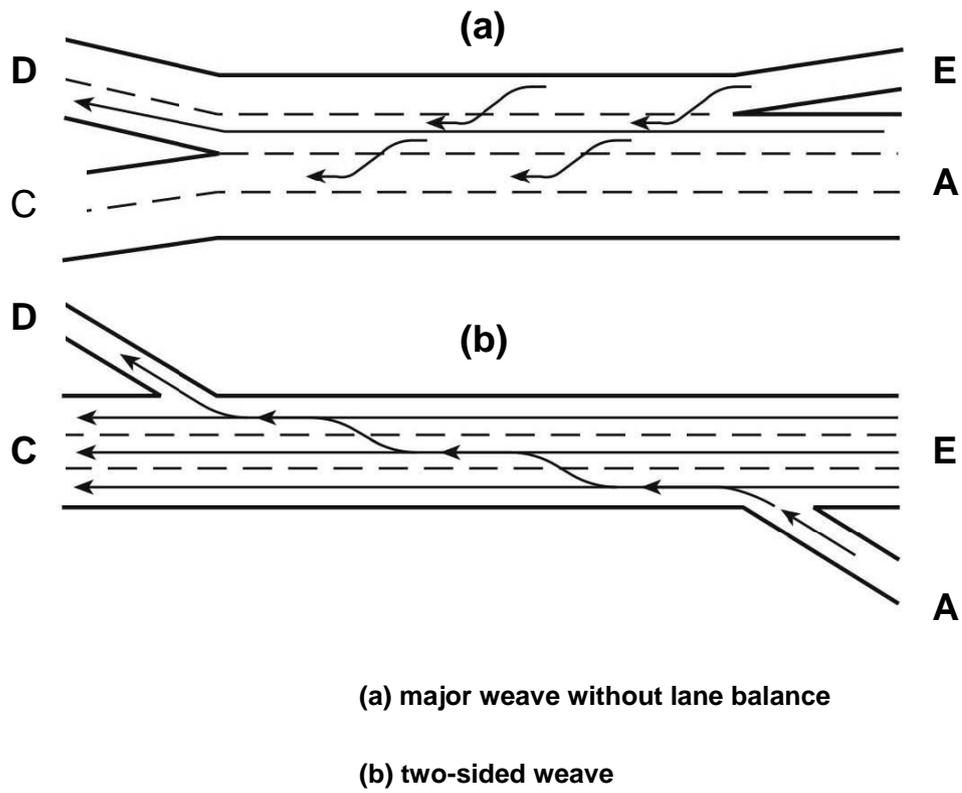


Figure 39: Type C Weaves

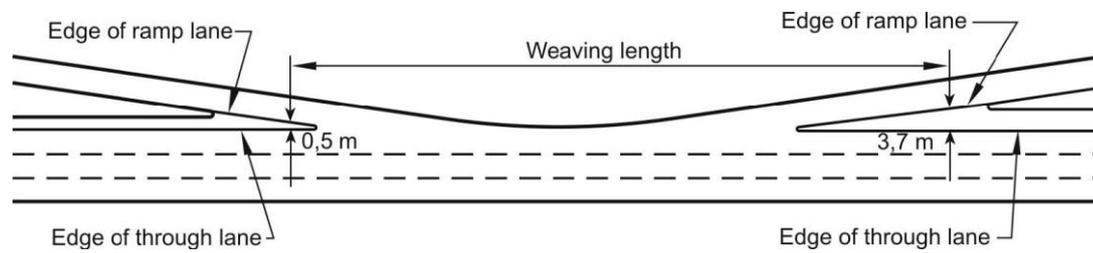


Figure 40: Weaving Distance



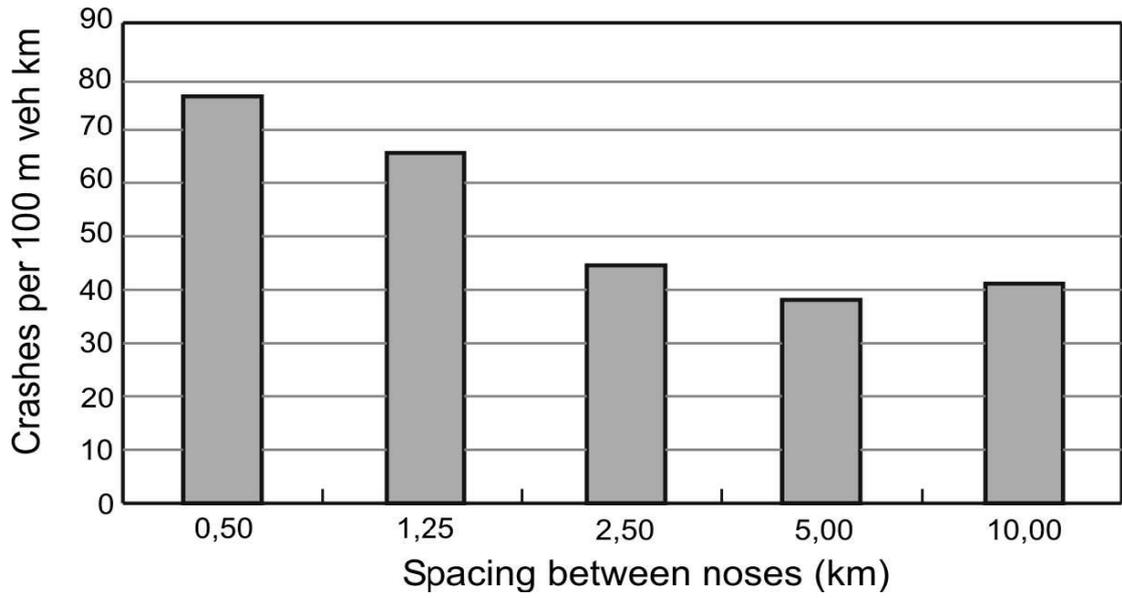
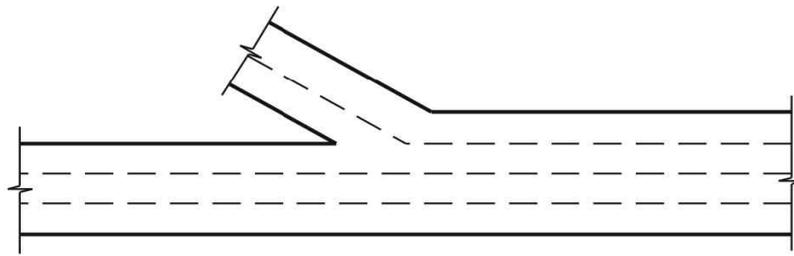
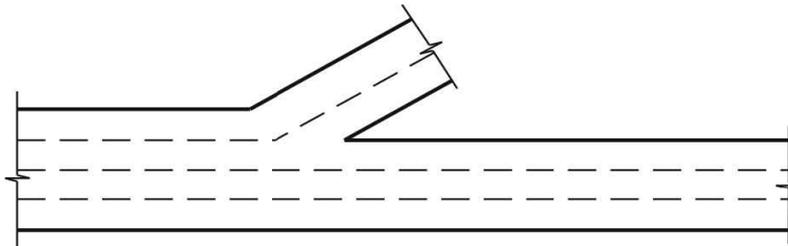


Figure 41: Relationship between Interchange Spacing and Accident Rate



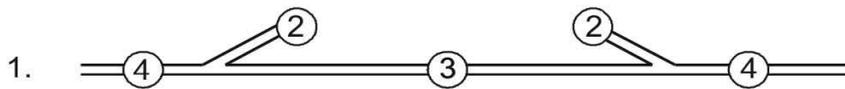
Through lanes upstream + ramp lanes - 1
= through lanes downstream

Merge

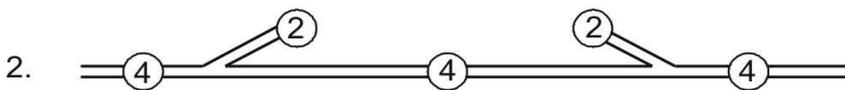


Through lanes upstream
= through lanes downstream + ramp lanes - 1

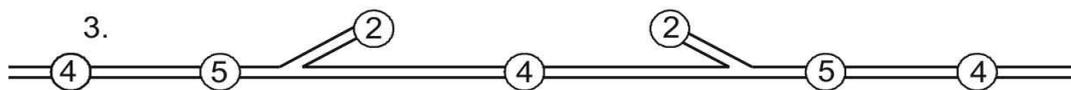
Diverge



Lane balance, but no continuity of basic number of lanes

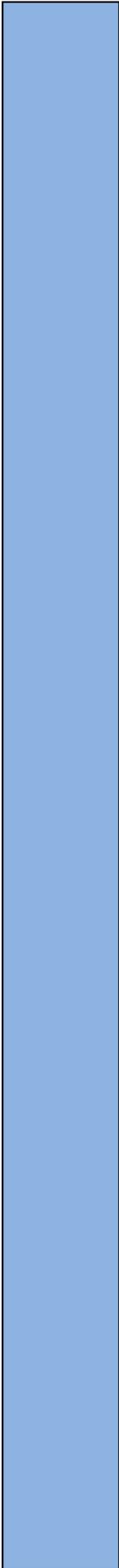


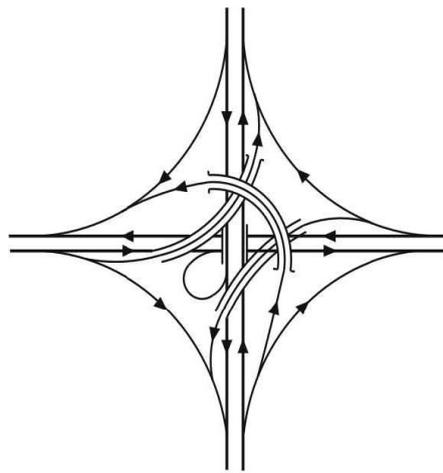
Basic number of lanes but no compliance with lane balance



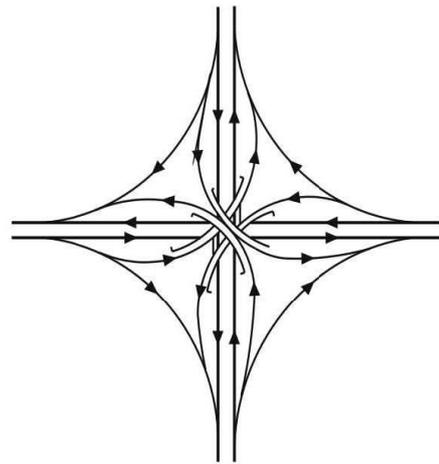
Continuity of basic number of lanes and lane balance

Figure 42: Coordination of lane balance with basic number of lanes

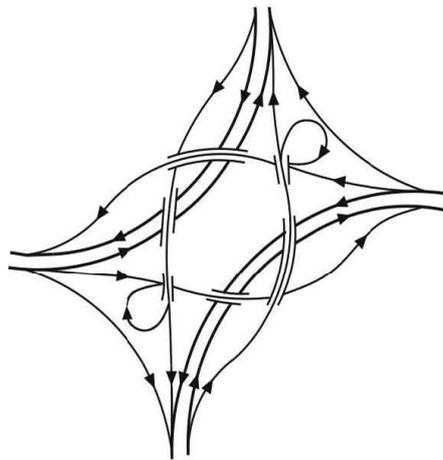




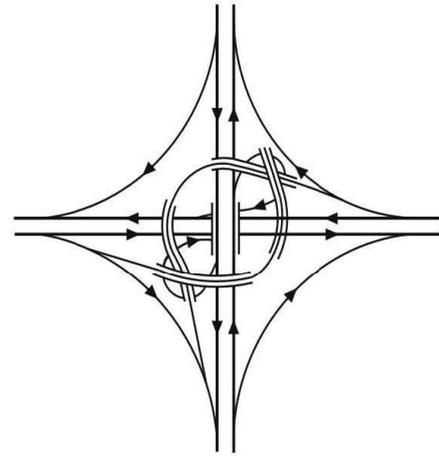
(ii) Partially-directional



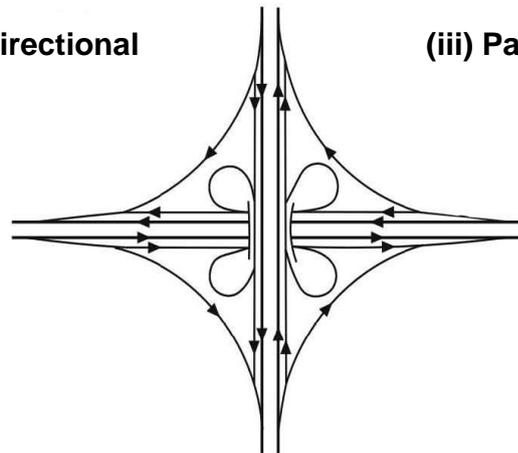
(i) Fully directional



(iv) Partially-directional



(iii) Partially-directional



(v) Full-cloverleaf

Figure 43: Four Legged System Interchanges

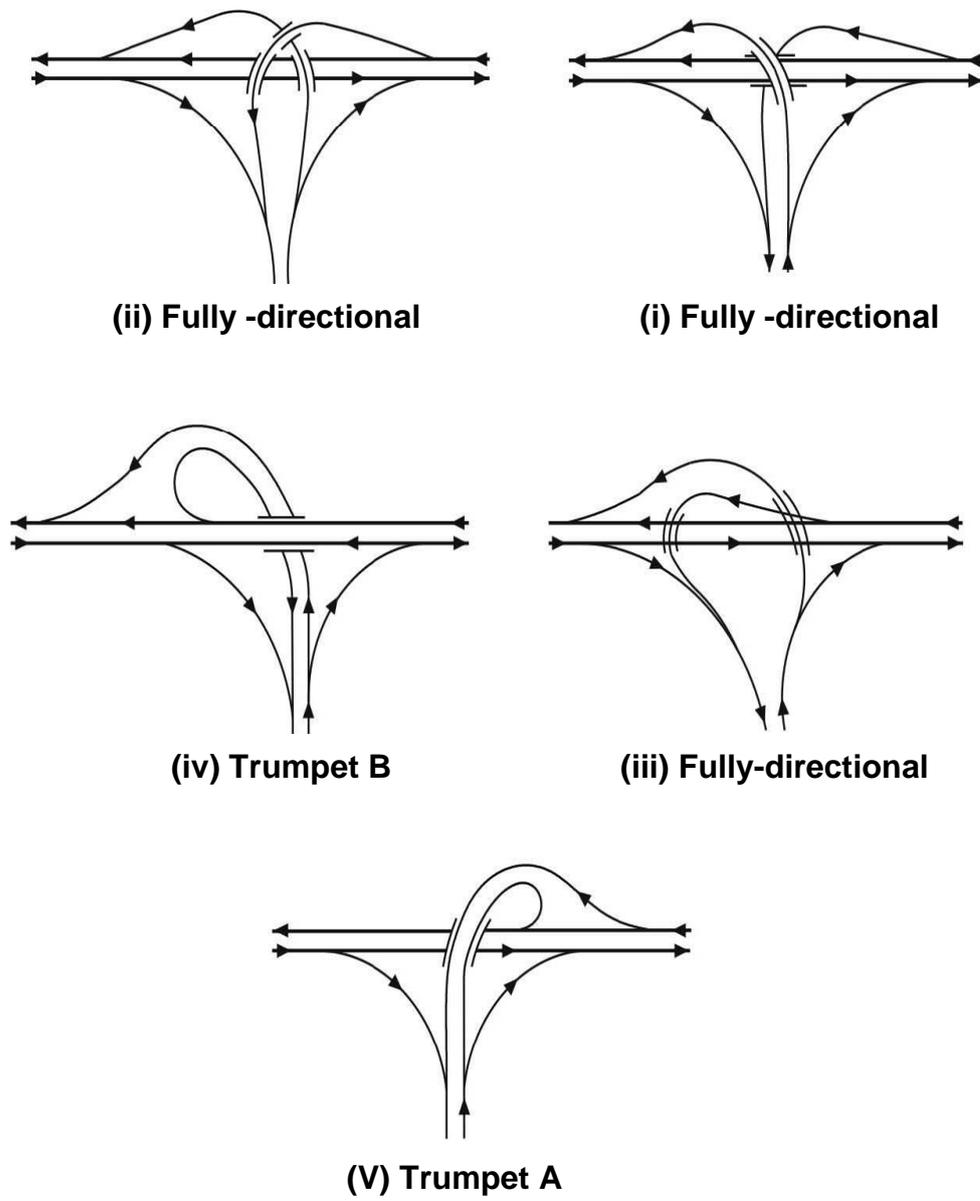
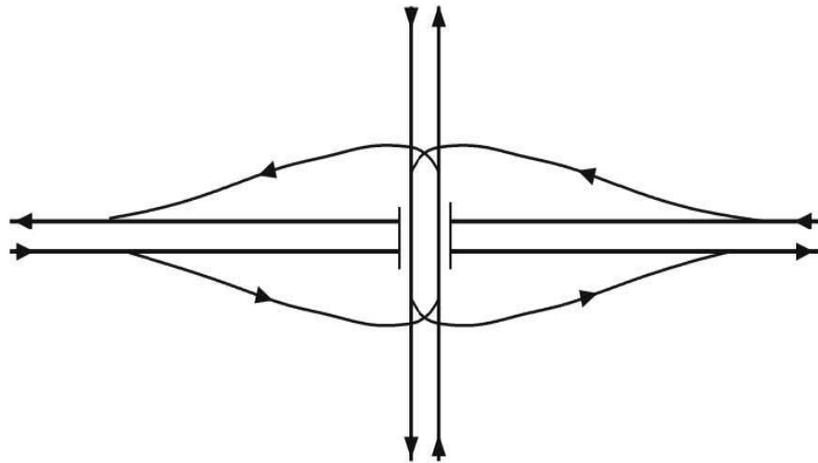
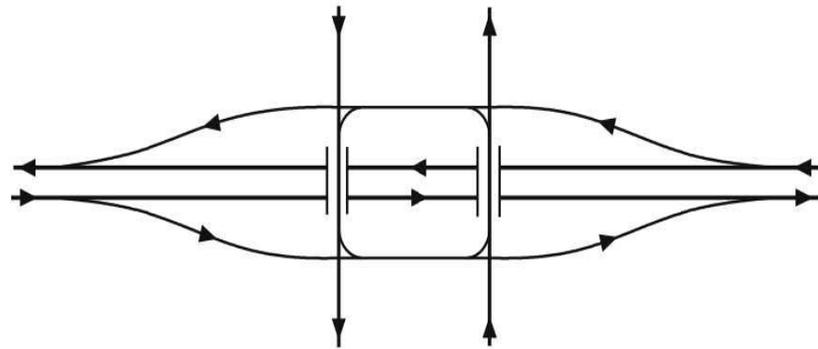


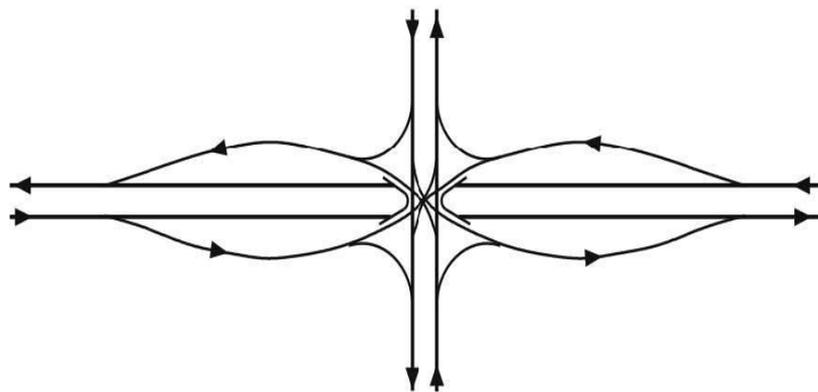
Figure 44: Three Legged System Interchanges



Simple diamond



Split diamond



Single point diamond

Figure 45: Diamond Interchanges

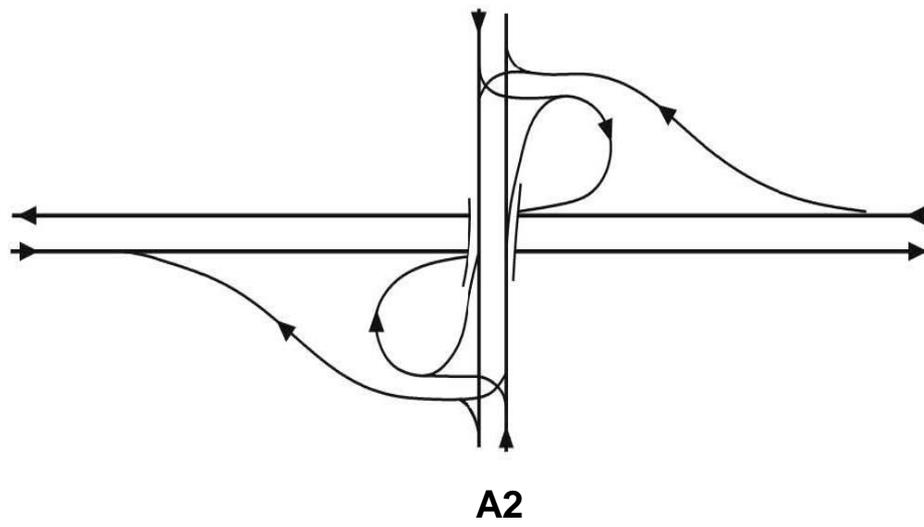
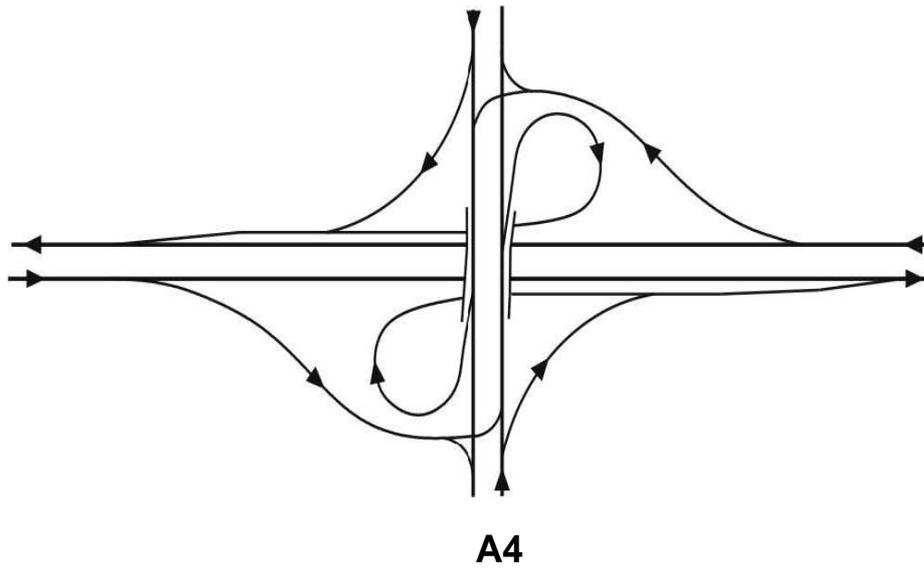
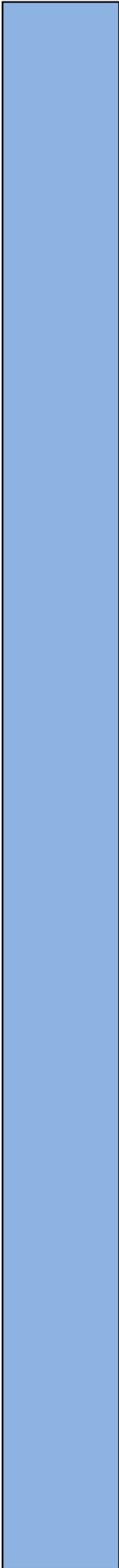
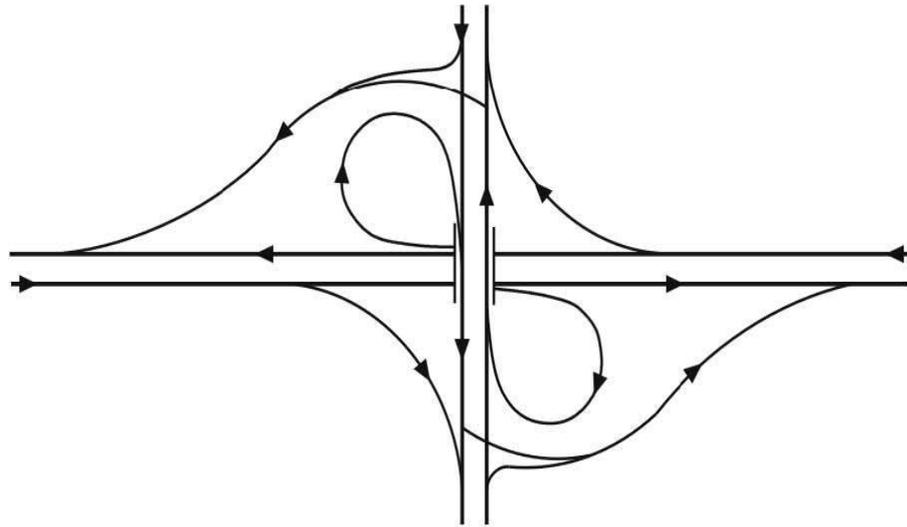
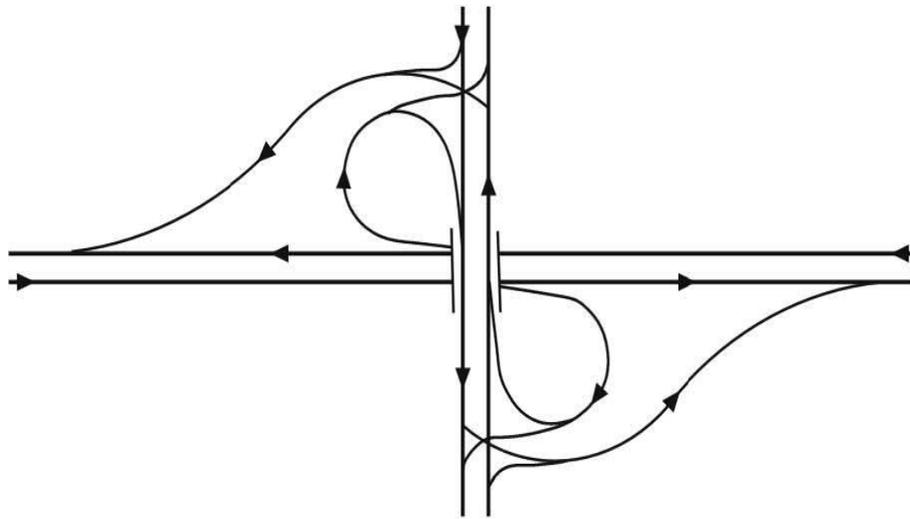


Figure 46: Par-Clo A Interchanges





B4



B2

Figure 47: Par-Clo B Interchanges

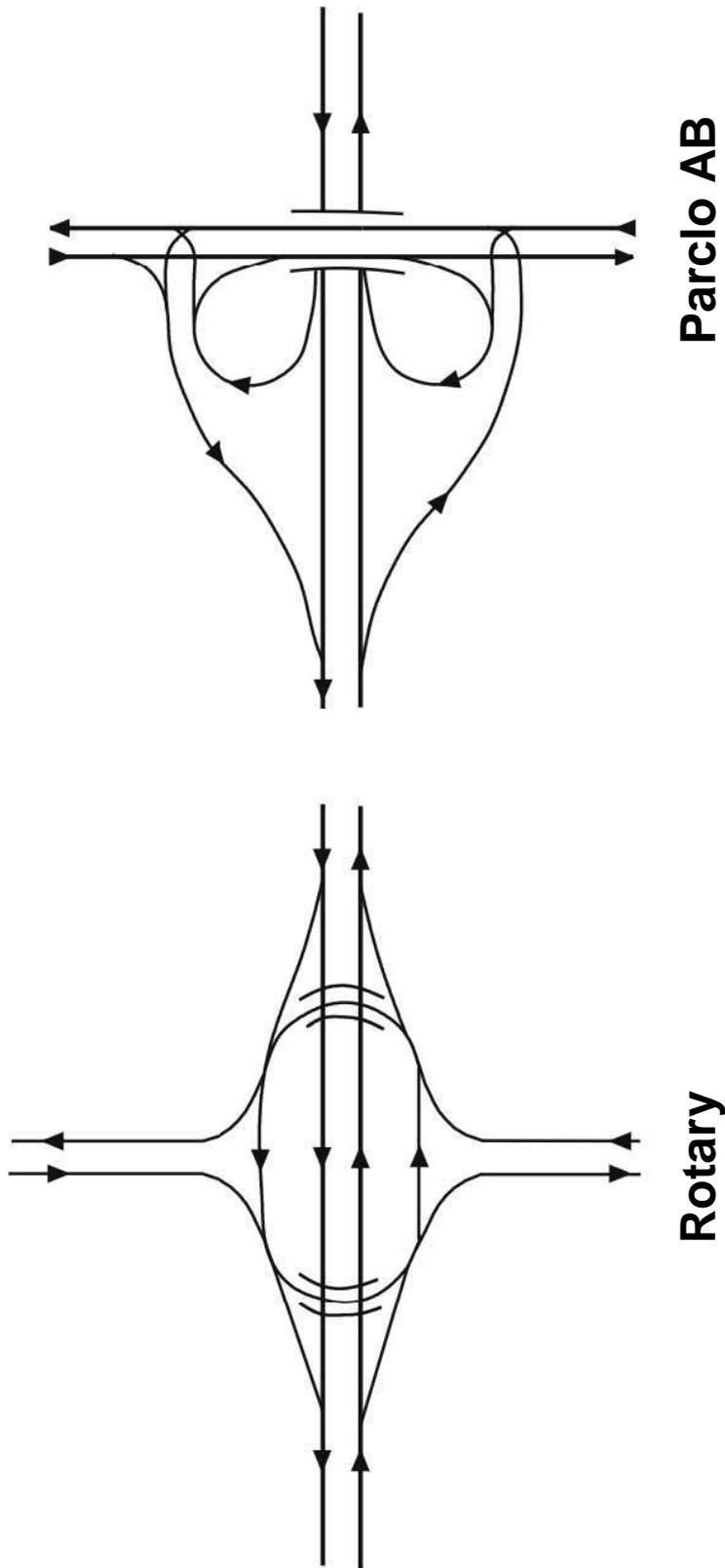


Figure 48: Par-Clo AB Interchanges and Rotaries



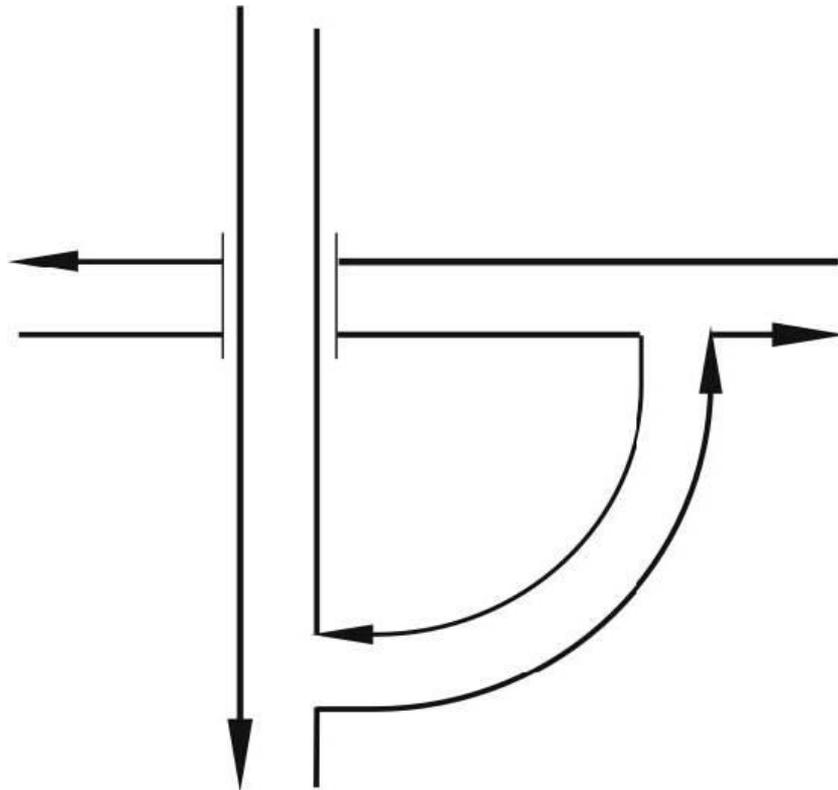


Figure 49: Jug Handle Interchange

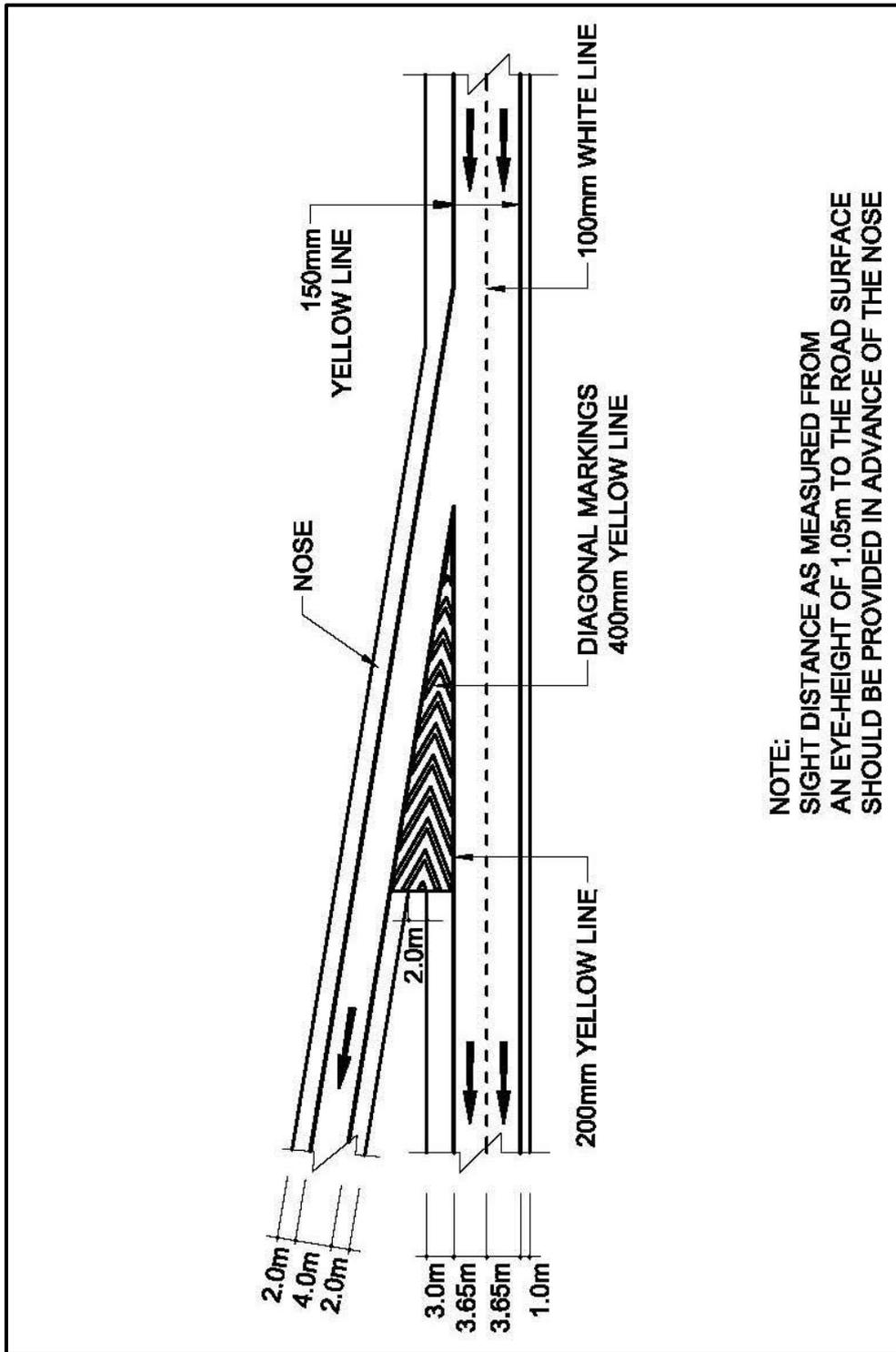


Figure 50: Single Lane Exit

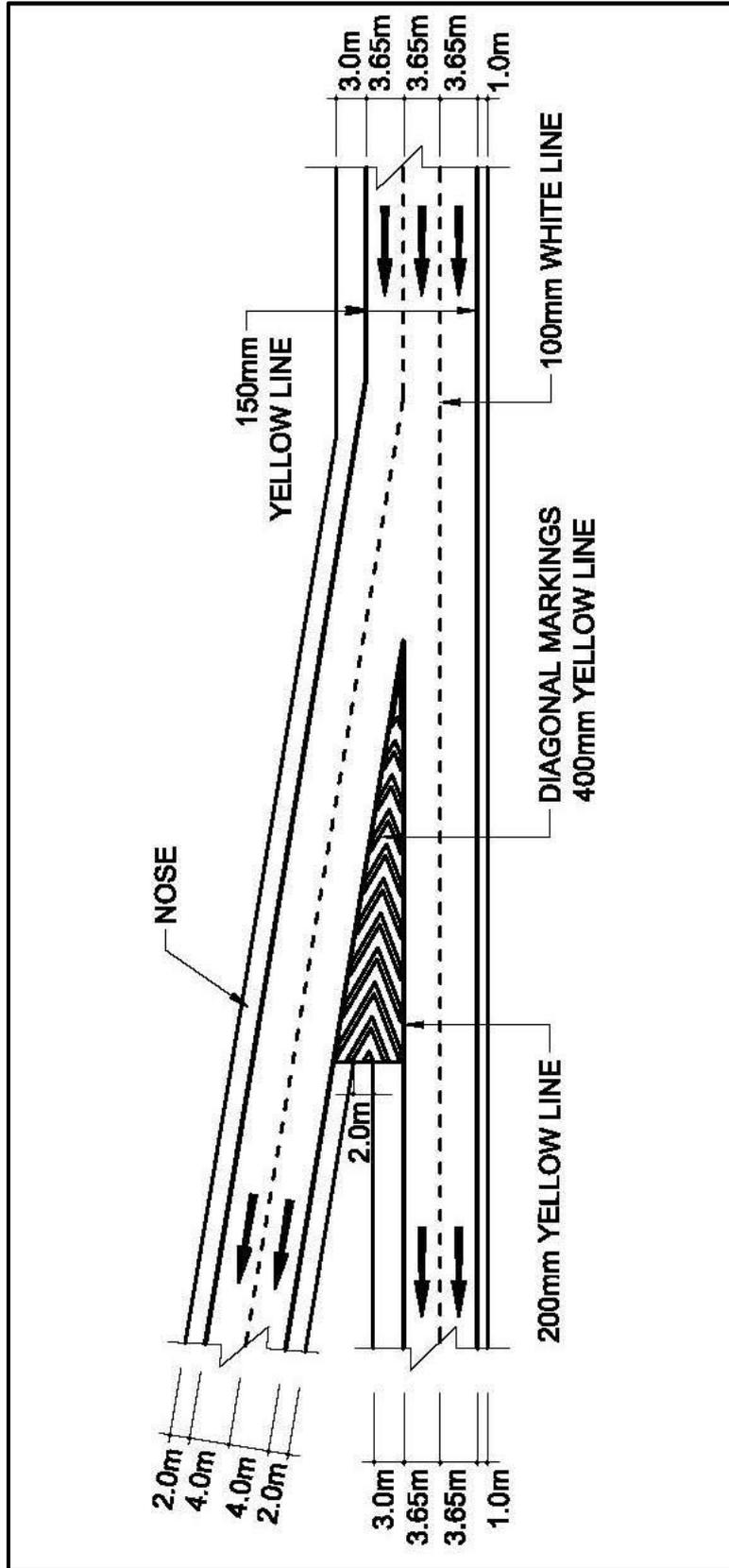


Figure 51: Two Lane Exit

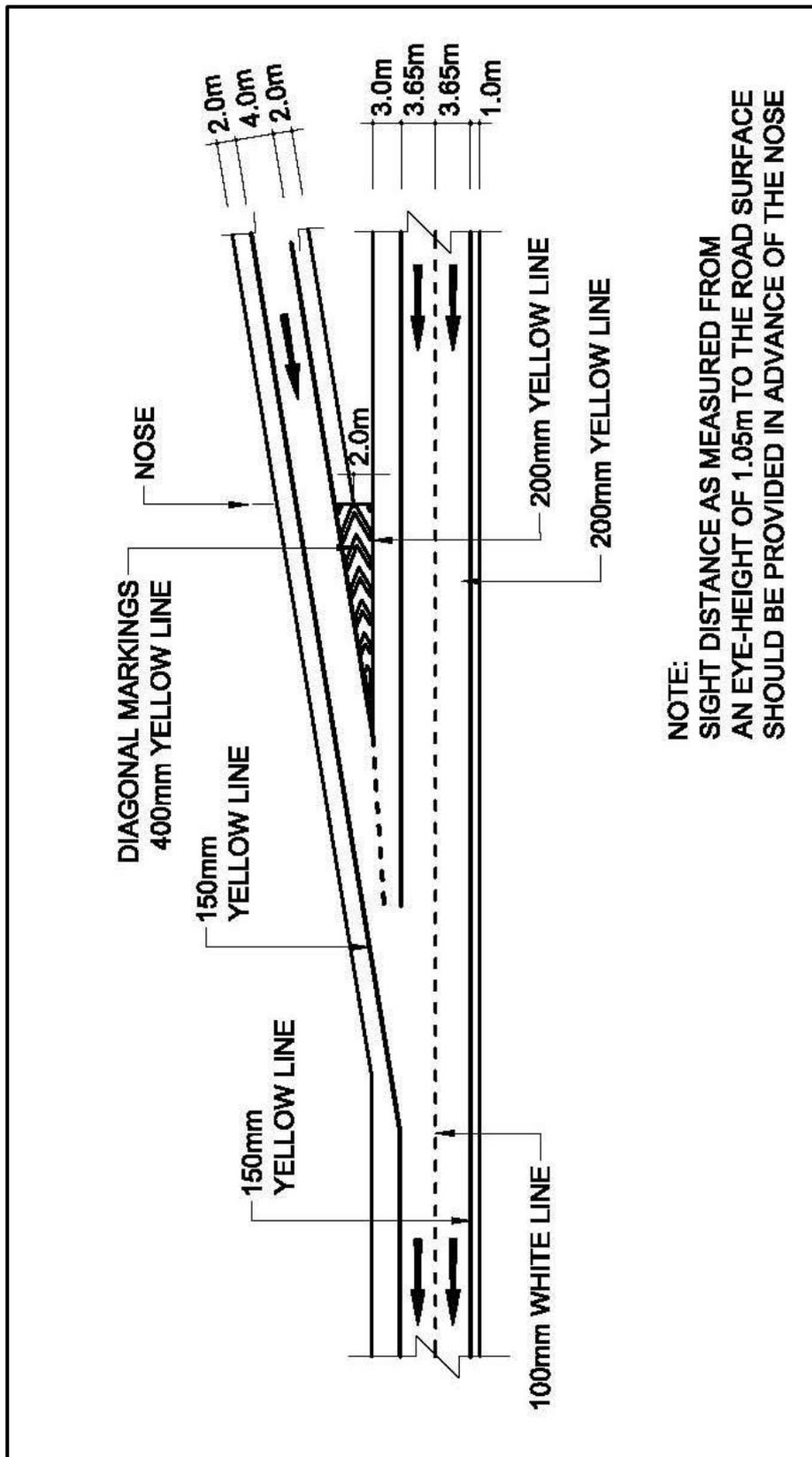
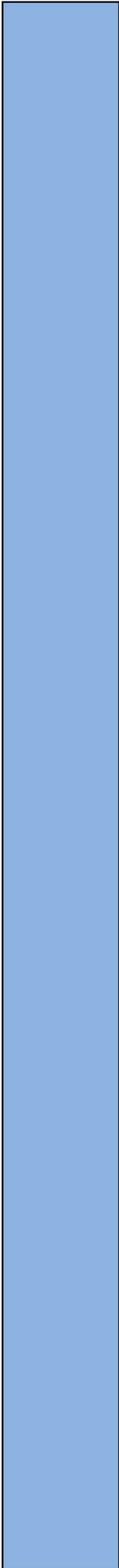


Figure 52: One Lane Entrance



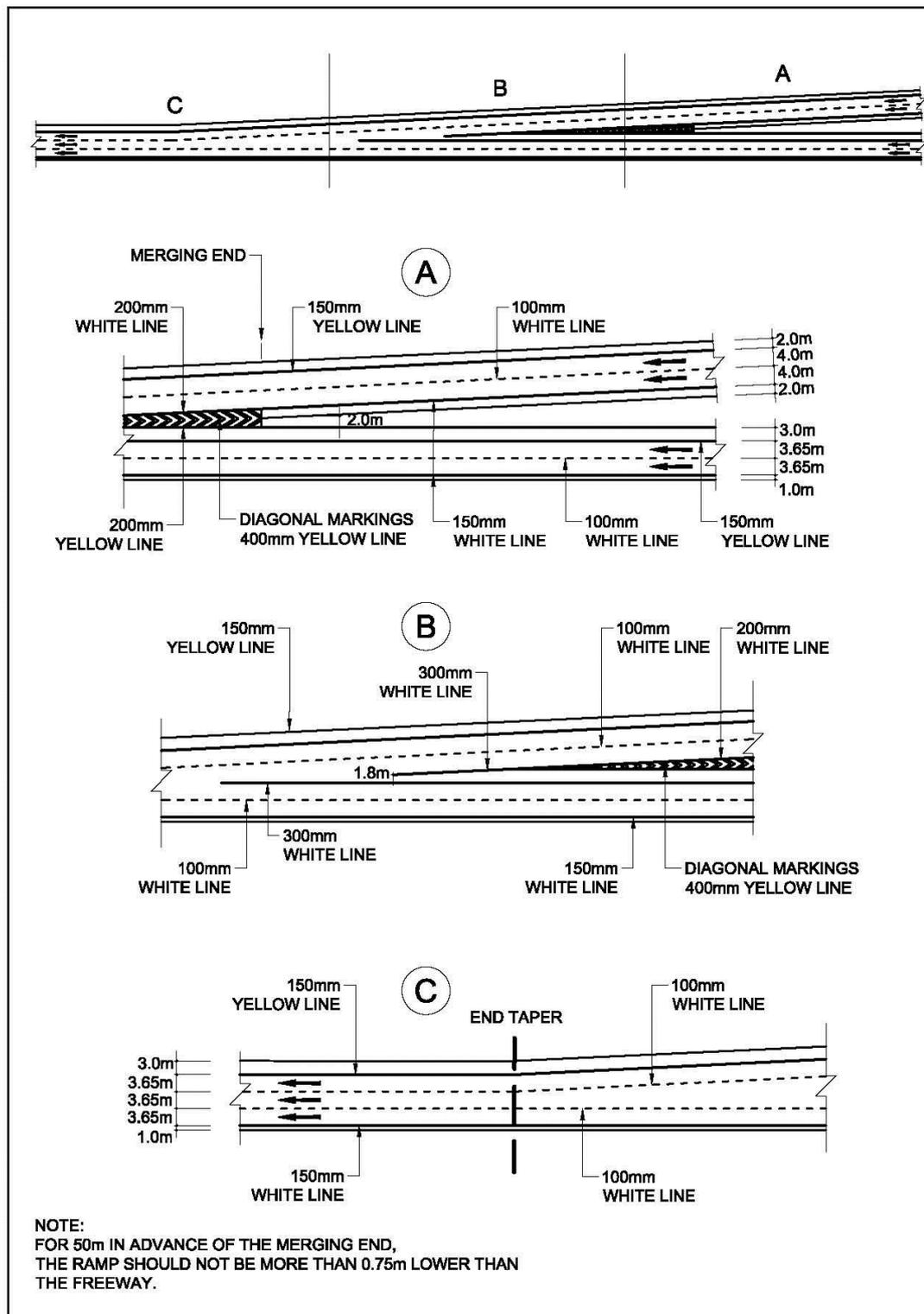


Figure 53: Two Lane Entrance

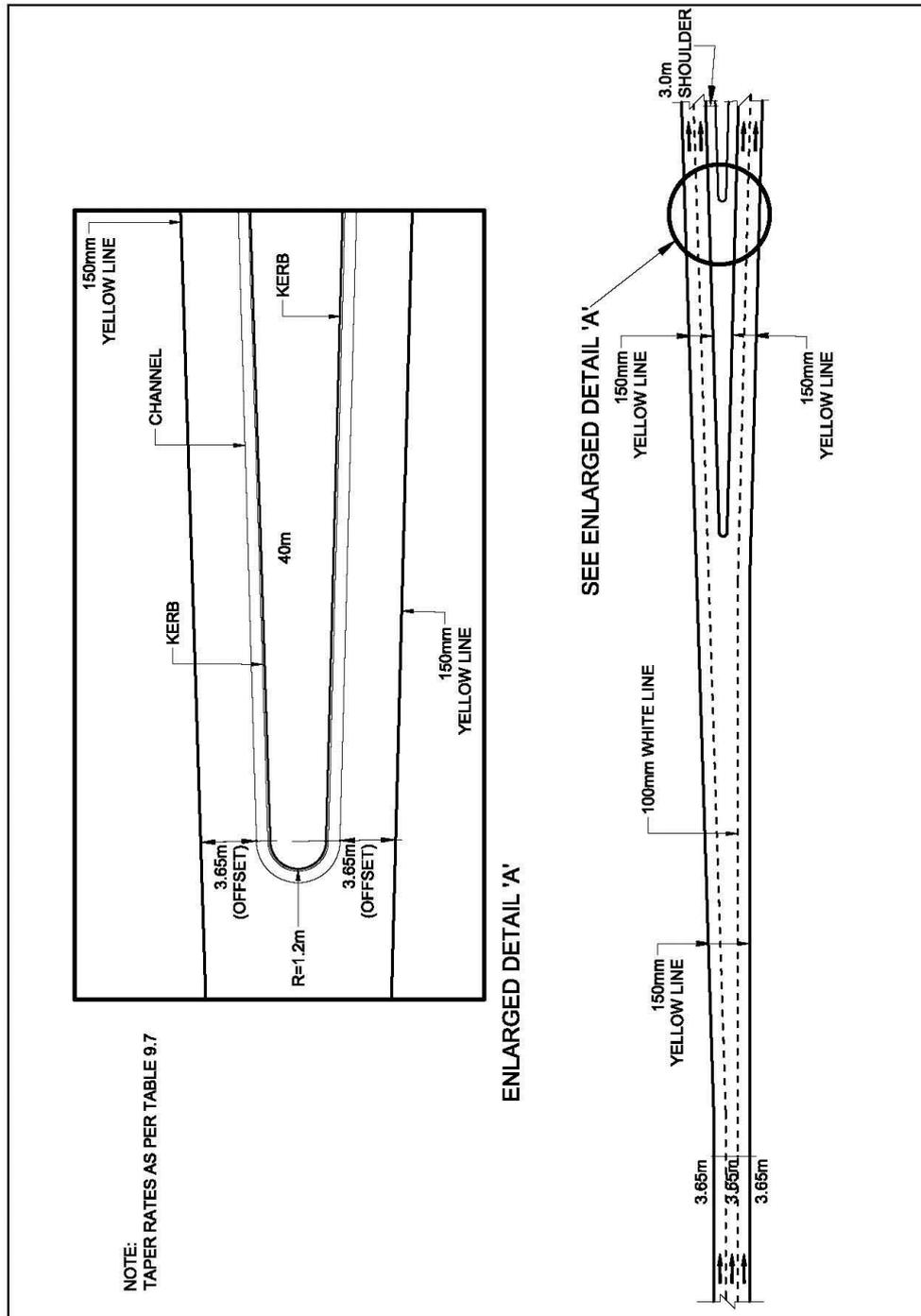


Figure 54: Major Fork



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