A Guide on Geometric Design of Roads

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Jabatan Kerja Raya

Cawangan Jalan

ATJ 8/86 (Pindaan 2015)

A GUIDE ON GEOMETRIC DESIGN OF ROADS

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KERAJAAN MALAYSIA

A GUIDE ON GEOMETRIC DESIGN OF ROADS

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FOREWORD

There has been tremendous progress in the road design methodology and process within JKR which are underlined in the numerous revised Technical Design Guides produced by JKR and REAM that updates the road design requirements in line with the current international standards and practices worldwide.

This Arahan Teknik (Jalan) ATJ 8/86 (Pindaan 2015), A Guide on Geometric Design of Roads, is the revision of the existing Arahan Teknik (Jalan) 8/86 which also includes input from REAM GL 2/2002: A Guide on Geometric Design of Roads, Malaysia Highway Capacity Manual MHCM 2011, Austroad: Guide to Traffic Engineering Practice: Part 13 – Pedestrians (1995), AASHTO - A Policy of Geometric Design of Highways and Streets, 2001 & 2011 and LLM/GP/T5-08 – Guideline For Toll Expressway System-Design Standard in preparing the guideline.

The preparation of this guideline was carried out through many discussions and deliberations by the committee members and also a working committee workshop specially held for this purpose. Feedbacks and comments received were carefully considered and incorporated into this guideline wherever appropriate. This guideline had also been presented and approved in the Mesyuarat Jawatankuasa Pemandu Pengurusan Bil. 1/2015 on (7 January 2015).

This guideline will be reviewed and updated from time to time to cater for and incorporate the latest development in road geometric design, as and when necessary. Any comments and feedback regarding this guideline should be forwarded to The Unit Standard, Bahagian Pembangunan Inovasi & Standard, Pakar Kejuruteraan Jalan & Jambatan, Cawangan Jalan, JKR.

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1.0 INTRODUCTION AND SUMMARY

1.1 Introduction

This Guideline is limited to the geometric aspect of road design as distinguished from that of structural design. It is intended as a manual on the geometric design of road, inclusive of both rural and urban conditions. The geometric design of road is only applicable to Rural or Urban areas as specifically indicated in this Guideline.

This Guideline is to be applied to all new construction and improvements of roads for vehicular traffic. Comments from users will be most welcomed as this Guideline will continue to be updated from time to time as and when deemed necessary.

This Guideline is to be used in conjunction with other Guidelines that have been or will be produced by the Jabatan Kerja Raya.

1.2 Roles And Responsibilities

Road Designer	:	To produce road design according to the Guideline and best engineering practice with adherence to road safety requirements and taking into account of social, environmental and traffic impact to the surrounding area, with due consideration of cost implication to the client.
Design Checker	:	To verify road design follows the Guideline and best engineering practice inclusive of road safety requirements, taking into account of the social, environmental and traffic impacts to the surrounding area.
Independent Checker	:	Third party to recheck and verify road design follows the Guideline and best engineering practice, inclusive of road safety requirements, without being biased towards any party.
Road Safety Auditor	:	To ascertain that the road design takes into consideration the road safety requirements and the incorporation of mitigation measures required due to circumstances which may compromise certain aspects of the design due to some constraints.

2.0 ROAD CLASSIFICATION AND DESIGN STANDARDS

2.1 Road Classification / Hierarchy

2.1.1 The Importance of Road Classification

The importance of defining a hierarchy of roads is that it can help clarify policies concerning the highway aspects of individual planning decisions on properties served by the road concerned. Specific planning criteria could be developed and applied according to a road's designation in the hierarchy; for example design speed, width of carriageway, control over pedestrian, intersections, frontage access etc. In this way the planning objectives would be clear for each level of road in the hierarchy and policies on development control and traffic management would reinforce one another.

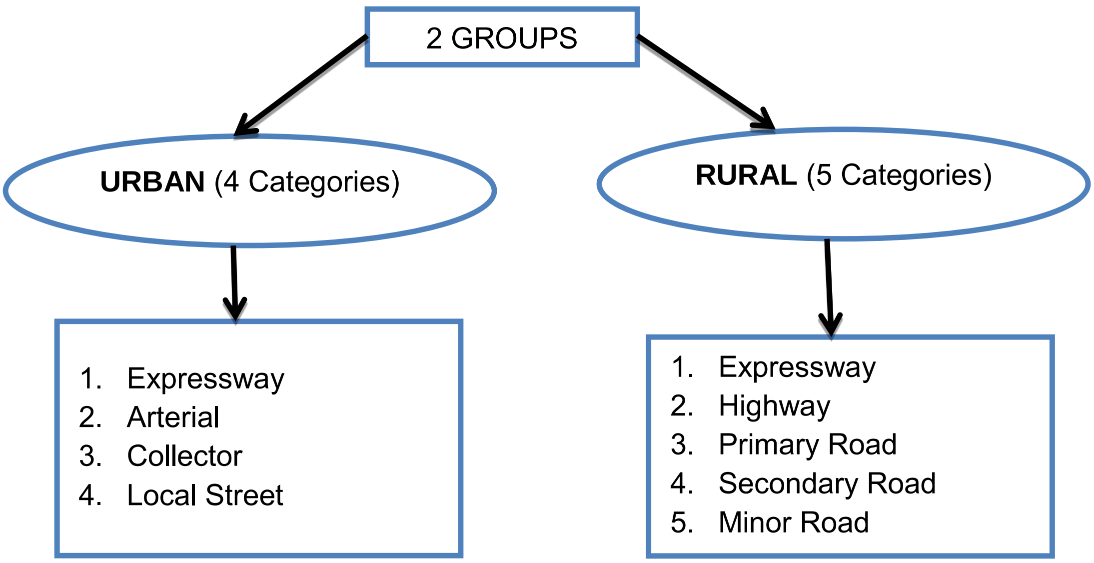
2.1.2 Functions of Road

Each road has its function according to its role either in the National Network, Regional Network, State Network or City/Town Network. The most basic function of a road is transportation. This can be further divided into two sub-functions; namely mobility and accessibility. However, these two sub-functions are in trade off. To enhance one, the other must be limited.

In **rural** areas, roads are divided into **five (5) categories**, namely, **EXPRESSWAY**, **HIGHWAY**, **PRIMARY ROAD**, **SECONDARY ROAD** and **MINOR ROAD**. In **urban** areas, roads are divided into **four (4) categories**, namely **EXPRESSWAY**, **ARTERIAL**, **COLLECTOR** and **LOCAL STREET**. They are in ascending order of accessibility and descending order of mobility.

2.1.3 Road Category and Their Application

Categories of Road



Categories of roads in Malaysia are defined by their general functions as follows:

(a) Expressway

An Expressway is a divided highway for through traffic with full control of access and always with grade separations at all intersections.

In rural areas, they apply to the interstate highways for through traffic and form the basic framework of National Road Transportation for fast travel. They serve long trips and provide higher speed of travelling and comfort. To maintain this, they **are fully access controlled** and are designed to the highest standards. In urban areas, they form the basic framework of road transportation system in urbanized area for through traffic. They also serve relatively long trips and smooth traffic flow and with full access control and complements the Rural Expressway.

All expressways including the ramps will have full access control. However, in urban areas, it may be appropriate to allow left in – left out access with service interchange ramps to enhance connectivity to the existing road network. Any such connections on entry ramps should provide for appropriate acceleration distances onto the expressway, and any access on an exit ramp shall have adequate deceleration distance so that safety is not affected. Any such connections should be assessed for the likely usage and the traffic capacity determined.

(b) Highway

They constitute the interstate national network for intermediate traffic volumes and complement the expressway network. They usually link up directly or indirectly the Federal capital, State capitals, large urban centers and points of entry/exit to the country. They serve long to intermediate trip lengths. Speed of travel is not as important as in an Expressway but relatively high to medium speed is necessary. Smooth traffic is provided with **partial access control**.

(c) Primary Roads

They constitute the major roads forming the basic network of the road transportation system within a state. They serve intermediate trip lengths and medium travelling speeds. Smooth traffic is provided **with partial access control**. They usually link the State capitals and District capitals or other major towns.

(d) Secondary Roads

They constitute the major roads forming the basic network of the road transportation system within a District or Regional Development Area. They serve intermediate trip lengths with **partial access control**. They usually link up the major towns within the District or Regional Development Area.

(e) Minor Roads

They apply to all roads other than those described above in the rural areas. They form the basic road network within a Land Scheme or other sparsely populated rural area. They also include roads with special functions such as holiday resort roads, security roads or access roads to microwave stations. They serve mainly local traffic with short trip lengths with no access control.

(f) Arterials

An arterial is a continuous road within partial access control for through traffic within urban areas. Basically it conveys traffic from residential areas to the vicinity of the central business district or from one part of a city to another which does not intend to penetrate the city center. Arterials do not penetrate identifiable neighbourhoods. Smooth traffic flow is essential since it carries large traffic volumes.

(g) Collectors

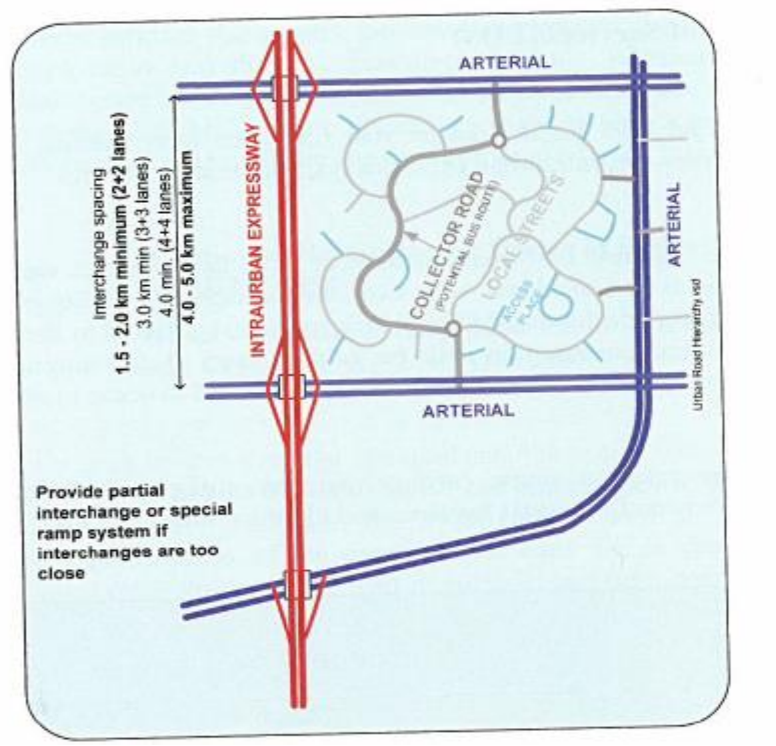
A collector road is a road with partial access control designed to serve as a collector or distributor of traffic between the arterial and the local road systems. Collectors are the major roads which penetrate and serve identifiable neighbourhoods, commercial areas and industrial areas.

(h) Local Streets

The local street system is the basic road network within a neighbourhood and serves primarily to offer direct access to abutting land. They are links to the collector road and thus serve short trip lengths. Through traffic should be discouraged.

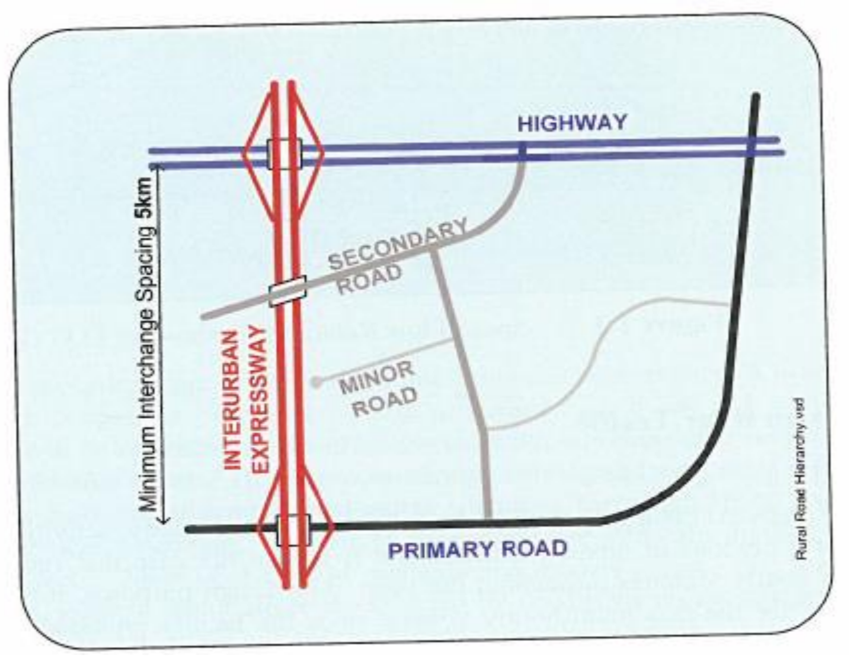
The characteristics for road categories under the Urban and Rural condition are summarized as in **Table 2.1 and depicted in Figures 2.1A and 2.1B** respectively.

FIGURE 2.1A: FUNCTIONAL CLASSIFICATION SYSTEM – URBAN AREA



Source: LLM/GP/T5-08 – Guideline for Toll Expressway System-Design Standard, Figure 2-1

FIGURE 2.1B: FUNCTIONAL CLASSIFICATION SYSTEM – RURAL AREA



Source: LLM/GP/T5-08 – Guideline for Toll Expressway System-Design Standard, Figure 2-2

TABLE 2.1: CHARACTERISTICS OF ROAD CATEGORIES

AREA	ROAD CATEGORIES	Trip Length			Design Volume			Speed			NETWORK
		Long	Med	Short	High	Med	Low	High	Med	Low	
RURAL	Expressway	■	■		■	■		■			National network
	Highway	■	■		■	■		■	■		National network
	Primary Road	■	■		■	■			■		State network
	Secondary Road		■	■		■	■		■	■	District network
	Minor Road			■			■			■	Supporting network
URBAN	Expressway	■	■		■				■		National network
	Arterial		■		■	■			■		Major links to Urban centres
	Collector		■	■		■	■		■	■	Major streets within urban centres
	Local Street			■			■			■	Minor streets / town network

Source: REAM GL 2/2002: A Guide on Geometric Design of Roads, Table 2.1

2.1.4 Road Administration

For the purpose of road administration, roads are classified as Federal, State, Local Authority (City Hall, Municipal or Local Council), or Village (Kampung) Roads depending upon the jurisdiction they fall under.

(a) Federal Roads

Federal Roads are roads that are gazetted under the Federal Road Ordinance and are usually roads linking the State Capitals, Airports, Railway Stations and Ports. Roads within Felda Land Schemes and those in other Regional Land Schemes constructed under the Federal Fund shall also fall under this category. The maintenance of these roads is the responsibility of the Federal Government and is done through the State JKR with funds from the Federal Government.

Federal Roads can fall into the rural category under the national highway network, or the urban category under the arterial roads which link major urban centers.

(b) State Roads

State Roads are all the other roads within the State outside the jurisdiction of the Local Authority or District Office, built to JKR Standard. They are normally constructed with State Funds. The maintenance of these roads are the responsibility of the State Government and is done through the State JKR.

State roads can fall into the rural category under primary road network in the state, or the urban category under collector roads within urban centers.

(c) Local Authority Roads

Local Authority Roads are all those roads within the limits and boundaries of the Local Authority and are normally maintained by the responsible Local Authority.

Local roads can fall under the rural category if it is a minor road supporting the local network, or the urban category if it serves as minor streets in a local town network.

(d) Village (Kampung) Roads

Village or Kampung Roads are all those roads directly under the jurisdiction of the District Office. They can be earth or metaled roads, usually with no right of way, and maintained by the District Office.

Village or Kampong Roads are under the rural category and serve predominantly the local rural areas only.

2.2 Design Standards for Roads

2.2.1 The Importance of Standardization

Technically, the geometric design of all roads needs to be standardized for the following reasons:-

(a) To provide uniformity in the design of roads according to their performance requirements.
(b) To provide consistent, safe and reliable road facilities for movement of traffic.

(c) To provide a guide for less subjective decision on road design.

2.2.2 Rural and Urban Areas

Urban areas are defined as roads within a gazetted Municipality limit or township having a population of at least 10,000 where buildings and houses are gathered and business activity is prevalent. Any roads outside the Municipality limit is considered rural, including roads connecting Municipalities that are more than 5 kilometers apart.

There is no fundamental difference in the principles of design for rural and urban roads. Roads in urban areas, however, are characterized by busy pedestrian activities and frequent stopping of vehicles owing to short intersection spacing and congested built-up areas. Lower design speeds are usually adopted for urban roads and different cross sectional elements are applied to take into account the nature of traffic and adjoining land use. It is for these reasons that variations in certain aspects of geometric design are incorporated for these two broad groups of roads.

2.2.3 Application of Design Standards for Roads

The design standard is classified into six (6) groups (R6, R5, R4, R3, R2 & R1) for rural (R) areas and into six (6) groups (U6, U5, U4, U3, U2 & U1) for urban (U) areas. Each of these standards are listed below with descending order of hierarchy.

(a)Standard R6/U6: Provides the highest geometric design standards for rural or urban areas. Roads under this category usually serve long trips with high travelling speed (90 kph or higher) of travelling, comfort and safety. It is always designed with divided carriageways and with full access control to ensure comfort and safety. The Rural and Urban Expressway falls under this standard.

(b)Standard R5/U5: Provides high geometric standards and usually serve long to intermediate trip lengths with high to medium travelling speeds (80 kph or higher). It is usually with partial access control. The Highway, Primary Road and Arterial fall under this standard. It is sometimes designed as divided carriageways with partial access control.

(c)Standard R4/U4: Provides medium geometric standards and serve intermediate trip lengths with medium travelling speeds (70 kph or higher). It is also usually with partial access control. The Primary Road, Secondary Road, Minor Arterial and Major Collector fall under this standard.

(d)Standard R3/U3: Provides low geometric standard and serves mainly local traffic. There is partial or no access control. The Secondary Road, Collector or Major Local Streets are within this standard. The travelling speed is usually 60 kph.

(e)Standard R2/U2: Provides low geometric standards for two way flow. It is applied only to local traffic with low volumes of commercial traffic. The Minor Roads and Local Streets fall under this standard. The travelling speed is usually 50 kph.

(f)Standard R1 /U1: Provides the lowest geometric standards and is applied to road where the volumes of commercial vehicles are very low in comparison to passenger traffic. The travelling speed is 40 kph or less. In cases where commercial traffic is not envisaged such as private access road in low cost housing area, the geometry standards could be lowered especially the lane width and gradient.

2.3 Access Control

2.3.1 Degree of Control

Access control is the condition where the right of owners or occupants of abutting land or other persons to access, in connection with a road is fully or partially controlled by the public authority.

Control of access is usually classified into three types for its degree of control, i.e. namely full control, partial control and non-control of access.

Full Control of Access means that preference is given to through traffic by providing access connection with selected public roads only and prohibiting at-grade crossings or direct private driveway connections. The access connections with public roads varies in the spacing between them from 2 km in the highly developed central business areas to 8 km or more in the sparsely developed urban fringes.

Full control of access means no driveways from adjacent properties are permitted except to a service road. Limited access means property access may be restricted by the construction of a median that only allow left turns into properties and left turns out. Criteria for the location of property access to new local streets include distance from intersections, sight distance required, and a possible requirement for vehicles not to reverse onto the road.

For the various road classes and the category of property access control for the road abutting it, refer to **Table 2.2**.

TABLE 2.2: PROPERTY ACCESS CONTROL FOR EACH ROAD CLASS

Road Class	Degree of Property Access Control	
	New Road	Upgrading of Existing Road
Expressway through carriageways	Full	Full
Expressway ramps and system interchange ramps	Full	Full
Urban expressway service interchange ramps	Full/Limited	Full/Limited
Primary road	Limited/Full	Limited/Nil
Secondary roads	Limited	Nil
Minor roads	Meet design criteria	Nil

Source: LLM/GP/T5-08 – Guideline for Toll Expressway System-Design Standard

Partial Control of Access means that preference is given to through traffic to a degree that, in addition to access connection with selected public roads, there may be some other crossings necessary along the trafficked roads. At-grade intersections should be limited and only allowed at selected locations. The spacing of at-grade intersections preferably signalized may vary from 0.4 km to 1.0 km.

To compensate for the limited access to fully or partially access controlled roads, frontage or service roads are sometimes provided alongside the main roads.

In **Non-Control Access**, there is basically no limitation of such accesses.

2.3.2 Selection of Access Control

The selection of the degree of control required is important so as to preserve the 'as-built capacity' of the road as well as to improve overall safety for all road users. Two aspects pertaining to the degree of control is to be noted, i.e.: -

(a) Consideration of accesses to existing developments during design stage.

(b) Control of accesses to future developments after road being constructed.

The selection of degree of access control depends on traffic volumes, function of the road and the road network around the areas. **Tables 2.3A** and **2.3B** are general guides for the selection of degree of access control for the Urban and Rural road condition respectively.

TABLE 2.3A: SELECTION OF ACCESS CONTROL (RURAL)

Road Category \ Design Standard	R6	R5	R4	R3	R2	R1
Expressway	F	-	-	-	-	-
Highway	-	P	-	-	-	-
Primary Road	-	P	P	-	-	-
Secondary Road	-	-	P	P	-	-
Road	-	-	-	-	N	N

TABLE 2.3B: SELECTION OF ACCESS CONTROL (URBAN)

Road Category \ Design Standard	U6	U5	U4	U3	U2	U1
Expressway	F	-	-	-	-	-
Arterial	-	P	P	-	-	-
Collector	-	P	P	P	-	-
Local Street	-	-	N	N	N	N

Note: **F =** Full Control of Access
P = Partial Control of Access
N = No Control of Access

2.4 Selection of Design Standards

The selection of the required design standard should begin with the assessment of the function of the proposed road and the area it traverses. This should generally be done in conjunction with The Highway Planning Unit, Ministry of Works. If there is an overlapping of functions, the ultimate function of the road shall be used for the selection criteria. The Road Designer should approach the road design utilizing the desirable design values and not merely taking the minimum preset values for the road design parameters.

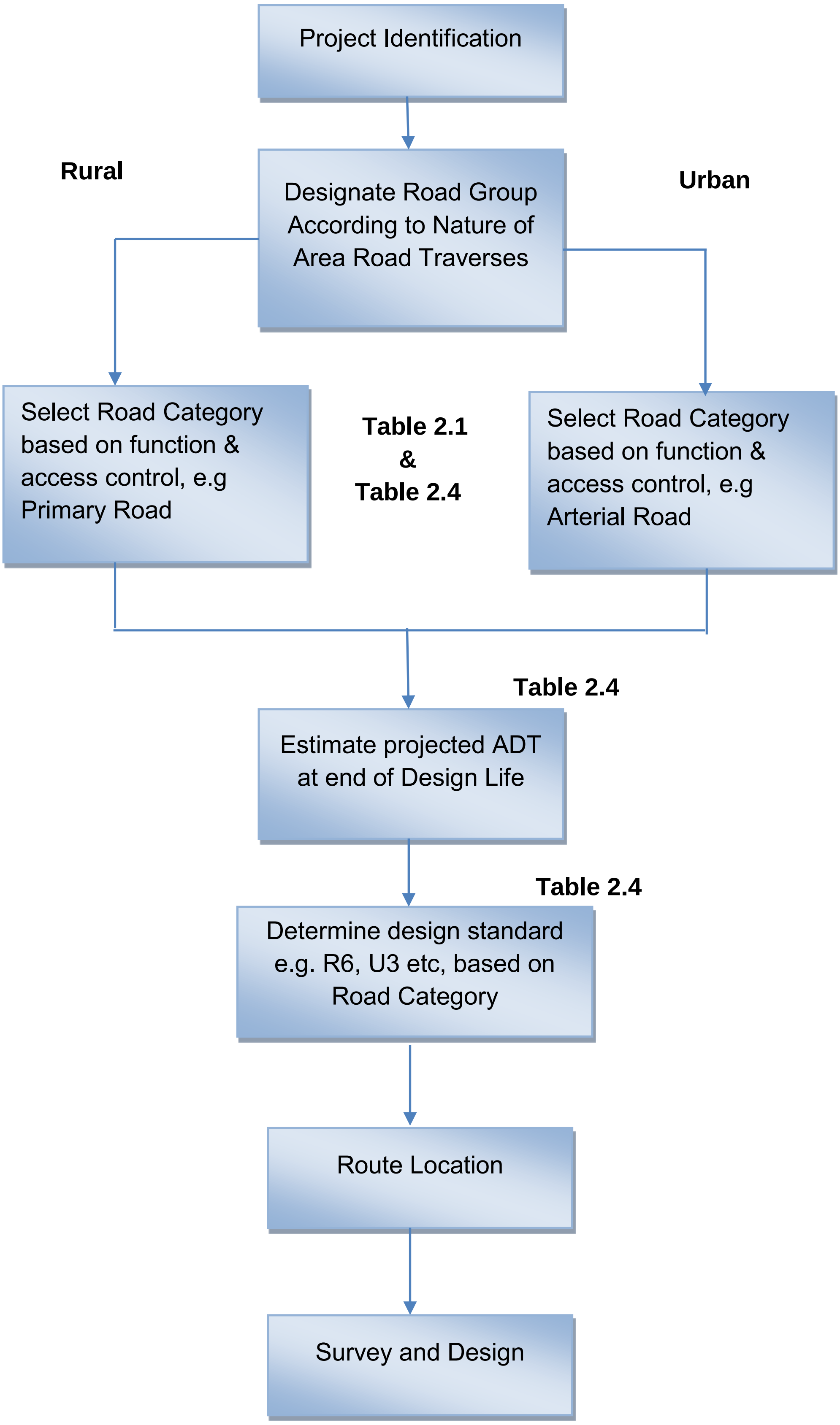
The selection of design standards used for various categories of roads is as shown in **Tables 2.4** for the Rural and Urban condition respectively, while the process flow chart for the selection of design standards is as shown in **Figure 2.2**.

The projected **Average Daily Traffic (ADT)** at the end of the **design life (20 years after completion of the road)** should then be calculated and from **Table 2.4**, the required design standard can be obtained. From the capacity analysis the required number of lanes can then be calculated.

TABLE 2.4: SELECTION OF DESIGN STANDARD

Area	Projected ADT / Road Category	All Traffic Volume	≥ 10,001	3,001 To 10,000	1,001 To 3,000	151 To 1,000	≤150
RURAL	Expressway	R6	-	-	-	-	-
	Highway	R5	-	-	-	-	-
	Primary Road	-	R5	R4	-	-	-
	Secondary Road	-	-	R4	R3	-	-
	Minor Road	-	-	-	-	R2	R1
URBAN	Expressway	U6	-	-	-	-	-
	Arterials	-	U5	U4	-	-	-
	Collector	-	U5	U4	U3	-	-
	Local Street	-	-	U4	U3	U2	U1

FIGURE 2.2: FLOW CHART FOR SELECTION OF DESIGN STANDARDS



2.5 Road Safety

Road safety consideration is an important aspect in the design of roads, With an increasing number of accidents each year, attention to road safety should be emphasized, While the road element is only one of the three groups (road, human and vehicle element) of influences causing accidents, it is nonetheless the responsibility of the designer to provide as safe a road environment as possible.

In this aspect, the road design should be one with uniformly high standards applied consistently over a section. It should avoid discontinuities such as abrupt major changes in design speeds, transitions in roadway cross section, the introduction of a short radius curve in a series of longer radius curves, a change from full to partial control of access, constrictions in roadway width by narrow bridges or other structures, intersections without adequate sight distances, or other failures to maintain consistency in the roadway design. The highway should offer no surprises to the driver, in terms of either geometries or traffic controls.

Road design may not always be based upon minimum standards. A more liberal and optimum design must always be considered to increase the level of safety of roads substantially, even if the construction cost may be higher.

It is to be borne in mind that safety deficiencies caused by always adopting minimum design criteria may not be corrected by any known traffic device or appurtenance. Warning sign or other added roadside appurtenance is a poor substitute for adequate geometric design of roads.

In the effort to improve road safety, **Road Safety Audit (RSA)** was introduced in 1994. It is a formal process to identify deficiencies in road design at different stages as well as to identify for elimination hazardous features on existing roads so as to make the road safer for all road users.

RSA is carried out in five clearly defined stages i.e., **the feasibility and planning stage (Stage 1), preliminary design stage (Stage 2), detailed design stage (Stage 3), pre-opening stage (Stage 4) and operational/audit of existing roads (Stage 5).**

Reference on RSA should be made to the relevant document published by Public Works Department Malaysia.

3.0 DESIGN CONTROL AND CRITERIA

This chapter discusses those characteristics that govern the various elements in the design of highway. These include the topography and land use through which the route is to be located, the traffic that will use the route, design vehicles characteristics, design speed of the route and the capacity to be provided along the route.

3.1 Topography and Land Use

The location of a road and its design are considerably influenced by the topography, physical features, and land use of the area traversed. Geometric design elements such as alignment, gradients, sight distance and cross- section are directly affected by topography, and must be selected so that the road designed will reasonably fit into those natural and man-made features and economize on construction and maintenance.

The topography of the profile of a road can generally be divided into three groups, namely, **FLAT, ROLLING and MOUNTAINOUS**, where: -

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

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

MOUNTAINOUS terrain: The topographical condition where longitudinal and transverse changes in the elevation of the ground with respect to the road or street are abrupt and where benching and side hill excavation are frequently required to obtain acceptable horizontal and vertical alignment. The natural ground cross-slopes in a mountainous terrain are generally **above 25%.**

The topography of a route may affect the operation characteristic of the roadway and this may affect the type of road to be built. For example, steep gradient and restrictive passing sight distance will greatly reduce the capacity of a 2-lane road (single carriageway) and lower the operation speed of this link. However, on a dual carriageway or a wider road, the effects will be much less and hence the operation speed and characteristics of the road will be enhanced. Consequently, the nature of the terrain, to a great extent, determines the type of road to be built.

Topographic conditions may also affect the cross-sectional arrangement of divided roads. In some cases a facility provided on a single road formation may be appropriate; in others it may be more fitting to locate the facility on a road with two separate formations due to economic or environmental considerations. The existing topographical condition may affect the design of formation level as the main design is to follow the existing terrain in order to reduce the earthworks and environmental issues. In some cases, if the alignment crosses through hilly terrain, the formation level in the opposing directions may be at different levels so as to suit the terrain as well as to avoid cutting the hill and disturbing its stability.

In urban areas, land development for residential, commercial and industrial purposes will restrict choice of road location, lower running speed, generate more turning movements, and require more frequent intersections than in open rural areas. Geologic and climatic conditions must also be considered for the location and geometric design of a road.

Since topography and land use have pronounced effect on road geometrics, information regarding these features should be obtained in the early stages of planning and design. Aerial surveys generally can expedite the collection of these data. Topographic maps of suitable scale form the necessary base for preliminary location. In the preparation of final plans a scale of 1:1000 is generally used, and sometimes a scale of 1:500 on supplemental drawings to show particular details.

The topographic maps should be supplemented by further data regarding subsurface and drainage conditions, the value of land, size, type and value of buildings, planning for the improvement of the area, and other information that may affect or be affected by the road.

3.2 Traffic

The design of a road should be based on traffic data which serves to establish the "loads" for geometric design. Traffic data for a road or section of road are generally available from the most recent edition of "Traffic Volume Peninsular Malaysia" published by the Highway Planning Unit of the Ministry Of Works.

3.2.1 Average Daily Traffic (ADT)

ADT represents the total traffic for the year divided by 365, or the average volume per day. Knowledge of the ADT is important for many purposes, such as determining the annual usage as justification for proposed expenditures or for design of structural elements of a road. The projected ADT is also used to designate the standard of road as shown in **Table 2-3: Selection of Design Standards**. However, the direct use of ADT in geometric design is not appropriate because it does not indicate the significant fluctuation in the traffic occurring during various months of the year, days of the week and hours of the day. A more appropriate measurement is by hourly volume which is used to determine the capacity requirement of the road.

3.2.2 Design Hourly Volume (DHV)

The traffic pattern on any road shows considerable variation in traffic volumes during the different hours of the day and in hourly volumes throughout the year. It is difficult to determine which of these hourly traffic volumes should be used for design. It would be wasteful to base the design on the maximum peak hour traffic of the year, yet the use of the average hourly traffic would result in an inadequate design.

To determine the hourly traffic best fitted for design, a curve showing the variation in hourly traffic volumes during the year is used. The Highway Planning Unit of the Ministry of Works should be consulted for the survey data if available. In the absence of the traffic survey data, the hourly traffic used in design is the 30th highest hourly volume of the year, abbreviated as 30 HV. The design hourly volume, abbreviated DHV is the 30HV of the future year chosen for design.

The above criteria is applicable to most rural and urban roads. However, for roads on which there is an unusual or highly seasonal variation in traffic flow such as holiday resort roads, the 30HV may not be applicable. It may be desirable, to choose an hourly volume for design (about 50 percent of the hourly volumes) expected to occur during a very few maximum hours (15 to 20) of the design year whether or not it is equal to 30HV.

3.2.3 Design Hourly Volume Ratio (K)

K, is the ratio of DHV to the designed ADT. K's value ranges from 7% to 20% and the actual highest peak hour value should be obtained from traffic data. The Highway Planning Unit of the Ministry of Works and other relevant authorities should be consulted for the traffic data if available.

In the absence of information, k = 12% for urban roads and k = 15% for rural roads can be used. For roads with highly distinct fluctuations of traffic, whether seasonal, daily or hourly, it is recommended that traffic surveys be carried out as the k values may be unrealistic.

3.2.4 Directional Distribution Ratio (D)

For 2-lane roads, the DHV is the total traffic in both directions of travel. On roads with more than two lanes and on 2-lane roads where important intersections are encountered or where additional lanes are to be provided later, knowledge of the hourly traffic load in each direction of travel is essential for design.

The directional distribution of traffic during the design hour should be determined by field measurements on the facilities. The Highway Planning Unit of the Ministry of Works and other relevant authorities should be consulted for the survey data if available.

Generally, in the absence of field data, D value of 60% in urban areas and 65% in rural areas can be used. Traffic distribution by directions is generally consistent from year to year and from day to day except on some roads serving holiday resort areas or retreat enclaves.

3.2.5 Traffic Composition

This is the percentage of various classes of vehicles in the DHV. Vehicles of different sizes and weights have different operating characteristics, which must be considered in geometric design. Commercial vehicles generally are heavier, slower and occupy more roadway space and consequently impose a greater traffic effect on the road than the passenger vehicles.

Vehicles of various sizes and weights are classified into the following six (6) groups under the Annual National Traffic Census conducted by HPU:

(a) motorcycles
(b) cars and taxis
(c) light vans and utility vehicles
(d) medium lorries (2 axles)
(e) heavy lorries (3 or more axles)
(f) buses

On existing road network, the traffic composition at peak hour can be established from "Road Traffic Volume Malaysia" published by the Highway Planning Unit, Ministry of Works.

3.2.6 Projection of Traffic

New roads or improvements of existing roads should be based on future traffic expected to use the facilities. Desirably, a road should be designed to accommodate the traffic that might occur within the life of the facility under reasonable maintenance. This is seldom economically feasible and is difficult to estimate.

The projection of traffic for use in the design should be based on a period of 20 years after completion of the road. In areas where traffic estimation is difficult due to uncertainty in land use, planning or roadside interference, the design of the formation width shall be based on a period of 20 years, but pavement construction may be staged basing on a 10 year period for the first stage.

The Highway Planning Unit and other relevant authorities should be consulted for the projection of traffic over the design period. Where construction is to be staged, the designer's attention is drawn to the problem of relocation of services. It is for this reason that utility services be located at the edge of the R.O.W., within the services corridor (if need to install) at the outset of the road construction works so as to cause the least disturbance, even if the road pavement needs to be widened later.

3.3 Design Vehicles and Characteristics

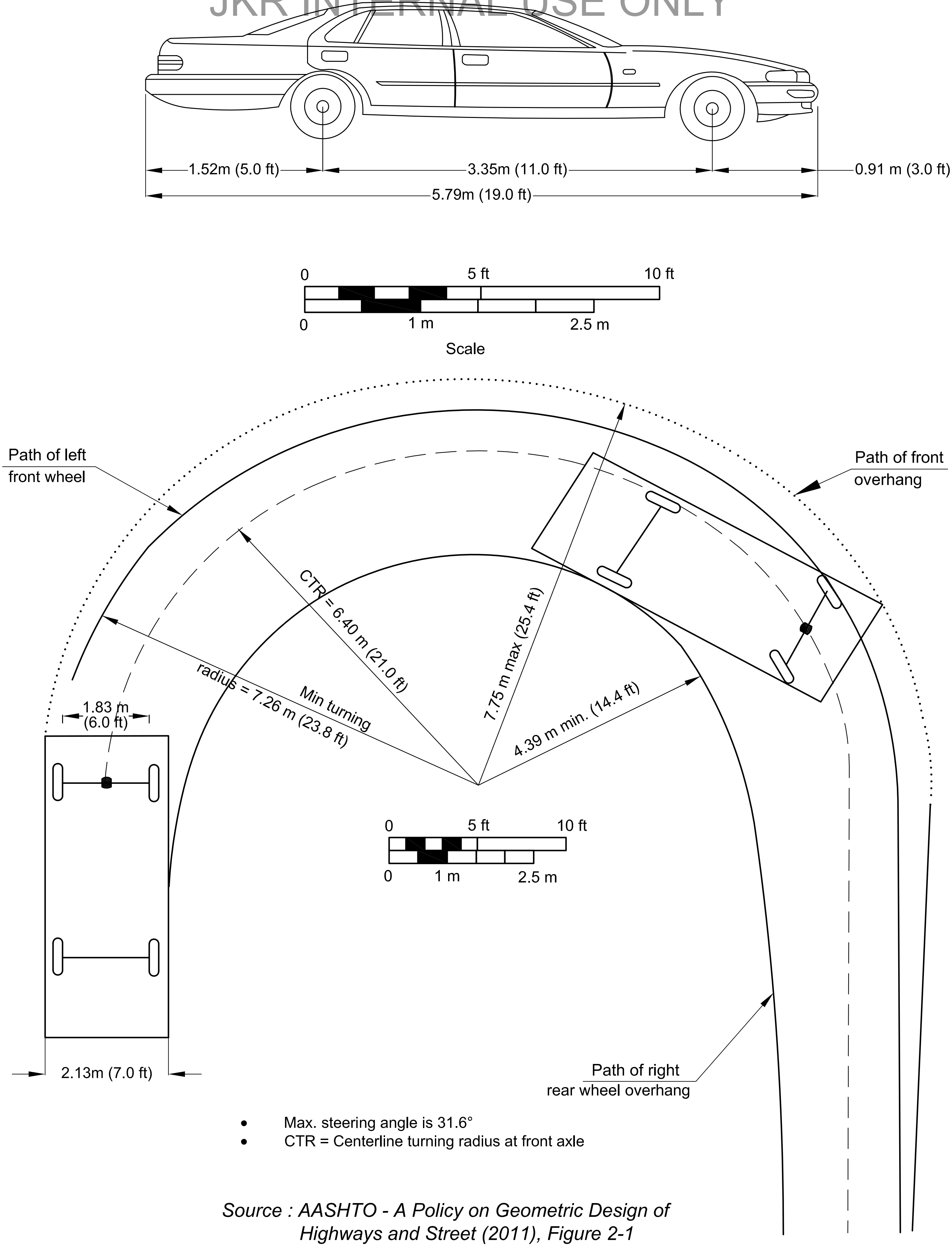
The physical characteristics of vehicles and the proportions of various size vehicles using the roads will affect the geometric design of roads. A design vehicle is a selected motor vehicle, the weight, dimensions and operating characteristics of which are used to establish highway design controls to accommodate vehicles of a designated type.

For purposes of geometric design, the design vehicle should be one with dimensions and minimum turning radius larger than those of almost all vehicles in its class. Since roads are designed for future traffic the sizes of vehicles used in design should be determined by analyzing the trends in vehicle dimensions and characteristics.

3.3.1 Design Vehicles

The design vehicles to be used for geometric design follows that used by AASHTO as in Chapter II of AASHTO " Design Vehicle" - A Policy of Geometric Design of Highways and Streets, 2001 and Chapter II of AASHTO "Design Vehicles" - A Policy of Geometric Design of Highways and Streets, 2011.

Figures 3.1, 3.2 and 3.3 show the dimensions and turning characteristics for the P, SU and WB-15 design vehicles respectively.

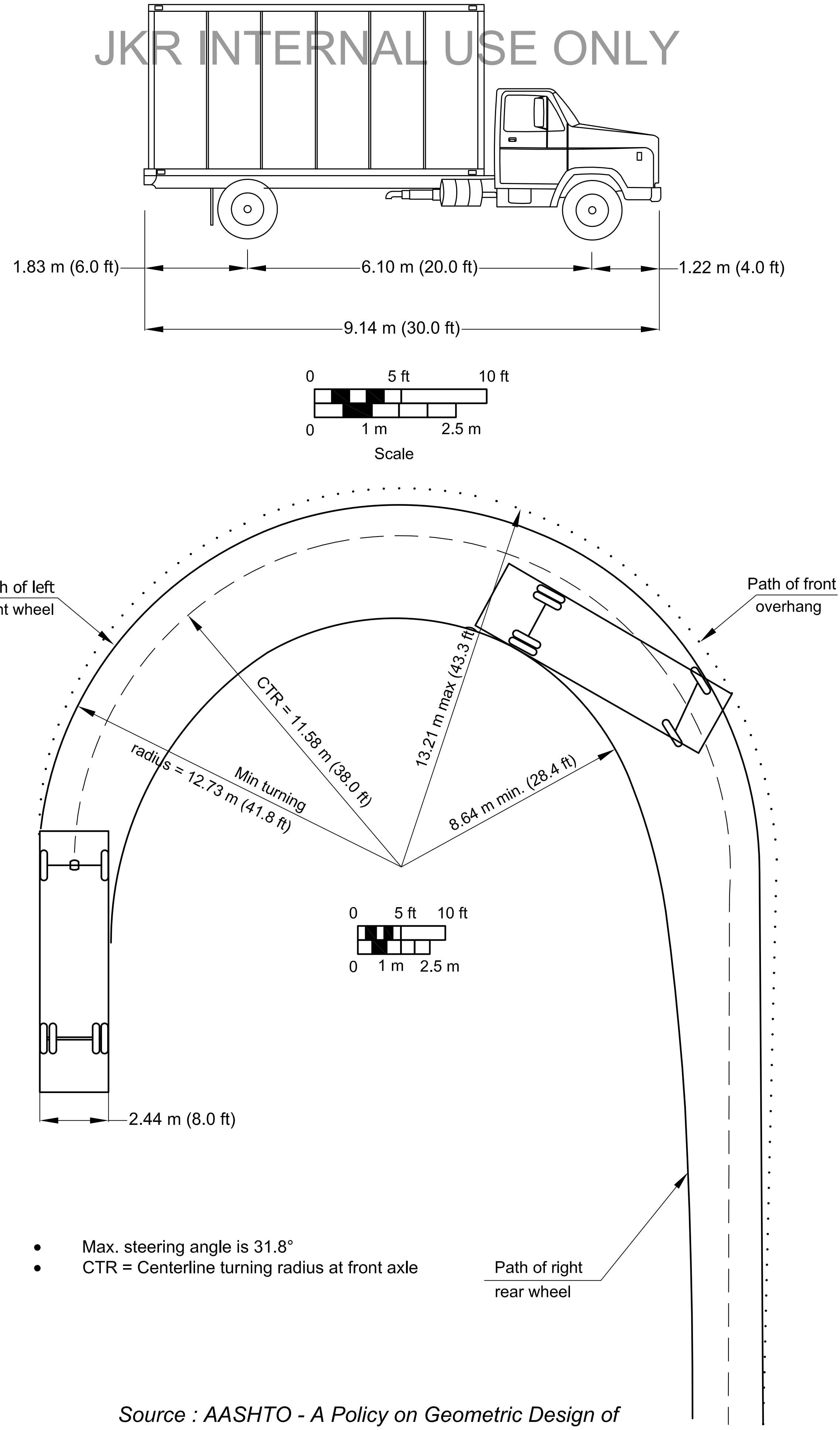


- Max. steering angle is 31.6°
- CTR = Centerline turning radius at front axle

Source : AASHTO - A Policy on Geometric Design of Highways and Street (2011), Figure 2-1

FIGURE 3.1: P DESIGN CURVE

Scale : Not to scale

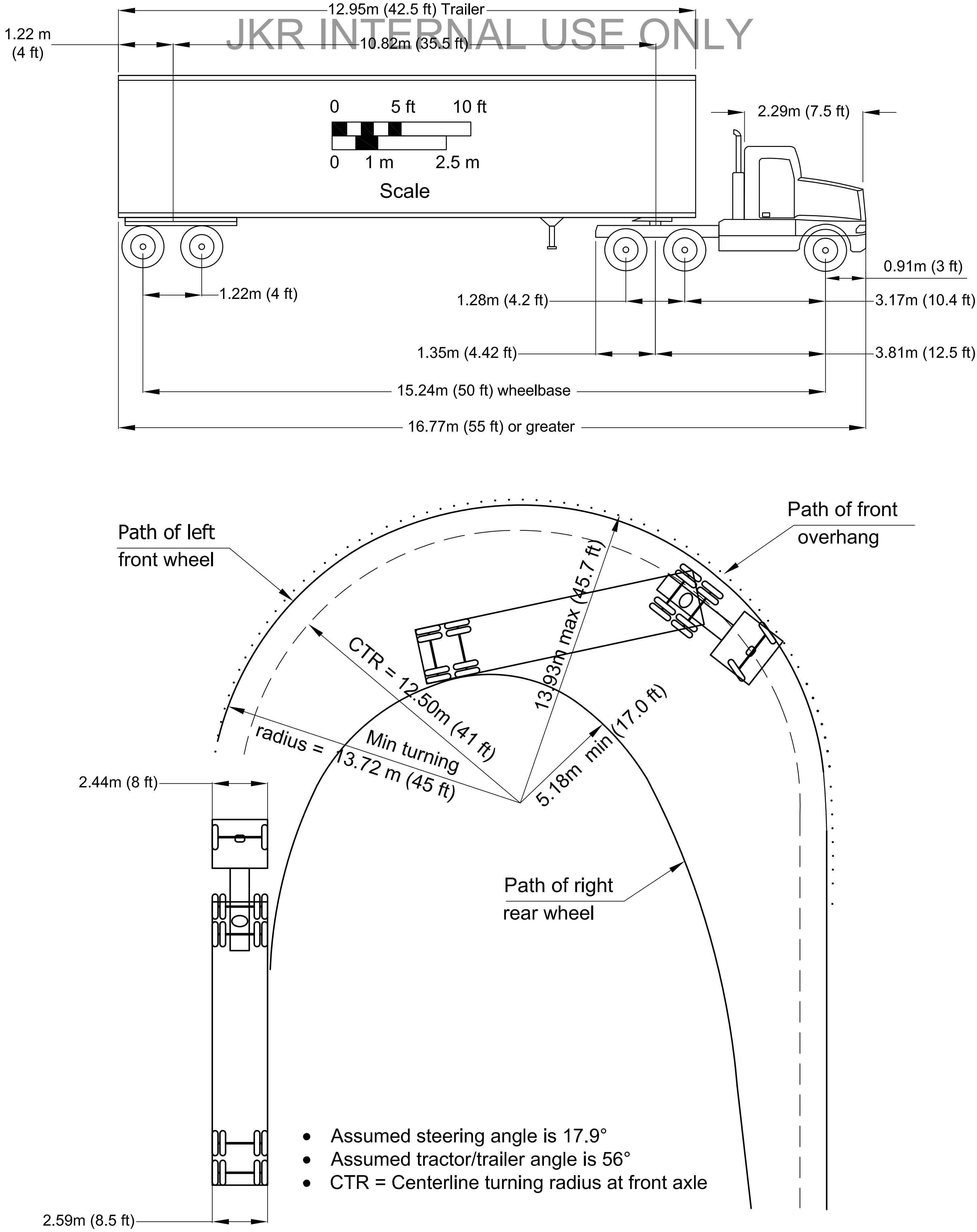


- Max. steering angle is 31.8°
- CTR = Centerline turning radius at front axle

Source : AASHTO - A Policy on Geometric Design of Highways and Street (2011), Figure 2-2

FIGURE 3.2: SU DESIGN CURVE

Scale : Not to scale



- Assumed steering angle is 17.9°
- Assumed tractor/trailer angle is 56°
- CTR = Centerline turning radius at front axle

Source : AASHTO - A Policy on Geometric Design of Highways and Street (2011), Exhibit 2-14

FIGURE 3.3 : WB-15 DESIGN CURVE

Scale : Not to scale

3.3.2 Summary of Dimension of Design Vehicles

Table 3.1 below summarizes the design vehicle dimensions and characteristics.

TABLE 3.1: DESIGN VEHICLE DIMENSIONS

Design Vehicle		Dimension in meter (m)						Turning Radius (m)
Type	Symbol	Wheel Base (m)	Overhang (m) Front*	Overhang (m) Rear **	Overall Length (m)	Overall Width (m)	Height (m)	
Passenger Car	P	3.35	0.91	1.52	5.79	2.13	1.3	7.26
Single Unit Truck	SU	6.10	1.22	1.83	9.12	2.44	3.4-4.1	12.73
Truck Combination	WB-15	WB1: 4.45 WB2: 10.79	0.91	0.6	16.77	2.59	4.1	13.72

Note:

a) Maximum allowable overall lengths under current Malaysian Legislation are 16m, or 25 m, if with special approval.

b) Maximum allowable overall width under current Malaysian Legislation is 2.5 m.

c) Maximum overall height control under current Malaysian Legislation is 4.2 m.

* This is the length of the front overhang from the front axle
**This is the length of the rear overhang from the back axle of the tandem axle assembly

3.4 Speed

Speed is a primary factor in all modes of transportation, and is an important factor in the geometric design of roads. The speed of vehicles on a road depends, in addition to capabilities of the drivers and their vehicles, upon general conditions such as the physical characteristics of the highway, the weather, the presence of other vehicles and the legal speed limitations.

The speed is selected to meet the needs of the road to fulfill its function. Therefore roads which are planned to provide long distance travel will be designed with a higher speed while those which provides short distance travel can be given a lower design speed.

3.4.1 Operating Speed

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

3.4.2 Design Speed

Design speed is the maximum safe speed that can be maintained over a specified section of the road when conditions are so favorable that the design features of the road governs. The assumed design speed should be a logical one with respect to the topography, the adjacent land use and the type of road. Every effort should be made to use as high a design speed as reasonably practicable while maintaining the desired degree of safety, mobility and efficiency.

Tables 3.2A and **3.2B** indicate the selection of design speeds with respect to rural and urban standards respectively.

Design speed is defined as: "a speed selected to establish specific minimum geometric design elements for a particular section of highway". These design elements include vertical and horizontal alignment, and sight distance. Other features such as widths of pavement and shoulders, horizontal clearances, etc., are generally indirectly related to design speed.

The choice of design speed is influenced primarily by factors such as the terrain, economic, environmental factors, type and anticipated volume of traffic, functional classification of the highway, and whether the area is rural or urban. A highway in level or rolling terrain justifies a higher design speed than one in mountainous terrain.

Where a difficult location is obvious to approaching drivers, they are more likely to accept a lower design speed than where there is no apparent reason for it. Where it is necessary to reduce design speed, many drivers may not perceive the lower speed condition ahead, and it is important that they be warned in advance. The changing conditions should be indicated by such traffic devices as speed zone signs and appropriate warning signs depicting the conditions ahead.

A highway carrying a large volume of traffic may justify a higher design speed than a less important facility in a similar topography, particularly where the savings in vehicle operation and other costs are sufficient to offset the increased cost of right of way and construction. On the contrary, a lower design speed should not be assumed for a secondary road where the topography is such that drivers are likely to travel at high speeds.

Taking into account the above considerations, as high a design speed as feasible should be used. It is preferable that the design speed for any section of highway be a constant value. However, during the detailed design phase of a project, special situations may arise in which engineering, economic, environmental, or other considerations can make it impractical to provide the minimum elements established by the design speed. The most likely examples are partial or brief horizontal sight distance restrictions, such as those imposed by bridge rails, bridge columns, retaining walls, sound walls, cut slopes, and median barriers.

The cost to correct such restrictions may not be justified. Technically, this will result in a reduction in the effective design speed at the location in question. Such technical reductions in design speed shall be discussed with and approved by Road Authorities.

TABLE 3.2A: DESIGN SPEED (RURAL)

Design Standard	Design Speed (km/ hr)		
	Terrain		
	Flat	Rolling	Mountainous
R6	120	100	80
R5	100	80	60
R4	90	70	60
R3	70	60	50
R2	60	50	40
R1	40	30	20

TABLE 3.2B: DESIGN SPEED (URBAN)

Design Standard	Design Speed (km/ hr)		
	Area Type		
	I	II	III
U6	100	80	60
U5	80	60	50
U4	70	60	50
U3	60	50	40
U2	50	40	30
U1	40	30	20

Note:

Type I - Relatively free in road location with very little problems as regards land acquisition, affected buildings or other socially sensitive areas.

Type II - Intermediate between I and III.

Type III - Very restrictive in road location with problems as regards to land acquisition, affected buildings and other sensitive areas.

3.4.3 Design Sections

In the design of a substantial length of road, it is desirable, though not always feasible, to allow for a constant design speed. Changes in terrain and other physical controls may dictate a change in the design speed on certain-sections. If a different design speed is introduced, the change should not be abrupt, but there should be a transition section of at least 1 km per every 10 km/h drop in design speed to permit drivers to change speed gradually before reaching the section of the road with a different design speed.

The transition sections are sections with intermediate design speeds, and where the magnitude of the change in the design speeds are large, more than one transition section will be required.

The intermediate design speeds shall follow the sequence as established in **Table 3.2A** and **Table 3.2B**, i.e. if the design speed of section A is 80 km/hr and that of section B is 60 km/hr, then the design speed of the transition section between them should be 70 km/hr.

3.5 Capacity

The term highway capacity pertains to the ability of a roadway to accommodate traffic and is defined as the maximum number of vehicles that can pass over a given section of a lane or a roadway during a given time period under prevailing roadway and traffic conditions. Capacity considered here is only applicable to uninterrupted flow or open roadway conditions. Capacity for interrupted flow as at intersections will be dealt with separately.

For the purpose of determining the road capacity and Level of Service (LOS) for various types of road facilities, reference shall be made to Malaysia Highway Capacity Manual (HCM) 2011.

3.5.1 Design Volume

Design volume is the volume of traffic estimated to use the road during the design year, which is taken as 20 years after the completion of the road. The derivation of the design hour volume (DHV) is as discussed in Section 3.2.2.

Design of new highways or improvement of existing ones usually should not be based on current traffic volume alone. Considerations should also be given to the future traffic expected to use the facilities. A highway should be designed to accommodate the traffic that might occur within the life span of the facility under reasonable maintenance.

It is difficult to define the life of a highway because major segments may have different lengths of physical life. Furthermore, the design of highway based on life expectancy is greatly influenced by economics.

3.5.2 Service Volume

The service volume is the maximum volume of traffic that a designed road would be able to serve without the degree of congestion falling below a preselected level as defined by the level of service which is the operating conditions (freedom to maneuver) at the time the traffic is at the design hour volume. **Table 3.3** gives an indication of the Levels of Service used while **Figure 3.4** gives a pictorial definition of the relationship of Level Of Service to operating speed.

TABLE 3.3: LEVEL OF SERVICE

Level of Service	Remarks
A	Free Flow with individual users virtually unaffected by the presence of other vehicles in the traffic stream. This is a condition of free flow with low volume and high speed of vehicle travel on the highways.
B	Stable traffic flow with a high degree of freedom to select speed and operating condition but with some influence from the other users.
C	Restricted flow that remains stable but with significant interaction with others in the traffic stream. The general level of comfort and convenience decline noticeable at this level. Speed and maneuverability are closely controlled by the higher volume. Most of the drivers are restricted in their freedom to select their own speed, change lane or pass.
D	High density flow in which speed and freedom to maneuver are severely restricted and comfort and convenience have decline even though flow remains stable. This level represents unstable flow with operating speed being maintained, though considerably affected by changes in operating condition.
E	Unstable flow at or near capacity levels with poor level of comfort and convenience. This level represents operation at lower operating speed with volume at or near the capacity of the highways. Flow is unstable and stoppage may occur for a momentary duration.
F	Forced traffic flow in which the amount of traffic approaching a point exceeds the amount that can be served. LOS F is characterized by poor time travel with low comfort, convenience and increase accident exposure. This condition describes a forced flow operation at low speed where volumes are below the capacity. Speed is reduced substantially and stoppage may occur for short or long periods of time because of the downstream traffic condition.

Source: Malaysia HCM 2011

FIGURE 3.4: ILLUSTRATION OF LEVEL OF SERVICE (LOS)

3.5.3 Design Level of Service

The selection of the design level service is as given in **Table 3.4A** and **Table 3.4B** which gives the optimum design Levels of Service in rural and urban areas respectively.

TABLE 3.4A: DESIGN LEVEL OF SERVICE (RURAL)

Road Category	Design Level of Service
Expressway	C
Highway	C
Primary Road	D
Secondary Road	D
Minor Road	E

Source: REAM GL 2/2002: A Guide on Geometric Design of Roads, Table 3-5A

TABLE 3.4B: DESIGN LEVEL OF SERVICE (URBAN)

Road Category	Design Level of Service
Expressway	D
Arterial	D
Collector	D
Local Street	E

Source: REAM GL 2/2002: A Guide on Geometric Design of Roads, Table 3-5B

4.0 ELEMENTS OF DESIGN

4.1 Sight Distance

4.1.1 General

Sight distance is the length of road ahead visible to drivers. The ability of a driver to see ahead is of utmost importance to the safe and efficient operation of a road. The designer must provide sight distance of sufficient length in which drivers can control the speed of the vehicles so as to avoid striking an unexpected obstacle on the travelled way. Also, at frequent intervals and for a substantial portion of the length, 2-lane undivided roads should have sufficient sight distance to enable drivers to overtake vehicles without hazard.

Decision sight distance, is the distance required for a driver to detect an unexpected hazard and to select an appropriate speed or path thereby completing a maneuver which avoids this hazard. Sight distance thus includes stopping sight distance, passing sight distance and decision sight distance.

4.1.2 Stopping Sight Distance

The stopping sight distance is the length required to enable a vehicle travelling at or near the design speed to stop before reaching an object in its path. Minimum stopping sight distance is the sum of two distances i.e.,

(i) The distance traversed by a vehicle from the instant the driver sights an object for which a stop is necessary, to the instant the brakes are applied; and

(ii) The distance required to stop the vehicle after the brake application begins.

(a) Perception and Brake Reaction Time

For safety, a reaction time that is sufficient for most operators, rather than for the average operator is used in the determination of minimum sight distance. A perception time value of 1.5 seconds and a brake reaction time of a full second are assumed. This makes the total perception and brake reaction time to be 2.5 seconds.

(b) Braking Distance

The approximate braking distance of a vehicle on a level roadway is determined by the use of standard formula: -

$$d = 0.039\frac{V^2}{a}$$

Where, **d** = Brake distance (m)
V = Initial speed (kph)
a = Deceleration rate (m/s)

[Studies (Ref: Fambro, D. B., K. Fitzpatrick, and R. J. Koppa. Determination of Stopping Sight Distances, NCHRP Report 400, Washington, D.C.: Transportation Research Board, 1997) showed that most drivers decelerate at a rate greater than 4.5 m/s^2 when confronted with the need to stop for an unexpected object in the roadway. Approximately 90 percent of all drivers decelerate at rates greater than 3.4 m/s^2. Such decelerations are within the driver's capability to stay within his or her lane and maintain steering control during the braking maneuver on wet surfaces. Therefore, 3.4 m/s^2 (a comfortable deceleration for most drivers) is recommended as the deceleration threshold for determining stopping sight distance.]

(c) Design Values

The sum of the distance traversed during perception and brake reaction time and the distance required to stop the vehicle is the minimum stopping sight distance. The values to be used for minimum stopping sight distances are as shown in **Table 4.1**.

TABLE 4.1: MINIMUM STOPPING DISTANCE

Design Speed (kph)	Min. Stopping Sight Distance (m)
120	250
110	220
100	185
90	160
80	130
70	105
60	85
50	65
40	50
30	35

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-1

(d) Effect of Grades on Stopping Sight Distance

When a road is on a grade the standard formula for braking distance is

$$D = \frac{V^2}{254\,\left[\left(\frac{a}{9.8}\right) \pm g\right]}$$

in which *g* is the percentage of grade divided by 100.

The effect of grade on stopping sight distance is as shown in **Table 4.2.**

TABLE 4.2: EFFECTS OF GRADES IN STOPPING SIGHT DISTANCE - (WET CONDITIONS)

Design Speed (kph)	Stopping Sight Distance (m) for Downgrades			Stopping Sight Distance (m) for Upgrades		
	3%	6%	9%	3%	6%	9%
30	32	35	35	31	30	29
40	50	50	53	45	44	43
50	66	70	74	61	59	58
60	87	92	97	80	77	75
70	110	116	124	100	97	93
80	136	144	154	123	118	114
90	164	174	187	148	141	136
100	194	207	223	174	167	160
110	227	243	262	203	194	186
120	263	281	304	234	223	214

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-2

(e) Every effort should be made to provide stopping distances greater than the minimum design values shown in **Table 4.1** especially at locations where there are restrictions to sight in the horizontal plane.

(f) The recommended stopping sight distances are based on passenger car operation. Trucks generally need longer stopping distances from a given speed than passenger vehicles. However, a truck driver is able to see substantially farther beyond vertical sight obstructions because of the higher position of the seat in the vehicle. Separate stopping sight distances for trucks and passenger cars, therefore, are not generally used in highway design. However, where horizontal sight restrictions occur on downgrades, particularly at the ends of long downgrades where truck speeds closely approach or exceed those of passenger cars, the greater height of eye of the truck driver is of little value, even when the horizontal sight obstruction is a cut slope. Although the average truck driver tends to be more experienced than the average passenger car driver and quicker to recognise potential risks, it is desirable under such conditions to provide stopping sight distance that exceeds the values in **Table 4.1** or **Table 4.2**.

4.1.3 Decision Sight Distance

Stopping sight distances are usually sufficient to allow reasonably competent and alert drivers to come to a hurried stop under ordinary circumstances. However, these distances are often inadequate when drivers must make complex or instantaneous decisions, when information is difficult to perceive, or when unexpected or unusual maneuvers are required. Limiting sight distances to those provided for stopping may also preclude drivers from performing evasive maneuvers, 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 distances for drivers to corroborate advance warning and to perform the necessary maneuvers. It is evident that there are many locations where it would be prudent to provide longer sight distances. In these circumstances, decision sight distance provides the greater length that drivers need.

Decision sight distance is the distance required for a driver to detect an unexpected object or otherwise difficult-perceived information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard or its potential threat, select an appropriate speed and path, and initiate and complete the required safety maneuver safely and efficiently. Because decision sight distance gives drivers additional margin for error and affords them sufficient length to maneuver their vehicles at the same or reduced speed rather than to just stop, its values are substantially greater than those of stopping sight distance.

Drivers need decision sight distance whenever there is a likelihood for error in either information reception, decision-making, or control actions. The following are examples of critical locations where these kinds of errors are likely to occur, and where it is desirable to provide decision sight distance; i.e at interchange and intersection locations where unusual or unexpected maneuvers are required and changes in cross section such as toll plaza and lane drops.

The decision sight distances in **Table 4.3** provide values to be used by designers for appropriate sight distances at critical locations and serve as criteria in evaluating the suitability of the sight lengths at these locations. Because of the additional safety and maneuverability these lengths yield, it is recommended that decision sight distances be provided at critical locations or that these points be relocated to locations where decision sight distances are available. If it is not possible to provide these distances because of horizontal or vertical curvature or if relocation is not possible, special attention should then be given to the use of suitable traffic control devices for providing advance warning of the conditions that are likely to be encountered.

TABLE 4.3: DECISION SIGHT DISTANCE

Design Speed (kph)	Decision Sight Distance for Avoidance Maneuver (meters)			
	A	B	C	D
50	70	155	145	195
60	95	195	170	235
70	115	235	200	275
80	140	280	230	315
90	170	325	275	360
100	200	370	315	400
110	235	420	330	430
120	265	470	360	470

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-3

Decision sight distance values that will be applicable to most situations have been developed from empirical data. The decision sight distances vary depending on whether the location is on a rural or urban road, and on the type of maneuver required to avoid hazards and negotiate the location properly. Table 4-3 shows decision sight distance values for various situations rounded for design. As can be seen in **Table 4.3**, generally shorter distances are required for rural roads and when a stop in the maneuver is involved. The decision sight distance are determined as:-

$$d = 0.278Vt + 0.039\frac{V^2}{a} \qquad \text{for maneuver A and B}$$

$$d = 0.278Vt \qquad \text{for maneuver C and D}$$

Where, **d** = Decision sight distance (m)
V = Design speed (kph)
t = Brake reaction time (s)
a = Deceleration rate (m/s)

The following avoidance maneuvers are covered in **Table 4.3**:

- Avoidance Maneuver A : Stop on rural road. [t =3.0s]
- Avoidance Maneuver B : Stop on urban road. [t = 9.1s]
- Avoidance Maneuver C : Speed/path/direction change on rural road. [t varies between 10.2s and 11.2s]
- Avoidance Maneuver D : Speed/path/direction change on urban road. [t varies between 14.0s and 14.5s

In computing and measuring decision sight distances, the 1050 mm eye height and 200 mm object height criteria used for stopping sight distance have been adopted. Although drivers may have to be able to see the entire roadway situation, including the road surface, the rationale for the 200 mm object height is as applicable for decision as it is for stopping sight distance.

4.1.4 Passing Sight Distance

Most roads in rural areas are two-lane two way on which vehicles frequently overtake slower moving vehicles, the passing of which must be accomplished on a lane regularly used by the opposing traffic. Passing sight distance for use in design should be determined on the basis of the length needed to safely complete a normal passing maneuver.

The minimum passing sight distance for two-lane highways is determined as the sum of four distances:-

(i) Distance traversed during the perception and reaction time and during the initial acceleration to the point of encroachment on the passing lane.

(ii) Distance travelled while the passing vehicle occupies the passing lane.

(iii) Distance between the passing vehicle at the end of its maneuver and the opposite vehicles.

(iv) Distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the passing lane.

(a) Design Values

The total passing sight distance is determined by the sum of the above four element. **Table 4.4** gives the minimum values to be used for each design speed.

In designing a road, these distances should be exceeded as much as practically possible and such sections provided as often as can be done with reasonable costs so as to present as many passing opportunities as possible.

TABLE 4.4: MINIMUM PASSING SIGHT DISTANCES (2-LANE, 2-WAY)

Design Speed (kph)	Min. Passing Sight Distance (m)
120	775
110	730
100	670
90	615
80	540
70	485
60	410
50	345
40	270
30	200

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-7

(b) Effect of Grade on Passing Sight Distance

Specific adjustment for design use is not available. The effect of grade is not considered in design as the effect is compensated in the upgrade or downgrade. However, it should be realized that greater distances are needed for passing on grades as compared to level conditions. The designer should recognize the desirability of increasing the minimum values as shown in **Table 4.4**.

(c) Frequency and Length of Passing Sections

Sight distance adequate for passing should be provided frequently on 2-lane roads. Designs with only minimum sight distance will not assure that safe passing can always be made. Even on low volume roads a driver desiring to pass may, upon reaching the section, find vehicles in the opposing lane and thus be unable to utilize the section or at least not be able to begin to pass at once.

The percentage of length of road or section of road with sight distance greater than the passing minimum should be computed. The adequacy of sight distance is determined by analysis of capacity related to this percentage, and this would indicate whether or not alignment and profile adjustments are necessary to accommodate the design hour volumes. Generally this percentage should be about 60% for flat terrain, 40% for rolling terrain and 20% for mountainous terrain. The available passing sight distances should not be concentrated in one section but be evenly distributed throughout the entire road. It is important to note that in order to cater for high volume of traffic at high Level of Service, it requires that frequent and nearly continuous passing sight distances be provided.

It is not necessary to consider passing sight distance on roads that have two or more traffic lanes in each direction of travel. Passing maneuvers on multilane roads are expected to occur within the limits of each carriageway allocated for that direction of travel. Thus passing maneuvers that require crossing the centerline of four-lane undivided road are reckless acts of driving.

4.1.5 Criteria for Measuring Sight Distance

(i) Height of Driver's Eye

The eyes of the average driver in a passenger vehicle are considered to be 1050mm above the road surface.

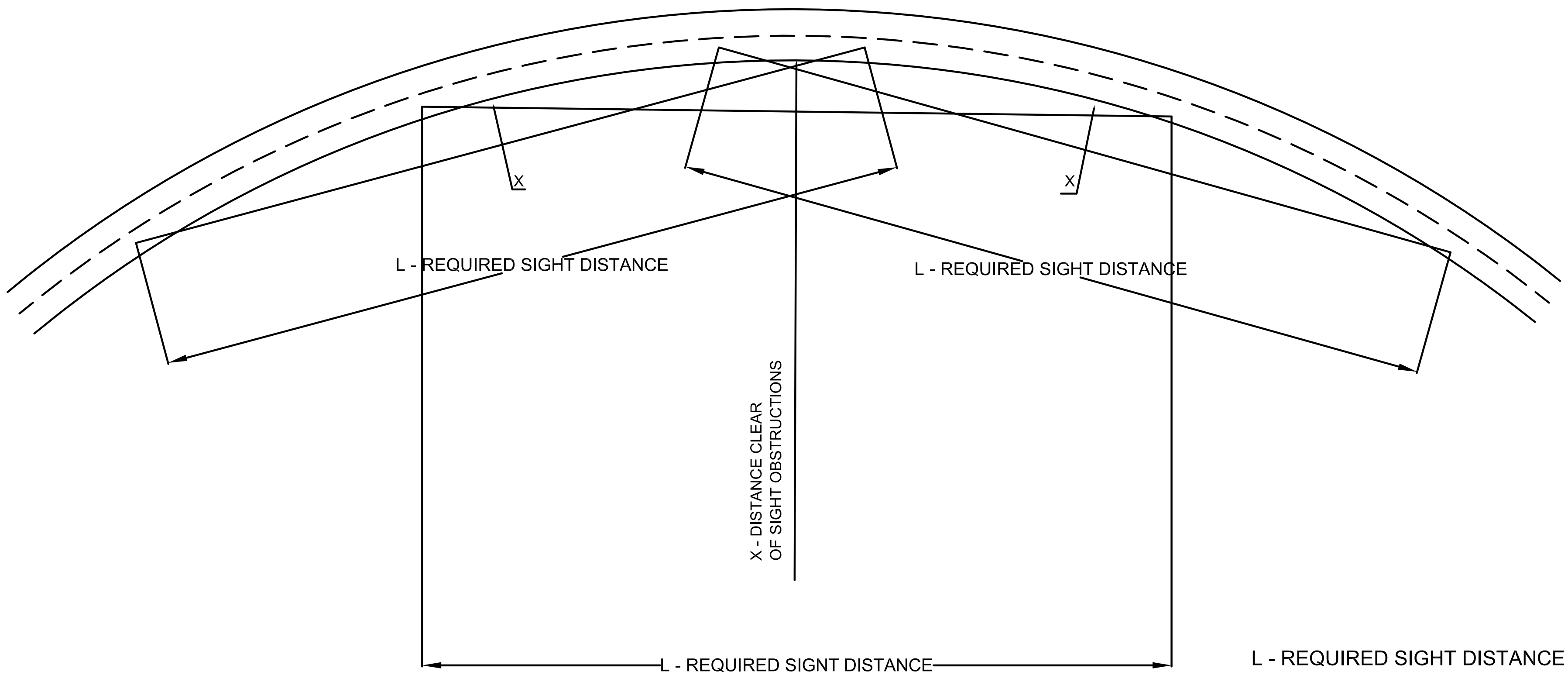
(ii) Height of Object

A height of object of 200mm is assumed for measuring stopping sight distance and the height of object for passing sight distance 1330mm both measured from the road surface.

Sight distance should be considered in the preliminary stages of design when both the horizontal and vertical alignment are still subject to adjustment. The sight distance should be determined graphically on the alignment plans and recorded at frequent intervals, for both directions of travel. This will enable the designer to appraise the overall layout and effect a more balanced design by minor adjustments in the plan or profile.

Horizontal sight distance should be measured on the inside of a curve at the center of the inside lane. Vertical sight distance should be measured along the longitudinal profile of the central line, using the height of driver's eye and the object height. **Figure 4.1** and **Figure 4.2** provide examples of measuring sight distance in both plan and profile respectively. For two lane two- way roads passing sight distance in addition to stopping sight distance and decision sight distance should be measured and recorded. **Sight distances, in this case, are to be checked from the centerline of the road**.

HORIZONTAL CURVE



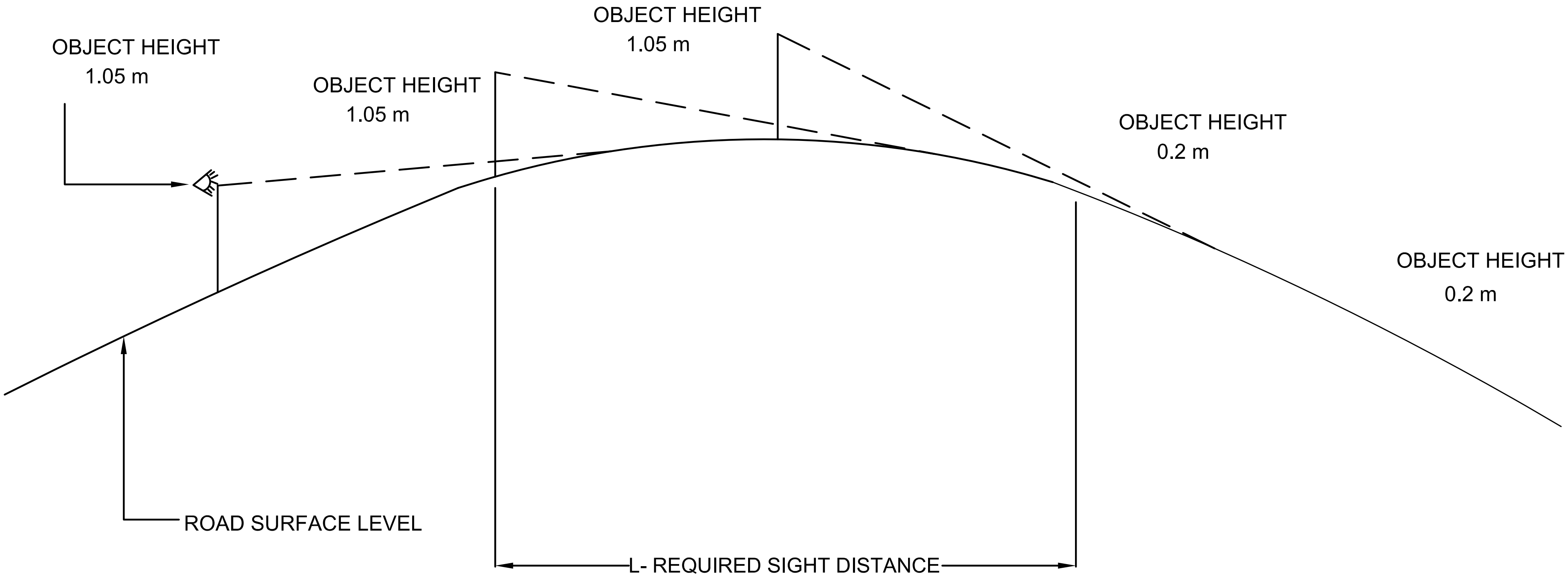
L - REQUIRED SIGHT DISTANCE
X - DISTANCE CLEAR OF SIGN OBSTRUCTIONS

Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 4-1

FIGURE 4.1 : MEASURING SIGHT DISTANCE ON PLAN

Scale : Not to scale

VERTICAL PROFILE



Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 4-2

FIGURE 4.2 : MEASURING SIGHT DISTANCE ON PROFILE

Scale : Not to scale

4.2 Horizontal Alignment

4.2.1 General

In the design of horizontal curves, it is necessary to establish the proper relationship between the design speed and curvature and also their joint relations with superelevation and side friction. From research and experience, limiting values have been established for the superelevation (e) and the coefficient of friction (f).

4.2.2 Superelevation Rates

The maximum rates of superelevation usable are controlled by several factors such as climatic conditions, terrain conditions and frequency of very slow moving vehicles that would be subjected to uncertain operation.

While it is acknowledged that a range of values should be used, for practical purposes in establishing the design criteria for horizontal alignment, a superelevation rate of between 6% and 10% can be used. For areas where climatic condition often causes slippery road surfaces and vehicles to operate at slow speed the superelevation should not exceed 8%.

4.2.3 Minimum Radius

The minimum radius is a limiting value of curvature for a given speed and is determined from the maximum rate of superelevation and the maximum allowable side friction factor.

The **minimum safe radius (Rmin)** can be calculated from the standard curve formula.

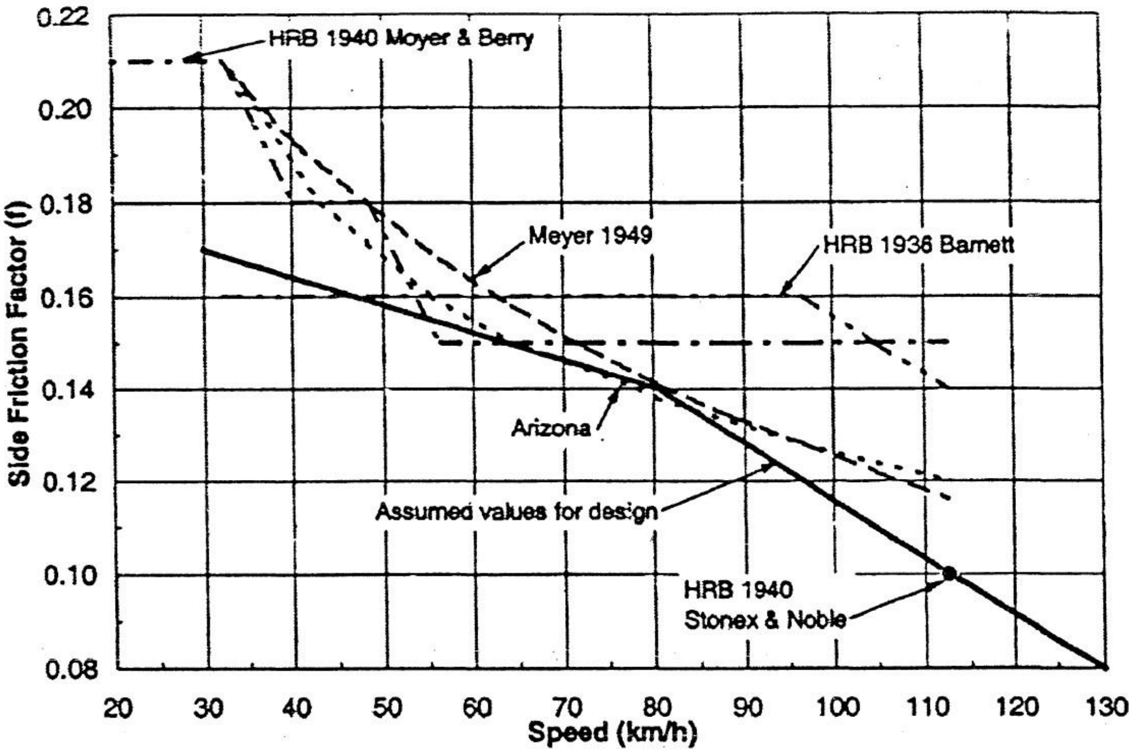
$$R_{min} = \frac{V^2}{127(e+f)}$$

Where,

	Rmin	=	Minimum radius of circular curve (m)
	V	=	Design Speed (kph)
	e	=	Maximum superelevation rate
	f	=	Maximum allowable side friction factor

Figure 4.3 illustrates the range of side friction factors (f) that has been derived using empirical methods for the various type of roads.

FIGURE 4.3: COMPARISON OF SIDE FRICTION FACTORS ASSUMED FOR DESIGN OF DIFFERENT TYPES OF ROADS



Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-13

Table 4.5 lists the minimum radius to be used for the designated speeds and maximum superelevation rates.

While the values in Table 4-5 lists the minimum radius that can be used, it should be noted, that the above minimum radius will not provide the required stopping sight distance and passing site distance within typical cross-sections. Hence, all efforts should be made to design the horizontal curves with radius larger than the minimum values shown for greater comfort and safety.

TABLE 4-5: MINIMUM RADIUS

Design Speed (kph)	Minimum Radius (m)		
	e = 0.06	e = 0.08	e = 0.10
120	755	665	595
110	560	500	455
100	435	395	360
90	335	305	275
80	250	230	210
70	195	175	160
60	135	125	115
50	90	80	75
40	55	50	45
30	30	30	25

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-14

4.2.4 Transition Design Control

The superelevation transition section consists of the superelevation runoff and tangent runout sections. The superelevation runoff section consists of the length of roadway needed to accomplish a change in outside lane cross slope from zero to full superelevation or vice versa. The tangent runout section consists of the length of roadway needed to accomplish a change in outside lane cross slope from normal cross slope rate to zero or vice versa. For reasons of safety and comfort, the pavement rotation in the superelevation transition section should be effected over a length that is sufficient to make such rotation imperceptible to drivers. To be pleasing in appearance, the pavement edges should not appear distorted to the driver.

(i) Spiral Curve Transition

Vehicles follow a transition path as it enters or leave a circular horizontal curve. To design a road with built-in safety, the alignment should be such that a driver travelling at the design speed will not only find it possible to confine his vehicle to the occupied lane but will be encouraged to do so. Spiral transition curves are used for this purpose. Generally, the Eulers spiral, also known as the clothoid is used. The degree of curve varies from zero at the tangent end of the spiral to the degree of the circular arc at the circular curve end.

Length of Spiral (L):-

The following formula is used by some for calculating the minimum length of a spiral :

$$L = \frac{0.0214V^3}{RC}$$

Where,	**L**	=	Minimum length of spiral
	V	=	Speed (kph)
	R	=	Curve radius
	C	=	Rate of increase of centripetal acceleration (m per sec^3)

The factor C is an empirical value and a range of 0.3 to 0.9 has been used for highways. Highway design does not normally require the level of precision achieved using this formula. A more practical control for determining the length of spiral is that which equals the length required for superelevation runoff. Current practice indicates that the appearance aspect of superelevation runoff largely governs the length.

(ii) Minimum Length of Superelevation Run-off

For appearance and comfort, the length of superelevation runoff should be based on a maximum acceptable difference between the longitudinal grades of the axis of rotation and the edge of pavement. The axis of rotation is generally represented by the alignment centerline for undivided roadways. The length of superelevation runoff should not exceed a longitudinal slope as indicated in **Table 4.6**.

TABLE 4-6: RELATIONSHIP OF DESIGN SPEED TO MAXIMUM RELATIVE PROFILE GRADIENTS

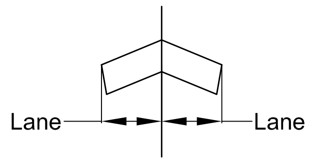
Design Speed V (kph)	Maximum Relative Gradient (and Equivalent Maximum Relative Slopes) for Profiles between the Edge of Two-Lane Travelled Way and the Centerline (%)
30	0.75 (1:133)
40	0.70 (1:143)
50	0.65 (1:154)
60	0.60 (1:167)
70	0.55 (1:182)
80	0.50 (1:200)
90	0.47 (1:213)
100	0.44 (1:227)
110	0.41 (1:244)
120	0.38 (1:263)

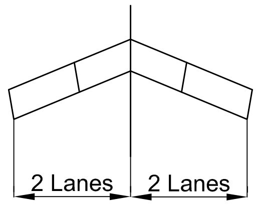
Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-27

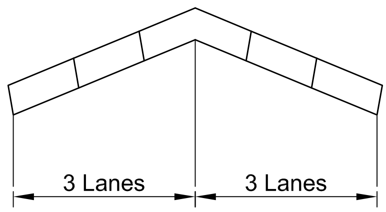
Table 4.7A, Table 4.7B and **Table 4.7C** gives the various design speeds and degree of curvature, the minimum lengths of superelevation runoff, the superelevation rates and the limiting curvatures for which superelevation is not required.

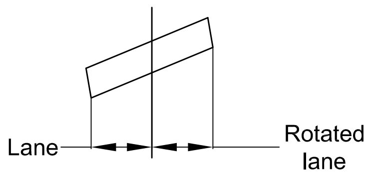
For pavements with more than 2 lanes and standard lane width of 3.6m, the super elevation runoff lengths should be as follows: -

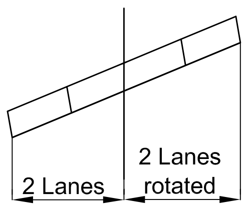
a) 3 lane pavements (1.5 lane rotated) - 1.25 times the length for 2-lane roads.
b) 4 lane undivided pavements (2 lanes rotated) - 1.5 times the length for 2-lane roads.
c) 6 lane undivided pavements (3lane rotated) - 2.0 times the length for 2-lane roads.

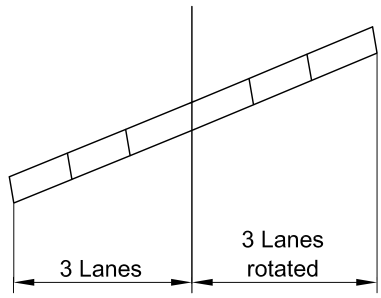
<table>
<tr><th>One Lane Rotated</th><th>Two Lanes Rotated</th><th>Three Lanes Rotated</th></tr>
<tr>
<td>

Normal Section</td>
<td>

Normal Section</td>
<td>

Normal Section</td>
</tr>
<tr>
<td>

Rotated section</td>
<td>

Rotated section</td>
<td>

Rotated section</td>
</tr>
</table>

Source : AASHTO - A Policy on Geometric Design of Highways and Street (2011), Table 3-16

FIGURE 4.4: ILLUSTRATION OF NUMBER OF LANES ROTATED

Scale : Not to scale

TABLE 4-7A : DESIGN SUPERELEVATION TABLE

R (m)	V = 30kph				V = 40kph				V = 50kph				V = 60kph				V = 70kph			
	e (%)	2 L(m)	4 L(m)	6 L(m)	e (%)	2 L(m)	4 L(m)	6 L(m)	e (%)	2 L(m)	4 L(m)	6 L(m)	e (%)	2 L(m)	4 L(m)	6 L(m)	e (%)	2 L(m)	4 L(m)	6 L(m)
7000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
5000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
3000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
2500	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
2000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
1500	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	17	26	34
1400	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	15	23	30	RC	17	26	34
1300	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	15	23	30	RC	17	26	34
1200	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	15	23	30	RC	17	26	34
1000	NC	0	0	0	NC	0	0	0	RC	14	21	28	RC	15	23	30	2.6	17	26	34
900	NC	0	0	0	NC	0	0	0	RC	14	21	28	RC	15	23	30	2.8	18	27	36
800	NC	0	0	0	NC	0	0	0	RC	14	21	28	RC	15	23	30	3.1	20	30	40
700	NC	0	0	0	RC	13	19	26	RC	14	21	28	2.8	17	25	34	3.4	22	33	44
600	NC	0	0	0	RC	13	19	26	RC	14	21	28	3.1	19	28	38	3.8	25	37	50
500	NC	0	0	0	RC	13	19	26	2.8	15	23	30	3.5	21	32	42	4.2	27	41	54
400	RC	10	14	20	RC	13	19	26	3.3	18	27	36	4	24	36	48	4.7	31	46	62
300	RC	10	14	20	3.1	16	24	32	3.9	22	32	44	4.6	28	41	56	5.4	35	53	70
250	RC	10	14	20	3.5	18	27	36	4.2	23	35	46	5	30	45	60	5.8	38	57	76
200	2.8	13	20	26	3.9	20	30	40	4.7	26	39	52	5.5	33	50	66	6	39	59	78
175	3	14	22	28	4.1	21	32	42	5	28	42	56	5.8	35	52	70	R min = 195			
150	3.3	16	24	32	4.4	23	34	46	5.3	29	44	58	6	36	54	72				
140	3.5	17	25	34	4.5	23	35	46	5.4	30	45	60	6	36	54	72				
130	3.6	17	25	34	4.6	24	35	48	5.6	31	47	62	R min = 135							
120	3.8	18	27	36	4.8	25	37	50	5.7	32	47	64								
110	3.9	19	28	38	5	26	39	52	5.8	32	48	64								
100	4.1	20	30	40	5.2	27	40	54	6	33	50	66								
90	4.2	20	30	40	5.4	28	42	56	6	33	50	66								
80	4.5	22	32	44	5.6	29	43	58	R min = 90											
70	4.7	23	34	46	5.8	30	45	60												
60	5	24	36	48	6	31	46	62												
50	5.4	26	39	52	R min = 55															
40	5.8	28	42	56																
30	6	29	43	58																
	R min = 30																			

R (m)	V = 80kph				V = 90kph				V = 100kph				V = 110kph				V = 120kph			
	e (%)	2 L(m)	4 L(m)	6 L(m)	e (%)	2 L(m)	4 L(m)	6 L(m)	e (%)	2 L(m)	4 L(m)	6 L(m)	e (%)	2 L(m)	4 L(m)	6 L(m)	e (%)	2 L(m)	4 L(m)	6 L(m)
7000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
5000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
3000	NC	0	0	0	NC	0	0	0	RC	20	31	40	RC	23	34	46	RC	24	37	48
2500	NC	0	0	0	RC	19	28	38	RC	20	31	40	RC	23	34	46	2.7	26	38	52
2000	RC	18	27	36	RC	19	28	38	RC	20	31	40	2.8	25	37	50	3.3	31	47	62
1500	RC	18	27	36	2.7	21	31	42	3.1	25	38	50	3.6	32	47	64	4.2	40	60	80
1400	RC	18	27	36	2.8	21	32	42	3.3	27	41	54	3.8	33	50	66	4.4	42	63	84
1300	2.7	19	29	38	3	23	34	46	3.5	29	43	58	4	35	53	70	4.7	45	67	90
1200	3.1	22	33	44	3.2	25	37	50	3.7	30	45	60	4.2	37	55	74	5	47	71	94
1000	3.4	24	37	48	3.6	28	41	56	4.2	34	52	68	4.8	42	63	84	5.6	53	80	106
900	3.6	26	39	52	3.9	30	45	60	4.5	37	55	74	5.1	45	67	90	5.8	55	82	110
800	4	29	43	58	4.2	32	48	64	4.9	40	60	80	5.4	47	71	94	6	57	85	114
700	4.3	31	46	62	4.6	35	53	70	5.2	43	64	86	5.8	51	76	102	R min = 755			
600	4.8	35	52	70	5	38	57	76	5.6	46	69	92	6	53	79	106				
500	5.3	38	57	76	5.4	41	62	82	5.9	48	72	96	R min = 560							
400	5.9	42	64	84	5.9	45	68	90	R min = 435											
300	6	43	65	86	R min = 335															
250	R min = 250																			

e max	=	6%
R	=	Radius of curve
Vd	=	Design speed
L	=	Minimum length of runoff (excluding tangent runout) length are for 3.6m lane width and the respective maximum relative slope for each design speed
NC	=	Normal crown 2.5%
RC	=	Remove adverse crown, SE at normal crown slope

Notes:-

The RC row presents minimum radii for a computed SE rate of 2.5%

The row NC designates a travel way cross section used on curves that are so flat that the normal cross slope can be used.

For curve radii falling between NC and RC, a plane slope across the entire pavement equal to NC should be used.

Source: Adapted from AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-22

TABLE 4-7B : DESIGN SUPERELEVATION TABLE

R (m)	V = 30kph				V = 40kph				V = 50kph			
	e	2	4	6	e	2	4	6	e	2	4	6
	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)
7000	NC	0	0	0	NC	0	0	0	NC	0	0	0
5000	NC	0	0	0	NC	0	0	0	NC	0	0	0
3000	NC	0	0	0	NC	0	0	0	NC	0	0	0
2500	NC	0	0	0	NC	0	0	0	NC	0	0	0
2000	NC	0	0	0	NC	0	0	0	NC	0	0	0
1500	NC	0	0	0	NC	0	0	0	NC	0	0	0
1400	NC	0	0	0	NC	0	0	0	NC	0	0	0
1300	NC	0	0	0	NC	0	0	0	NC	0	0	0
1200	NC	0	0	0	NC	0	0	0	NC	0	0	0
1000	NC	0	0	0	NC	0	0	0	RC	14	21	28
900	NC	0	0	0	NC	0	0	0	RC	14	21	28
800	NC	0	0	0	NC	0	0	0	RC	14	21	28
700	NC	0	0	0	RC	13	20	26	RC	14	21	28
600	NC	0	0	0	RC	13	20	26	2.6	14	21	28
500	NC	0	0	0	RC	13	20	26	3.0	17	25	34
400	RC	12	18	24	2.7	14	21	28	3.6	20	30	40
300	RC	12	18	24	3.4	17	26	34	4.5	25	37	50
250	2.5	12	18	24	4.0	21	31	42	5.1	26	42	52
200	3.0	14	22	28	4.6	24	35	48	5.8	32	48	64
175	3.4	16	24	32	5.0	26	39	52	6.2	34	52	68
150	3.8	18	27	36	5.4	28	42	56	6.7	37	56	74
140	4.0	19	29	38	5.6	29	43	58	6.9	38	57	76
130	4.2	20	30	40	5.8	30	45	60	7.1	39	59	78
120	4.4	21	32	42	6.0	31	46	62	7.4	41	61	82
110	4.7	23	34	46	6.3	32	49	64	7.6	42	63	84
100	5.0	24	36	48	6.6	34	51	68	7.8	43	65	86
90	5.2	25	37	50	6.9	35	53	70	7.9	44	66	88
80	5.5	26	40	52	7.2	37	56	74	8.0	44	66	88
70	5.9	28	42	56	7.5	39	58	78	R min = 80			
60	6.4	31	46	62	7.8	40	60	80				
50	6.9	33	50	66	8.0	41	62	82				
40	7.5	36	54	72	R min = 50							
30	8	38	58	76								
	R min = 30											

R (m)	V = 60kph				V = 70kph				V = 80kph			
	e	2	4	6	e	2	4	6	e	2	4	6
	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)
7000	NC	0	0	0	NC	0	0	0	NC	0	0	0
5000	NC	0	0	0	NC	0	0	0	NC	0	0	0
3000	NC	0	0	0	NC	0	0	0	NC	0	0	0
2500	NC	0	0	0	NC	0	0	0	NC	0	0	0
2000	NC	0	0	0	NC	0	0	0	RC	18	27	36
1500	RC	15	23	30	RC	17	25	34	RC	18	27	36
1400	RC	15	23	30	RC	17	25	34	RC	18	27	36
1300	RC	15	23	30	RC	17	25	34	2.7	19	29	38
1200	RC	15	23	30	RC	17	25	34	2.9	21	31	42
1000	RC	15	23	30	2.8	18	27	36	3.4	24	37	48
900	RC	15	23	30	3.1	20	30	40	3.7	27	40	54
800	RC	15	23	30	3.4	22	33	44	4.1	30	44	60
700	2.7	15	23	30	3.8	25	37	50	4.5	32	49	64
600	3.0	18	27	36	4.3	28	42	56	5.1	37	55	74
500	3.4	20	31	40	4.9	32	48	64	5.8	42	63	84
400	3.9	23	35	46	5.7	37	56	74	6.6	48	71	96
300	4.7	28	42	56	6.7	44	66	88	7.6	55	82	110
250	5.6	34	50	68	7.4	48	73	96	7.9	57	85	114
200	6.2	37	56	74	7.9	52	78	104	R min = 230			
175	7.0	42	63	84	8.0	52	79	104				
150	7.4	44	67	88	R min = 175							
140	7.8	47	70	94								
130	7.9	47	71	94								
120	8.0	48	72	96								
110	R min = 125											

R (m)	V = 90kph				V = 100kph				V = 110kph				V = 120kph			
	e	2	4	6	e	2	4	6	e	2	4	6	e	2	4	6
	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)
7000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
5000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
3000	NC	0	0	0	RC	21	32	42	RC	22	33	44	RC	24	36	48
2500	RC	19	29	38	RC	21	32	42	RC	22	33	44	2.9	27	41	54
2000	RC	19	29	38	2.6	21	32	42	3.0	26	40	52	3.5	33	50	66
1500	2.8	21	32	42	3.4	28	42	56	3.9	34	51	68	4.6	44	65	88
1400	3.0	23	34	46	3.6	29	44	58	4.1	36	54	72	4.9	46	70	92
1300	3.2	25	37	50	3.8	31	47	62	4.4	39	58	78	5.2	49	74	98
1200	3.4	26	39	52	4.1	34	50	68	4.7	41	62	82	5.6	53	80	106
1000	4.0	31	46	62	4.8	39	59	78	5.5	48	72	96	6.5	62	92	124
900	4.4	34	51	68	5.2	43	64	86	6.0	53	79	106	7.1	67	101	134
800	4.8	37	55	74	5.7	47	70	94	6.6	58	87	116	7.6	72	108	144
700	5.3	41	61	82	6.3	52	77	104	7.2	63	95	126	8.0	76	114	152
600	6.0	46	69	92	6.9	55	85	110	7.7	68	101	136	R min = 665			
500	6.7	51	77	102	7.6	62	93	124	8.0	70	105	140				
400	7.5	57	88	114	8.0	65	98	130	R min = 500							
300					R min = 396											
250	R min = 305															

e max	=	8%
R	=	Radius of curve
Vd	=	Design speed
L	=	Minimum length of runoff (excluding tangent runout) length are for 3.6m lane width and the respective maximum relative slope for each design speed
NC	=	Normal crown 2.5%
RC	=	Remove adverse crown, SE at normal crown slope

Notes:-

The RC row presents minimum radii for a computed SE rate of 2.5%

The row NC designates a travel way cross section used on curves that are so flat that the normal cross slope can be used.

For curve radii falling between NC and RC, a plane slope across the entire pavement equal to NC should be used.

Source: Adapted from AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-23

TABLE 4-7C : DESIGN SUPERELEVATION TABLE

R (m)	V = 30kph				V = 40kph				V = 50kph				V = 60kph				V = 70kph				V = 80kph				V = 90kph				V = 100kph				V = 110kph				V = 120kph			
	e	2	4	6	e	2	4	6	e	2	4	6	e	2	4	6	e	2	4	6	e	2	4	6	e	2	4	6	e	2	4	6	e	2	4	6	e	2	4	6
	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)	(%)	L(m)	L(m)	L(m)
7000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
5000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0
3000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	20	30	40	RC	22	33	44	RC	24	36	48
2500	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	19	28	38	2.7	22	33	44	3.1	27	41	54	3.6	34	51	68
2000	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	17	26	34	RC	19	28	38	3.5	29	43	58	4.1	36	54	72	4.8	46	68	92
1500	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	15	23	30	RC	17	26	34	RC	17	26	34	2.9	22	33	44	3.8	31	47	62	4.3	38	57	76	5.1	48	72	96
1400	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	15	23	30	RC	17	26	34	2.6	19	28	38	3.1	24	36	48	4	33	49	66	4.6	40	61	80	5.5	52	78	104
1300	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	15	23	30	RC	17	26	34	2.8	20	30	40	3.3	25	38	50	4.3	35	53	70	5	44	66	88	5.9	56	84	112
1200	NC	0	0	0	NC	0	0	0	NC	0	0	0	RC	15	23	30	RC	17	26	34	3	22	32	44	3.6	28	41	56	5.1	42	63	84	5.9	52	78	104	7	66	99	132
1000	NC	0	0	0	NC	0	0	0	RC	15	22	30	RC	15	23	30	2.9	19	28	38	3.5	25	38	50	4.2	32	48	64	5.6	46	69	92	6.4	55	84	110	7.7	73	109	146
900	NC	0	0	0	NC	0	0	0	RC	15	22	30	2.6	15	23	30	3.2	21	31	42	3.9	28	42	56	4.6	35	53	70	6.2	51	76	102	7.1	62	94	124	8.5	81	121	162
800	NC	0	0	0	NC	0	0	0	RC	15	22	30	2.7	16	24	32	3.5	23	34	46	4.3	31	46	62	5.1	39	59	78	6.9	56	85	112	8	70	105	140	9.5	90	135	180
700	NC	0	0	0	RC	14	22	28	RC	15	22	30	3.1	19	28	38	4	26	39	52	4.8	35	52	70	5.8	44	67	88	7.8	64	95	128	9	79	119	158	10	95	142	190
600	NC	0	0	0	RC	14	22	28	2.7	15	22	30	3.6	22	32	44	4.5	29	44	58	5.5	40	59	80	6.5	50	75	100	8.9	73	109	146	9.9	87	130	174	R min = 595			
500	NC	0	0	0	RC	14	22	28	3.1	17	26	34	4.2	25	36	50	5.3	35	52	70	6.4	46	69	92	7.6	58	87	116	9.8	80	120	160	R min = 455							
400	RC	12	19	24	2.8	14	22	28	3.8	21	32	42	5	30	45	60	6.3	41	62	82	7.5	54	81	108	8.8	67	101	134	R min = 360											
300	RC	12	19	24	3.6	19	28	38	4.8	27	40	54	6.3	38	57	76	7.8	51	77	102	9	65	97	130	9.9	76	114	152												
250	2.6	12	19	24	4.2	22	32	44	5.6	31	47	62	7.1	43	64	86	8.7	57	85	114	9.7	70	105	140	R min = 275															
200	3.1	15	22	30	5	26	39	52	6.6	37	55	74	8.2	49	74	98	9.6	63	94	126	R min = 210																			
175	3.5	17	25	34	5.6	29	43	58	7.1	39	59	78	8.8	53	79	106	9.9	65	97	130																				
150	4	19	29	38	6.2	32	48	64	7.8	43	65	86	9.4	56	85	112	R min = 160																							
140	4.3	21	31	42	6.4	33	49	66	8.1	45	67	90	9.7	58	87	116																								
130	4.5	22	32	44	6.7	34	52	68	8.5	47	71	94	9.8	59	88	118																								
120	4.8	23	35	46	7	36	54	72	8.8	49	73	98	10	60	90	120																								
110	5.1	24	37	48	7.4	38	57	76	9.1	50	76	100	R min = 115																											
100	5.5	26	40	52	7.7	40	59	80	9.5	53	79	106																												
90	5.9	28	42	56	8.2	42	63	84	9.8	54	81	108																												
80	6.4	31	46	62	8.6	44	66	88	10	55	83	110																												
70	6.9	33	50	66	9.1	47	70	94	R min = 75																															
60	7.5	36	54	72	9.6	49	74	98																																
50	8.2	39	59	78	10	51	77	102																																
40	9.1	44	65	88	R min = 45																																			
30	9.9	48	71	96																																				
	R min = 25																																							

e max	=	10%
R	=	Radius of curve
Vd	=	Design speed
L	=	Minimum length of runoff (excluding tangent runout) length are for 3.6m lane width and the respective maximum relative slope for each design speed
NC	=	Normal crown 2.5%
RC	=	Remove adverse crown, SE at normal crown slope

Notes:-

The RC row presents minimum radii for a computed SE rate of 2.5%

The row NC designates a travel way cross section used on curves that are so flat that the normal cross slope can be used.

For curve radii falling between NC and RC, a plane slope across the entire pavement equal to NC should be used.

Source: Adapted from AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-24

(iii) Desirable Length of Spiral

Recent study of operational effects of spiral curve transitions found that spiral length is an important design control. Specifically, the most desirable operating conditions were noted when the spiral curve length was approximately equal to the length of the natural spiral path adopted by drivers. Differences between these two lengths resulted in operational problems associated with large lateral velocities or shifts in lateral position at the end of the transition curve.

Based on these considerations, desirable lengths of spiral transition curve are as shown in **Table 4.8**. These lengths correspond to 2 secs of travel time at the design speed of the roadway. This travel time has been found to be representative of the natural path for most drivers.

Table 4-8: Desirable Length of Spiral Curve Transition *

Design Speed (km/h)	Spiral Length (m)
30	17
40	22
50	28
60	33
70	39
80	44
90	50
100	56
110	61
120	67

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-34

*** Note:**
All effort should be made to use higher values

4.2.5 Methods of Attaining Superelevation

Three specific methods of profile design in attaining superelevation are (a) revolving the pavement about the centerline profile (b) revolving the pavement about the inside edge profile, and (c) revolving the pavement about the outside edge profile. **Figure 4.5** illustrates these three methods diagrammatically. The rate of cross slope is proportional to the distance from start of superelevation runoff.

(i) **Figure 4.5A** illustrates the method where the pavement section is revolved about the centerline profile. This general method is the most widely used in design since the required change in elevation of edge of pavement is made with less distortion than by the other methods. The centerline profile is the base line and one-half of the required elevation change is made at each edge.

(ii) **Figure 4.5B** illustrates the method where the pavement section is revolved about the inside edge profile. The inside edge profile is determined as the line parallel to the calculated centerline profile, One half of the required change in cross slope is made by raising the center line profile with respect to the inside pavement edge and the other half by raising the respect to the centerline profile.

(iii) **Figure 4.5C** illustrates the method where the pavement section is revolved about outside edge profile and involves similar geometrics as (b), except that the change is affected below the upper control profile.

Except when site condition specifically requires, method (a) shall be adopted for undivided roads.

4.2.6 Superelevation Runoff with Medians

In the design of divided roads, the inclusion of a median in the cross section alters somewhat the superelevation runoff treatment. The three general cases for superelevation runoff design are as follows:

(i) The whole of the travelled way, including the median, is super elevated as a plane section. This case is limited to narrow medians and moderate superelevation rates to avoid substantial differences in elevation of the extreme pavement edges because of the median tilt. Diagrammatic profile controls is similar to **Figure 4.5A** except that the two median edges will appear as profiles only slightly removed from centerline.

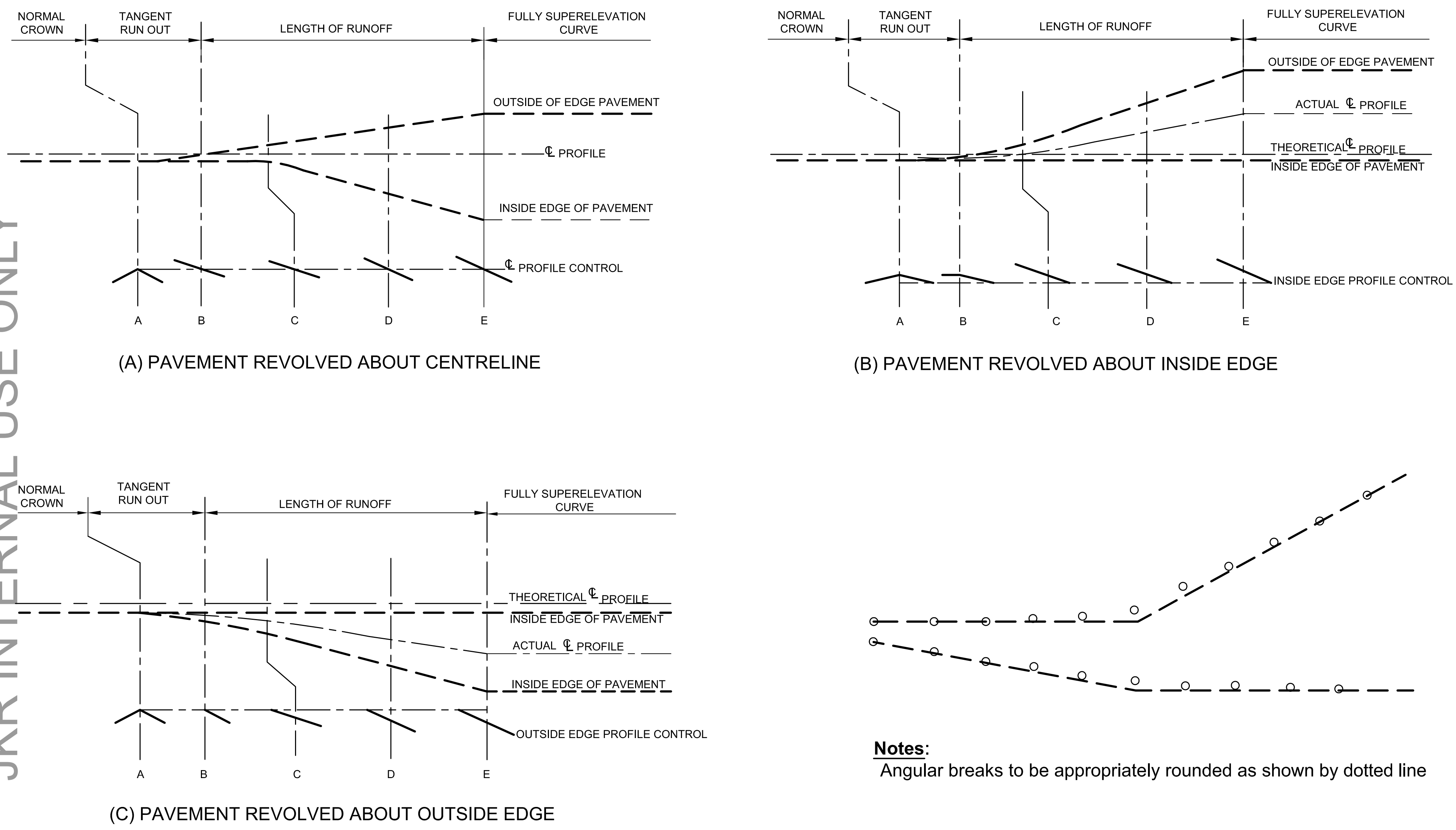
(ii) The median is held in a horizontal plane and the two pavements separately are rotated around the median edges. This case has most application with medians of intermediate width. Runoff design uses the median edge profile as the control. One pavement is rotated about its lower edge and the other about its higher edge. The diagrammatic profile control is similar to **Figure 4.5B** and **Figure 4.5C** with the centerline grade control the same for the two pavements.

(iii) The two pavements are separately treated for run-off with a resultant variable difference in elevation at the median edges. The differences in elevation of the extreme pavement edges are minimized by a compensating slope across the median. A fairly wide section is necessary to develop right shoulder areas and desired gentle slope between. Thus this case is more applicable to wide median of 9m or more. The pavement rotation can be made by any of the methods as shown in **Figure 4.5**.

4.2.7 Pavement Widening on Curves

Pavements on curves are widened to make operating conditions on curve comparable to those on tangents. Pavement widening on curves is the difference in pavement width required on a curve and that used in a tangent. **Table 4.9** gives the widths of pavement widening that are required on open road curves.

Widening should be attained gradually on the approaches to the curve to ensure a reasonable smooth alignment on the edge of pavement and to fit the paths of vehicles entering or leaving the curve. Preferably, widening should be attained over the superelevation runoff length with most of all of the widening attained at the start of circular curve point



Notes:
Angular breaks to be appropriately rounded as shown by dotted line

Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 4-4

FIGURE 4.5 : DIAGRAMMATIC PROFILES SHOWING METHODS OF ATTAINING SUPERELEVATION

Scale : Not to scale

TABLE 4.9: PAVEMENT WIDENING ON OPEN ROAD CURVES

Pavement Width (m)	7				6.5				5.5				Required Widening (m)
Design Speed (kph)	80	60	50	40	60	50	40	30	50	40	30	20	
	R ≥ 470	R ≥ 340	R ≥ 280	R ≥ 230	R ≥1100	R ≥ 880	R ≥ 680	R ≥ 520	R ≥ 100	R ≥ 68	R ≥ 52	R ≥ 39	None
	470 > R ≥ 280	340 > R ≥ 180	280 > R ≥ 150	230 > R ≥ 130	1100 > R ≥ 340	880 > R ≥ 280	670 > R ≥ 230	520 > R ≥ 190		68 > R ≥ 60	52 > R ≥ 35	39 > R ≥ 27	0.50
		180 > R ≥ 150	150 > R ≥ 100	130 > R ≥ 130	340 > R ≥ 180	280 > R ≥ 150	230 > R ≥ 130	190 > R ≥ 110				27 > R ≥ 20	0.75
				86 > R ≥ 64	180 > R ≥ 150	150 > R ≥ 100	130 > R ≥ 86	110 > R ≥ 74				20 > R ≥ 16	1.00
				64 > R ≥ 60			86 > R ≥ 65	57 > R ≥ 45				16 > R ≥ 15	1.25
							65 > R ≥ 60	45 > R ≥ 38					1.50
								38 > R ≥ 35					1.75
													2.00

* Note:

R = Radius in m.

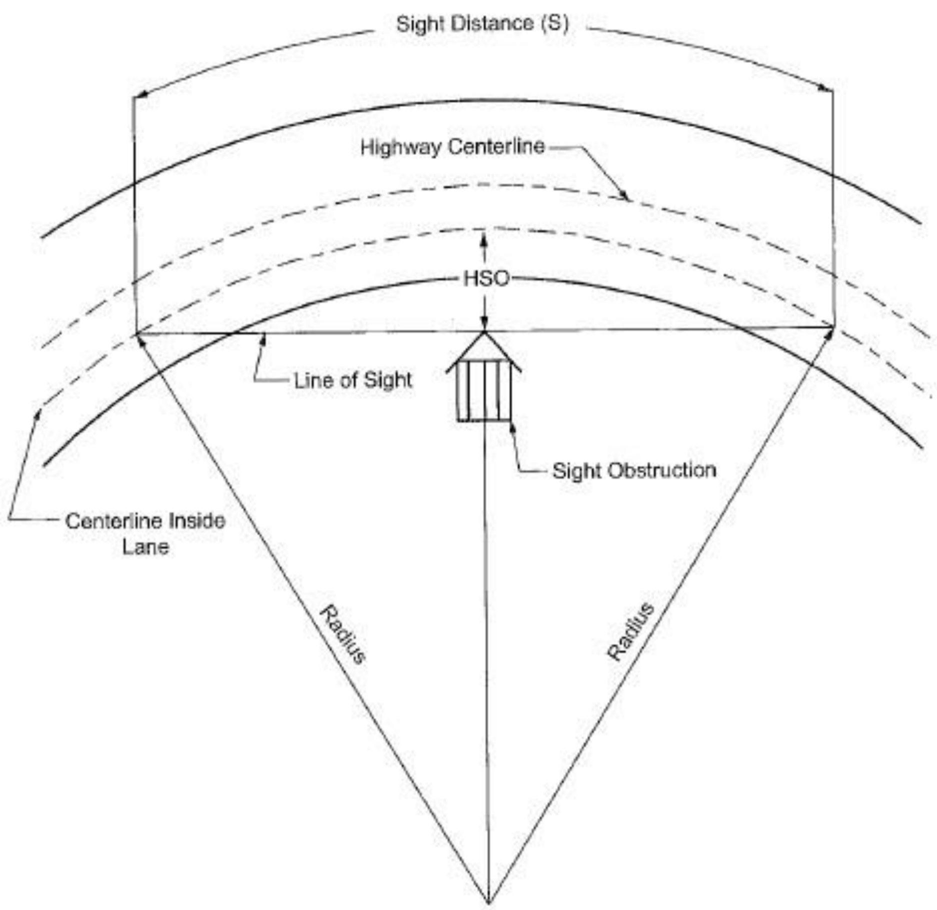
Source: REAM GL 2/2002: A Guide on Geometric Design of Roads, Table 4-8

4.2.8 Sight Distance on Horizontal Curves

Another element of horizontal alignment is the sight distance across the inside of curves. Where there are sight obstructions (such as walls, cut slopes, buildings and guardrails), a design to provide adequate sight distance may require adjustment if the obstruction cannot be removed. Using the design speed and a selected sight distance as a control, the designer should check the actual condition and make necessary adjustment in the manner most fitting to provide adequate sight distance.

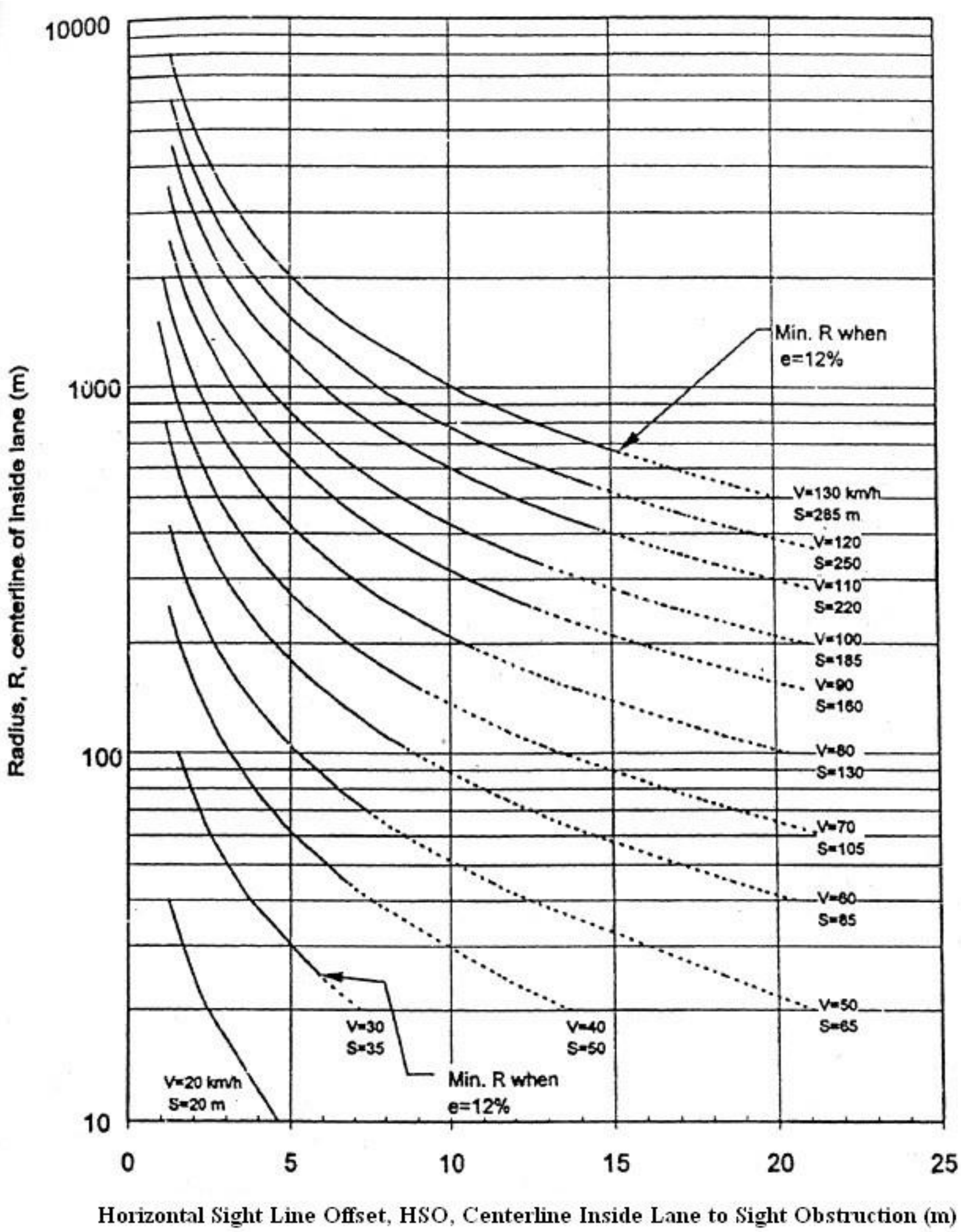
Figure 4.6A and **Figure 4.6B** can be used to easily check the Horizontal Sight Line Offset (HSO) needed for clear sight areas that satisfy SSD criteria for horizontal curve of various radii.

Figure 4.6A: Components for Determining Sight Distance on Horizontal Curve



Source: AASHTO – A Policy on Geometric Design of Highways and Street (2011), Figure 3-23

Figure 4.6B: Design Control for SSD on Horizontal Curve



Source: AASHTO – A Policy on Geometric Design of Highways and Street (2011), Figure 3-22a

4.2.9 General Controls for Horizontal Alignment

In addition to the specific design elements for horizontal alignment, a number of general controls are recognized and should be used. These controls are not subject to empirical or formula derivation but are important for the attainment of efficient and smooth-flowing roads. These are :

(i) The horizontal alignment should be consistent with the topography and width, preserving developed properties and community values. Winding alignment composed of short curves should be avoided. On the other hand very long straights should, likewise, also be avoided. The maximum length of straight section of the highway should be limited, where possible, to 2 minutes travelling time.

(ii) The use of the minimum radius for the particular design speed should be avoided wherever possible. Generally flat curves should be used, retaining the maximum curvature for the most critical conditions.

(iii) Consistent alignment should always be sought. Sharp curves should not be introduced at the end of long tangents. Where sharp curves must be introduced, it should be approached, where possible, by successively sharper curves from the generally flat curvature.

(iv) For small deflection angles, curves should be sufficiently long to avoid the appearance of a kink. Curves should be at least 150m long for a central angle of 5° and the length should be increased 30m for each 1°decrease in the centerline angle. The minimum length of horizontal curve on main roads, should be about 3 times the design speed, L_{cmin}= 3V (in meter). On high speed controlled access facilities the desirable minimum length of horizontal curve should be double the minimum length i.e. L_{cdes}= 6V.

(v) Any abrupt reversal in alignment should be avoided. The distance between reverse curves should be the sum of the superelevation runoff lengths and the tangent run-out lengths.

(vi) The 'broken back' arrangement of curves should be avoided. Use of spiral transitions or a compound curve alignment is preferable for such conditions if it is unavoidable.

(vii) Other than tangent, flat curvature should also be avoided on high, long embankment fills. In the absence of cut slopes, shrubs, and tree above the roadway, it is difficult for drivers to perceive the extent of curvature and adjust their vehicle operation to suit the prevalent conditions.

(viii) Caution should be exercised in the use of compound circular curves. While the use of compound curves affords flexibility in fitting the highway to the terrain and other ground controls, the simplicity with which such curves can be used often tempts the designer to use them without restraint. Preferably their use should be avoided where curves are sharp. Compound curves with large differences in curvature will introduce the same problems that arise at a tangent approach to a circular curve. Where topography or right-of-way restriction make their use necessary, the radius of the flatter circular arc, R_1, should not be more than 50 percent greater than the radius of the sharper circular arc, R_2, i.e., R_1 should not exceed 1.5 R_2. A several-step compound curve on this basis is suitable as a form of transition to sharp curves. A spiral transition between flat curves and sharp curves is even more desirable. On one-way roads such as ramps, the difference in radii of compound curves is not so important if the second curve is flatter than the first. However, the use of compound curves on ramps, which results in a flat curve between two sharper curves, is not good practice.

(ix) The designer should ensure that the required sight distance (i.e. decision sight distance) is provided when approaching interchanges and intersections.

4.3 Vertical Alignment

4.3.1 Maximum Grades

The vertical profile of road affects the performance of vehicles. The effect of grades on trucks which have weight power ratio of about 180 kg/kW, is considered. The maximum grade controls in terms of design speed is summarised in **Table 4.10A** to **Table 4.10F**.

TABLE 4.10A: MAXIMUM GRADES FOR R1, R2, U1 & U2 STANDARD ROADS

Type of Terrain / Area	Design Speed (kph)					
	30	40	50	60	70	80
Flat & Type I (%)	8	7	7	7	7	6
Rolling & Type II (%)	11	11	10	10	9	8
Mountainous & Type III (%)	16	15	14	13	12	10

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 5-4

TABLE 4.10B: MAXIMUM GRADES FOR R3 & R4 STANDARD ROADS

Type of Terrain	Design Speed (kph)					
	50	60	70	80	90	100
Flat (%)	7	7	7	6	6	5
Rolling (%)	9	8	8	7	7	6
Mountainous (%)	10	10	1	9	9	8

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 6-4

TABLE 4.10C: MAXIMUM GRADES FOR U3 & U4 STANDARD ROADS

Area Type	Design Speed (kph)				
	40	50	60	70	80
Type I (%)	9	9	9	8	7
Type II (%)	12	11	10	9	8
Type III (%)	13	12	12	11	10

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 6-8

TABLE 4.10D: MAXIMUM GRADES FOR R5 STANDARD ROADS

Type of Terrain	Design Speed (kph)					
	60	70	80	90	100	110
Flat (%)	5	5	4	4	3	3
Rolling (%)	6	6	5	5	4	4
Mountainous (%)	8	7	7	6	6	5

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 7-2

TABLE 4.10E: MAXIMUM GRADES FOR U5 STANDARD ROADS

Area Type	Design Speed (kph)					
	50	60	70	80	90	100
Type I (%)	8	7	6	6	5	5
Type II (%)	9	8	7	7	6	6
Type III (%)	11	10	9	9	8	8

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 7-10

TABLE 4.10F: MAXIMUM GRADES FOR R6 & U6 STANDARD ROADS

Type of Terrain / Area	Design Speed (kph)			
	80	90	100	110
Flat & Type I (%)	4	4	3	3
Rolling & Type II (%)	5	5	4	4
Mountainous &Type III (%)	*6*	*6*	*6*	*5*

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 8-1

The total upgrade for any section of road should not exceed 3000 m unless the grade is less than 4 %.

4.3.2 Minimum Grades

A desirable minimum grade or 0.5 percent should be used. A grade of 0.35 percent may be allowable where a high type pavement accurately crowned is used. On straight stretches traversing across wide areas of low lying swamps, use of even flatter grades may be allowable with prior acknowledgement and approval. However, the design of the storm water drainage outlet should be considered carefully at these locations to ensure that flooding of the travelled lanes is avoided.

Where less than 0.5% longitudinal gradient is necessary, the superelevation runoff length used should be the minimum value permitted. This will minimise flat area along the carriageway where the cross falls are less than 1%. The use of open texture wearing courses at these locations should be considered; they will help in preventing hydroplaning.

4.3.3 Critical Grade Length

The term "critical grade length" indicates the maximum length of a designated upgrade upon which a loaded truck can operate without an unreasonable reduction in speed. To establish the design values for critical grade lengths, for which gradeability of trucks is the determining factor, the following assumptions are made:

(i) The weight-power ratio of a loaded truck is 180 kg/kW.

(ii) The average running speed as related to design speed is used to approximate the speed of vehicles beginning an uphill climb.

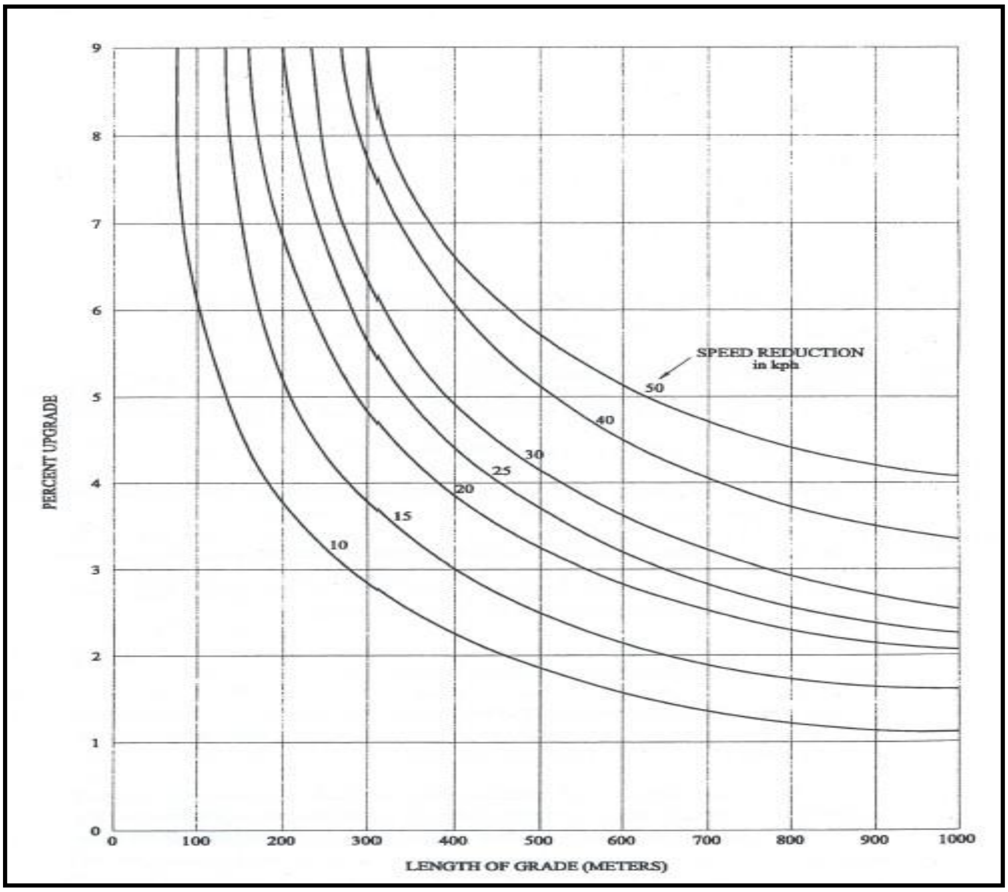
(iii) The common basis for determining the critical grade length is a reduction in speed of trucks below the average running speed. Studies show that accident involvement rate increases significantly when truck speed reduction exceeds 15kph. For example accident involvement rate is 2.4 times greater for 25kph reduction than for a 15kph reduction.

The length of any given grade that will cause the speed of a representative truck (180 kg/kW) entering the grade at 90km/h to reduce by various amounts below the average running speed is shown graphically in **Figure 4.7**. The 25kph speed reduction curve should be used as the general design guide for determining the critical length of grade.

Where a particular section is made up of a combination of upgrades, the length of critical grade, measured from tangent point to tangent point, should take into consideration the entire section of the combination.

Where the length of critical grade is exceeded and especially where grade exceeds 5%, consideration should be given to providing an added uphill lane, that is, the climbing lane, for slow moving vehicles, particularly where the volume is at or near capacity and the truck volume is high.

FIGURE 4.7: CRITICAL LENGTH OF GRADE FOR DESIGN FOR TYPICAL HEAVY TRUCK OF 180kg/kW



Source: REAM GL 2/2002: A Guide on Geometric Design of Roads, Figure 4-5

4.3.4 Climbing Lanes for Two Lane Roads

The following conditions and criteria, which reflect economic consideration, should be satisfied to justify a climbing lane: -

(i) Upgrade peak traffic flow rate in excess of 200 vehicles per hour

(ii) Upgrade truck peak flow rate in excess of 20 vehicles per hour

(iii) One or more of the following conditions exist :

- A 25kph or greater speed reduction is expected for typical heavy truck.

- Level of service E or F exists on the grade
- A reduction of two or more levels of services is experienced on approaching the start of the grade.

Where climbing lanes are provided, the following requirements are to be followed:-

a) The climbing lanes should begin near the foot of the grade and should be proceeded by a tapered section of at least 50m long.

b) The width of the climbing lane should be the same as the main carriageway lane width and in any case should not be less than 3.25m.

c) The section of the road must be separated by a New Jersey Type Concrete Median with adequate signage and markings and the opposing lane also widened to 2 lanes to make it a four lane divided carriageway.

d) The climbing lane should end at least 60m beyond the crest and in particular should be at a point where the sight distance is sufficient to permit passing with safety. In addition, a corresponding taper length as in (a) of 100m should be provided.

e) The shoulder on the outer edge of the climbing lane should be as wide as the shoulder on the normal two lane section. Where conditions dictate otherwise, a useable shoulder width of 1.25m is acceptable.

4.3.5 Passing Lane Sections on Two-Lane Roads

Where a sufficient number and length of safe passing sections cannot be obtained in the design of horizontal and vertical alignments alone, an occasional section of four lanes may be introduced to provide more sections and length safe for passing. Such sections are particularly advantageous in rolling terrain, especially where the alignment is winding or where the vertical profile includes critical lengths of grade. Four lane sections should be sufficiently long to permit its effective usage.

The sections of four lanes introduced need not be divided. The use of a median, however is advantageous and should be considered on roads carrying 500 vehicles per hour or more.

The transition between the two lane and four lane pavements should be located where the gradual change in width is in full view of the driver. Sections of four-lane road, particularly divided sections should not be longer than 3 km so as to ensure that the driver does not lose his awareness that the road is basically a two lane facility.

Where four lane sections are not practical, passing one direction lane sections shall be introduced at regular intervals. The passing lane must be at least 125m long and of full lane width and proceeded by a taper of 50m at the beginning and 100m at the end.

4.3.6 Climbing Lanes on Multilane Roads

Multilane roads more frequently have sufficient capacity to handle their traffic load, including the normal percentage or slow-moving vehicles without becoming congested. However where the volume is at or near capacity (1700 vehicle per hour per lane) and the truck volume is high (≥10% of total flow) so as to interfere with the normal flow of traffic then addition of climbing lanes should be considered.

4.3.7 Vertical Curves

Vertical curves are used to effect a gradual change between tangent grades. They should be simple in application and should result in a design that is safe, comfortable in operation, pleasing in appearance and adequate for drainage. For simplicity, the parabolic curve with an equivalent vertical axis centered on the vertical point of intersection is used.

The rate of change of grade to successive points on the curve is a constant amount for equal increments of horizontal distance, and equals the algebraic difference between the intersecting tangent grades divided by the length of curve or A/L in percent per meter. The reciprocal L/A is the horizontal distance in metre required to effect a 1 percent change in gradient and is a measure of curvature. This quantity (L/A), termed k, is used in determining the horizontal distance from the beginning of the vertical curve to the apex or low point of the curve. The k value is also useful in determining the minimum lengths of vertical curves for the various design speeds.

The lengths of vertical curves used should be as long as possible and above the minimum values for the design speeds where economically feasible.

(i) Crest Vertical Curves

Minimum lengths of crest vertical curves are determined by the sight distance requirements. The stopping sight distance is the major control for the safe operation at the design speed chosen. Passing sight distances are not used as it provides for an uneconomical design. An exception may be at decision areas such as sight distance to ramp exit gores where longer lengths are necessary.

The basic formula for length of a parabolic vertical curve in terms of algebraic difference in grade and sight distance (using an eye height of 1.05m and object height of 0.20m) are as follows:

Where, S **is less than L,**

$$L = \frac{AS^2}{404}$$

Where, S **is greater than L,**

$$L = 2S - \frac{404}{A}$$

Where,
L = Length of vertical curve (m)
S = Sight distance (m)
A = Algebraic difference in grades (percent)

Table 4.11A indicates the required K values that are to be used in design for the various design speeds.

TABLE 4.11A: CREST VERTICAL CURVE (K VALUES)

Design Speed (km/h)	120	110	100	90	80	70	60	50	40	30
Minimum K Value	144	112	78	59	39	26	17	10	10	5

(ii) Sag Vertical Curves

At least four different criteria for establishing lengths of sag vertical curves are recognized. These are (1) headlight sight distance, (2) rider comfort, (3) drainage control and (4) a rule of thumb for general use and this criterion is used to establish the design values for a range of lengths of sag vertical curves. It is again convenient to express the design control in terms of the K value. **Table 4.11B** indicates the minimum K values that are to be used.

Longer curves are desired wherever feasible and should be used, but where K values in excess of 55 are used, special attention to drainage must be exercised. Shorter sag vertical curves may be justified for economic reasons, in cases where an existing element, such as a structure which is not ready for replacement, controls the vertical profile.

Drainage of kerbed pavements are especially important on sag vertical curves where a grade line of not less than 0.3 percent within 15m of the level point must be maintained.

TABLE 4.11B: SAG VERTICAL CURVE (K VALUES)

Design Speed (km/h)	120	110	100	90	80	70	60	50	40	30
Minimum K Value	63	55	45	38	30	23	18	13	9	6

Source: AASHTO – A Policy on Geometric Design of Highways and Street (2001), Exhibit 3-79

4.3.8 General Controls for Vertical Alignment

In addition to the specific controls, there are several general controls that should be considered.

(i) A smooth gradeline with gradual changes should be strived for in preference to a line with numerous breaks and short lengths of grade. While the maximum grade and the critical length are controls, the manner in which they are applied and fitted to the terrain on a continuous line determines the suitability and appearance of the finished product.

(ii) The 'roller coaster' or the 'hidden-dip' type of profile should be avoided. They are avoided by the use of horizontal curves or by more gradual grades.

(iii) A broken back gradeline should be avoided, particularly in sags where the full view of both vertical curves is not pleasing. This effect is very noticeable on divided roadways with open median sections.

(iv) On long grades, it is preferable to place the steepest grades at the bottom and reduce the grades near the top of the ascent or to break the sustained grade by short intervals of higher grade instead of a uniformed sustained grade that might be only slightly below the allowable minimum.

(v) Where intersections at grade occur on sections with moderate to steep grades, it is desirable to reduce the gradient through the intersection.

4.4 Combination of Horizontal and Vertical Alignment

Horizontal and vertical alignment should not be designed independently. They complement each other. Excellence in their design and in the design of their combination increases utility and safety, encourage uniform speed, and improve appearance, almost always without additional cost.

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 design. 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 analysed 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, poses a hazardous condition.

(iii) Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. Their 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 curve is made longer than the vertical curve. Also, suitable design can be by using design values above the minimum for the design speed.

(iv) Somewhat allied to the above, 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.

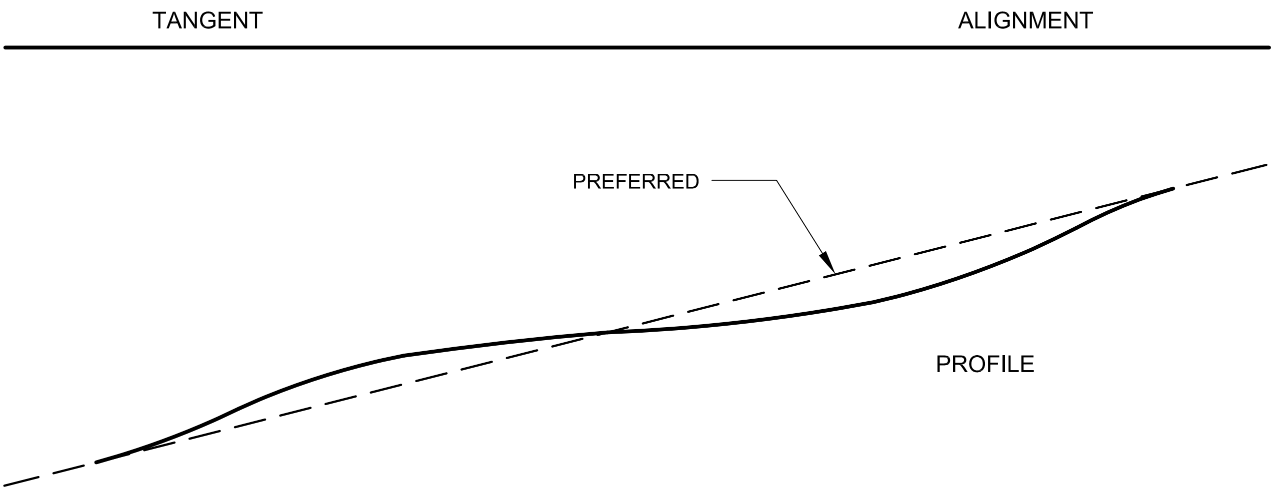
(v) On 2-lane roads the need for safe passing sections at frequent intervals and for an appreciable percentage of the length of the road often supersedes the general desirability for combination of horizontal and vertical alignment. In these cases it is necessary to work towards long tangent sections to secure sufficient passing sight distance in design.

(vi) Horizontal curvature and profile should be made as flat as feasible at intersections where sight distance along both roads is important and vehicles may have to slow down or stop.

(vii) On divided roads, 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 results from the concept and logical design basis of one-way roadways.

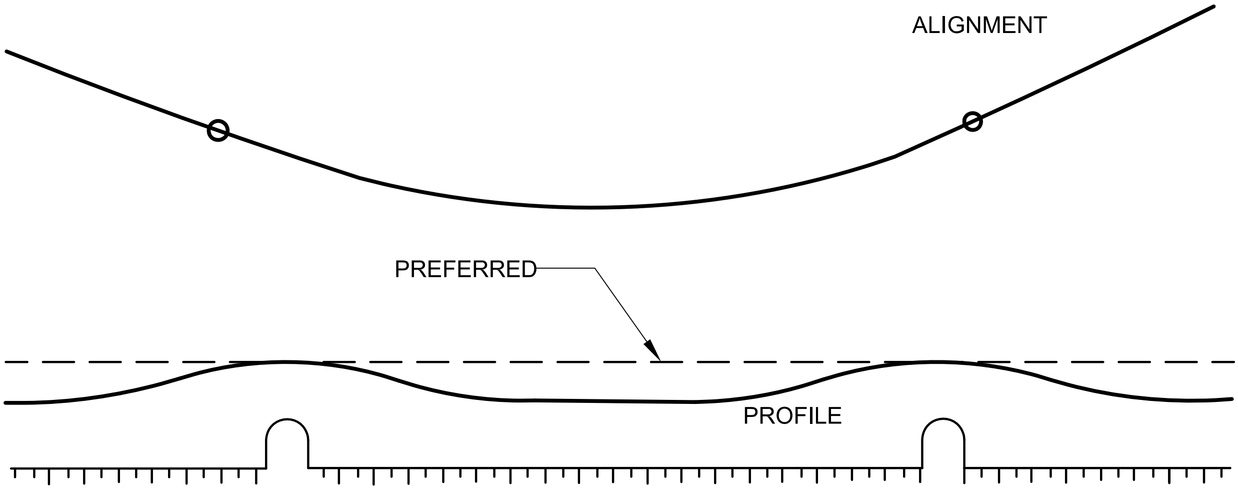
(viii) In residential areas the alignment should be designed to minimise nuisance factors to the neighbourhood. Generally, a depressed road makes it less visible and less noisy to adjacent residents. Minor horizontal alignment adjustments can sometimes be made to increase the buffer zone between the road and the residential areas.

(ix) Examples of poor and good practices are illustrated in **Figure 4.8A to Figure 4.8G (Alignment & Profiles A** to **N)** below :



(A) PROFILE WITH TANGENT ALIGNMENT

Notes : Avoid designing little local dips in an otherwise long uniform grade



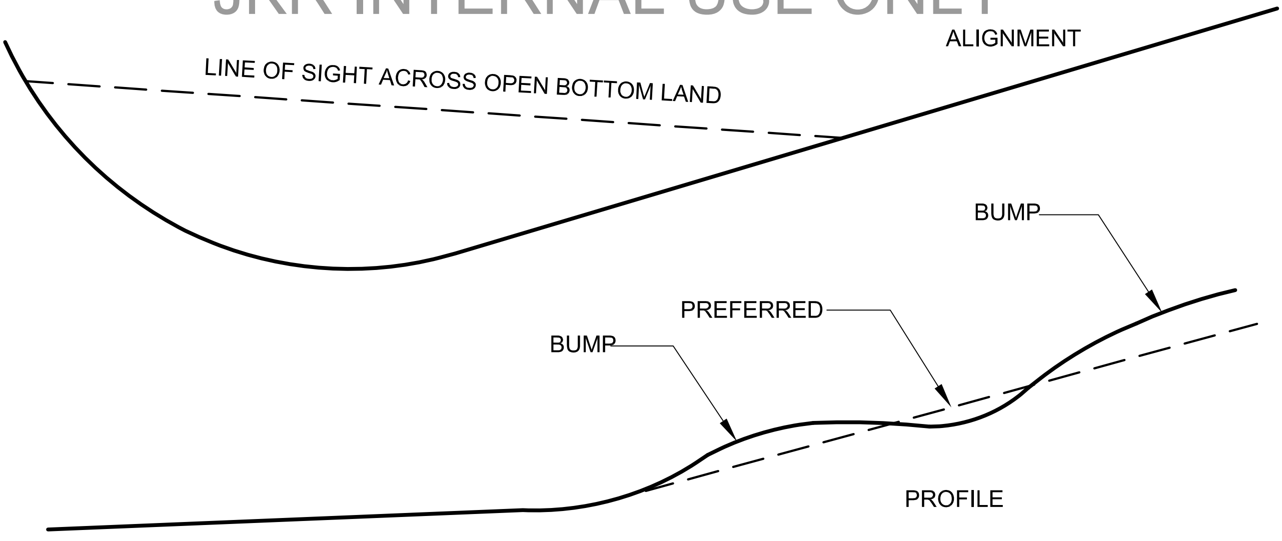
(B) PROFILE WITH TANGENT ALIGNMENT

Notes : Short humps in the grade should be avoided.

Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 4-6

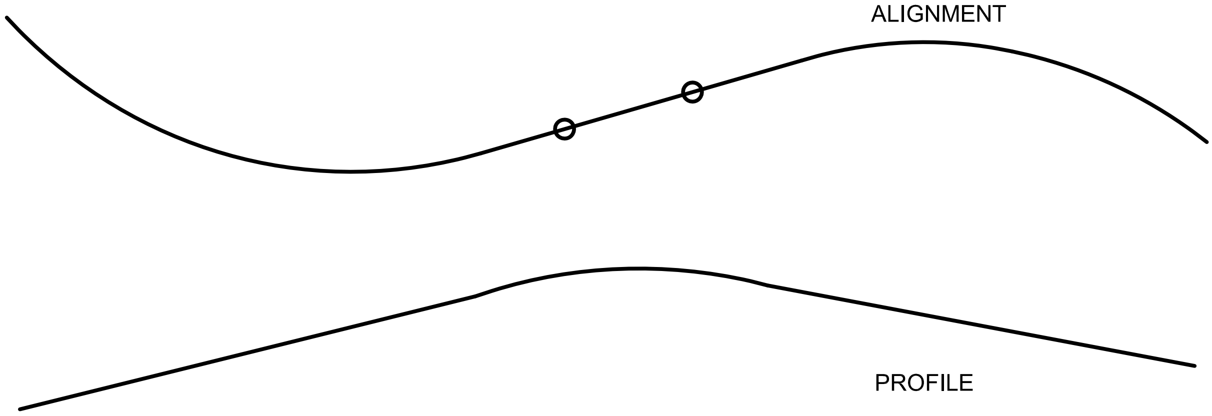
FIGURE 4.8A: A & B

Scale : Not to scale



(C) DISTANCE VIEW SHOWING BUMPS IN PROFILE GRADE LINE

Notes : A distant side view of a long grade tangent will reveal every bump on it



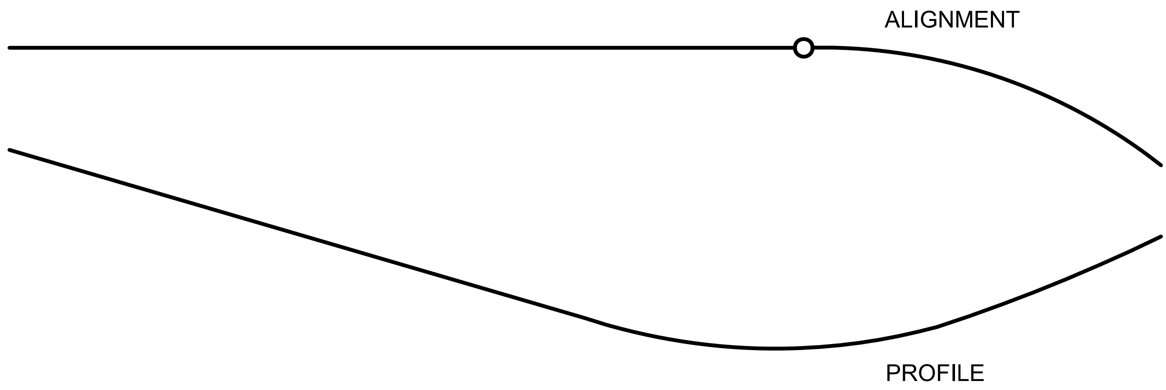
(D) SHORT TANGENT ON A CREST BETWEEN TWO HORIZONTAL CURVES

Notes : This combination is deficient for two reasons. The tangent between the curves is too short, and the reverse occurs on a crest.

Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 4-6

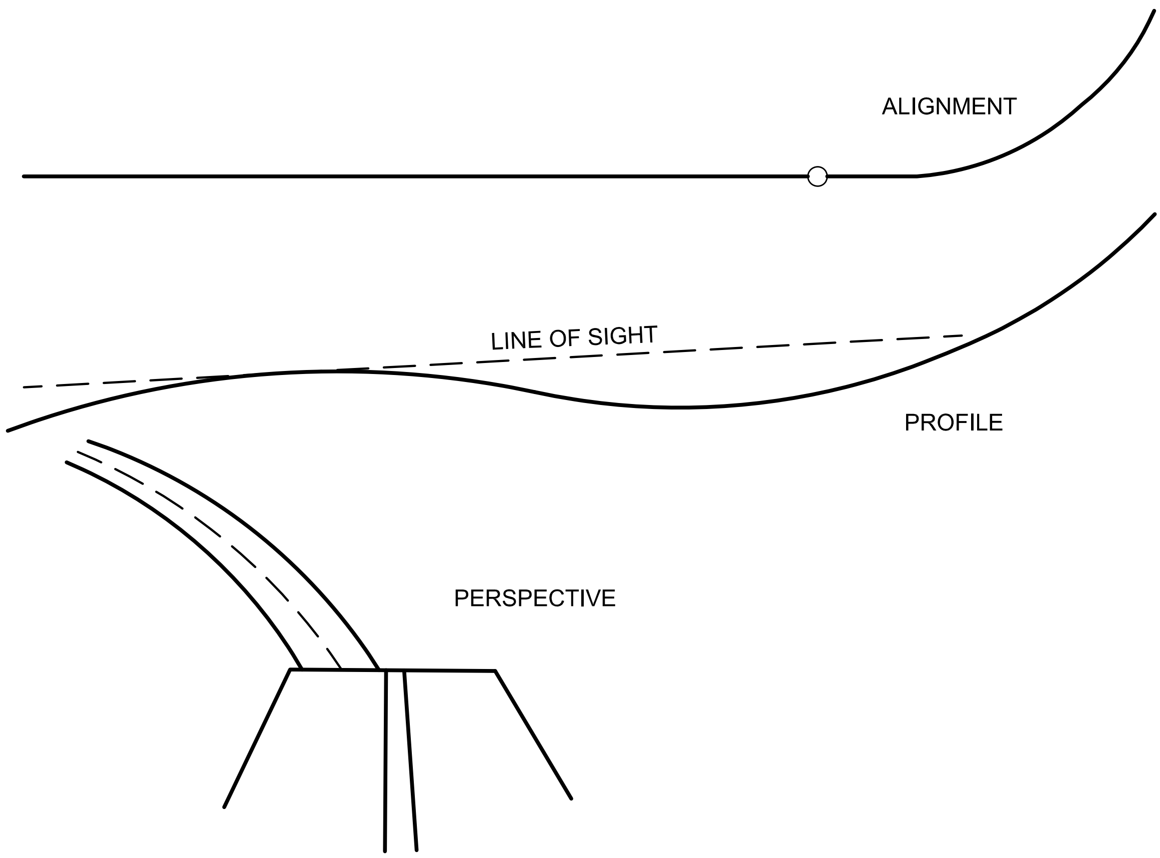
FIGURE 4.8B: C & D

Scale : Not to scale



(E) SHARP ANGLE APPEARANCE

Notes :- This combination presents a poor appearance. The horizontal curve looks like a sharp angle.



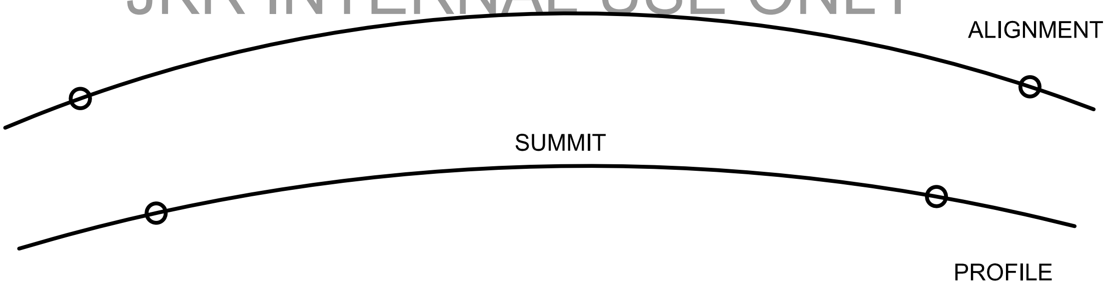
(F) DISJOINTED EFFECT

Notes : A disjointed effect occurs when the beginning of a horizontal curve is hidden from the driver by an intervening crest while the continuation of the curve is visible in the distance beyond the intervening crest.

Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 4-6

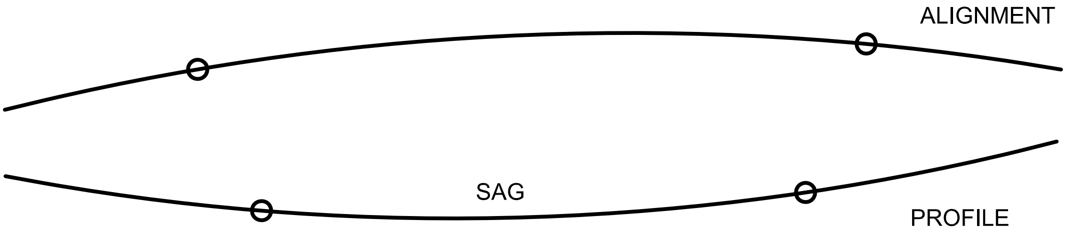
FIGURE 4.8C : E & F

Scale : Not to scale



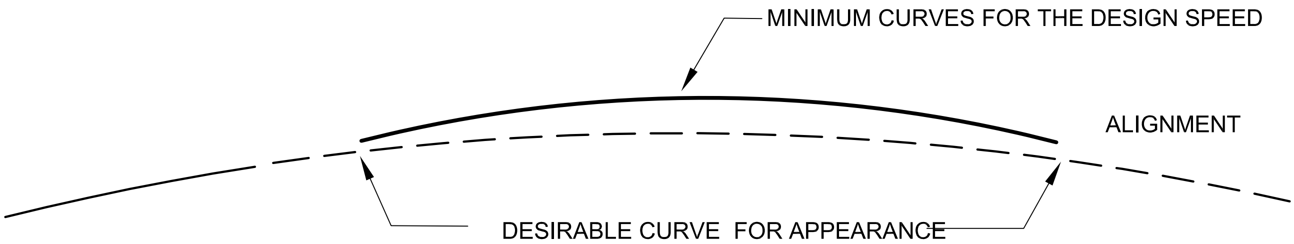
(G) COINCIDING CURVES IN HORIZONTAL AND VERTICAL DIMENSIONS

Notes : When horizontal and vertical curves coincide, a very satisfactory appearance results.



(H) OPPOSING CURVES IN HORIZONTAL AND VERTICAL DIMENSIONS

Notes : When horizontal and vertical curves oppose, a very satisfactory appearance result.



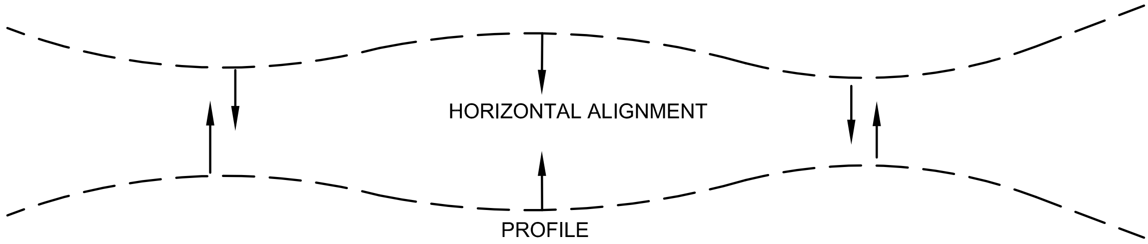
(I) FLAT CURVES APPROPRIATE FOR HORIZONTAL WITH SMALL CENTRAL ANGLE REGARDLESS OF PROFILE

Notes : Very long flat curves, even where not required by the design speed, also have a pleasing appearance when the central angle is very small.

Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 4-6

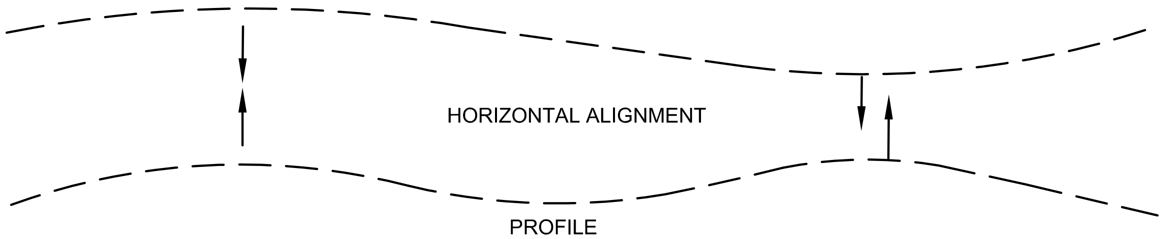
FIGURE 4.78D G , H & I

Scale : Not to scale



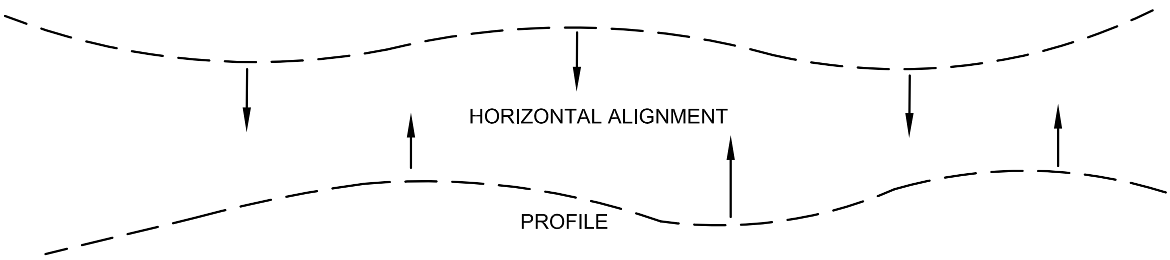
(J) COINCIDING CURVES VERTICES IN HORIZONTAL AND VERTICAL DIMENSIONS

Notes : The classic case of coordination between horizontal and vertical alignment in which the vertices of horizontal and vertical curves coincide, creating a rich effect of three-dimensional s-curves, composed of convex and concave helixes



(K) COINCIDING VERTICES WITH SINGLE-PHASE SKIP

Notes : A legitimate case of coordination : one phase is skipped in the horizontal plane, but vertices still coincide. The long tangent in plane is softened by vertical curvature.

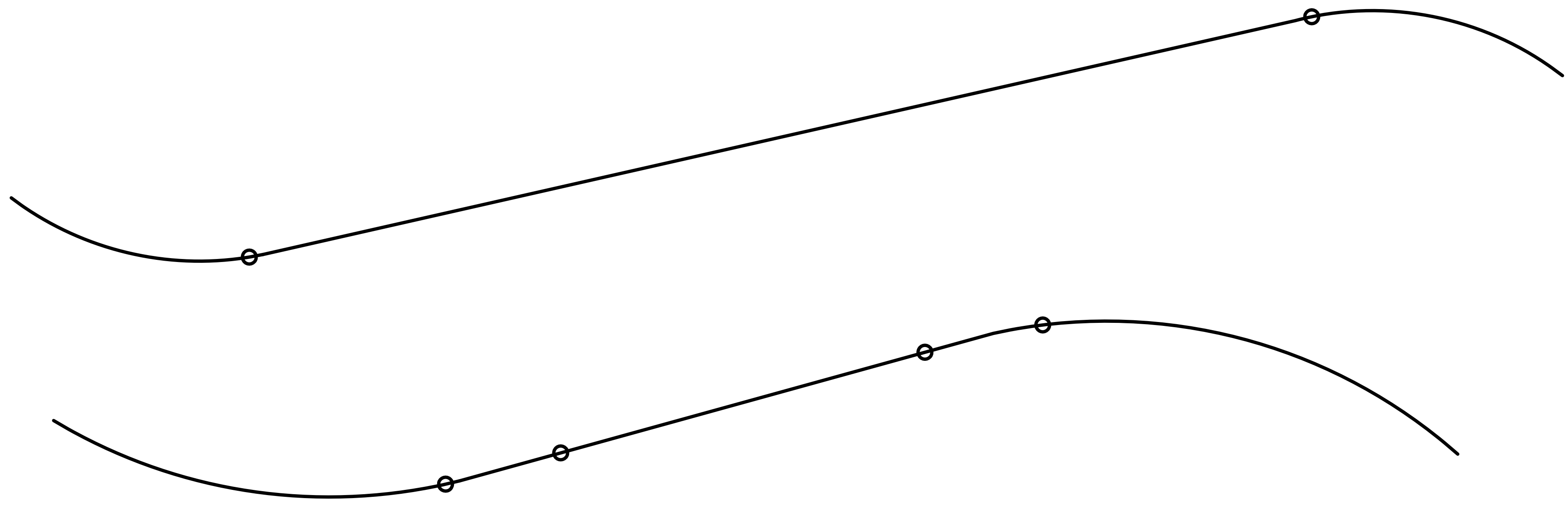


(L) WEAK COORDINATION OF HORIZONTAL AND VERTICAL ALIGNMENTS

Notes : A case with weak coordination where the vertical alignment is shifted half a phase with respect to horizontal alignment so the vertices coincide with point of inflection. The superelevation in this case occurs on grade, while crests and sags have normal cross sections in the first case, superelevation occurs on crests sags, while grades have normal cross sections.

Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 4-6

FIGURE 4.8E: J, K & L



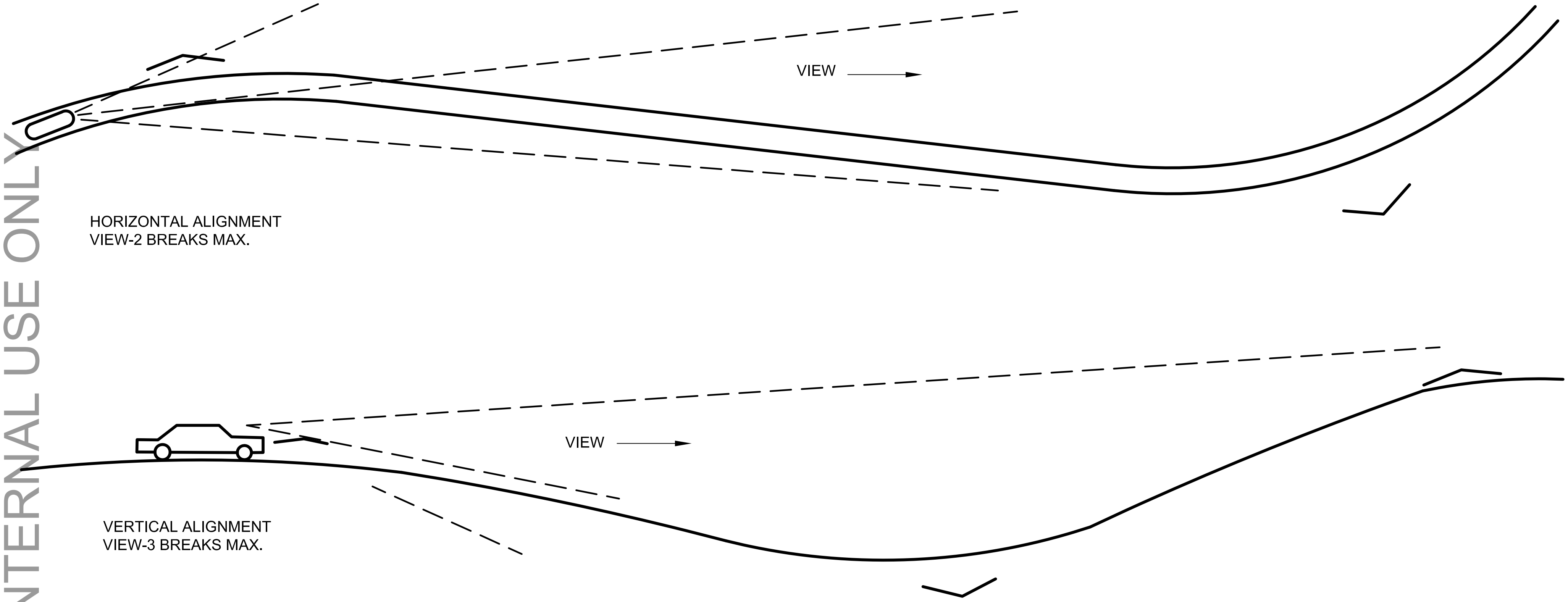
(M) HORIZONTAL ALIGNMENT SHOULD BE BALANCE

Notes : The upper line is an example of poor design because the alignment consists of a long tangent with short curves, where as the balance between the curves and tangents in the lower alignment is the preferred design.

Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 4-6

FIGURE 4.8F : M

Scale : Not to scale



(N) GOOD COORDINATION OF HORIZONTAL AND VERTICAL ALIGNMENT

Notes: Guideline to be used for coordination of horizontal and vertical alignment reverse occurs on a crest

Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 4-6

FIGURE 4.8G : N

Scale : Not to scale

5.0 CROSS SECTION ELEMENTS

5.1 Pavement

5.1.1 Surface Type

The selection of the pavement type is determined by the volume and composition of traffic, soil characteristics, weather, availability of materials, the initial cost and the overall annual maintenance and service life cost.

The important characteristics of surface type in relation to geometric design are the ability of a surface to retain the shape and dimensions, the ability to drain, and the effect on driver's behavior. **Table 5.1** gives the general selection of the pavement surface types for the various road standards.

For minor roads of grades exceeding 8%, the surface and shoulder should be sealed to prevent erosion.

The structural design of the pavement should be in accordance to **Arahan Teknik (Jalan) – 5/85 (Pindaan 2013) "Manual for The Structural Design of Flexible Pavement"** and **"Design Guide For Alternative Pavement Structures, Low-Volume Roads "**.

TABLE 5.1: PAVEMENT SURFACE TYPE

Design Standard	Description
R6 / U6	Asphaltic Concrete / Concrete / Specialty Mix
R5 / U5	Asphaltic Concrete / Concrete / Specialty Mix
R4 / U4	Asphaltic Concrete / Specialty Mix
R3 / U3	Concrete / Specialty Mix / Asphaltic Concrete
R2 / U2	Surface Treatment / Semigrout / Asphaltic Concrete
R1 / U1	Gravel / Surface Treatment

Source: Adapted from REAM GL 2/2002: A Guide on Geometric Design of Roads, Table 5-1

5.1.2 Normal Cross Slope

Cross slopes are an important element in the cross-section design and a reasonably steep lateral slope is desirable to minimise water ponding on flat sections of unkerbed pavements due to pavement imperfections or unequal settlements and to control the flow of water adjacent to the kerb on kerbed pavements. The range of cross slopes for various pavement types varies from 2.5% - 6.0%.

The rate of cross slope is an important element in cross-section design. Superelevation on curves is determined by the speed-curvature relationships given in Chapter 4, but cross slope or crown on tangents or on long-radius curves are complicated by two contradictory controls. On one hand, a reasonably steep lateral slope is desirable to minimize ponding of water on pavements with flat profile grades as a result of pavement imperfections or unequal settlement. Horizontal and vertical alignment should also be coordinated to avoid flat spots where crest vertical curves and superelevation transitions coincide. A steep cross slope is also desirable on kerbed pavements to confine water flow to a narrow width of pavement adjacent to the kerb.

On the other hand, steep cross slope are undesirable on tangents because of the tendency of vehicles to drift toward the low edge of the traveled way. This drifting becomes a major concern in areas where snow and ice are common. Cross slopes up to and including 2 percent (2%) are barely perceptible in terms of vehicle steering. However, cross slopes steeper than 2 percent (2%) are noticeable and may require a conscious effort in steering. Furthermore, steep cross slopes increase the susceptibility to lateral skidding when vehicles brake on wet pavements or when stops are made on dry pavements under emergency conditions.

5.2 Lane Widths and Marginal Strip

Lane widths and the condition of the pavement surface are the most important features of a road pertaining to the safety and comfort of driving. The capacity of a highway is markedly affected by the lane width and in a capacity sense. The effective width of a travelled way is further reduced when adjacent obstructions such as retaining walls, bridge piers and parked cars restrict the lateral clearance.

Marginal strip is a narrow pavement strip attached to both edges of a carriageway. It is paved to the same standard as the pavement structures. For divided roads, the marginal strips are provided on both sides of the carriageway in both directions. The marginal strip is included as part of the shoulder width and is demarcated from the through lane by lane edge markings on the marginal strip.

Table 5.2 indicates the lane and marginal strip widths that are to be used for the various road standards.

TABLE 5.2: LANE & MARGINAL STRIP WIDTHS

Design Standard	Lane Width (m)	Marginal Strip Width (m)
R6 / U6	3.65	0.50
R5 / U5	3.50	0.50
R4 / U4	3.50	0.25
R3 / U3	3.25	0.25
R2 / U2	3.00	0.25
R1 / U1	5.00*	0.00
Interchange Ramps		
Single Lane	4.50	Lt 1.50 Rt 0.50
Multi Lanes	3.50	Lt 0.50 Rt 0.50
Single Lane Loop	4.50	Lt 1.50 Rt 0.50

Source: Adapted from REAM GL 2/2002: A Guide on Geometric Design of Roads, Table 5-2

Note: * denotes the total two-way width

5.3 Shoulders

5.3.1 General Characteristics

A shoulder is the portion of the roadway continuous with the travelled way purpose for the accommodation of stopped vehicle, emergency use and also lateral support of the pavement. A typical cross section showing the road shoulder and verge as shown in **Figure 5.2**.

Their main functions are:-

(i) Space is provided for emergency stopping free of the traffic lane.

(ii) Space is provided for the occasional motorist who desires to stop for various reasons.

(iii) Space is provided to escape potential accidents or reduce their severity.

(iv) The sense of openness created by shoulders of adequate width contributes to driving ease and comfort.

(v) Sight distance is improved in cut sections, thereby improving safety.

(vi) Highway capacity is improved and uniform speed is encouraged.

(vii) Lateral clearance is provided for signs and guardrails.

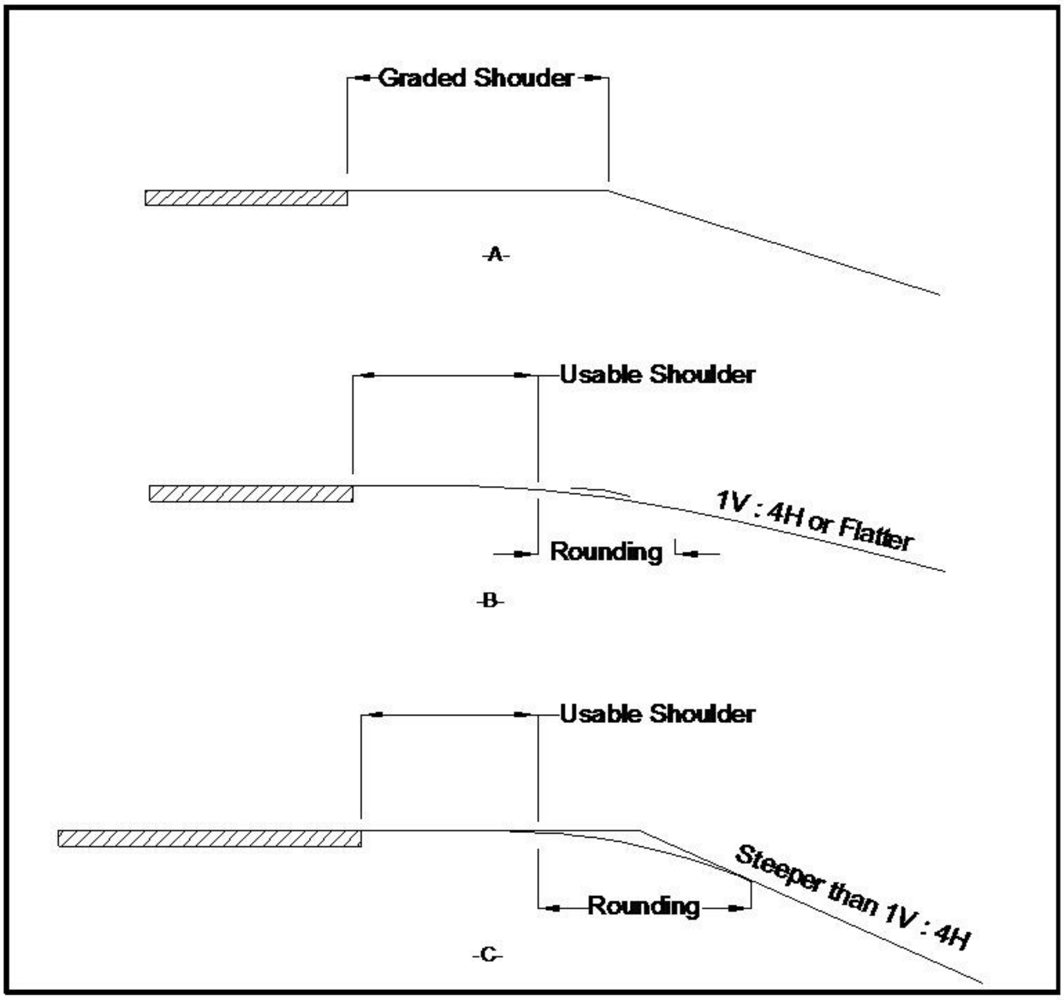
(viii) Structural support is to the pavement is enhanced.

The term "shoulder" is variably used with a modifying adjective to describe certain functional or physical characteristics. The following applies to the terms used here:

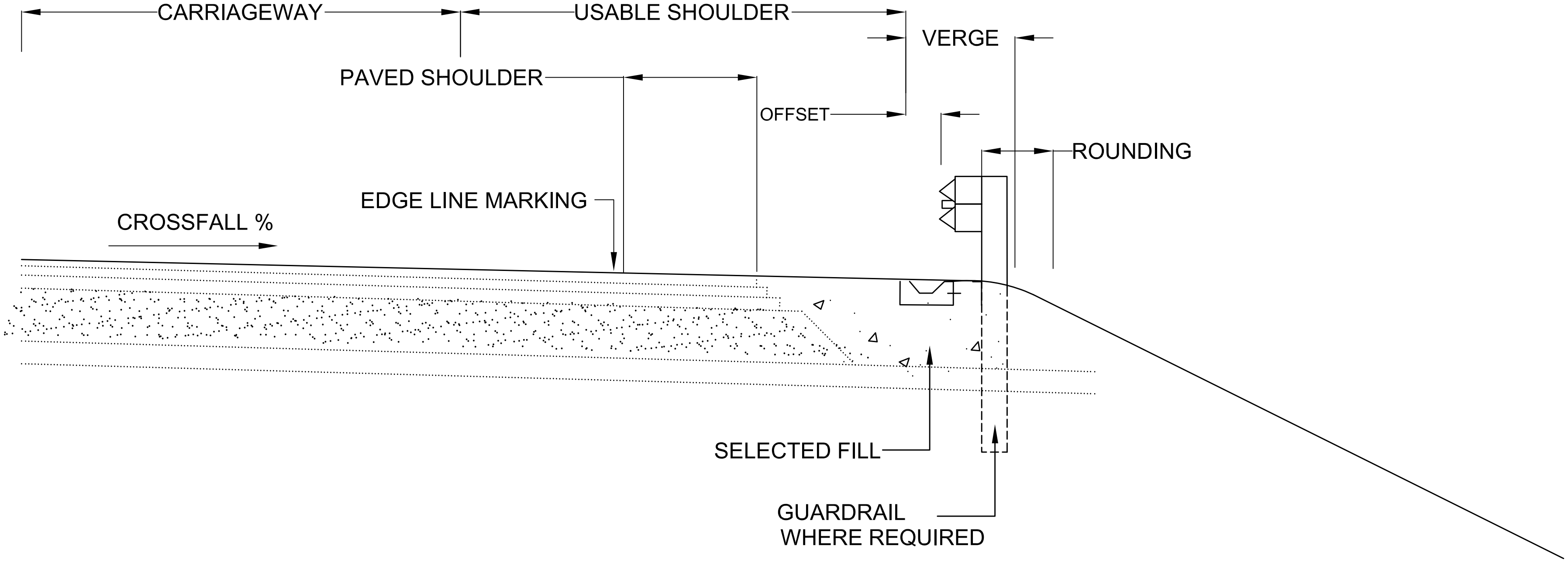
(a) The "graded" width of shoulder is that measured from the edge of the traveled way to the intersection of the shoulder slope and the foreslope planes, as shown in **Figure 5.1A**.

(b) The "usable" width of shoulder is the actual width that can be used when a driver makes an emergency or parking stop. Where the sideslope is 1V:4H or flatter, the "usable" width is the same as the "graded" width since the usual rounding of 1.2m to 1.8m width at the shoulder break will not lessen its useful width appreciably. **Figures 5.1B** and **5.1C** illustrate this clearly.

FIGURE 5.1: GRADED AND USABLE SHOULDERS



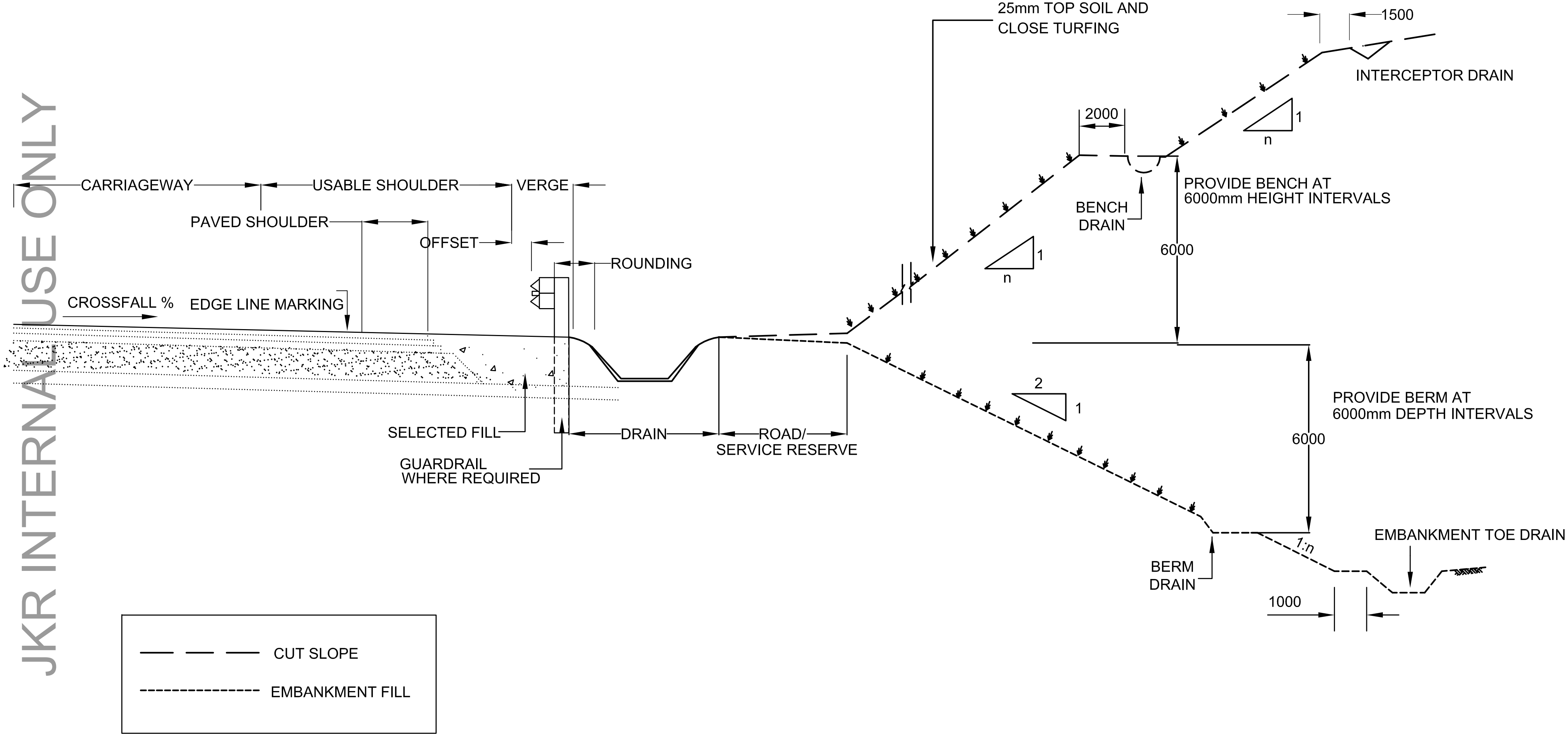
Source: AASHTO – A Policy on Geometric Design of Highways and Street (2011), Figure 4.4



Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 5-1

FIGURE 5.2 : TYPICAL CROSS SECTION OF SHOULDER AND VERGE

Scale : Not to scale



Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 5-1

FIGURE 5.2a : TYPICAL ROAD CROSS SECTION OF CUT SLOPE AND EMBANKMENT FILL

Scale : Not to scale

5.3.2 Width of Shoulders

The normal usable shoulder width that should be provided along high type facilities is 3m. However, in difficult terrain and on low volume roads, usable shoulders of this width may not be feasible. A minimum usable shoulder width of 0.6m should be considered in such cases.

Table 5.3A and **Table 5.3B** gives the widths of shoulders for the various road standards in rural and urban areas.

TABLE 5.3A: USABLE SHOULDER WIDTH (RURAL)

Design Standard	Usable Shoulder Width (m)		
	Terrain		
	Flat	Rolling	Mountainous
R6	3.00	3.00	2.50
R5	3.00	3.00	2.50
R4	3.00	3.00	2.00
R3	2.50	2.50	2.00
R2	2.00	2.00	1.50
R1	1.50	1.50	1.50

Source: REAM GL 2/2002: A Guide on Geometric Design of Roads, Table 5-3A

TABLE 5-3B: PAVED SHOULDER WIDTH (URBAN)

Design Standard	Paved Shoulder Width (m)		
	Area Type *		
	I	II **	III **
U6	3.00	3.00	2.50
U5	3.00	3.00	2.50
U4	3.00	2.50	2.00
U3	2.50	2.00	1.50
U2	2.00	1.50	1.50
U1	1.50	1.50	1.50

Source: REAM GL 2/2002: A Guide on Geometric Design of Roads, Table 5-3B

Notes:

* For Area Type definition, see Table 3-2B
** For Areas Type II & III, U1 to U4, shoulder may be replaced by sidewalk

5.3.3 Type of Shoulders

There are basically two types of shoulders, namely paved and unpaved. Paved shoulders can be either bituminous or concrete surfaced while unpaved shoulders can be either crushed rock, earth or turf shoulders.

The paved shoulder widths for rural roads are as shown in **Table 5.4**. The figures given are in addition to the width required for parking.

TABLE 5-4: PAVED SHOULDER WIDTH (RURAL)

Design Standard	Paved Shoulder Width (m)
R6	2.5
R5	2.5
R4	1.5
R3	1.5

Note:

(i) For R5 & R6 in mountainous terrain, road shoulder should be paved even though minimum usable shoulder width might not be attainable.

(ii)For R1 & R2, there is no requirement of minimum shoulder width to be paved.

The structure and the cross slope of these shoulders are different and they are described below.

5.3.4 Shoulder Cross Slope

All shoulders should be sloped sufficiently to rapidly drain surface water but not to the extent that vehicular use would be hazardous. Since the type of shoulder construction has a bearing on the cross-slope, the two (slope and type) should be determined jointly.

Because of the nature of the surfacing materials used and surface irregularities, unpaved surfaces such as earth, gravel, or crushed stone need an even greater cross slope on tangents to prevent the absorption of water into the surface. Therefore, cross slopes greater than 2 percent (2%) may be used on these types of surfaces.

Bituminous and concrete surfaced shoulders should be sloped from 2 to 6 percent (%), gravel or crushed rock shoulders from 4 to 6 % and turf shoulders 6%. Where kerbs are used on the outside of the shoulders, the minimum cross-slope should not be less than 4 percent to prevent ponding on the roadway.

At super elevated areas, the shoulder cross-slopes should be adjusted to ensure that the maximum "roll over" does not exceed 8 percent.

5.3.5 Shoulder Structure

For shoulders to function effectively, they must be sufficiently stable to support occasional vehicle loads, in all kinds of weather without rutting. Paved or stabilised shoulders offer many advantages in this aspect. The structure of the paved shoulders should be similar to that of the carriageway for all design standards. However, where gradients exceed 8% the shoulders should also be close turfed or sealed to prevent erosion for R2 and R1 roads.

5.3.6 Shoulder Stability

If shoulders are to function effectively, they should be sufficiently stable to support the occasional vehicle loads which they are subjected to, without easily giving in to rutting or other modes of failure. Evidence of rutting, skidding, or vehicles being mired down, even for a brief seasonal period, may discourage and prevent the shoulder from being used as intended.

All types of shoulders should be constructed and maintained flushed with the traveled way pavement if they are to fulfill their intended function. Regular maintenance is needed to provide a flush shoulder. Unstabilized shoulders generally undergo consolidation with time, and the elevation of the shoulder at the traveled-way edge tends to become lower than the traveled way after some time. The drop-off can adversely affect driver control when driving onto the shoulder at any appreciable speed. In addition, when there is no visible assurance of a flush stable shoulder, the operational advantage of drivers staying close to the pavement edge is reduced.

Paved or stabilized shoulders offer numerous advantages including: -

(i) Provisional of refuge for vehicles during emergency situations.
(ii) Elimination of rutting and drop-off adjacent to the edge of the traveled way.
(iii) Provision of adequate cross slope for drainage of roadways.
(iv) Reduction of maintenance.
(v) Provision of lateral support for roadway base and surface course.

Shoulders with turf growth may be appropriate, under favorable climatic and soil conditions, for local roads and some collectors. Turf shoulders are subject to a buildup that may inhibit proper drainage of the traveled way unless adequate cross slope is provided. When wet, the turf may be slippery unless closely mowed and on granular soil. Turf shoulders offer good traveled-way delineation and do not invite use as a traffic lane. Stabilized turf shoulders need little maintenance other than mowing.

5.4 Kerbs

5.4.1 General Consideration

The type and location of kerbs will appreciably affect driver's behaviour and in turn the safety and usage of a road. Kerbs are used for drainage control, pavement edge delineation, aesthetics, delineation of pedestrian walkways and to assist in the orderly development of the roadside.

Kerbs are needed mostly on roads in urban areas. In rural areas, the use of kerbs should be avoided as far as possible except in localized areas which has predominant aspects of an urban condition.

5.4.2 Types of Kerbs

The three general classes of kerbs are **BARRIER KERBS, MOUNTABLE/SEMI-MOUNTABLE KERBS** and **CHANNEl KERBS** has numerous types and detail design. Each may be designed as a separate unit or integrally with the pavement. They may also be designed with a gutter to form a combination of kerb and gutter section.

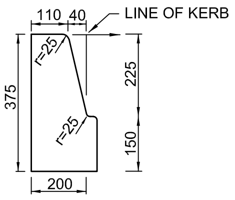
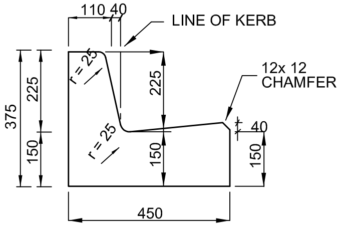
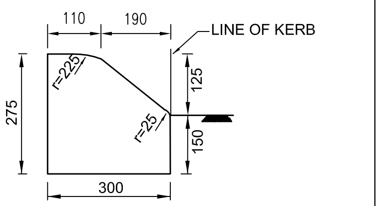
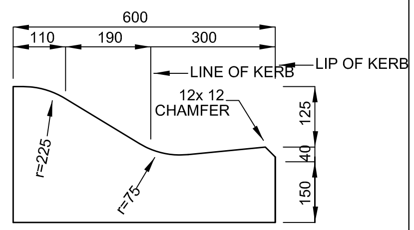
Barrier kerbs are relatively high and steep faced and are designed to inhibit or discourage vehicles from leaving the roadway. They should not be used on expressways. They should also not be used where the design speeds exceed 70 km/hr. or in combination with traffic barriers. They are recommended for use in built-up areas adjacent to footpaths with considerable pedestrian traffic **(Type B1).** Where pedestrian traffic is light, a semi-barrier type **(Type B2)** may be used.

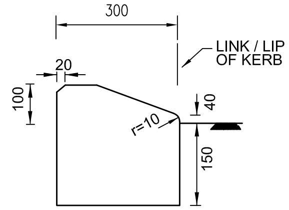
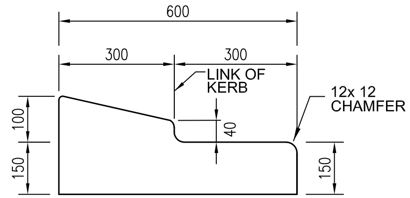
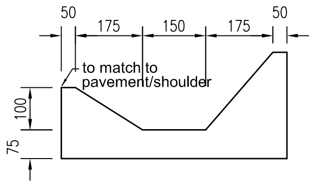
Mountable kerbs are used to define pavement edges of through carriageways **(Type M1 & M2).** For channelization islands, medians, outer separators or any other required delineation within the roadway, a semi-mountable type may be used **(Type SM1 & SM2).**

Channel kerbs should be considered for roads built on high embankment and for roads with wide roadway, more for the purpose of delineation and drainage to cater for road surface run off at intersections and also expressway.

Figure 5.3A and **Figure 5.3B** shows the various standard types of kerbs that are to be used.

The width of kerbs is considered as cross section elements entirely outside the traffic lane width. However, for drainage kerbs, the gutter section may be considered as part of the marginal strip. Where roadways do not have any marginal strip, kerbs should be offsetted by a minimum of 0.25 m.

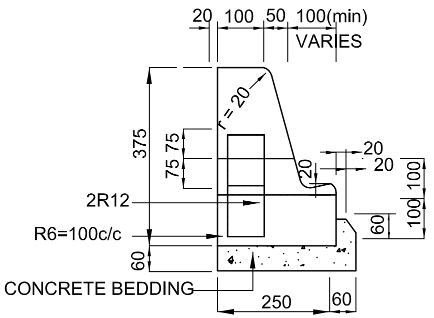
TYPE OF KERB	USE
1) BARRIER KERBS	
 B1	Barrier kerbs are used to inhibit vehicles from leaving the roadway. They are recommended for use in built-up areas adiacent tp footpaths or car parks with considerable pestrian traffic and the design speed should be less than 70kph. Type B1 kerb is generally used at outer curve of the superelevated sections.
 B2	Type B2 kerb shoulder be used for normal cross section stretch and the superelevated sections at inner curve to collect surface run off.
2) SEMI-MOUNTABLE KERBS	
 SM1  SM2	Normally used for delineation and drainage on all intersections. It is suitable for all roads including expressways. It is also for use on bridges which is offset from bridge railing.

TYPE OF KERB	USE
3) MOUNTABLE KERBS	
 M1	Mountable kerbs type M1 and M2 are designed to discourage most traffic most traffic from mounting a traffic island. Roundabout or median except for long or over-dimensional vehicles.
	Types M2 kerbs are designed to collect surface runoff and to allow vehicles to cross over and to park clear of the carriageway during emergency.
4) CHANNEL KERBS	
	Channel kerb could be used along the paved shoulder where the surface runoff is considerably large. It is normally used on embankment where drain has to be placed along the paved shoulder immediately in front of barrier (Guardrail / New Jersey Barrier). It was designed to minimize the averse effects of the channel on the steering and vehicles shoulder be able to drive over this channel safely.

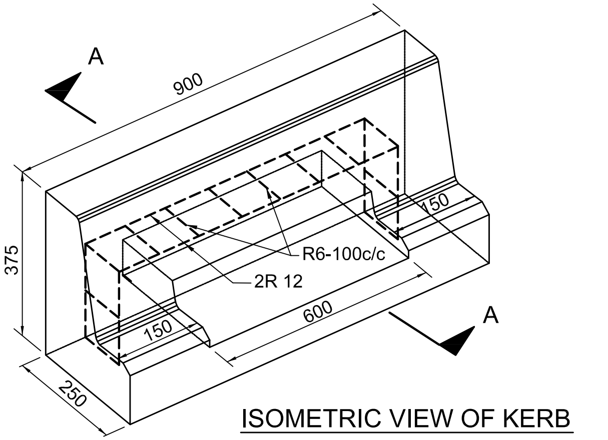
Notes:
1. Concrete for cast insitu or precast concrete kerb shall be grade 30/20.
2. Concrete bedding shall be grade 15/20.
3. The finished kerb shall be true to line, grade and level within +/-5mm and shall present a smooth appearance free from kinks and distortion.
4. All dimensions are in millimetres unless stated otherwise.

FIGURE 5.3A : ROAD KERB DETAILS

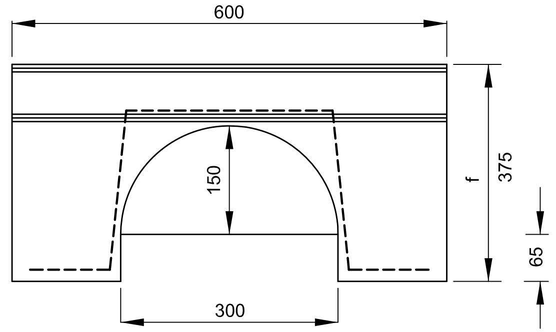
Scale : Not to scale



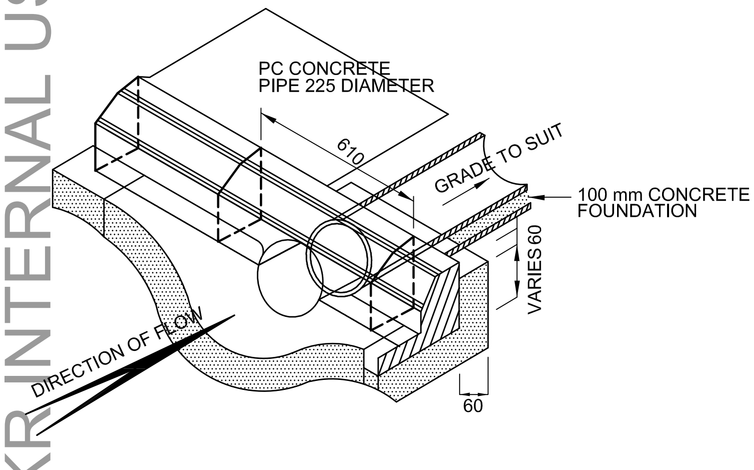
SECTION A-A



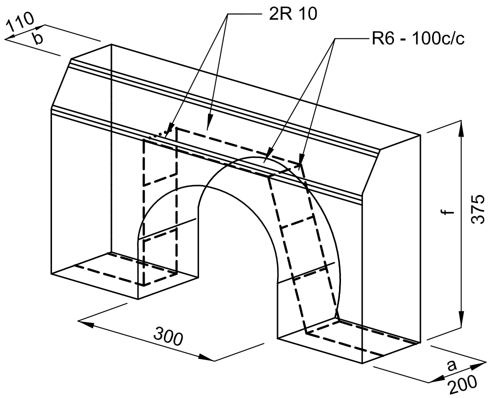
ISOMETRIC VIEW OF KERB
WITH RECTANGULAR OPENING



FRONT VIEW OF KERB



ISOMETRIC VIEW-KERB
INLET WITH CONCRETE PIPE



ISOMETRIC VIEW OF KERB
WITH ROUND OPENING

FIGURE 5.3B : ROAD KERB DETAILS

Scale : Not to scale

5.5 Sidewalks

Sidewalks are accepted as integral parts of urban roads and should be provided except on Urban Expressways or Major Arterials where the presence of pedestrians is minimal. However, the need for sidewalks in many rural areas is great because of the high speed and general lack of adequate lighting and due consideration must be given for it especially at points of community development such as schools, local business and industrial plants that results in high pedestrian concentrations.

While there are no numerical warrants, the justification for a sidewalk depends on the vehicle-pedestrian hazard which is governed by the volumes of pedestrian and vehicular traffic, their relative timing and the speed of the vehicular traffic.

In urban areas, sidewalks can be placed adjacent to the kerb and raised above the pavement. In the absence of kerbs, a strip of a minimum width of 1.0m must be provided between the sidewalk and the travelled way to allow for planting of trees or safety barriers.

In rural areas, sidewalks must be placed well away from the travelled way and separated from the shoulder by at least 1.0m.

A desirable minimum width of 2.0m is to be provided for all sidewalks. Where there are restrictions on right of way, a minimum of 1.50m can be considered. When provided, sidewalks must have all weather surfaces.

5.6 Traffic Barriers

Traffic barriers are used to minimize the severity of potential accidents involving vehicles leaving the travelled way. Because barriers are a hazard in themselves, emphasis should be on minimizing the number of such installations**. Latest edition of REAM GL 9/2006: Guidelines on Design and Selection of Longitudinal Traffic Safety Barrier** should be used for the design of longitudinal traffic barriers.

5.7 Medians

5.7.1 General

A median is a highly desirable element on all roads carrying four or more lanes and should be provided wherever possible. The principal functions of a median are to provide the desired freedom from the interference of opposing traffic, to provide a recovery area for out-of-control vehicles, to provide for speed changes and storage of right-turning and U-turning vehicles and to provide for future lanes.

For maximum efficiency, a median should be highly visible both night and day and in definite contrast to the through traffic lanes.

5.7.2 Median Types and Width

Medians may be depressed, raised or flushed with the pavement surface. They should be as wide as feasible but of a dimension in balance with other components of the cross-section. The general range of median width varies from a minimum of 1.5 m in a Type III urban situation to a desirable width of 10m on a rural expressway. On wide medians, it is essential to have a depressed centre or swale to provide for drainage.

Tables 5.5A and **5.5B** give the minimum and desirable width and types of medians that are to be applied to the various road standards for the Rural and Urban roads respectively. The median widths as expressed are the dimensions between the through lane edges and include the right shoulders if any.

TABLE 5.5A: MEDIAN WIDTH AND TYPES (RURAL)

Design Standard	Median Width (m)						Median Type
	Terrain						
	Flat		Rolling		Mountainous		
	Min.	Des.	Min.	Des.	Min.	Des.	
R6	4.0	10.0	4.0	10.0	4.0	10.0	**B,C,E,F**
R5	4.0	6.0	3.0	5.0	2.0	4.0	**E,F**
R4	3.0	5.0	2.0	4.0	1.5	3.0	**E,F**

Source: Adapted from REAM GL 2/2002: A Guide on Geometric Design of Roads, Table 5-5A

Note:
Min. - Minimum
Des. - Desirable (for consideration of landscaping or other aesthetic features)

TABLE 5.5B: MEDIAN WIDTH AND TYPES (URBAN)

Design Standard	Median Width (m)						Median Type
	Area Type						
	I		II		III		
	Min.	Des.	Min.	Des.	Min.	Des.	
U6	4.0	9.0	3.5	6.0	2.0	4.0	**B,C,E,F**
U5	3.0	6.5	2.5	4.0	2.0	3.0	**B,C,E**
U4	2.5	5.0	2.0	3.0	1.5	2.0	**A,B,C,D**
U3	2.0	4.0	1.5	2.0	1.5	2.0	**A,B,D**

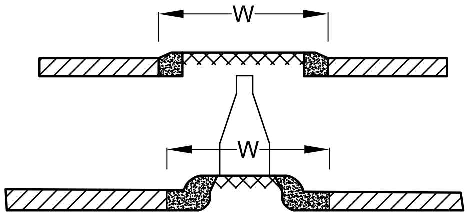
Source: Adapted from REAM GL 2/2002: A Guide on Geometric Design of Roads, Table 5-5B

Note:

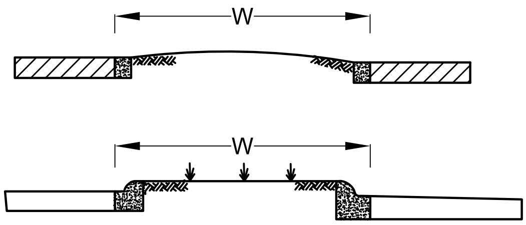
Min. - Minimum

Des. - Desirable (for consideration of landscaping or other aesthetic features)

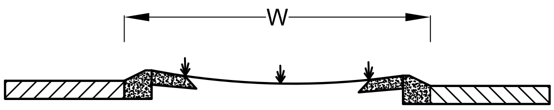
A) KERBED (TYPE A,B & C)



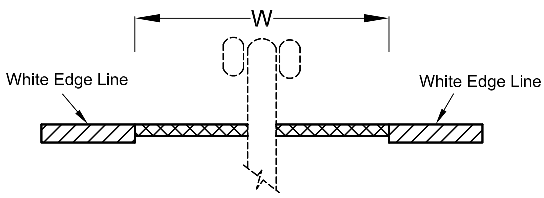
KERBED AND CROWNED PAVED
TYPE A



KERBED AND CROWNED TURF COVER
TYPE B

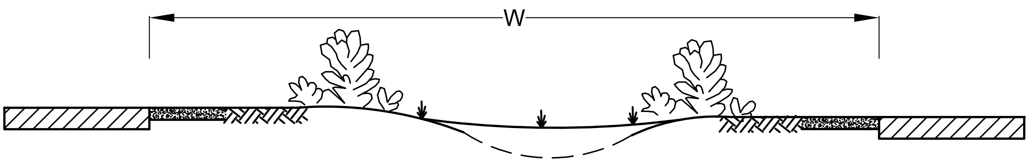


KERBED AND DEPRESSED TURF COVER
TYPE C

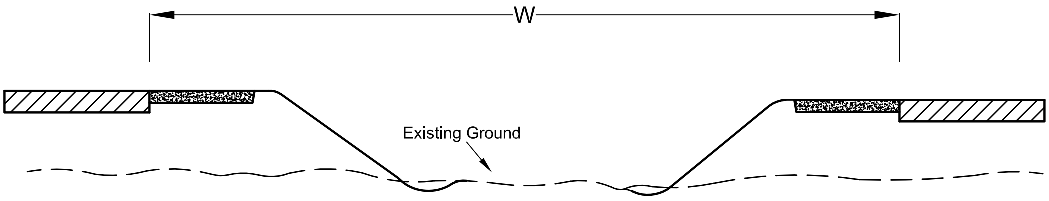


PAVED FLUSH SOMETIMES WITH BARRIER
TYPE D

B) NON KERBED (TYPE D,E & F)



FLUSH AND DEPRESSED TURF COVER
TYPE E



SEPARATE ROADWAYS NATURAL GROUND MEDIAN
TYPE F

W = Width of median

Notes :

1. Bushes and shrubs should not be allowed to be planted on the center median. Planting of trees and other decorative element for landscaping purposes should be referred to the local council or the approving authority for the latest guideline and approval.

2. For type A kerb i) Minimum values, w = 1.5m - 2.5m to utilize concrete barrier (NJB) only.
   ii) Desirable values, w_{DES} to be kerbed & crowned paved. (No NJB required).

FIGURE 5.4 : MEDIAN TYPES

Scale : Not to scale

5.8 Service Roads

5.8.1 General

Service roads are generally found in urban areas and they can have numerous functions, depending on the type of road they serve and the character of the surrounding area. They may be used to control access or function as a street facility serving adjoining property.

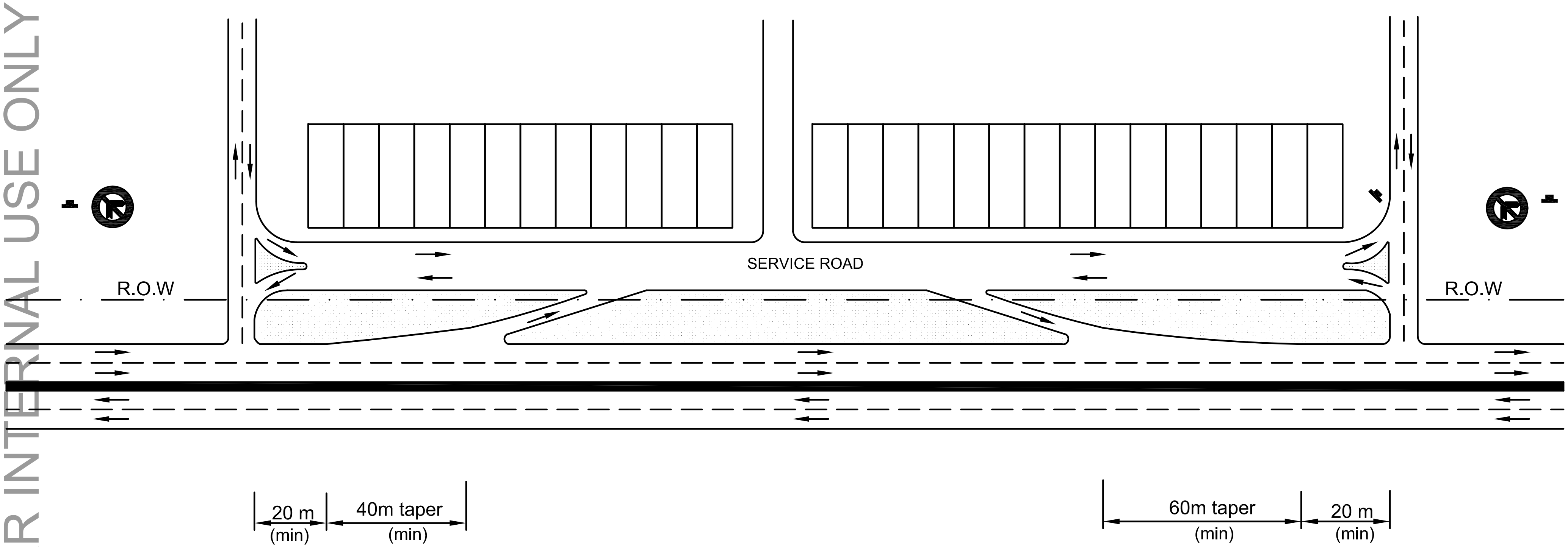
They segregate local traffic from the higher speed through traffic and intercept driveways of residences and commercial establishments along the road. Service roads also not only provide more favorable access for commercial and residential development than the faster moving arterials but also help to preserve the safety and capacity of the latter.

5.8.2 Design Requirements

From an operational and safety standpoint, one way service roads are much preferred to two-way and should be considered. One-way operation inconveniences local traffic to some degree, but the advantages in reduction in vehicular and pedestrian conflicts at intersecting streets often fully compensate for this inconvenience.

Two-way service roads may be considered for partially developed urban areas where the adjoining road system is so irregular and disconnected that one-way operation would introduce considerable added travel distance and cause undue inconvenience. Two-way service roads may also be necessary for suburban or rural areas where points of accesses to the through facility are infrequent i.e., where only one service road is provided, where roads connecting with the service roads are widely spaced or where there is no parallel street within reasonable distance of the service roads in urban areas that are developed or likely to be developed.

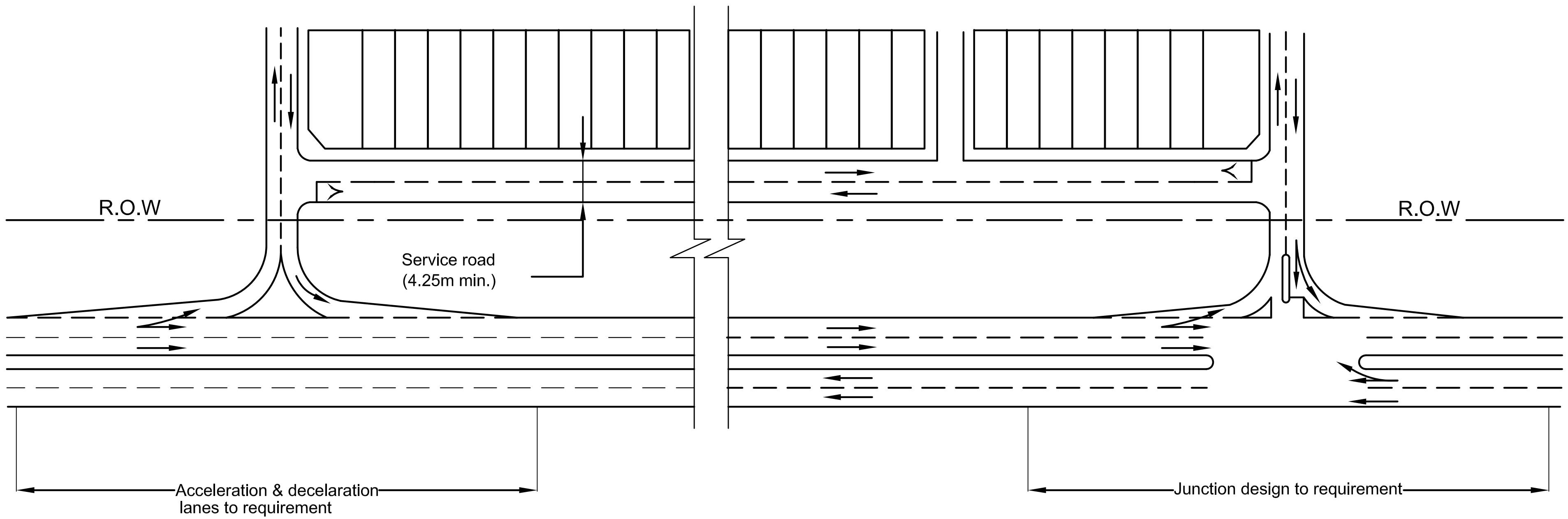
The design of a service road is affected by the type of service it is intended to provide. When provided, they should always cater for at least one side on-street parking. They should be at least 7.25m wide for one-way operation and 9.25m for two-way operation. **Figures 5.5A** and **5.5B** give the typical recommended layouts for a two-way operation service road fronting an urban arterial where the distance between junctions is greater than 0.5 km. The minimum reserve width for a service road is 12m.



Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 5-3

FIGURE 5.5A : TYPICAL LAYOUT OF A TWO-WAY SERVICE ROAD WITH MULTIPLE INGRESS AND EGRESS POINTS ON THE MAJOR ROAD

Scale : Not to scale



Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 5-3

FIGURE 5.5B : TYPICAL LAYOUT OF A TWO-WAY SERVICE ROAD WITH A LEFT-IN AND LEFT-OUT JUNCTION ON THE MAJOR ROAD

Scale : Not to scale

5.9 Pedestrian Crossings

5.9.1 General Considerations

Pedestrian crossings (whether level, overpass or underpass) should be provided where pedestrian volumes, traffic volumes, intersection capacity and other conditions favour their use. They may be warranted in areas of heavy peak pedestrian movements such as factories, schools, athletic fields or central business districts or where abnormal hazards or inconveniences to pedestrians would otherwise result.

Table 5-6 gives the general guidelines for determining the type of crossing that is required. Where the pedestrian and vehicle volumes do not fit into any of the category shown, judgement is needed in the assessment of the type of crossing required.

TABLE 5-6: GUIDELINE FOR TYPE OF CROSSING REQUIRED

Pedestrian Volume at Peak Hour (pedestrian/hr - pph)	Traffic Volume (total) at Peak Hour (vehicle/hr - vph)	Type of Crossing
<600	>600	Zebra Crossing
> 350 (each of three 1-hr periods) Or >175 (each of any eight 1-hr periods)	>600 (no central median) Or > 1000 (with central median)	SignalisedPedestrian Crossing

Source: NTJ 18/97, Austroad: Guide to Traffic Engineering Practice:Part 13 – Pedestrians (1995)

Where justified, the location and design of the pedestrian crossing would require an individual study. Where overhead pedestrian crossings are provided, side barriers must be installed to prevent jay-walking. Such barriers shall be installed for a distance of 75m on both sides of the location of the crossing. The minimum spacing between crossings is 400m.

For overhead pedestrian crossings, standard JKR designs shall be used as far as possible. These are available from the Road Design Unit.

5.9.2 School Level Crossings

The installation of a full traffic signal school level crossing should be considered when the following warrants are met:-

Either (a) 500 vehicles/hour on the road and 100 school children/hour crossing during the peak hours.

or, (b) 500 vehicles/hour on the road during the peak hour and a minimum of 500 school children during the entire day.

Where the above warrant for a full traffic signal school level crossing is not met, a partially signalized crossing, at least of the push-button type, or the un-signalized version with the aid of traffic warden may be provided.

5.10 U-Turns

5.10.1 General Considerations

Divided highways require median openings to accommodate vehicles making U turns in addition to right turning and cross traffic. Separate U-turn median openings may be required at the following locations:-

(i) Locations beyond intersections to accommodate minor turning movements not otherwise provided in the intersection or interchange area. The major intersection area is kept free for the important turning movements, in some cases obviating expensive ramps or additional structures.

(ii) Locations just ahead of an intersection to accommodate U-turn movements that would interfere with through and other turning movements at the intersection. Where a fairly wide median on the approach roadway has few openings, U-turning is necessary to reach roadside areas. Advance separate openings to accommodate them outside the intersection proper will reduce interference.

(iii) Locations occurring in conjunction with minor crossroads where traffic is not permitted to cross the major road but instead is required to turn left, enter the through traffic stream, weave to the right, U-turn, then return. On high-speed or high-volume roads, the difficulty and long lengths required for weaving with safety usually make this design pattern undesirable unless the volumes intercepted are light and the median is of adequate width. This condition may occur where a crossroad with high volume traffic, a shopping area, or other traffic generator that requires a median opening nearby and additional median openings would not be practical.

(iv) Locations occurring where regularly spaced openings facilitate maintenance operations, policing, repair service of stalled vehicles, or other highway-related activities. Openings for this purpose may be needed on controlled-access roads and on divided roads through undeveloped areas.

(v) Locations occurring on roads without control of access where median openings at optimum spacing are provided to serve existing frontage developments and at the same time minimize pressure for future median openings.

5.10.2 Design Considerations

U-turning vehicles interfere with through traffic by encroaching on part or all of the through traffic lanes. They are made at low speeds and the required speed change is normally made on the through traffic lanes with weaving to and from the outer lanes. As such U-turn facilities are potential hazard areas. Careful consideration should be made not only on the need for it but also on the type and location of the U-turns.

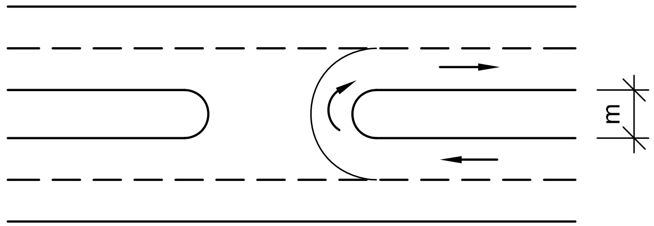
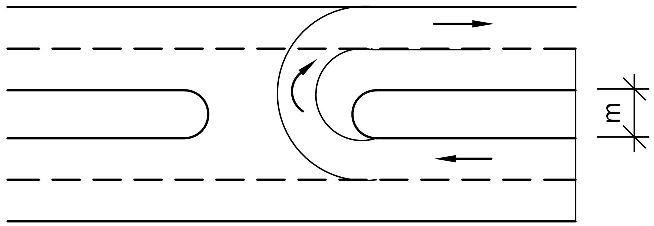
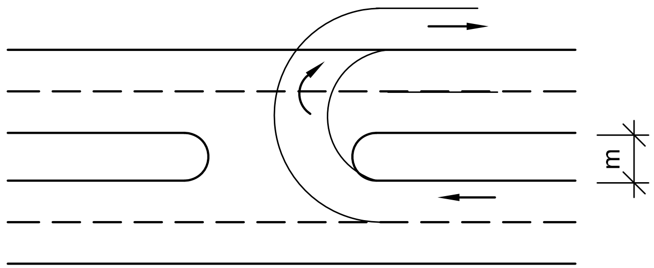
(i) Minimum Design for Direct U-Turns

For direct U-Turns, the width of the highway, including the median should be sufficient to permit the turn to be made without encroachment beyond the outer edges of the pavements. **Figure 5.6** gives the minimum width of the medium required for the various types of maneuvers for direct U-turns.

For direct U-Turns, the layout in **Figure 5.7A** and **Figure 5.7B** is suggested. Where direct U-Turns will not be feasible because of median restrictions, indirect U-Turns can be used.

(ii) Indirect U-Turns

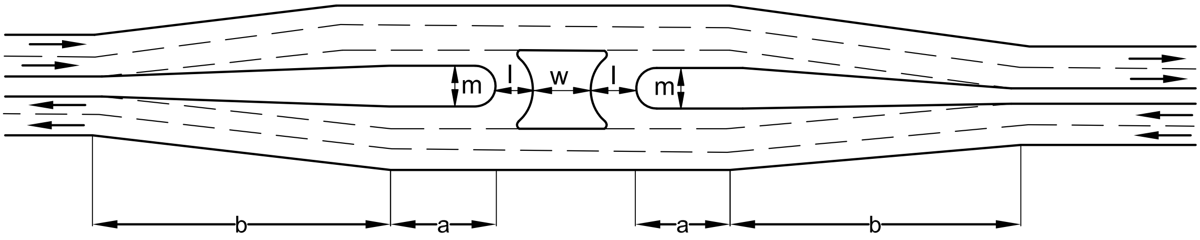
Indirect U-Turns can take many forms, the simplest being to allow traffic to use existing local streets or to go around the block for their turning movements. Where medians are narrow, the special indirect U-turn as shown in **Figure 5.8** can be used. Indirect U-Turns are only to be used for Road Standards R4, U4 and U3 only.

TYPE OF MANEUVER		MINIMUM WIDTH OF MEDIAN - (m) METRE FOR DESIGN VEHICLE		
		P	SU	WB-15
INNER LANE TO INNER LANE		9.75	19.50	21.25
INNER LANE TO OUTER LANE		6.00	15.75	17.75
INNER LANE TO SHOULDER		3.00	12.75	14.50

Source : REAM GL 2/2002 A Guide on Geometric Design of Roads, Figure 5-4

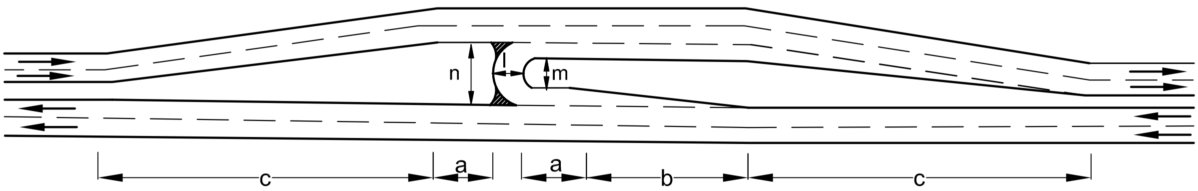
FIGURE 5.6 : MINIMUM DESIGN OF DIRECT U-TURNS

Scale : Not to scale



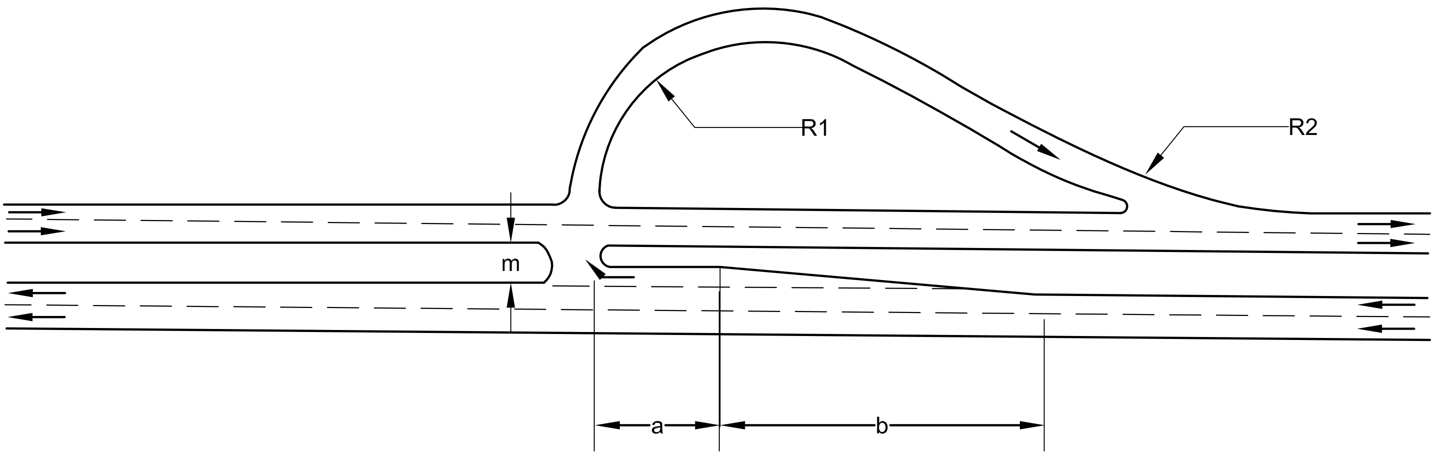
DIMENSION	MINIMUM DISTANCE (m)
m	10.0
l	7.5
w	10.0
a	20.0
b	60.0

FIGURE 5-7A: RECOMMENDED LAYOUT FOR DIRECT U-TURN (LOW SPEEDS)



DIMENSION	MINIMUM WIDTH / DISTANCE (m)
a	20
b	60
c	120
l	7.5
m	10.0
n	17.0

FIGURE 5-7B: RECOMMENDED LAYOUT FOR DIRECT U-TURN (HIGH SPEEDS)



DIMENSION	MINIMUM DISTANCE (m)
m	4
l	8
a	20

DIMENSION	MINIMUM DISTANCE (m)
R1	50
R2	100
b	60

FIGURE 5.8: SPECIAL INDIRECT U-TURN

Scale : Not to scale

(iii) Locations for U-Turns

The locations of U-Turns are very important and proper consideration should be given to it. As a general guide, **Table 5.7** can be used to determine the minimum distances between U-turns.

TABLE 5.7: DISTANCES BETWEEN U-TURNS

Design Standard	Distance Between U-Turns	Design Standard	Distance Between U-Turns
R6	No U-turns allowed	U6	No U-turns allowed
R5	3 km	U5	2 km
R4	2 km	U4	1km
		U3	1km

For road of design standards other than those in **Table 5.7**, the provision of U-Turns has to be carefully considered, if necessary, and has to take into account of sufficient median and lane widths for the turning radius of vehicles.

5.11 Bridge and Structure Cross Section

5.11.1 Width of Shoulders

The width of the usable shoulders should follow that as indicated in **Table 5.3A** and **Table 5.3B**. The width of the shoulder of the bridges and structures should be the same as that of the carriageway.

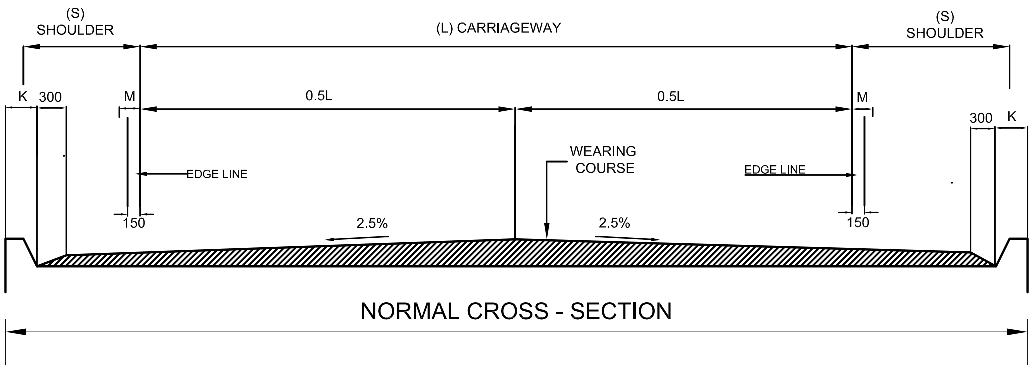
Figure 5.9 shows the typical bridge cross-section to be used.

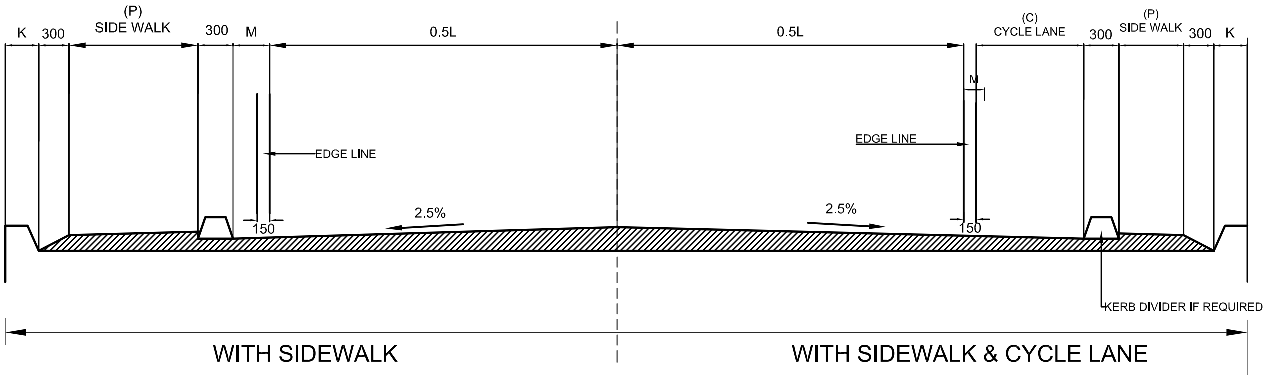
5.11.2 Required Clearance

(i) The clear vertical height of all structures above the roadway shall be a minimum of 5.4 meters over the entire width of traffic lanes, auxiliary lanes and it is recommended that an additional 0.1 meter be allowed for future pavement resurfacing. Whenever resurfacing will reduce the clearance to less than 5.4 meters, milling will be required to maintain the minimum clearance.

(ii) For clearance requirement over railways and waterways, reference should be made to the relevant authorities.

(iii) **Figure 5.10** indicates the clearance required for underpasses of various cross-sections.





DESIGN STANDARD	CARRIAGEWAY (L)	SHOULDER (S)	MARGINAL STRIP (M)
R6/U6	7300	3000	0
R5/U5	7000	3000	0
R4/U4	7000	3000	0
R3/U3	6500	2500	0
R2/U2	6000	2000	0
R1/U1	5000	1500	0

Where,

K = Kerb
C = Cycle lane
P = Pedestrian walk
M = Marginal Strip

Notes:

1. All dimensions are in millimetres unless stated otherwise.
2. Desirable width of sidewalk 'P' is 2.0m. A minimum width to be used is 1.25m.
3. Width of cycle lane 'C' is as indicated in Table 5-12.
4. Minimum width of 'K' to be used is 500mm. For provision of service ducts, this value is to be increased.
5. Kerbs for bridges are left to discretion.

FIGURE 5.9 : TYPICAL BRIDGE CROSS-SECTION

Scale : Not to scale

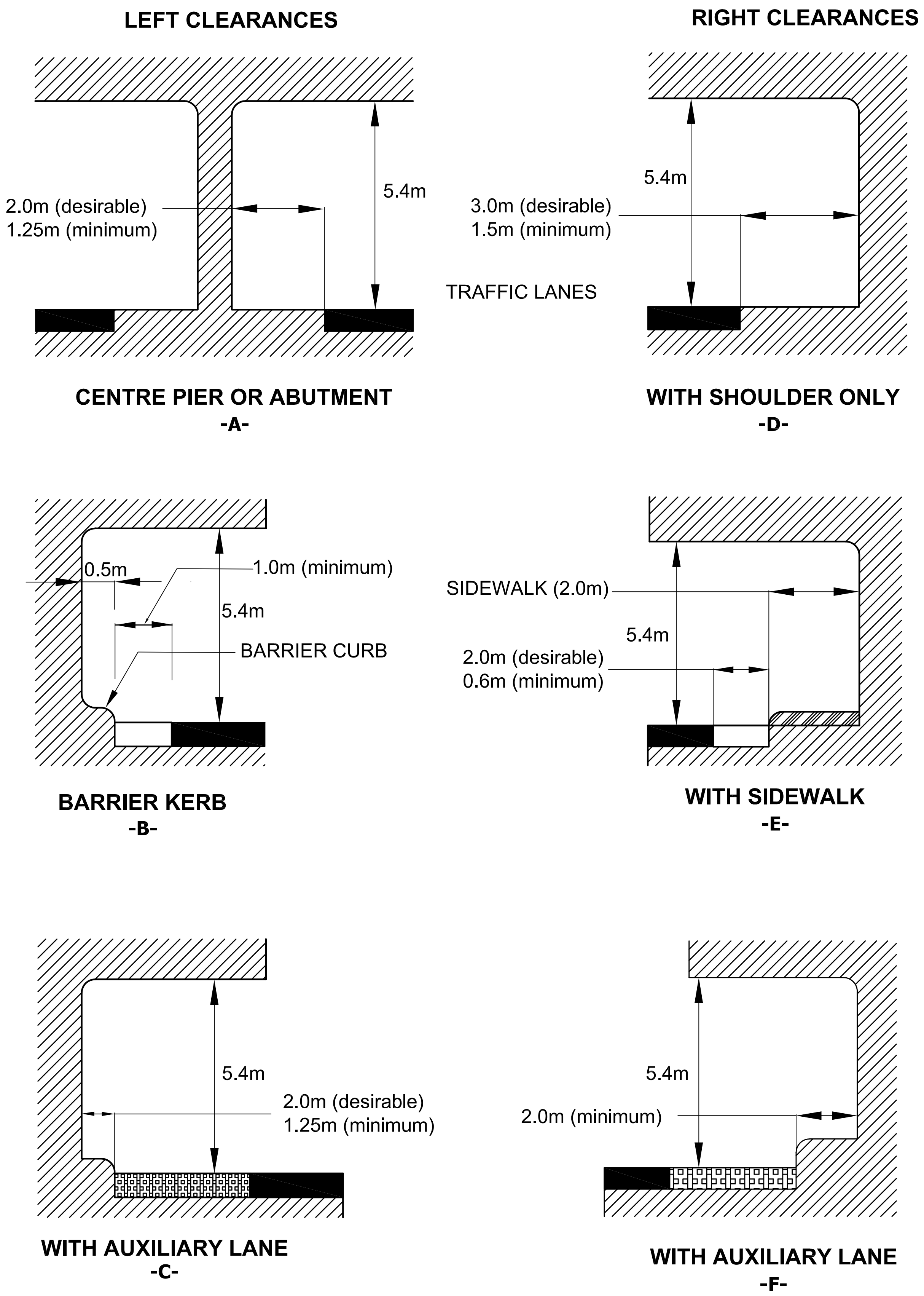


FIGURE 5.10 : CLEARANCES AT UNDERPASS

Scale : Not to scale

5.12 Bus Lay-bys

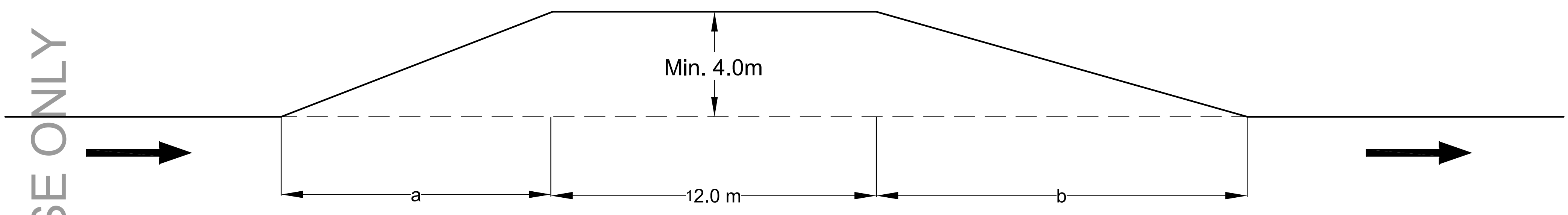
Bus lay-bys serve to remove the bus from the through traffic lanes. Its location and design should therefore provide ready access in the safest and most efficient manner possible. The basic requirement is that the deceleration, standing and acceleration of the buses be effected on pavement areas clear of and separated from the through traffic lanes.

The locations of bus lay-bys are important so as not to impede the normal flow of traffic. In this respect, bus lay-bys must not be located on any interchange ramps or structures, slip roads or within 60m of any junction or intersection. The distance between lay-bys too should not be less than 150m. Liaison will need to be made also with the relevant bus companies and the respective Local Authority.

For school areas, provision of bus lay-bys and parking areas should be specifically studied so as to ensure the unimpeded flow of traffic.

Figure 5.11 shows the typical layout and dimensions of bus lay-bys that are to be used in both rural and urban areas. If possible, a dividing area between the outer edge of the roadway shoulder and the edge of the bus layby lane should be provided. This dividing area should be as wide as possible but not less than 0.6m.

The pavement areas of the lay-bys should be of concrete for contrast in colour and texture with the through traffic lanes so to discourage through traffic from encroaching on or entering the bus stop.



DESIGN STANDARD	DIMENSION (m)	
	b	a
R5	70	35
R4	50	35
R3	35	25
R2	25	25
R1	20	10

DESIGN STANDARD	DIMENSION (m)	
	b	a
U5	50	35
U4	35	25
U3	25	25
U2	20	10
U1	20	10

FIGURE 5.11 : LAYOUT OF BUS LAY-BYS

Scale : Not to scale

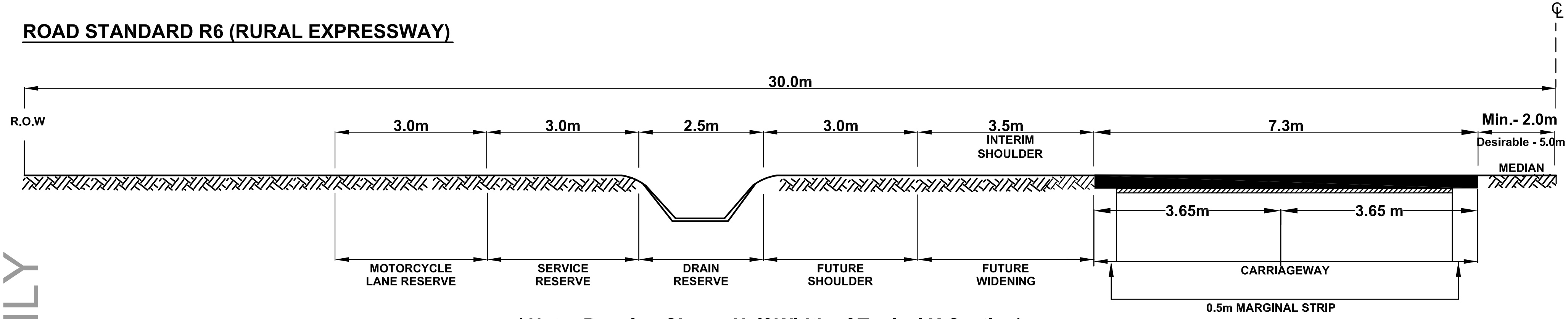
5.13 Minimum Reserve Width

An adequate road reserve width is important to cater not only for the present demands of traffic but more so for the future requirements as by then, it may not be possible to acquire any more additional land. **Figures 5.12A** and **5.12B** show examples of the typical cross-sections for various road standards in rural and urban areas together with the minimum reserve required, **Figure 5.12C** a typical cross-section of service reserve to cater for the necessary utility services, while **Table 5.8** lists the minimum reserve widths for the various road standards. The values as shown in **Table 5.8** are for road standards in flat areas and will need to be increased accordingly for areas involving deep cuts or fills.

TABLE 5.8: MINIMUM RESERVE WIDTH

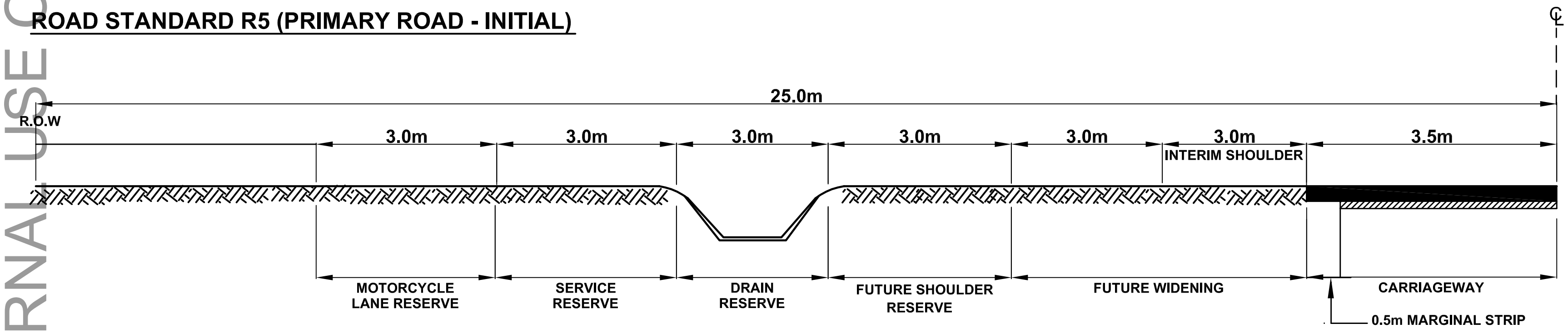
Area	Road Category	Design Standard	Minimum Reserve Width (m)
RURAL	Expressway	R6	60
	Highway	R5	60
	Primary Road	R5 R4	50 40
	Secondary Road	R4 R3	30 25
	Minor Road	R2 R1	20 20
URBAN	Expressway	U6	65
	Arterials	U5 U4	65 40
	Collector	U4 U3	40 40
	Local Street	U3 U2 U1	40 30 25

ROAD STANDARD R6 (RURAL EXPRESSWAY)



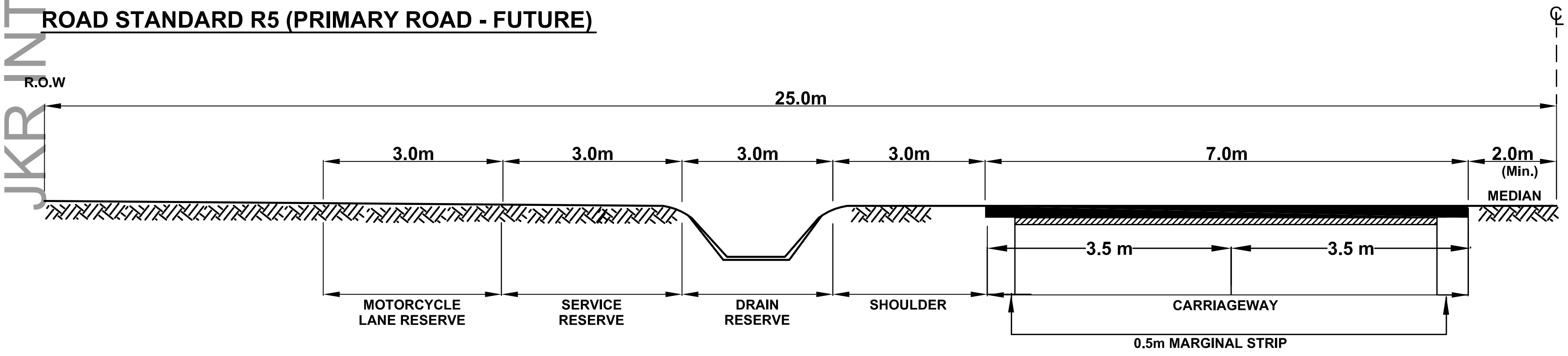
(Note: Drawing Shows Half Width of Typical X-Section)

ROAD STANDARD R5 (PRIMARY ROAD - INITIAL)



(Note: Drawing Shows Half Width of Typical X-Section)

ROAD STANDARD R5 (PRIMARY ROAD - FUTURE)



(Note: Drawing Shows Half Width of Typical X-Section)

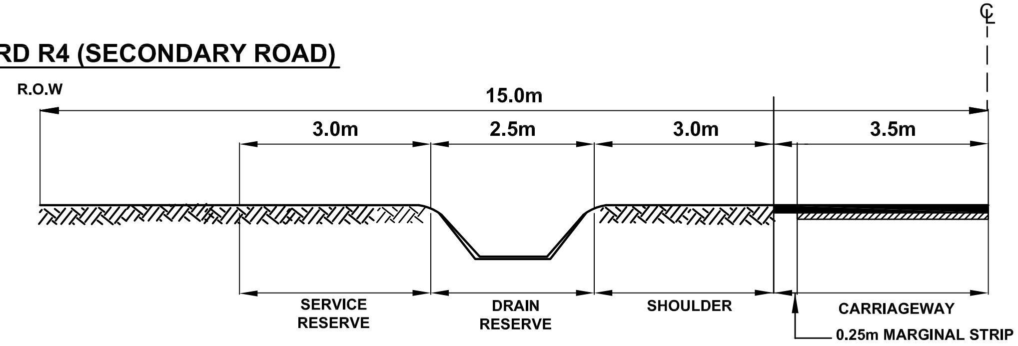
Notes:

The typical cross-section represents the ideal case to be adopted. However, actual cross-sectional elements to be constructed has to be referred to and approved by the S.O/P.D. before commencement of works.

FIGURE 5.12A : TYPICAL CROSS SECTION (RURAL AREA)

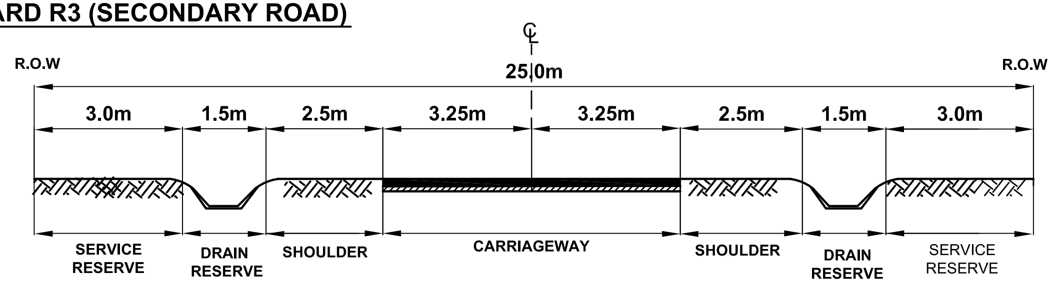
Scale : Not to scale

ROAD STANDARD R4 (SECONDARY ROAD)



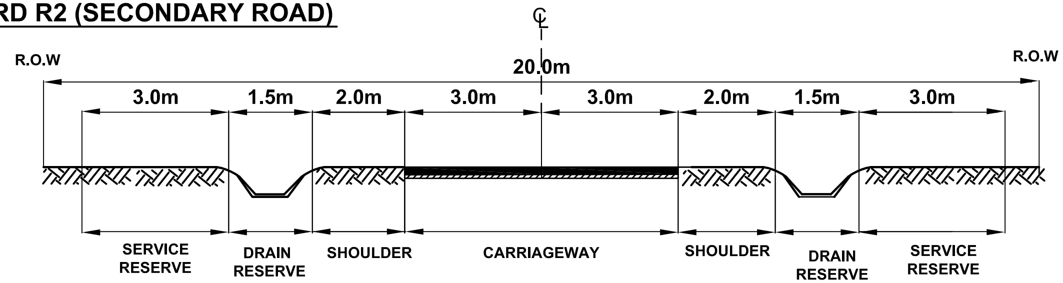
(Note: Drawing Shows Half Width of Typical X-Section)

ROAD STANDARD R3 (SECONDARY ROAD)



(Note: Drawing Shows Full Width of Typical X-Section)

ROAD STANDARD R2 (SECONDARY ROAD)

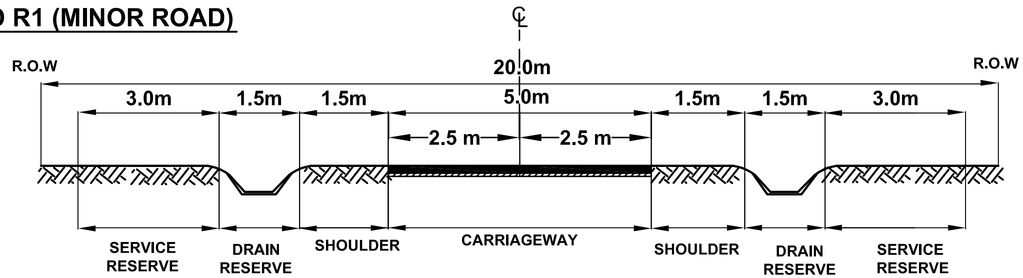


(Note: Drawing Shows Full Width of Typical X-Section)

Notes:

The typical cross-section represents the ideal case to be adopted. However, actual cross-sectional elements to be constructed has to be referred to and approved by the S.O/P.D. before commencement of works.

ROAD STANDARD R1 (MINOR ROAD)

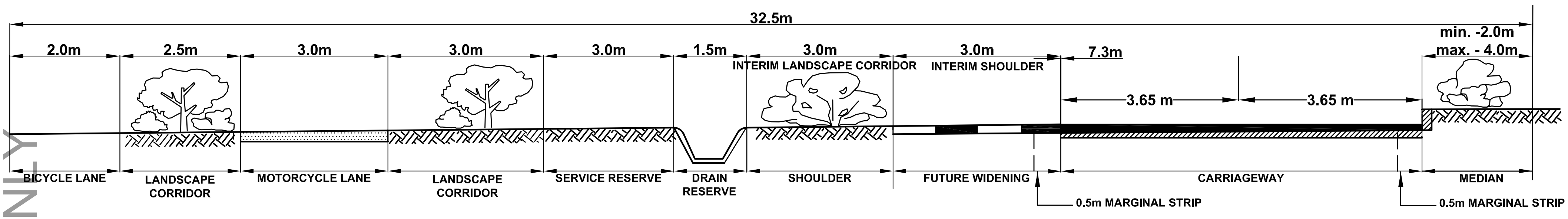


(Note: Drawing Shows Full Width of Typical X-Section)

FIGURE 5.12A : TYPICAL CROSS SECTION (RURAL AREA) (CONT.)

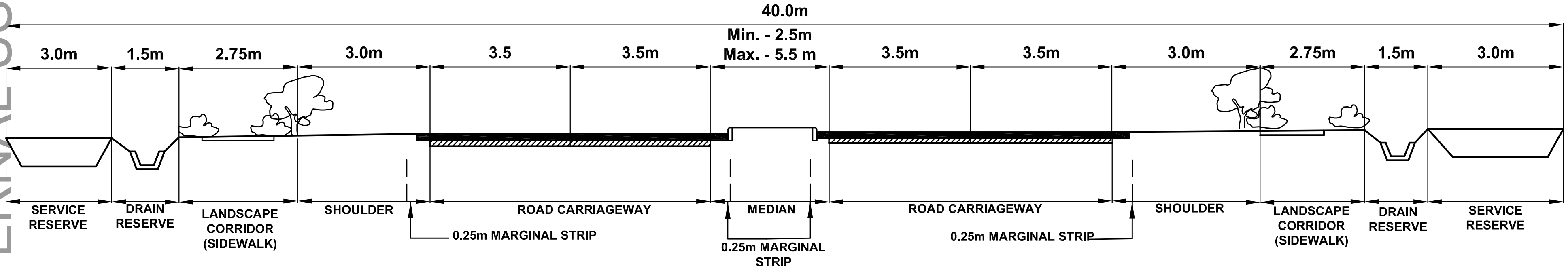
Scale : Not to scale

ROAD STANDARD U6 (URBAN EXPRESSWAY)



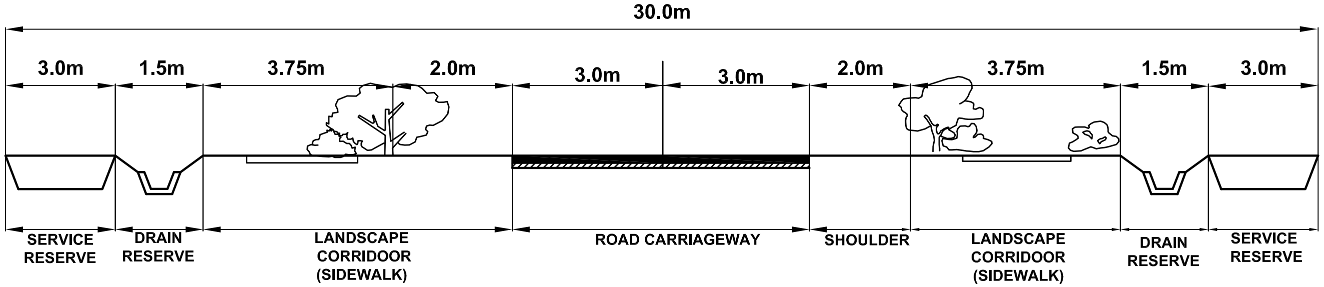
(Note: Drawing Shows Half Width of Typical X-Section)

ROAD STANDARD U4 (MAJOR COLLECTOR)



(Note: Drawing Shows Full Width of Typical X-Section)

ROAD STANDARD U2 (LOCAL STREET)



(Note: Drawing Shows Full Width of Typical X-Section)

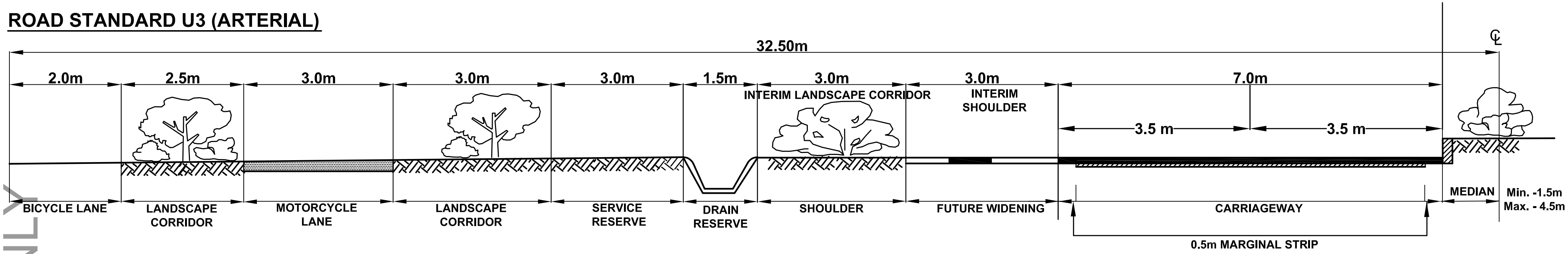
Notes:

The typical cross-section represents the ideal case to be adopted. However, actual cross-sectional elements to be constructed has to be referred to and approved by the S.O/P.D. before commencement of works.

FIGURE 12.B : TYPICAL CROSS SECTION (URBAN AREA)

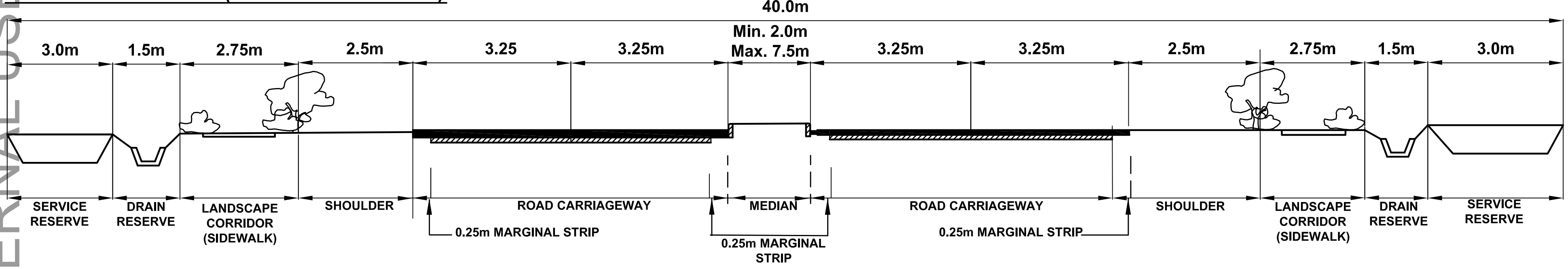
Scale : Not to scale

ROAD STANDARD U3 (ARTERIAL)



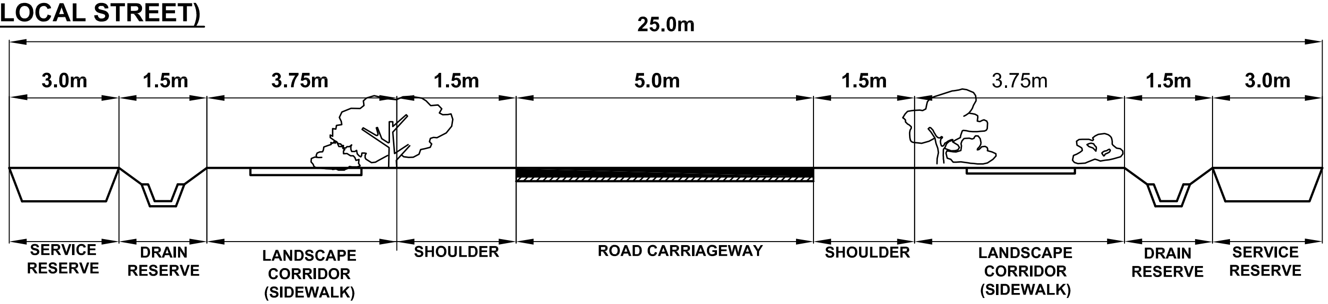
(Note: Drawing Shows Half Width of Typical X-Section)

ROAD STANDARD U3 (MAJOR COLLECTOR)



(Note: Drawing Shows Full Width of Typical X-Section)

ROAD STANDARD U1 (LOCAL STREET)



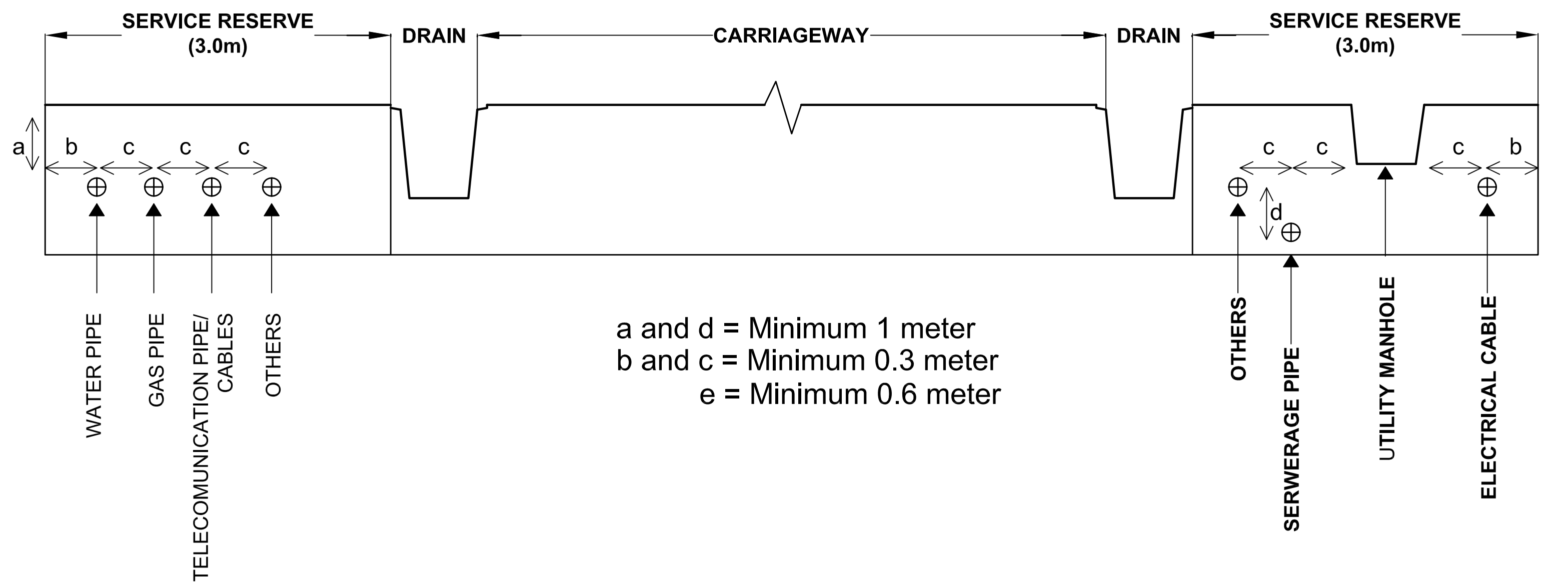
(Note: Drawing Shows Full Width of Typical X-Section)

Notes:

The typical cross-section represents the ideal case to be adopted. However, actual cross-sectional elements to be constructed has to be referred to and approved by the S.O/P.D. before commencement of works.

FIGURE 12.B : TYPICAL CROSS SECTION (URBAN AREA)(CONT.)

Scale : Not to scale



Typical cross section showing an example of spacing & arrangement of utilities in utility services reserve
(not to scale)

Notes: Actual spacing to refer to relevent utility agency

Source : GP006-A Garis Panduan Perancangan Laluan Kemudahan Utiliti, Figure 3

FIGURE 5.12C :TYPICAL CROSS SECTION (SERVICE RESERVE)

Scale : Not to scale

5.14 Motorcycle Lanes

5.14.1 Types of roads

Motorcycle facilities are to be provided along two (2) types of roads: -

(i) **Expressways:** Defined as a multi-lane high speed high volume road with full/partial control of access and grade separated interchanges all along the road. These include tolled roads and major Federal Roads.

(ii) **Non-expressways**: Defined as roads other than expressways and include Federal Roads, State Roads, Municipal Roads and other major roads.

5.14.2 Types of facilities

There are two (2) types of facilities provided for motorcycles: -

(i) Exclusive motorcycle lane

Defined as a roadway meant exclusively for use by motorcycles (motorcyclists are compelled by law to use it and other vehicles are prohibited by law from using it). It is physically separated from the main carriageway and is grade separated from the main carriageway at intersections/interchanges/points of conflicts. Details of the design of exclusive motorcycle lane should be made to the relevant document published by Public Works Department Malaysia.

(ii) Non-exclusive motorcycle lane

Defined as the extra lane or verge/ paved shoulder or marginal strip on the left hand side of a road where motorcycles are encouraged/required to use while riding along a road.

5.14.3 Non Exclusive Motorcycle Lane

5.14.3.1 When to Consider this Type of Facility

(i) On rural roadways where motorcycle travel is common, such as interstate routes, paved shoulders are highly desirable.

(ii) On secondary roadways where there are few commercial driveways and intersections with other roadways, many motorcyclists prefer riding on wide and smoothly paved shoulder.

(iii) On Major Federal and State Roads meeting the warrants specified in 5.14.3.2 of this guideline.

(iv) On urban streets, wide outside lanes and motorcycle lanes are usually not the preferred facilities due to proximity of the junctions.

5.14.3.2 Warrants

Non-exclusive motorcycle lane can be considered when the following conditions are met: -

(i) Two lane single carriageway with more than 15,000 vehicles per day/Dual carriageway with more than 10,000 vehicles per lane/day.

(ii) The composition of motorcyclist is more than 30% of the total traffic volume.

(iii) Five (5) numbers of accidents involving motorcyclist per year (within any one kilometer stretch).

5.14.3.3 Planning and Design Considerations

It is recommended that the shoulder for motorcycle lane be constructed of the same elements of design, materials and structure as the mainline pavement in order to facilitate construction, improve pavement performance and reduce maintenances costs. Widened lanes for motorcycles reduce edge stresses and the potential for edge drop-offs, increase safety and reduce maintenance costs.

Sufficient right of way is needed to accommodate the additional width of the non-exclusive motorcycle/paved shoulders and the relocation of drainage ditches that run parallel to the roadway.

Partial depth paved shoulders are rarely recommended because of the tendency to crack under vehicular loads. The non-exclusive motorcycle lane should be smoothly paved, have adequate strength and stability to support vehicle loads without rutting. The minimum width of the paved shoulder to accommodate motorcycles is 2.0 m - 2.5 m. The slope of the road should continue across the paved shoulder to maintain adequate drainage.

The non-exclusive motorcycle lane is adjacent to the traffic lane and separated by chevron markers with red colored surfacing (refer to Standard Specification for Road Work (JKR/SPJ/2008-S4 Section 4: Flexible Pavement) of 0.75 m width to indicate segregation or separation from the traffic main stream. The typical cross section of the non-exclusive motorcycle lanes are shown in the **Figure 5.13**.

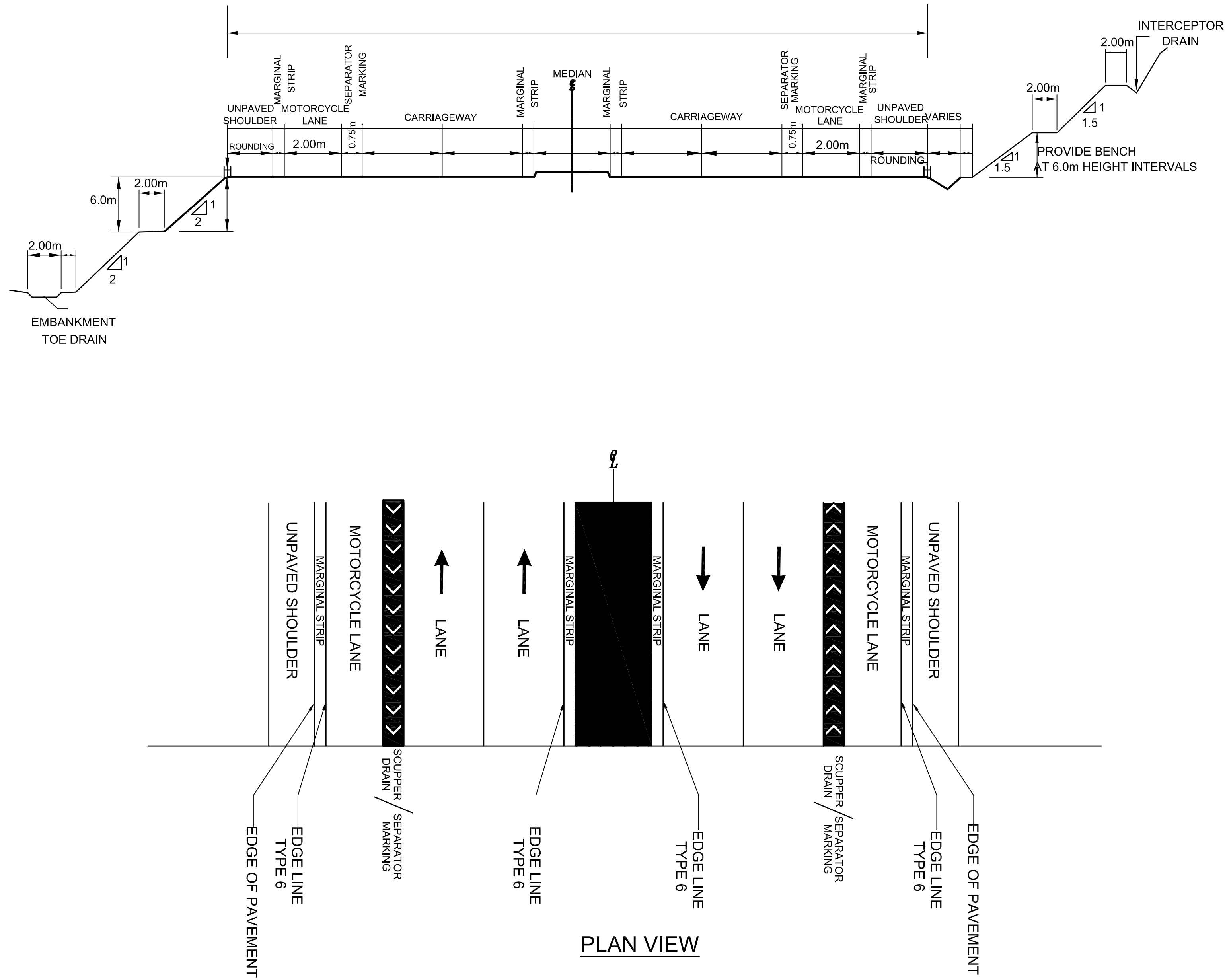


FIGURE 5.13 :TYPICAL PLAN AND CROSS SECTION OF THE NON-EXCLUSIVE CYCLE LANE

Scale : Not to scale

5.14.3.4 Treatment of Non-exclusive Motorcycle Lane at Points of Conflict

At intersections or interchanges, some form of channelization with specific routes for the motorcyclists should be provided to minimize conflicts that could arise. Possible intersection treatment types include those of at-grade or grade-separated. The type of intersection treatment will depend on the volume of the traffic and the volume of motorcycles.

Details of the treatment of non-exclusive motorcycle lane at points of conflict should be referred to the relevant document published by Public Works Department Malaysia.

6.0 OTHER ELEMENTS AFFECTING GEOMETRIC DESIGN

6.1 Drainage

Road drainage facilities provide for carrying water across the right of way and for removal of storm water from the road itself (surface run-off). These facilities include bridges, culverts, channels, gutters and various types of drains. Drainage design considerations are an integral part of geometric design and frequent flood plain encroachments will affect the highway alignment and profile if not done so.

The cost of drainage is neither incidental nor minor on most roads. Careful attention to requirements for adequate drainage and protection of the highway from floods in all phases of design and locations will prove to be effective in reducing costs in both construction and maintenance.

Reference should be made **to latest edition of ATJ 15/97, JPS MSMA Rev.2.0** and relevant JKR manual on this subject.

6.2 Lighting

Lighting may improve the safety of a road and the ease and comfort of operation thereon. Lighting of rural highways may be desirable but the need is much less than on roads in urban areas. They are seldom justified except at critical locations such as, interchanges, intersections, railroad grade crossings, narrow or long bridges, tunnels and areas where roadside interference is a factor.

Where lighting is being considered for future installation, considerable savings can be affected through design and installation of necessary conduits under the pavements and kerbs as part of the initial construction and should be considered.

Reference should be made to **latest edition of JKR/SPJ/2011-S7- Standard Specification for Road Works Section 7: Road Lighting** and relevant JKR manual on this subject.

6.3 Utilities

All road improvements, whether upgraded within the existing Right of Way or entirely on new Right of Way, generally involves the shifting of utility facilities. Although utilities generally have little effect on the geometric design of the road, full consideration should be given to measures necessary to preserve and protect the integrity and visual quality of the road, its maintenance efficiency and the safety of traffic.

Utilities relocation should form part of design works and the designer should liaise closely with the relevant Service Authorities in determining the existing utilities and their proposed relocation. Depending on the location of a project, the utilities involved could include Sewer Pipelines, Water Supply Pipelines, Drainage and Irrigation Lines, Oil and Gas Pipelines, Telecommunication Lines and Electric Cables.

Reference should be made to **latest edition of ATJ 4/85 (Pind. 1997) – Installation of Public Utility in Road Reserve and ATJ 3/2011 – Garis Panduan Memproses Permohonan Pembangunan Tepi Jalan Persekutuan.**

6.4 Signages and markings

Signages and markings are directly related to the design of the road and are features of traffic control and operation that the engineer must consider in the geometric layout of such facility. The signing and marking should be designed concurrently with the geometrics as an integral part as this can significantly reduce the possibility of future operational problems. The signages and markings should follow the standards that have been established. Reference should be made to the **latest edition Arahan Teknik (Jalan) 2A, 2B, 2C and 2D and REAM - GL 8/2004: (Guidelines on Traffic Control and Devices, Part 4: Pavement Marking and Delineation)** on the design, usage and application of signs and markings.

6.5 Traffic Signals

Traffic control signals are devices that control vehicular and pedestrian traffic by assigning the right-of-way to various movements for certain pretimed or traffic actuated intervals of time. They are one of the key elements in the function of many urban roads and should be integrated with the geometric design so as to achieve optimum operational efficiency.

Reference should be made to latest edition of **Arahan Teknik (Jalan) 13/87 A Guide to the Design of Traffic Signals.**

6.6 Erosion Control, Landscape Development and Environmental Impacts

Erosion prevention is one of the major factors in the design, construction and maintenance of highways. It should be considered early in the design stage and possible high risk locations identified. Some degree of erosion control measures can be incorporated into the geometric design, particularly in the cross-section elements.

Landscape development should be in keeping with the character of the road and its environment. The general areas of improvement include the following: -

(i) Preservation of existing vegetation

(ii) Transplanting of existing vegetation where feasible

(iii) Planting of new vegetation

(iv) Selective clearing and thinning

(v) Regeneration of natural plant species and material.

Landscaping of urban roads assume an additional importance in mitigating the many nuisances associated with urban traffic.

Reference should be made to the relevant JKR Manual / relevant Authorities on this subject, namely: -

(i) Latest edition of ATJ 16/03 (Pind. 2008)

(ii) Cawangan Alam Sekitar & Tenaga

(iii) ESCP, MSMA (JPS)

(iv) Jabatan Alam Sekitar

(v) Jabatan Lanskap Negara

The highway can and should be located and designed to complement its environment and serves to cater for future environmental improvement. The area surrounding a proposed highway is an interrelated system of natural, manmade and sociological variables. Changes in one variable within this system will undoubtedly affect the others. Some of these consequences may be negligible, but others may have a strong and lasting impact on the environment, including those involving the sustenance and quality of human life.

Because highway location and design decisions have an effect on adjacent areas and its surrounding development, it is thus important that environmental variables be given full consideration. Environmental impacts include those of social, economic and physical. In geometric design, only physical impacts are assessed, while the social and economic impacts would have to be taken care of during the planning stage.

6.7 <u>Emergency Escape Ramps</u>

Roads through rolling and mountainous terrain are sometimes designed under constrained circumstances due to various reasons including environmental and economic considerations. Hence, adverse road geometric features are sometimes unintentionally introduced. Among such features is the design of substandard vertical profile featuring long steep downgrade. Long downgrade alignments may be quite common but some may present serious problems especially to heavy laden commercial or transport vehicles.

The inability of drivers to control vehicle speeds on downgrades is not only hazardous but it can also have costly consequences. Heavy commercial vehicles and long haul transport buses travelling on roads in rolling and mountainous terrain encounter the risk of losing control on these long steep downgrades. They may end up falling into ravines, crashing onto the faces of slope or populated build-up areas that are usually located at the bottom of valley areas.

Continuous use of brakes by drivers may develop excessive heating on the brake system that usually resulted in 'brake fading'. Brake fade is a term used to describe the partial or total loss of braking power used in a vehicle brake system. Brake fade occurs when the brake pad and the brake rotor no longer generates sufficient mutual friction to stop the vehicle at its preferred rate of deceleration and can happen on motorcycles, cars, buses and heavy commercial vehicles. The brake pad in any brake system is designed to work at certain operating temperatures. This certainly indicates the fitness of a brake pad for application and its general quality.

Runaway vehicles need special exit facilities to enable the vehicles to reduce its speed and regain control without causing serious casualties or affect other road users. These facilities, in the form of ramps, need to be properly located and adequately designed to ensure its effective use. Adequate length of ramps and correct entry alignment enhanced with sufficient alerting features, are some of the design elements of the emergency escape ramps including the use of suitable arrester bed material and containment features.

Details of the design of emergency escape ramps should be referred to the relevant document published by Public Works Department Malaysia.

GENERAL SUMMARY - GEOMETRIC DESIGN CRITERIA FOR ROADS IN URBAN AREAS (METRIC)

DESIGN CONTROL AND CRITERIA	1	DESIGN STANDARD	-	U6			U5			U4			U3			U2			U1		
	2	ACCESS CONTROL	-	FULL			PARTIAL			PARTIAL/NIL			PARTIAL/NIL			NIL			NIL		
	3	AREA TYPE	-	F	R	M	F	R	M	F	R	M	F	R	M	F	R	M	F	R	M
	4	DESIGN SPEED	km/hr	100	80	60	80	60	50	70	60	50	60	50	40	50	40	30	40	30	20
CROSS SECTION ELEMENTS	5	LANE WIDTH	m	3.65			3.50			3.50			3.25			3.00				(5.00) a	
	6	USABLE SHOULDER WIDTH	m	3.00	3.00	2.50	3.00	3.00	2.50	3.00	2.50	2.00	2.50	2.00	1.50	2.00	1.50	1.50	1.50	1.50	1.50
	7	MEDIAN WIDTH (MINIMUM)	m	4.00	3.50	2.00	3.00	2.50	2.00	2.50	2.00	1.50	2.00	1.50	1.50	N / A			N / A		
	8	MEDIAN WIDTH (DESIRABLE)	m	9.0	6.0	4.0	6.5	4.0	3.0	5.0	3.0	2.0	4.0	2.0	2.0	N / A			N / A		
	9	MARGINAL STRIP WIDTH	m	0.50			0.50			0.25			0.25			0.25			0.00		
	10	MINIMUM RESERVE WIDTH	m	65			65				40			40		30			25		
ELEMENTS OF DESIGN	11	STOPPING SIGHT DISTANCE	m	185	130	85	130	85	65	105	85	65	85	65	50	65	50	35	50	35	20
	12	PASSING SIGHT DISTANCE	m	670	540	410	540	410	345	485	410	345	410	345	270	345	270	200	270	200	200
	13	MINIMUM RADIUS (AT MAX SE 6%)	m	435	250	135	250	135	90	195	135	90	135	90	55	90	55	30	55	30	15
	14	MINIMUM LENGTH OF SPIRAL (AT MAX SE- 2 LANES)	m	48	43	36	43	36	33	39	36	33	36	33	31	33	31	29	31	29	25
	15	MAXIMUM SUPERELEVATION	Ratio	0.06			0.06			0.06			0.06			0.06			0.06		
	16	MAXIMUM GRADE (DESIRABLE)	%	3	4	5	4	5	6	5	6	7	6	7	8	7	8	9	7	8	9
	17	MAXIMUM GRADE	%	3	5	7	6	8	11	8	10	12	9	11	13	7	11	16	7	11	17
	18	CREST VERTICAL CURVE (K)	-	78	39	17	39	17	10	26	17	10	17	10	10	10	10	5	10	5	3
	19	SAG VERTICAL CURVE (K)	-	45	30	18	30	18	13	23	18	13	18	13	9	13	9	6	9	6	3

REMARK :
1) ALL VALUES SHOWN ABOVE ARE MINIMUM/MAXIMUM VALUES. ALL EFFORT SHOULD BE MADE TO ACHIEVE AS HIGH A VALUE AS POSSIBLE
2) FOR DEFINITION OF AREA TYPE, SEE TABLE 3 - 2
3) ABBREVIATION : N/A - NOT APPLICABLE
( ) a - TOTAL WIDTH OF PAVEMENT
( ) b - RESERVE WIDTH DEPENDS ON ROAD CATEGORY

GENERAL SUMMARY - GEOMETRIC DESIGN CRITERIA FOR ROADS IN RURAL AREAS (METRIC)

DESIGN CONTROL AND CRITERIA	DESIGN STANDARD	-		R6			R5			R4			R3			R2			R1	
	ACCESS CONTROL	-		FULL			PARTIAL			PARTIAL/NIL			PARTIAL/NIL			NIL			NIL	
	TERRAIN	-	F	R	M	F	R	M	F	R	M	F	R	M	F	R	M	F	R	M
	DESIGN SPEED	km/hr	120	100	80	100	80	60	90	70	60	70	60	50	60	50	40	40	30	20
CROSS SECTION ELEMENTS	LANE WIDTH	m		3.65			3.50			3.50			3.25			3.00			(5.00) a	
	USABLE SHOULDER WIDTH	m	3.00	3.00	2.50	3.00	3.00	2.50	3.00	3.00	2.00	2.50	2.50	2.00	2.00	2.00	1.50	1.50	1.50	1.50
	MEDIAN WIDTH (MINIMUM)	m	4.00	4.00	4.00	4.00	3.00	2.00	3.00	2.00	1.50		N / A			N / A			N / A	
	MEDIAN WIDTH (DESIRABLE)	m	10.0	10.0	10.0	6.0	5.0	4.0	5.0	4.0	3.0		N / A			N / A			N / A	
	MARGINAL STRIP WIDTH	m		0.50			0.50			0.25			0.25			0.25			0.00	
	MINIMUM RESERVE WIDTH	m		60			60 (50) b			40 (30) b			25			20			20	
ELEMENTS OF DESIGN	STOPPING SIGHT DISTANCE	m	250	185	130	185	130	85	160	105	85	105	85	65	85	65	50	50	35	20
	PASSING SIGHT DISTANCE	m	775	670	540	670	540	410	615	485	410	485	410	345	410	345	270	270	200	200
	MINIMUM RADIUS (AT MAX SE 10%)	m	595	360	210	360	210	115	275	160	115	160	115	75	115	75	45	45	25	10
	MINIMUM LENGTH OF SPIRAL (AT MAX SE- 2 LANES)	m	95	80	70	80	70	60	76	65	60	65	60	55	60	55	51	51	48	38
	MAXIMUM SUPERELEVATION	Ratio		0.10			0.10			0.10			0.10			0.10			0.10	
	MINIMUM RADIUS (AT MAX SE 8%)	m	665	396	230	396	230	125	305	175	125	175	125	80	125	80	50	50	30	10
	MINIMUM LENGTH OF SPIRAL (AT MAX SE-2 LANES)	m	76	65	57	65	57	48	57	52	48	52	48	44	48	44	41	41	38	32
	MAXIMUM SUPERELEVATION	Ratio		0.80			0.80			0.80			0.80			0.80			0.80	
	MAXIMUM GRADE (DESIRABLE)	%	2	3	4	3	4	5	4	5	6	5	6	7	6	7	8	7	8	9
	MAXIMUM GRADE	%	3	4	6	3	5	8	6	8	10	7	8	10	7	10	15	7	11	17
	CREST VERTICAL CURVE (K)	-	144	78	39	78	39	17	59	26	17	26	17	10	17	10	10	10	5	3
	SAG VERTICAL CURVE (K)	-	63	45	30	45	30	18	38	23	18	23	18	13	23	13	9	9	6	3

REMARK :
1) ALL VALUES SHOWN ABOVE ARE MINIMUM/MAXIMUM VALUES. ALL EFFORT SHOULD BE MADE TO ACHIEVE AS HIGH A VALUE AS POSSIBLE
2) FOR DEFINITION OF AREA TYPE, SEE TABLE 3 - 2
3) ABBREVIATION :
F = FLAT
R = ROLLING
M= MOUNTAINOUS
N/A - NOT APPLICABLE
( ) a - TOTAL WIDTH OF PAVEMENT
( ) b - RESERVE WIDTH DEPENDS IN ROAD CATEGORY

REFERENCES

1. REAM GL 2/2002: A Guide on Geometric Design of Roads

2. MHCM- Malaysia Highway Capacity Manual (2011)

3. Austroad: Guide to Traffic Engineering Practice: Part 13 – Pedestrians (1995)

4. AASHTO - A Policy of Geometric Design of Highways and Streets, (2001)

5. AASHTO - A Policy of Geometric Design of Highways and Streets (2011)

6. LLM/GP/T5-08 – Guideline For Toll Expressway System-Design Standard

7. GP006-A: Garis Panduan Perancangan Laluan Kemudahan Utiliti,