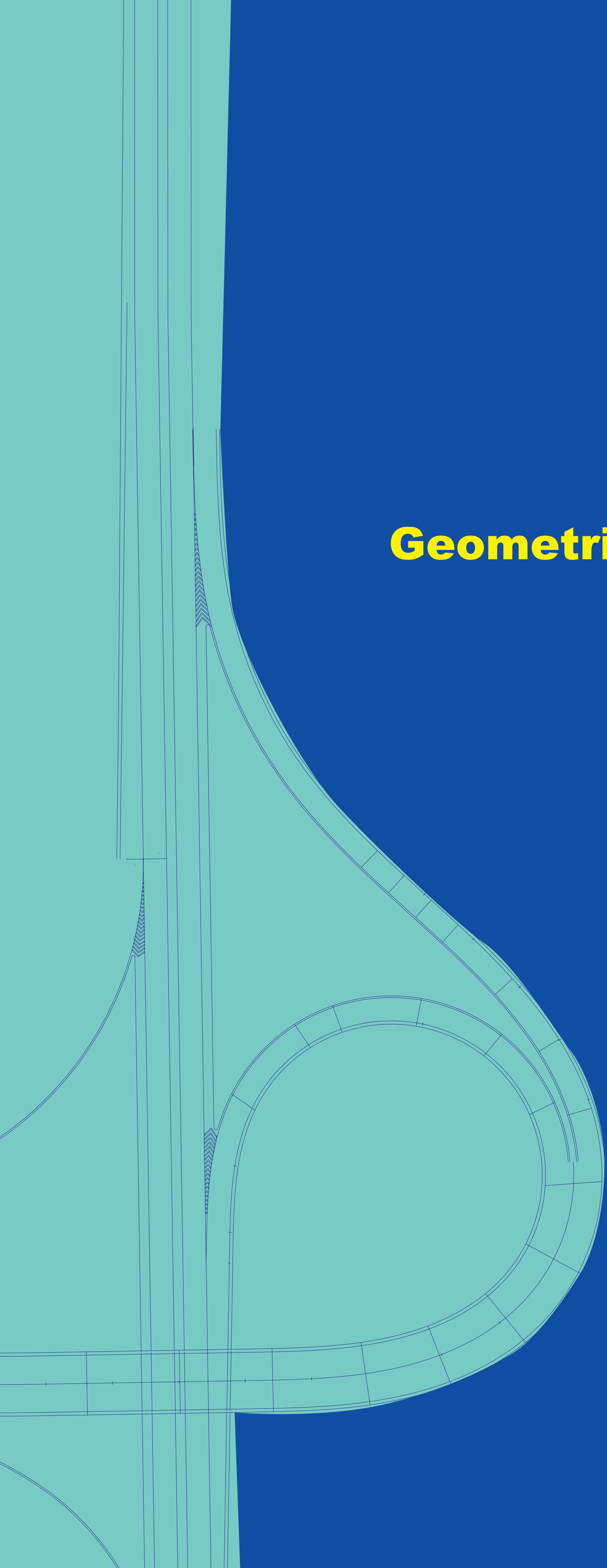

A Guide on Geometric Design of Roads

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Public Works Department Malaysia
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DEDICATION

This Arahan Teknik (Jalan) 8/86 'A Guide On Geometric Design Of Roads' is dedicated to the late

IR. TOH AH LEE

who had put in a lot of effort in the initial draft before his life was tragically ended in a road accident on 9TH JUNE, 1985.

ADDENDUM NO. 1

This Addendum to the 'Arahan Teknik Galan) 8/86 - A Guide on Geometric Design of Roads' shall be made part of this Arahan Teknik and users shall incorporate this Addendum into the Arahan Teknik (jalan) 8/86.

1.0 TABLE 3-2A DESIGN SPEED (RURAL) and

TABLE 3-2B : DESIGN SPEED (URBAN), page 25

Tables 3-2A and 3-?~B should read as table 3-2A : DESIGN SPEED (RURAL) (AMENDED SEPT. 1989) and table 3-2B : DESIGN SPEED (URBAN) (AMENDED SEPT. 1989) respectively, herewith as attached to this Addendum.

2.0 GENERAL SUMMARY - GEOMETRIC DESIGN CRITERIA FOR ROADS IN RURAL AREAS (METRIC), page 103

The above mentioned table is superceeded by table 'GENERAL SUMMARY - GEOMET RIC DESIGN CRITERIA FOR ROADS IN RURAL AREAS (METRIC) (AMENDED SEPT. 1989)' herewith attached to this Addendum.

3.0 GENERAL SUMMARY - GEOMETRIC DESIGN CRITERIA FOR ROADS IN URBAN AREAS (METRIC), t4Age 104

The above mentioned table is superceeded by table 'GENERAL SUMMARY - GEOMETRIC DESIGNED CRITERIA FOR ROADS IN URBAN AREAS (METRIC) (AMEND ED SEPT. 1989)' herewith attached to this Addendum.

TABLE 3-2A : DESIGN SPEED (RURAL) (AMENDED SEPT. 1989)

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
R1a	40	30	20

TABLE 3-213: DESIGN SPEED (URBAN) (AMENDED SEPT. 1989)

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	30
U1a	40	30	20

Note :

Type I -	Relatively free iri road location with very little problems ns regards land acquisition, affected buildings or other socially sensitive areas.
Type II -	Intermediate between I and II.
Type III -	Very restrictive in road location with problems as regards land acquisition, affected buildings and other sonsiltive areas.

GENERAL SUMMARY - GEOMETRIC DESIGN CRITERIA FOR ROADS IN RURAL AREAS (METRIC) x [4]

0	1	DESIGN STANDARD	-		R 6		R 5			R 4			R 3				R 2			R 1			R 1A	
n ...	2	ACCESS CONTROL	-		FULL		PARTIAL			PARTIAL			PARTIAL				NIL			NIL			NIL	
	3	TERRAIN	-	F	R	M	F	R	M	F	R	M	F	R	M	F	R	M	F	R	M	F	R	M
0	4	DESIGN SPEED	Km/hr	120	100	80	100	80	60	90	70	60	70	60	50	60	50	40	40	30	20	40	30	20
	5	| LANE WIDTH	m		3.50		! 3.50			(3.15			3.00		|		2.75 |			(5.00)[0]			(4.50)[0]	
	I 6	1 SHOULDER WIDTH	I m	13.00	3.00	2.50	13.00	3.00	1.50	13.00	3.00	2.00	12.50	2.50	2.00	12.00	1.00	1.50	1.50	1.50	1.50 ;	1.Y	1.50	I N I
	1	;HOULOrE~R~ allRI~trlVRLJ	m		1:00		1.00			1:00			0.50				0.50			0_50			0.50	
Z	8	| MEDIAN WIDTH (MINIMUM)	m ,	6.0	5.0	4.0	1 4.0	3.5	3.0	! 3.0	2.5	2.0 1	N/A				N/A			N/A j			N/A	
N	9	MEDIAN WIDTH (DESIRABLE)	m	18.0	12.5	8.0	12.0	9.0	6.0	9.0	6.5	4.0	N/A				N/A			N/A			N/A	
N	10	MARGINAL STRIP (WIDTH)	m		0.50		0.50			0.25			0.25				0.00			0.00			0.00	
	11	MINIMUM RESERVE WIDTH	m		60		60(50)		b	40(30)		b	20				20			12			12	
	12	STOPPING SIGHT DISTANCE	m	185	205	140	205	140	85	180	120	85	120	85	65	85	65	45	45	30	20	45	30	20
	13	PASSING SIGHT DISTANCE	m		N/A		700	550	450	625	500	450	500	450	350	450	350	300	300	250	200	300	250	200
Z	14	MINIMUM RADIUS	m	570	375	230	375	230	125	300	175	125	175	125	85	125	85	50	50	30	15	50	30	15
	15	MINIMUM LENGTH OF SPIRAL	m						SEE TABLE			4 -	4A							N/A			N/A	
0 n	16	MAXIMUM SUPERELEVATION	RATIO		0.10		0.10			0.10			0.10				0.10			0.10			0.10	
	17	MAXIMUM GRADE (DESIRABLE)	%	2	3	4	3	4	5	4	5	6	5	6	7	6	7	8	7	8	9		10	
W W	18	MAXIMUM GRADE	%	5	6	7	6	7	8	7	8	9	8	9	10	9	10	12	10	12	15		25	
	19	CREST VERTICAL CURVE (K)	-	120	60	30	60	30	15	45	22	15	22	15	10	15	10	10	10	5	5	10	5	5
	20	SAG. VERTICAL CURVE (K)	-	60	40	28	40	28	15	35	20	15	20	15	12	-15	--12-	10	-|-	8	8	10	8	8 |

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

s	I	**DESIGN STANDARD**	-		U 6			U 5		U 4				U 3			U 2			U i			U la	
0 ~	2	**ACCESS CONTROL**	-		**FULL**		**PARTIAL**			**PARTIAL**			**PARTIAL/NIL**				**NIL**			**NIL**			**NIL**	
uN	3	**AREA TYPE**	-	**I**	**II**	**III**	**i**	II	Itl	I II		III	i	il	III	I	II	III	I	II	III	I	11	III
0	4	**DESIGN SPEED**	**Km/hr**	**100**	**80**	**60**	80	60	50	70 60		50	60	50	40	50	40	30	40	30	20	40	30	10
1 ~~, ;	6 1 ;	1 A\1 uv,E 1	zl 1		Z **J.5u**			**J.uu**	**I**	J. L J		**I**		**a.uu**	**I**		**L.ia**			ir n i			**(4.50;** -	n I
	6 ! 7	**SHOULDER WIDTH !** **?sHOULDER WIDTH** **STRUCTURES** >1Wm	m m	3.00	3.00 1.00	2.50	3.00	3.00 1.00	2.50	3.00 2.50 1.00		2.00	2.50	2.00 0.50	1.50 1	2.00	1.50 0.50	1.50 1	1.50	1.50 0.50 ~	0.50	1.50	1.50 0.50	1.50 1
	8	**MEDIAN WIDTH (MINIMUM!)**	**m**	14.0 0	3.50	3.00	3.00	2.50	2.00	2.50 2.00		1.50	2.00	1.50	1.00		N/A			N/A			N/A	
	9	**MEDIAN WIDTH (DESIRABLE)**	m	112-009- 00		6.00	9.00	6.50	4.00	7.50 5.00		3.00	6.00	4.00	2.00		**N/A**			**N/A**			**N/A**	
V	10	**MARGINAL STRIP (WIDTH)**	**m**		0.50			0.50		0.25				0.25			0.00			0.00			0.00	
	11	**MINIMUM RESERVE WIDTH**	**m**		60			50		40(30)		b	30(20) b				20			11			12	
	12	**STOPPING SIGHT DISTANCE**	m	205	140	85	140	85	65	115 85		65	85	65	45	65	45	30	45	30	20	45~	30	20
	13	**PASSING SIGHT DISTANCE**	**m**		**N/A**		**550**	450	350	500 450		350	450	350	300	350	300	250	300	250	200	300	250	200
z	14	**MINIMUM RADIUS**	**m**	465	280	150	280	150	100	210 150		100	150	100	60	100	60	35	60	35	15	60	35	15
	15	**MINIMUM LENGTH OF SPIRAL**	**m**							**SEE TABLE**		**4** -	4B				**N/A**			**N/A**			**N/A**	
0 **0**	**16**	**MAXIMUM SUPERELEVATION**	**RATIO**		**0.06**			0.06		0.06				0.06			0.06			**0.06**			0.06	
	17	**MAXIMUM GRADE (DESIRABLE)**	**%**	3	4	5	4	5	6	5 6		7	6	7	8	7	8	9	7	8	9	7	8	9
w	18	**MAXIMUM GRADE**	**%**	6	7	8	7	8	9	8 9		10	9	10	12	10	12	15	10	12	15	10	12	15
w	19	**CREST VERTICAL CURVE (K)**	-	60	30	15	30	15	10	22 15		10	15	10	10	10	10	5	10	5	5	10	5	5
	20	**SAG. VERTICAL CURVE (K)**	**1** -	40	28	15	28	15	12	120 15		12	15	12	10	12	10	8	10	8	8	10	8	8

Replace section 5.14.2(c) (i) on pages 92 and 94 with the following :

Section 5.14.2 (c) Horizontal Alignment

(i) Minimum Radius

The following formula is used in determining the required. minimum radius for the curves.

R = V2
....
127(e+f)

Where R = Minimum radius of curve in meters V = Design speed, in km/hr
e = Maximum rate of superelevation
J- = Maximum allowable side friction factor.

Table 5-11 gives the minimum radius that are to be used in design. Flatter-curves should always be used wherever possible. There is no necessity to provide any transition (spiral) curves.

Table 5-11: MINIMUM RADIUS FOR DESIGN

Design Speed (km/hr)	Minimum Radius	
	e = 0.06	e = 0.10
20	15	15
30	35	30
40	60	50
50	100	85
60	150	125
80	280	230
100	465	375
120	710	570

CONTENTS

CHAPTER 6 - OTHER ELEMENTS AFFECTING. GEOMETRIC DESIGN

CHAPTER 1

INTRODUCTION AND SUMMARY

1.1 INTRODUCTION

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

This Arahan Teknik is to be applied to all new construction and improvements of roads for vehicular traffic undertaken by JKR. Modifications and updating will be carried out from time to time. In this respect, comments from users will be most welcomed.

This Arahan Teknik is to be used in conjunction with other Arahan Tekniks that have been or will be produced by Cawangan Jalan.

The design of at-grade intersections and interchanges are presented individually in separate Companion Arahan Tekniks.

CHAPTER 2.

DESIGN STANDARDS AND ROAD CLASSIFICATIONS

2.1 ROAD STANDARDS

2.1.1 Standardisation

The geometric design of all roads need to be standardised for the following; reasons:

(a) to provide a uniformity the design of roads according to their per formance requirements.
(b) to provide a consistent, safe and reliable road facilities for movement of traffic.
(c) to provide. a guide for less subjective deci sions on road design.

2.1.2 Rural and Urban Areas

Urban areas are defined as areas having a population of at least 1,000 where buildings and houses are gathered and business activity is prevalent. It covers all areas within the gazetted Municipality limits and also includes areas expected to become urbanised within the design period. Rural areas can be regarded as areas other than urban areas.

There is no fundamental dif ference in the principles of design for rural and urban roads. Roads in urban areas, however,are characterised by busy pedestrian activities and frequent stopping of vehicles owing to short intersection spacings 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 geomet ric design are incorporated for these two broad groups of roads.

2.1.3 Application of Standards

The design standard is classi-fied into seven groups (R6,R5, R4,R3,R2,:R1 & R1a) for rural areas and into seven groups (U6,U5,U4,it3,U'2,U1,& Uta) for urban areas. These are in descending order of hierarchy.

Roads which function to pro vide long distance travel, will require higher, design speeds whilst road which serve local traffic, where the effect of speed is less significant can have a lower design speed. Also roads with heavier traffic will be provided with a higher standard.

Each design standard is generally applicable to the road types as follows:-

(a) Standard. R6/U6 :

Provides the highest geometric design standard for rural or urban areas. They usually serve long trips with high speed of travelling, comfort and safety. It is always designed with divided carriage way and with full access control. The Rural and Urban Expressway falls under this standard.

(b) Standard. R5/U5:

Provides also high geometric standard and usually serve long to intermediate trip lengths with high to median travelling speeds. It is usually with partial access control, The Highway, Piimary Road and Arterial falls under this standard.

(c) Standard R4/U4:

Provides medium geometric standard and serve intermediate trip lengths with medium travelling speeds. It is also usually with partial access control. The Primary Road, Secondary Road, Minor Arterial and Major collector falls under this standard.

(d) Standard R:3/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 falls under this standard.

(e) Standard R2/U2:

Provides the lowest geometric standard 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.

(f) Standard R1/U1:

Provides very low geometric standard and is applied to very low traffic where the chances of two way flow is low.

(g) Standard R1a :

Applied to local access to restricted areas such as access to microwave stations and security areas.

(h) Srandard U1a :

Applied to local access to low cost housing areas.

2.2 ROAD CLASSIFICATION

2.2.1 Function 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 func tion of a road is transporta tion. This can be further divid-ed into two sub-functions; namely mobility and accessi bility. 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 cate gories, namely, EXPRESS WAY, HIGHWAY, PRIMARY ROAD, SECONDARY ROAD and MINOR ROAD and in urban areas, roads are divid-ed into four categories, name-ly, EXPRESSWAY, ARTERI-AL, COLLECTOR AND LOCAL STREET. They are in ascending order of accessibili-ty and thus in descending order of mobility.

2.2.2 Categories of Road

Roads are divided into two groups by area, i.e. rural and urban. Roads in rural areas are further classified into five categories by function namely Expressway, Highway, Primary Road, Secondary Road and Minor Road and into four categories in urban area, namely, Expressway, Arterial, Collector and Local Street. Their general. appli-cations are as follows.

(a) Expressway

An Expressway is a divided highway for through traffic with full control of access and always with grade sep arations at all intersec tions.

In rural, areas, they apply to the interstate highways for through traffic and make the basic framework of National road trans portation for fast travel ling. 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 frame work of road trans por tation system inur banised area for through traffic. They also serve relatively long trips and smooth traffic flow and with full access control and complements the Rural Expressway.

(b) Highways

They constitute the interstate national net work and complements the expressway net work. They usually link up directly or indirectly the Federal Capitals, State capitals and points of entry/exit to the country. They serve ong to intermediate trip lengths. Speed service is not so important as in an Expressway but relatively high to medi um speed is necessary. Smooth traffic is provid-ed 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 intermedi-ate trip lengths and medium travelling speeds. Smooth traffic is provided with partial access control They usually link up 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 Areas. They serve intermedi-ate trip lengths with partial access control.

They usually link up the raajor towns within the District or Regional Development Areas.

(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 inhabited areas in a 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 and are usually with partial or no access control.

(f) Arterials

An arterial is a continous road with 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 centre . Arterials do not penetrate identifiable neigh bourhoods. Smooth traffic flow is essential since it carries large traffic volume.

(g) Collectors

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

(h) Local Streets

The local street system is the basic road network within a neigh bourhood 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.

2.3 ROAD ADMINISTRATION

For the purposes of road administration, roads are classified as Federal, State, Local Authority (City Hall, Municipal or Local Council) or Kampong (District Office) Roads depending upon their jurisdiction.

(a) 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 the FELDA Land Schemes and those in other Regional Land Schemes constructed with Federal Funds also fall under this category. The maintnance of these roads are the responsibility of the Federal Government and is done through the State JKR with funds from the Federal Government.

(b) State Roads are all the other roads within the State outside the juris diction of the Local Authority or District Office, built to JKR standards. 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.

(c) Local Authority Roads are all those roads within the limits of the Local Authority and are normally maintained by the responsible local authority.

(d) Kampong (District Office) Roads are all those roads directly under the jurisdiction of the District Office. They are usually earth roads with no right of way. The maintenance of these roads are the responsibility of the District Office.

2.4 ACCESS CONTROL

2.4.1 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, namely full control, partial control and non-control of access

Full Control of Access mean that preference is given to through traffic by providing access connecting with select ed public roads only and by prohibiting crossings at grade or direct private driveway con nections.

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 crossings trafficked roads, at grade intersections should be limited and only allowed at selected locations. To compensate for the limited access to fully or partially access controlled roads, frontage or service roads are sometimes attached to the sides of the main roads.

In Non Control Access, there is basically no limitations of access.

2.4.2 Application

The selection of the degree of control required is important so as to preserve the as built capacity of the road as well as improved safety to all road users. Two aspects pertaining to the degree of control is to be noted.

(a) during the time of design in the consideration of accesses to existing developments.

(b) after the completion of the road in the control of accesses to future developments.

The selection of degree of access control depends on traffic volumes, function of the road and the road network around the areas. Table 2-2A and 2-2B is general guide for the selection of degree of access control.

TABLE 2--2A: ACCESS CONTROL (RURAL)

Road Catagory	Design Standard					
	R6	R5	R4	R3	R2	R1 / R1a
Expressway	F	-	-	-	-	-
Highway	-	P	-	-	-	-
Primary Road	-	P	P	-	-	-
Secondry Road	-	-	P	P	-	-
Minor Road	-	-	-	-	N	N

TABLE 2-2B: ACCESS CONTROL (URBAN)

Road Catagory	Design Standard					
	U6	U5	U4	U3	U2	U1 / U1a
Expressway	F	-	-	-	-	-
Arterial	-	P	P	-	-	-
Collector	-	-	P	P	-	-
Local Street	-	-	-	N	N	N

NOTE :

F = Full Control of Access
P = Partial Control of Access
N = No Control of Access

2.5 DESIGN STANDARDS

The design standards used for various categories of roads are as shown in Table 2-3

TABLE 2-3: DESIGN STANDARDS

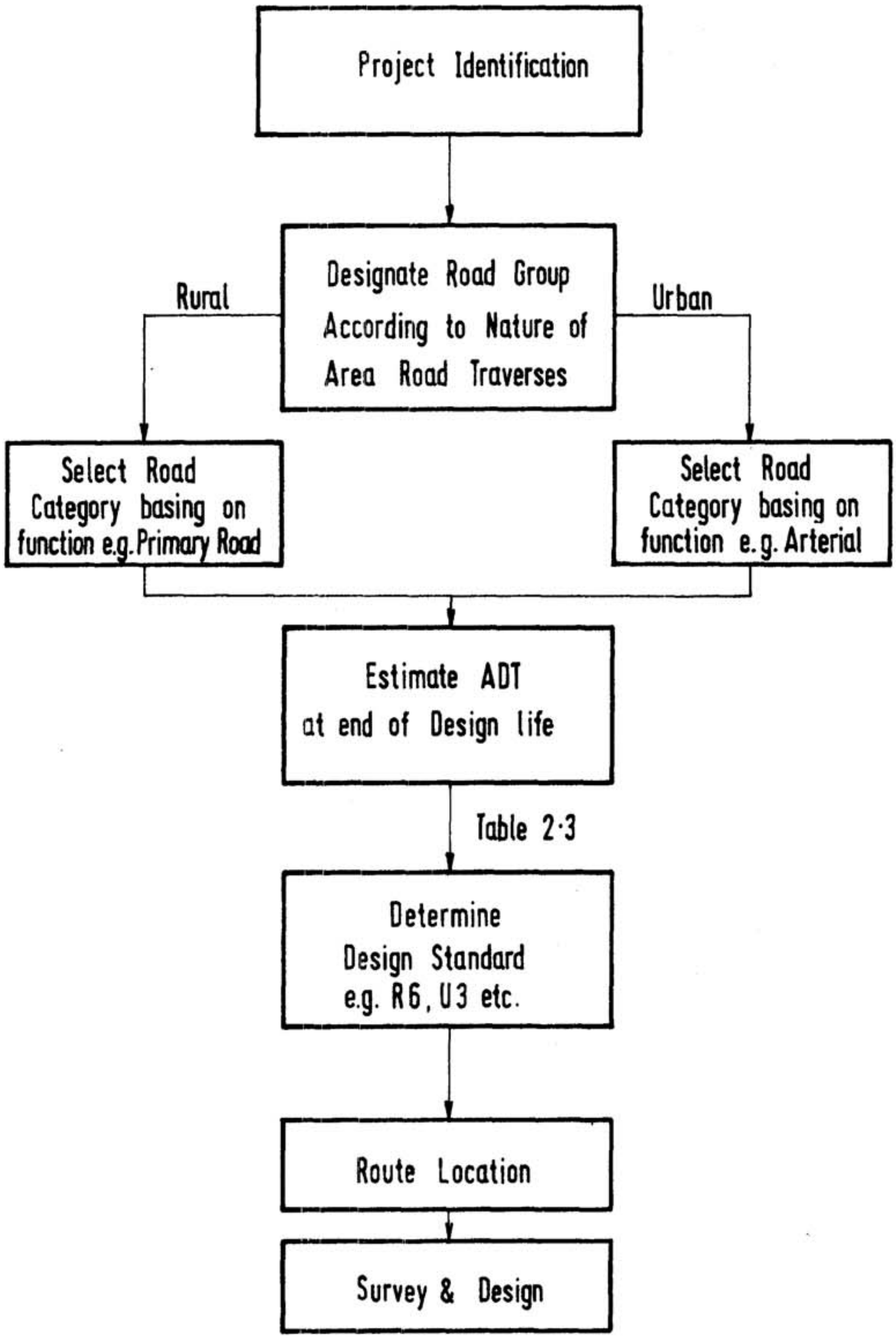
Area	Road Catagory	Projected ADT					
		All Traffic Volume	> 100,00	10,000 to 3,000	3,000 to 1,000	1,000 to 150	< 150
RURAL	Expressway	R6	-	-	-	-	-
	Highway	R5	-	-	-	-	-
	Primary Road	-	R5	R4	-	-	-
	Secondry Road	-	-	R4	R3	-	-
	Minor Road	-	-	-	-	R2	R1 / R1a
URBAN	Expressway	U6	-	-	-	-	-
	Arterials	-	U5	U4	-	-	-
	Collector	-	-	U4	U3	-	-
	Local Street	-	-	-	U3	U2	U1 / U1a

2.5.1 Selection of :Design Standard

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 of the Ministry of Works. If there is an overlapping of function, the ultimate function of the road shall be used for the selection criteria. The projected ADT at the end of the design life should then be cal culated and from Table 2-3, the required design standard can be obtained. From the capacity analysis (as in Chapter 3) the required number of lanes can then be calculat-ed.

Figure 2--1 gives a flow chart indicating the processes for the selection of the,required design standard.

FIGURE 2-1: FLOW CHART FOR SELECTION OF DESIGN STANDARDS



CHAPTER 3

DESIGN CONTROL AND CRITERIA

3.1 TOPOGRAPHY

The location of a road and its design are considerably influ enced by the topography, physical features, and land use of the area traversed. Geometric design elements such as alignment, gradients, sight distance and crosssec-tion 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 economise on construction and mainte-nance.

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

FLAT terrain means:
The topographical condition where highway sight dis tances, as governed by both horizontal and vertical restric-tions are generally long or could be made to be so with-out construction difficulty or expertise. The natural ground, cross slopes (i.e. perpendicu-lar to natural ground contours) in a flat terrain are generally below 3%.

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

MOUNTAINOUS terrain means:
The topographical condition where longitudinal and trans-verse 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 align-ment. The natural ground cross-slopes in a mountainous terrain are generally above 25%.

Steep grades and restrictive passing sight distance greatly reduce the capacity of a 2-lane road and lower the run-ning speed of traffic, whereas their effect on wider roads is much less. Consequently, the nature of the terrain some-times 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 on a single road formation may be appropriate; in others it may be more fitting to locate the road with two separate raad formation.

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 gener ally 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 sup plemental 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 serve to establish the "loads" for geometric design. Traffic data for a road or section of road generally are 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 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 DESIGN STANDARDS However, the direct use of ADT in geometric design is not appropriate because it does-not indicate the significant variation in the traffic occuring 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 deter mine 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 unusual or highly seasonal fluctuation in traffic flow such as holiday reasort 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 <u>Design Hourly Volume Ratio (K).</u>

K, is the ratio of DHV to the designed ADT. K's value ranges from 7% to 20% and the actual value should be obtained from traffic data. The Highway Planning Unit of the Ministry of Works 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, dayly or hourly, it is recommended that traffic surveys be carried out as the above k val ues may be unrealistic.

3.2.4 <u>Directional Distribution Ratio (D).</u>

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 latter, knowl edge 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 should be consulted for the survey data if available. Generally in the absence of field data D value of 6096 can be used in urban areas and 6590 in rural areas. Traffic distribution by directions is generally consistent from year to year and from day to day except on some roads serving holiday resort area.

3.2.5 <u>Traffic Composition</u>

This is the percentage of various class 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.

The various sizes and weights of vehicles as they affect traffic operation can be grouped into six(6) categories conforming to the classification for the National Traffic census:i.e=

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

For design purposes, the per centage of commercial vehicles during the peak hours should be known from field studies. Values can be obtained from the "Traffic Volume Peninsular Malaysia" published by the Highway Planning Unit of the Ministry of Works.

3.2.6 <u>Projection of Traffic</u>

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 accomodate 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 maybe staged basing on a 10 year period for the first stage. The Highway Planning Unit 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.

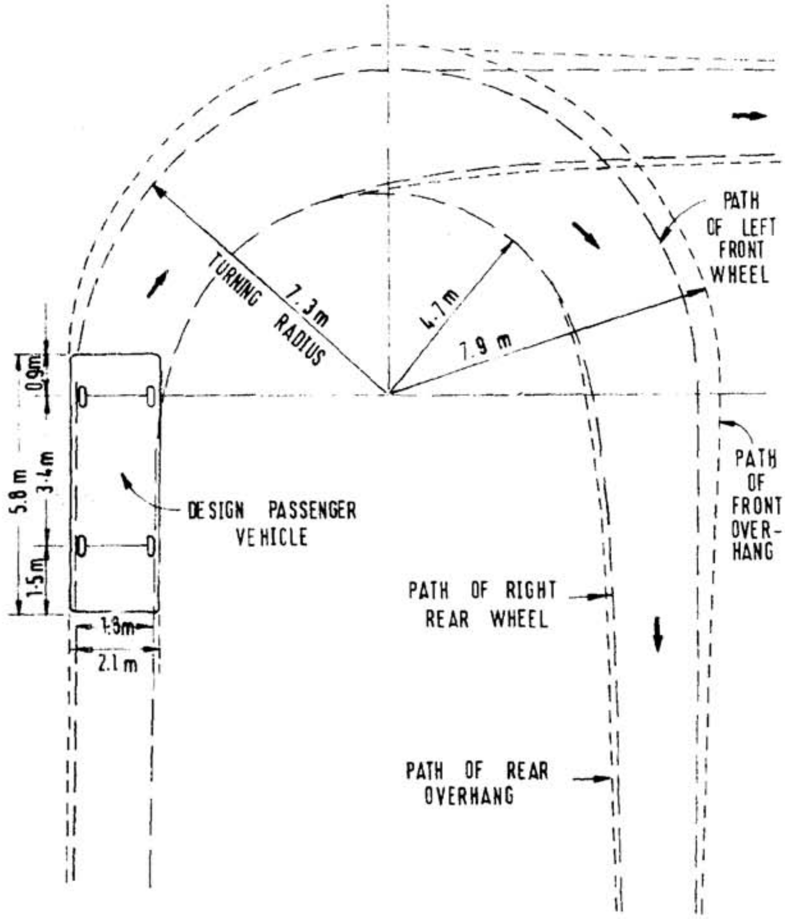
3.3 DESIGN VEHICLES AND CHARACTERISTICS

The physical. characteristics of vehicles and the proportions of various size vehicles using the roads 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 accomodate 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 analysing 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 Vehicles" - A Policy of Geometric Design of Highways and Streets (1984). Figure 3-1, 3-2 and 3-3. Shows the dimensions and turning characteristics for the P, SU and WB-50 design vehicles.

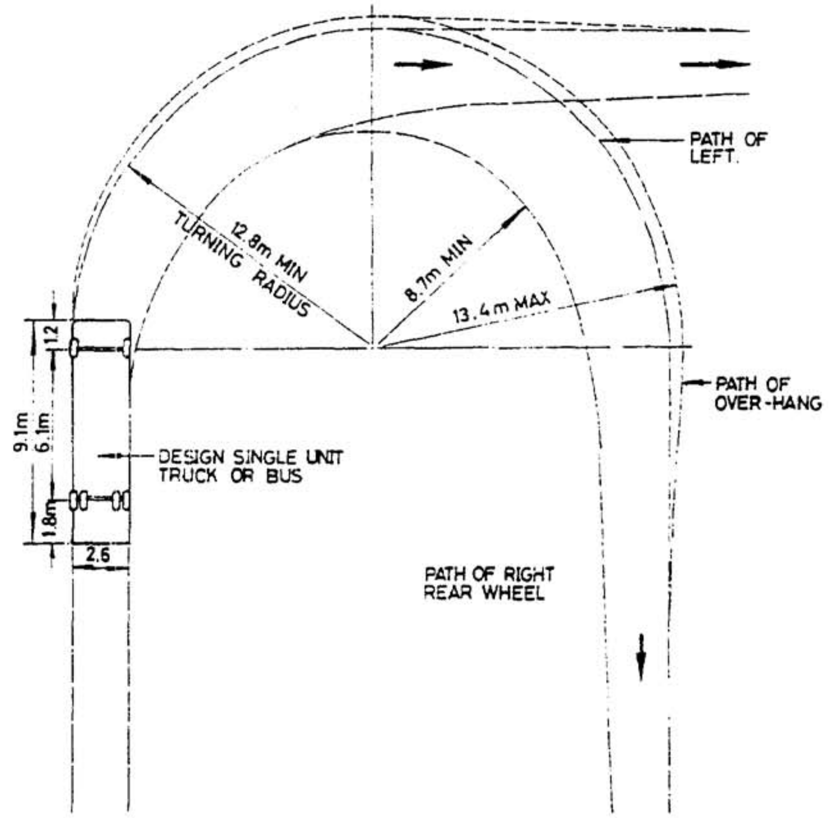
FIGURE 3-1 : P DESIGN VEHICLE



UNIT IN METER

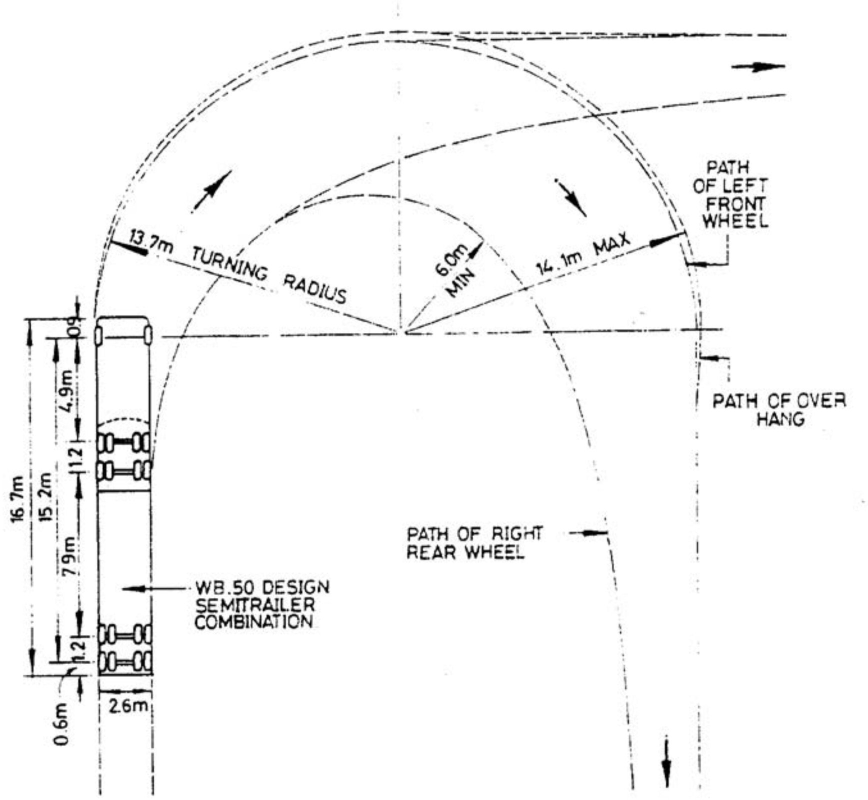
SOURCE: AASHTO - GEOMETRIC DISIGN OF HIGHWAYS AND STREET (1984) FUGURE II-1

FIGURE: 3-2 SU DESIGN VEHICLE



SOURCE : AASHTO - GEOMETRIC DESIGN OF HIGHWAYS AND STREET (1984) FIGURE II-2

FIGURE:3-3 WB-50 DESIGN VEHICLE



SOURCE: AASHTO - GEOMETRIC DESIGN OF HIGHWAYS AND STREET (1984) FIGURE 11-6

3.3.2 Summary of Dimension of Design Vehicles

Table 3-1 below,summarises the design vehicle dimensions and characteristics.

TABLE 3-1 : DESIGN VEHICLE DIMENSIONS

Design Vehicle		Dimension in metre						Turning Radius (m)
Type	Symbol	Wheel Base	Overall		Overall Lenght	Overall Width	Height	
			Front	Rear				
Passenger Car	P	3.4	0.9	1.5	5.8	2.1	1.3	7.3
Single Unit Truck	SU	6.1	1.2	1.8	9.1	2.6	4.1	12.8
Truck Combination	WB-50	7.9	0.9	0.6	16.7	2.6	4.1	13.7

NOTE:

a) Maximum allowable overall lengths under current Malaysian Legislation are 30'(9.2 m), or 40'(12 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 only for buses, which is 15 ft (4.57 m).

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 Speeds are selected to meet the needs of the road to fulfill its function. Thus 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 favourable weather conditions and under prevailing traffic conditions with out 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 favourable 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 or road. Every effort should be made to use as high a design speed as practicable while maintaining the desired degree of safety, mobility and efficiency. Table 3-2A and 3-2B indicates the selection of design speeds with respect to rural and urban standards.

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
R1a	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
U1a	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 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, although 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 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 3-2B, i.e. If the design speed of section A is 120 km/hr and that of section B is 81D km/hr, then. the design speed of the transition section between them should be 100 km/hr.

3.5 CAPACITY

The term highway capacity pertains to the ability of a roadway to accomodate 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 pre vailing roadway and traffic conditions. Capacity considered here is only applicable to uninterrupted flow or open roadway conditions. Capacity for interrupted flow as at inter sections will be dealt with separately.

Capacity is also usually stated in terms of passenger car units (p.c.u). Table 3-3 gives the conversion factors to be used in converting the various classes of vehicles to passenger car units.

TABLE 3-3 : CONVERSION FACTORS TO P.C.U'S.

Type of Vehicle	Equivalent Value in p.c.u’s			
	Rural Standards	Urban Standards	Round About Design	Traffic Signal Design
Passenger Cars	1.00	1.00	1.00	1.00
Motorcycle	1.00	0.75	0.75	0.33
Light Vans	2.00	2.00	2.00	2.00
Medium Lorries	2.50	2.50	2.80	1.75
Heavy Lorries	3.00	3.00	2.80	2.25
Buses	3.00	3.00	2.80	2.25

Source:

1) Road Research Laboratory "Research on Road Traffic", HMSO, London, 1965, pp 201

2) Highway Planning Unit, Ministry of Works, Malaysia.

3.5.1 Capacity under ideal conditions

Under ideal conditions, the possible capacity for uninterrupted flow are as follows:

a) For, 2-lane two way (total) = 2,800 pcu/hr.
b) For, multil.ane (per lane) = 2,000 pcu/hr.

Ideal conditions consist of the following for two-lane roads:

a) Design speed greater than or equal to 100 km/hr.
b) Lane widths greater than or equal to 3.65 m (12')
c) Clear shoulders wider than or equal to 1.83 m (6 ')
d) No "Nc passing zones" on the highway.
e) All passenger cars in the traffic stream
f) A 50/50 directional split of traffic.
g) No impediments to through traffic due to traffic control or turning vehicles.
h) Level terrain.

When one or more of these conditions are not met, the actual capacity will be reduced. The effects of each are discussed in the Highway Capacity Manual (1) which gives adjustment factors to be applied to the above possible capacity to determine the design capacity to be used for the design conditions.

(1) Highway Capacity Manual, Special Report No: 209 Transportation Research Board, Washington D.C., 1985.

3.5.2 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.

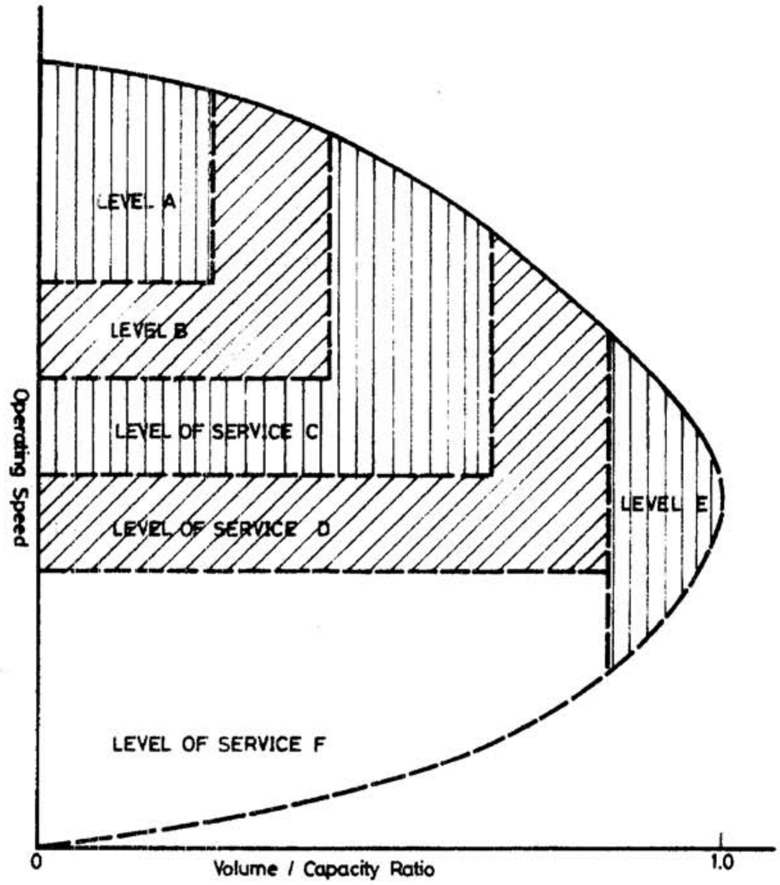
3.5.3 Service Volume

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 condi tions (freedom to manoeuver) at the time the traffic is at the design hour volume. Table 3-4 gives an indication of the 'Levels of service used while Figure 3-4 gives a schematic concept of the relationship of level of service to operating speed and volume/capacity ratio.

TABLE 3-4 : LEVELS OF SERVICE.

Level of Service	Remarks
A	Free Flow with low volumes, densities and high speeds. Drivers can maintain their disered speeds with little or no delay.
B	Stable Flow. Operating speeds beginning to be restricted somewhat by traffic conditions. Some slight delay.
C	Stable Flow. Speeds and maneuverability are more closely controlled by higher volumes. Acceptable delay.
D	Approaching Unstable Flow. Tolerable operating speeds which are considerably affected by operating conditions. Tolerable delay.
E	Unstable Flow. Yet lower operating speeds and perhaps stoppages of momentary duration. Volumes are at or near capacity congestion and intolerable delay.
F	Forced Flow. Speeds and volume can drop to zero. Stop pages can occur for long periods. Queues of vehicles backing up from,a restriction downstream.

FIGURE 3-4: RELATIONSHIP OF LEVEL OF SERVICE TO OPERATING SPEED AND VOLUME / CAPACITY RATIO.



CHAPTER 4

ELEMENTS OF DESIGN

4.1 SIGHT DISTANCE

4.1.1 General

Sight distance is the length of road ahead visible to drivers. Ability of a driver to see ahead is of the 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. Sight distance thus includes stopping sight distance and passing sight distance. Sight distance applied to multilane divided roads is only the safe stopping sight distance.

4.1.2 Stopping Sight Distance

The stopping sight distance is the length required to enable a vehicle traveling 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) 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 is assumed. This make 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 stan dard formula:

d = 0.00394 v 2/f

where :

d = brake distance, (m)

v = initial speed, (km/hr)

f = coefficient of friction between tires and roadway.

In this formula the f factor varies considerably due to many physical elements such as condition of tires, condition of pavement surface, the presence of moisture etc; The values of f for design are inclusive of all these factors.

(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 <u>Table 4-1</u>

TABLE 4-1 : MINIMUM STOPPING SIGHT DISTANCE

Design Speed (km / hr)	Min. Stopping Sight Dist (m)
120	285
100	205
80	140
60	85
50	65
40	45
30	30
20	20

d) Effect of Grades on Stopping Sight Distance

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

d = 0.00394 v2/(f k g)

in which g is the percentage of grade divided by 100

The safe stopping sight distances on upgrades are shorter. and those on downgrades are longer. However the effect of grade to sight distance need not be considered as the sight distance available on downgrades is larger than on upgrades and this more or less provides for the necessary corrections for grade.

(e) Every effort should be made to provide stopping sight 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

4.1.3 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 cf four distances:

(i) distance: traversed during the perception and reaction time and during the inital 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 opposing 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 elements. Table 4.2 gives the minimum values to be used for each design speed

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

TABLE 4-2 : MINIMUM PASSING SIGHT DISTANCE (2-LANE -2 WAY)

Design Speed (km / hr)	Min. Passing Sight Dist (m)
120	800
100	700
80	550
60	450
50	350
40	300
30	250
20	200

(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 either in upgrade or downgrade. However it should be realised that greater distances are needed for passing on grade as compared to level conditions. The designer should recognise the desirability of increasing the minimum as shown in Table 4_2

(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 alipiqiment and profile adjustments are necessary to accomodate the design hour volumes. Generally this percent age 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.

4.1.4 Criteria for Measuring Sight Distance

(a) Height of Driver's Eye

The eyes of the average driver in a passen ger vehicle are considered to be 0.92m (3.5ft) above the road surface.

(b) Height of Object :

A height of object of 150mm (6in) is assumed for measuring stopping sight distance and the height of object for passing sight distance is 1.32m(4.25ft) both measured from the road surface.

4.2 HORIZONTAL ALIGNMENT

4.2.1 General

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

4.2.2 Superelevation Rates

The maximum rates of superelevation usable are con trolled by several factors such as climatic conditions, terrain conditions and frequency of very slow moving vehicles that would tie subjected to uncertain operation. While it is acknowledged that a range of values should be used, for practical perpo.ses in establishing the design criteria for horizontal alignment, a maximum superelevation rate of 0.10 is used for, roads in rural areas and 0.06 for roads in urban areas.

4.2.3 Minimum Radius

The minimum radius is a limiting value of curvature for a given speed arid 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.

$$Rmin = \frac{V^2}{127\,(e + f)}$$

where :

Rmin = minimum radius of circular curve(m)

V = Design speed (km/hr)

e = maximum superelevation rate

f = maximum allowable side friction factor

Table 4.3 lists the minimum radius to be used for the designated design speed and maximum superelevation rates.

TABLE 4-3 : MINIMUM RADIUS

Design Speed (km/hr)	MINIMUM RADIUS (m)	
	e = 0.06	e = 0.10
120	710	570
100	465	375
80	280	230
60	150	125
50	100	85
40	60	50
30	35	30
20	15	15

While the values in Table 4-3 lists the minimum radius that can be used, all efforts should be made to design the horizontal curves with radii larger than the minimum values shown for greater comfort and safety.

4.2.4 Transition (Spiral) Curves

Vehicleis follow a transition path as it enters or leave a circular horizontal curve. To desi 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 Euler 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

a) Length of Spiral

The length of spirals to be used are normally calculated from the Spiral formula or from the empirical superelevation runoff' lengths. Superelevation runoff is the general term denoting the length of highway needed to accomplish the charige is cross slope from a section with ,adverse crown removed to a fully superelevated section or vice versa. Current practice indicates that the appeareance aspect of superelevation runoff .Largely governs the length. The length of superelevation runoff should not exceed a longitudinal slope (edge compared to centreline of a two-lane road) of 1:200

Table 4.4A, B and C gives, for the various design speeds and degree of curvature, the minimum lengths of spiral, the superelevation rates and the limiting curvature for which superelevation is not required. For pavements with more than 2 lanes, the superelevation runoff lengths should be as follows

(i) 3 lane pavements - 1.2 times the length for 2 lane roads

(ii) 4 lane undivided pavements - 1.5 times the length for 2 lane roads

(iii) 6 lane undivided pavements - 2.0 times the length for 2 lane roads

4.2.5 Methods of Attaining Superelevation

Three specific methods of profile design in attaining superelevation are (a) revolving the pavement about the centreline profile (b) revolving the pave ment about the inside edge profile, and (c) revolving the pavement about the outside edge profile. Fig 4-1 illustrates these three methods diagrammatically. The rate of cross slope is proportional to the distance from start of superelevation runoff.

(a) Fig. 4-1.A illustrates the method where the pavement section is revolved about the centreline profile. This general method is the most widely used in design since the required change in elevation of edge of

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TABLE 4-4B DESIGN SUPERELEVAT:ION TABLE (URBAN)

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pavement is made with less distortion than by the other methods. The centreline profile is the base line and one-half of the required elevation change is made at each edge.

(b) Fig. 4-1B 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 centreline profile. One half of the required change in cross slope is made by raising the centre line profile with respect to the inside pavement edge and the other half by raising the outside pavement edge and equal amount with respect to the centreline profile.

(c) Fig. 4-1C illustrates the method where the pavement section is revolved about outside edge profile involves similar geometrics as (b), except that the change is affected below the upper control profile.

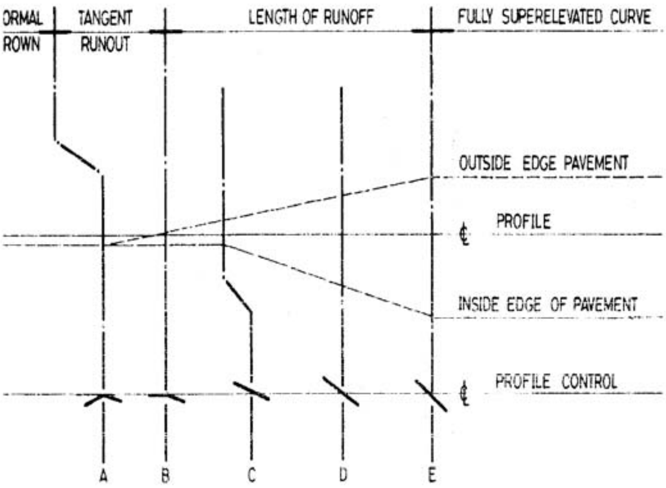
Except when site condition specifically requires , method (a) shall be adopted for undivided roads.

4.2.6 Superelevation Runoff with Medians

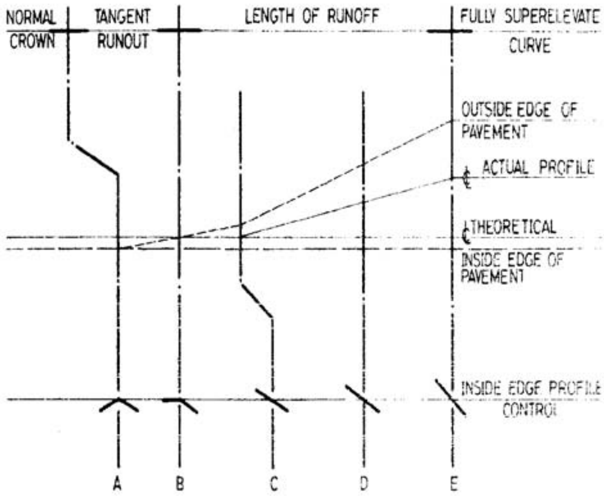
In the design of divided roads, the inclusion of a median in the cross section alters somewhat the superelevation runoff treatment. The three general cases for superelevation runoff design are as follows:

(a) The whole of the travelled way, including the median, is superelevated 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 Fig. 4-1A except that the two median

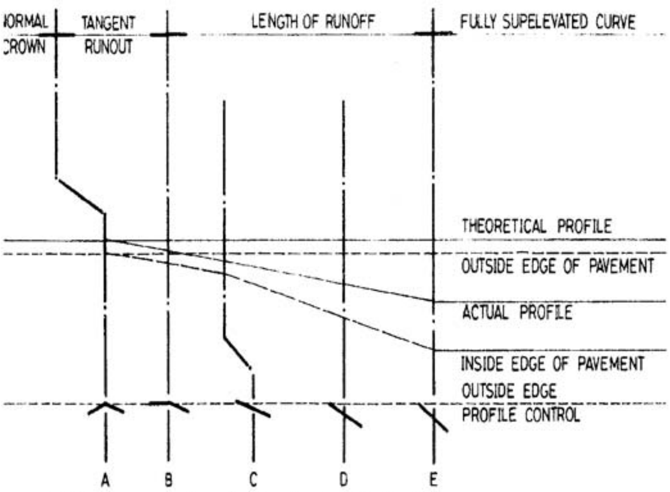
FIGURE: 4-1 DIAGRAMMATIC PROFILES SHOWING METHODS OF ATTAINING SUPERELEVATION



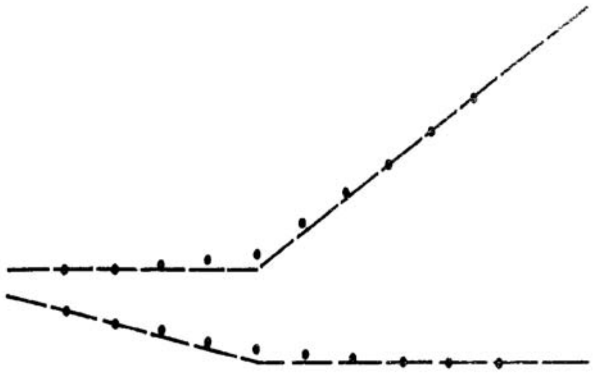
(A) Pavement Revolved about Centerline



(B) Pavement Revolved about Inside Edge



(C) Pavement Revolved about Outside Edge



NOTE: ANGULAR BREAKS TO BE APPROPRIATELY ROUNDED AS SHOWN BY DOTTED LINE.

edges will appear as profiles only slightly removed frog centre-line.

(b) 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 Fig. 4_1B and Fig. 4T1C with the centreline grade control the same for the two pavements.

(c) The two pavements are separately treated for runoff with a resultant variable difference in elevation at the median edges. The differences in elevation of the extreme pavement edges are minimised 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 ease is more applicable to wide median of 9 m or more. The pavement rotation can be made by any methods in Fig. 4-1

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 use in a tangent. Table 4-5 give 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 rea sonable smooth alignment on the edge of pavement and to fit the paths of vehicles entering or leaving the curve. Preferably, widening should be

TABLE 4-s P ~t Widening On Open Road Curves

PKNW Win (m)	7				5.5				5-5				REU8iE0 WMENING (M)	
DESM SPEED (Km/h)	80	50	50	l0	60	50	40	30	50	40	30	70	(M)	
	P--470	8-340	R> 180	R-1- 230	R'-' 1100	R 2 880	Rar 680	R2510	R R'100	R' 68	R> 52	M39 39	MORE	
	4M- ~ˉ8U	340- R-Ir	20- 11-- 1513	2309 R 130	1100>n R=340	880- R -2800	670} R ~ Z%	520'*- R ICU		68~ It- 4u	52=- R al.i	39> If -°	a	
		180~°' R~t150	150s- R~-100	130- 8286	340- R ~180	280- R~150	230y R'130	190- R'110				J7~'° R'`24	1	
				86' R264	180' R:!'150	150- R y100	130=- R286	110=- 8274				24 4'-16	, ·"	y
				64~ 8260			86a R265	74~ R251				16~° R' 15	1.25	
							65' R260	57 8245					154	
								45 R 238					1.75	
								35> R 2ˉ- 35					2, 4 e	

Note:

R rahm in meth s

attained owner the superelavation runoff length with most of all, of' the widening attained at the start of circular curve point.

4.2.8 Sight Distance on Horizontal Curves

Another element of horizontal lignment is the sight distance across the inside of curves. Where there are sight oba-tructions (such as walls, cut slopes, buildings and guardrails),a design to provide adequate sight distance may require adjustment in the nor-mal road cross-section or change) in alignment 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 the necessary adjustments in the manner most fitting to provide adequate sight distance

4.2.9 General Controls for Horizontal Alignment

In addition to the specific design elements for horicontal alignment, a number of gener-al controls are recognised and should be used. These con-trols are not subject to empiri-cal or formula derivation but are important for the attain-ment of safe and smooth-flow-ing roads. These are:

(a) The horizontal align-ment should,be consis-tent with the topogra-phy and with preserv-ing developed proper ties and community val-ues. Winding alignment composed of short curves should be avoided. On the other hand too long a straight should also be avoided.

(b) The use of the mini-mum radius for the par-ticular design speed should be avoided wherever possible. Generally flat curves should be used, retain-ing the maximum for the most critical condi-tions.

(c) Consistent alignment should always be sought. Sharp curves should not be intro-duced at the end of long tangent. Where sharp curves must be introduced, it should be approached, where possaible, by succes-sively sharper curves from the generally flat curvature.

(d) 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` 50 and the length should be increased 30 m for each 10 decrease in the central angle

(e) Any abrupt. reversal in alignment should be avoided. The distance between reverse curves should be the sum of the supreleva-tion runoff lengths and the tangent runout lengths.

(f) The 'broken back' arrangement of curves should be avoided. Ilse of spiral transitions or a compound curve align-ment is preferable for such conditions if it. is unavoidable.

4.3 VERTICAL ALIGNMENT

4.3.1 Maximum Grades

The vertical profile of road affect the performance of vehicles. The effect of grades on trucks which have weight power ratio of about 300 lb/hp, is considered. The maximum grade controls in terms of design speed is sum-marised in Table 4-6

TABLE 4-6 : MAXIMUM GRADES

Design Speed (km/hr)	Desirable Maximum Grade (%)	Maximum Grade (%)
120	2	5
100	3	6
80	4	7
60	5	8
50	6	9
40	7	10
30	8	12
20	9	15
Road Standard R1a	10	25

The desirable maximum should be aimed at in most cases. The maximum grades should be used infrequently. The total upgrade for any section of road should not exceed 3000m, 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 swamp use of even flatter grades may be allowable with prior approval.

4.3.3 Critical Grade Length

The term " critical grade length" indicate the maximum length of a designated upgrade upon which a loaded truck can operate without an unreasonable reduction in speed.

To establish the design vlaues for critical grade lengths" for which gradeability of trucks is the determining factor, the fol lowing assumptions are made:

(i) The weight-power ratio of a loaded truck is a bout 300 lb /hp .

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

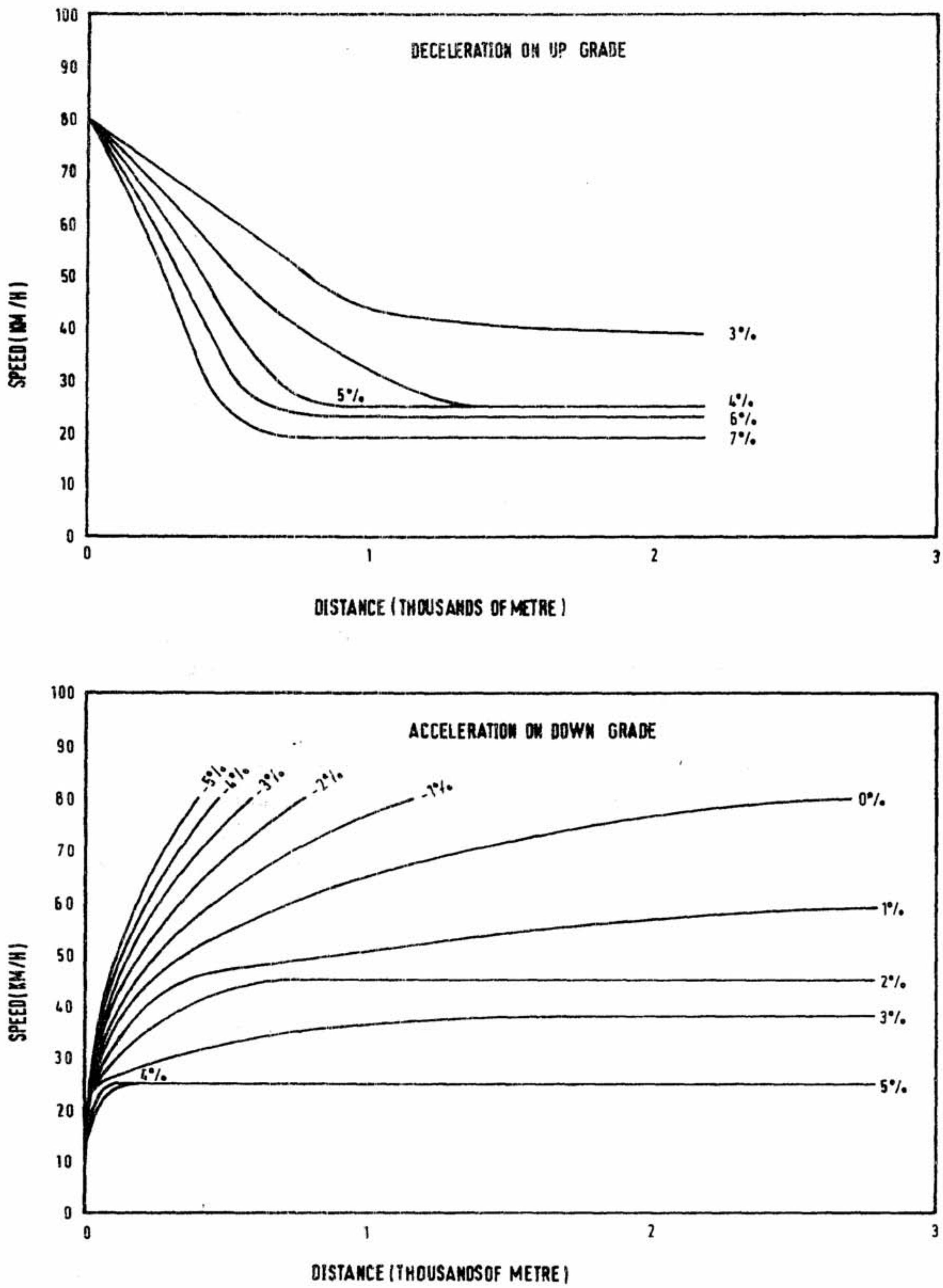
(iii) A maximum reduction in speed to half the design speed is allowed for design speeds of 80km/hr or above except for design speeds of 50 and 40km/hr where the allowable minimum speeds are 30 and 25km/hr respectively.

The critical grade lengths for the various speeds are as shodm in Table 4-7.

Where a particular section is made up of a combination of upgrades, the length of critical grade should take into consideration, the entire section of the combination. A speed reduction/recovery curve should be plotted to determine the critical grade length base on the assumptions used above. Figure 4-2 gives the acceleration and deceleration curves that can be used for this purpose.

Design Speed	Gradient (%)	Critical Grade Length (m)
120	3 4 5	500 400 300
100	4 5 6	500 400 300
80	5 6 7	500 400 300
60	6 7 8	300 250 200
50	7 8 9	250 200 170
40	8 9 10	200 170 150
30	-	Not defined - decision left to designer
20	-	Not defined - decision left to designer

FIGURE 4 - 2 : SPEED - DISTANCE CURVES



Where the length of critical grade is exceeded and especially where grades exceed 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.

4.3.4 Climbing Lanes for Two Laned Roads

Climbing lanes are desirable and should be considered where the critical grade lengths are exceeded and provided the volume of traffic and percentage of trucks justify the additional cost. An exception to the above would be where overidding safety considerations dictate the addition of the climbing lane regardless of grade or traffic volume. Where climbing lanes are provided, the following requirements are to be followed.

(a) the climbing lane should begin rear the foot of the grade and should be preceded by a tapered section of at least. 50m long.

(b) the width of the -lirnbing 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 signing and markings and the opposing lane widened also 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 N a5 wide as the shoulder on the normal two laved section, Where conditions dictate otherwise, a usable stsul der widtn of 1.25m is acceptable.

4.3.5 Passing Lane Sections and Laybyes on Two-Lane Roads

Where a sufficient number and length of safe passing section cannot be obtained in the design of horizontal and vertical alignment alone, an occasional section of four lanes may be introduced to provide more sections and length safe for passing. Such section 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 shoed be considered on roads carrying 500 vehicles per hour or more.

The transitions between the two lane and four lane pavements should be located where the 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 trie driver, does not lose his awareness that the road is basically a two lane facility.

Where four lane sections are riot practical, passing laybyes may be introduced at regular intervals. This passing laybye must be at least 1'000rri long and of full lane width and preceded by a taper of 50m at the beginning and 100m at the end.

4.3.6 Climbing Lanes on Multilane Roads

Multilane roats more frequently have sufficient capacity to handle their traffic load, including the normal percentage of slow-moving vehicles without becoming congested. However where the volume is at or neat, capacity and the truck volume is high so as to interfere with the normal flow of traffic, climbing lanes should be considered.

4.3.1 Vertical Curves

Vertical curves are used to effect a gradual charge 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 equivalen-G 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 metere. The reciprocal L/A is the horizontal distance in metre required to ef:fec;t 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 begining 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 des isan 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.

(a) 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 formulas for length of a parabclic vertical curve in terms of algebraic difference in grade and sight distance (using an eye height of 0.92m and object height of 0.15m) are as follows:

Where S is less than L,

$$L = \frac{AS2}{405}$$

Where S is greater than L,

$$L = 2S - \frac{405}{A}$$

Where :

L = length of vertical curve(m)
S = sight distance (m)
A = algebraic difference in grades (percent)

Table 4-8 indicates the minimum k values that are to be used design for the various design speeds.

TABLE 4-8 CREST VERTICAL CURVE (k values)

Design Speed (km/hr)	120	100	80	60	50	40	30	20
Minimum k value	120	60	30	15	10	10	5	5

(b) Sag Vertical curves

At least four different criteria for establishing lengths of snag vertical curves are recognised. These are (1) headlight sight distance, (2) rider comfort, (3) drainage control and (4) a rule of thumb for general appearance. However, the headlight sight distance basis appears to be the most logical 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 4-9 indicates the minimutr, 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 art-USF:d, 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 rei)lace:ment controls the vertical profile.

Drainage of curbed 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 4-9 SAG VERTICAL CURVE (k values)

Design Speed (km/hr)	120	100	80	60	50	40	30	20
Minimum k value	60	40	28	15	12	10	8	8

4.3. 8 General Controls For Vertical Alignment

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

(a) 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 critcal length are controls, the manner in which they are applied and fitted to the terrain on a. continous line determines the suitability and appearance of the finished product.

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

(c) A broken back gradeline should be avoiled, particularly in sags where the full view, of both vertical curves is not pleasing. This effect is very notice able en divided roadways with open median sections

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

(e) Where intersections at grad 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 indepeni-Jilently. They complement each other. Excellence in their design and in the design of their combination increase utility and safety, encourage uniform speed, and improve appeararce, almost always without additional cost.

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

(a) 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.

(b) Vertical curvature superimposed upon horizontal curvature, or vice versa, generally results in a more pleasing facility but it should be analyzed for effect upon traffic. Successive changes in profile rot in combination with horizontal curvature may, result in a series of humps visible to the driver for some distance, a hazardous condition.

(c) Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This condition is hazardous in that the driver carinot perceive the horizontal change in alignment, especially at night when the headlight beams go straight ahead into space. The hazard of this arrangemerit is avoided if the horizontal curvature leads the vertical curvature, i.e. the horizontal curve is made longer than the vertical curve. Also, suitable design can be made by using design values well above the minimums for the design speed.

(d) 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.

(e) 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 toward long tangent sections to secure sufficient passing sight distance in design.

(f) Horizontal curvature and profile should be made as Qat as feasible at intersections where sight distance a1mg both roads is important and vehicles may have to slow down or stop.

(g) 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 oneway roadways

CHAPTER 5

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.

Pavements may be considered as three general types - high, intermediate and low. High type pavements are justified for high volume traffic for which it is fitting that the surface have smooth riding qualities and good nonskid properties in all weather. Intermediate type pavement vary from those only slightly less costly and with somewhat less strength than high type pavements to surface treatments. Low type pavements range from surface treated earth roads and stabilized, materials to loose surface such as earth and gravel.

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 "Manual On Pavement Design".

TABLE 5-1 : PAVEMENT SURFACE TYPE

Design Standard	Pavement Type	Description
R6 / U6	H / H	Asphaltic Concrete / Concrete
R5 / U5	H / H	Asphaltic Concrete / Concrete
R4 / U4	I / H	Dense Bituminous Macadam / Asphaltic Concrete / Concrete
R3 / U3	I / I	Bitumenous Macadam / Concrete
R2 / U2	I / I	Surface Treatment / Semigrout
R1 / U1	L / I	Earth Gravel / Semigrout
R1a / U1a	L	Surface Treatment / Semigrout

Where :
- H : High Type Pavement
- I : Intermediate Type Pavemant
- L : Low Type Pavement

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 uncurbed pavements due to pavement imperfections or unequal settlements and to control the flow of water adjacent to the curb on curbed pavements. Table 5-2 indicates the range of cross slopes for various pavement types.

TABLE 5-2: NORMAL PAVEMENTS CROSS SLOPE

Surface Type	Cross Slope Rate (percent)
High	2.5
Intermediate	2.5 - 3.5
Low	2.5 - 6.0

5.2 LANE WIDTHS AND MARGINAL STRIPS

Lane widtlis 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 5-3 indicates the lane and marginal strip widths that are to be used for the various road standards.

TABLE 5-3 : LANE AND MARGINAL STRIP WIDTH

Design Standard	Lane Width (m)	Marginal Strip Width (m)
R6 / U6	3.50	0.50
R5 / U5	3.50	0.50
R4 / U4	3.25	0.25
R3 / U3	3.00	0.25
R2 / U2	2.75	0.00
R1 / U1	(5.00)	0.00
R1a / U1a	(5.40)	0.00

NOTE : ( ) denotes the total two-way lane width

On 6-lane: divided highways, the middle lane of the three lanes on each direction should have a wider lane of 3.75m when the commercial vehicle volume exceeds 20% of the total traffic.

Figure 5-10A and 15-10B shows examples of the usage of the above wider lama of 3.75m and the usage of the marginal strips.

5.3 SHOULDERS

5.3.1 General Characteristics

A shoulder is the portion of the roadway continuous with the travelled way for accomodation of stopped vehicle, for emergency use and for Lateral support of the pavement.

Their main functions are :

(a) space is provided for emergency stopping free of the traffic lane.

(b) space is provided for the occasional motorist who desires 'to stop for various reasons.

(c) space is provided to escape potential accidents or reduce their severity.

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

(e) sight distances is improved in cut sections, thereby improving safety.

(f) highway capacity is improved and uniform speed is encouraged.

(g) lateral clearance is provided for signs and guardrails.

(h) structural support is given to the pavement.

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 lout 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--4A and 5--4B gives the widths of shoulders for the various road standards in rural and urban areas.

TABLE 5-4 : 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
R1a	1.50	1.50	1.50

TABLE 5-4Ba SHOULDER WIDTH (URBAN)

Design Standard	Usable 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
U1a	1.50	1.50	1.50

(*) For Area Type see Table 3-2B

5.3.3 Shoulder Cross Slope

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

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

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

5,3.4 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 offers many advantages in this aspect, Paved shoulders of the same strength and standard as the pavement should be considered for road standards R6, R5 and U6, U5 while stabilised shoulders designed to cater for 50% of the design traffic should be considered for road standards R4, R3 and U4, U3. For road standards R2, R1, U7,, U1 and R1a, the normal earth type shoulder can be used. However, where gradients exceed 8% the shoulders should also be paved to prevent erosion.

5.4 KERBS

5.4.1 General Consideration

The type and location of kerbs 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 localised areas which has predominant aspects of an urban condition.

5.4.2 Types of Kerbs

The two general classes of kerbs are BARRIER KERBS and MOUNTABLE KERBS and each 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 te, form a combination kerb and gutter section.

Barrier kerb 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 alsc 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, where pedestrian traffic is light, a semibarrier type may be used.

Mountable kerbs are used to define pavement edges of through carriageways. For channelisation islands, medians, outer separators or any other required delineation within the roadway, a ,semi-mountable type may be used.

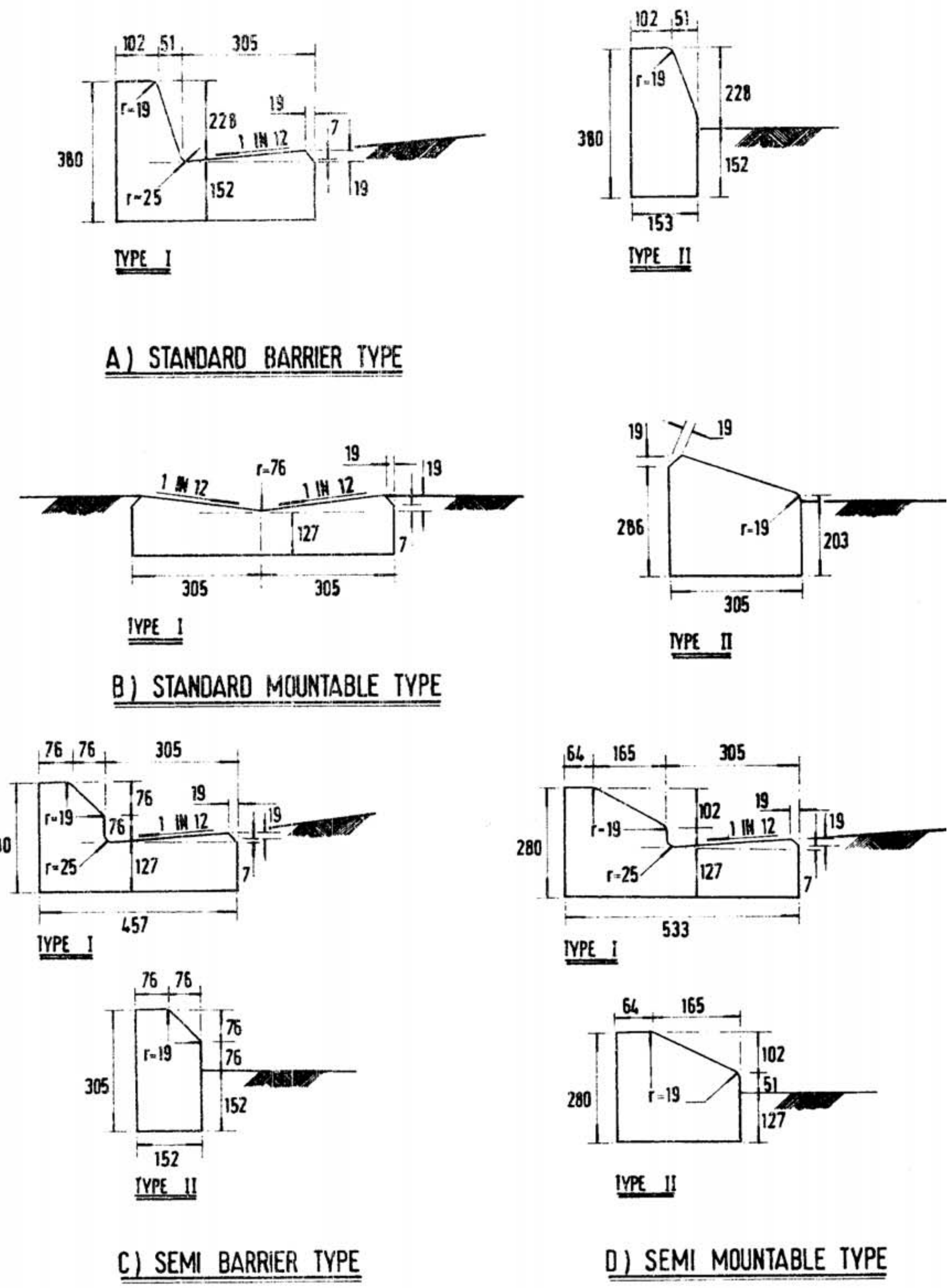
Figure 5-1 shows the various standard types of kerbs that are to be used.

The width of kerbs are con sidered as cross section ele ments 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, an offset of 0.3 to 01.6m is desirable.

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 are 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

FIGURE 5-1 STANDARD KERB TYPES



NOTE.

ALL DIMENSIONS ARE IN mm.

governed by the volumes of padestrian and vehicular. traffic, their relative timiag and the speed of the vehicular traffic.

In urban areas, sidewalks can be placed adjacent to the curb and raised above the pavement. In the absence of curbs, 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 desriable minimum width of 2.Om is to be provided for all sidewalks. Where there are restrictions on right of way, a minimum of 1.25m can be considered. When provided, sidewalks must have all weather surfaces.

5.6 TRAFFIC BARRIERS

Traffic barriers are used to minimise the severity of potential accidents involving vehicles leaving the travelled way. Because barriers are a hazard in themselves, emphasis should be on minimising the number of such installations. Arahan Teknik (Jalan) 1/8.5 "A2anual On Design Guidelines of Longitudinal Traffic Barrier" (May, 1984) 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 front 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 medium 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 flush with the pavement surface. They should be as wide as feasible but of a dimension in balanced with other components of the cross-section. The general range of median width varies from a minimum of 1.0 m in a Type III urban situation to a desirable width of 18m on a rural expressway. On wide medians, it is essential to have a depressed centre or swale to provide for drainage.

Figure 5-2 gives examples of kerbed and unkerbed medians while Table 5-5A and 5-5B gives the minimum and desirable widths and types of medians that are to be applied to the various road standards. The median widths as expressed are the dimensions between the through lane edges and includes the right shoulders if any.

TABLE5-5A : MEDIAN WIDTH AND TYPES (RURAL)

Design Standard	Median Width (M)						Median Type
	Terrain						
	Flat		Rolling		Mountainous		
	Min.	Des.	Min.	Des.	Min.	Des.	
R6	6.0	18.0	5.0	12.5	4.0	8.0	B, C, E, F
R5	4.0	12.0	3.5	9.0	3.0	6.0	E, F
R4	3.0	9.0	2.5	6.5	2.0	4.0	E, F

Note : Min. - Minimum
Des. - Desirable (for consideration of landscaping)

TABLE 5--5B : DREDIAN WIDTH AND TYPES (URBAN)

Design Standard	Medium Width (M)						Median Type
	Area Type						
	I		II		III		
	Min.	Des.	Min.	Des.	Min.	Des.	
U6	4.0	12.0	3.5	9.0	3.0	6.0	B, C, E, F
U5	3.0	9.0	2.5	6.5	2.0	4.0	B, C, E
U4	2.5	7.5	2.0	5.0	1.5	3.0	A, B, C, D
U3	2.0	6.0	1.5	4.0	1.0	2.0	A, B, D

Note: Min. - Minimum
Des. - Desirable (for consideration of landscaping)

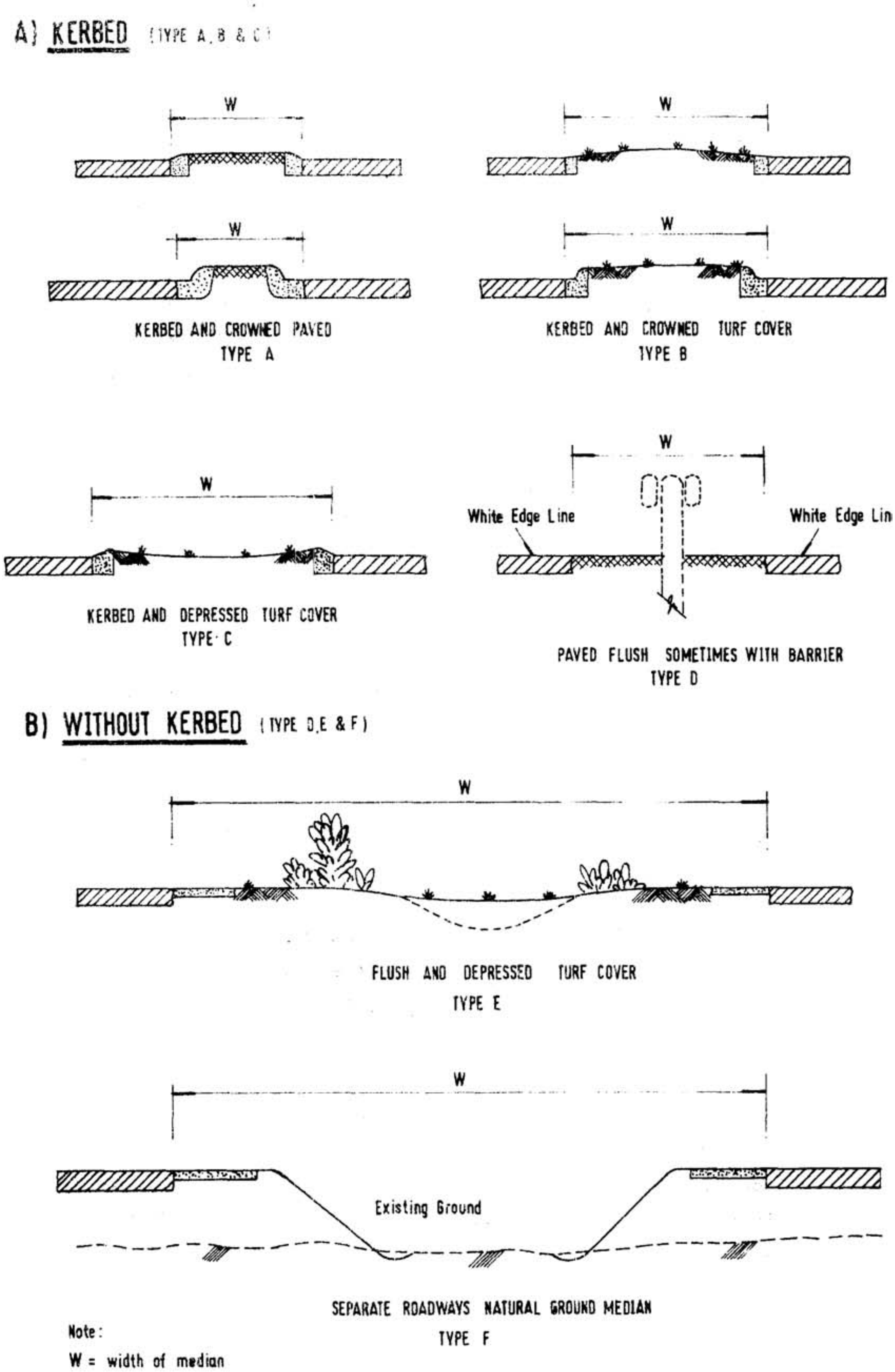


FIGURE 5-2 MEDIAN TYPES

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 favourable access for commercial and residential develovment than the faster moving arterials but also helps 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 nuch preiferred 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 access to the through facility are infrequent; 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 b e 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 or 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. Figure 5-3 gives the recommended layout for a tw-o-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.

5. 9 PEDESTRIAN CROSSINGS

5. 9.1 General Considerations

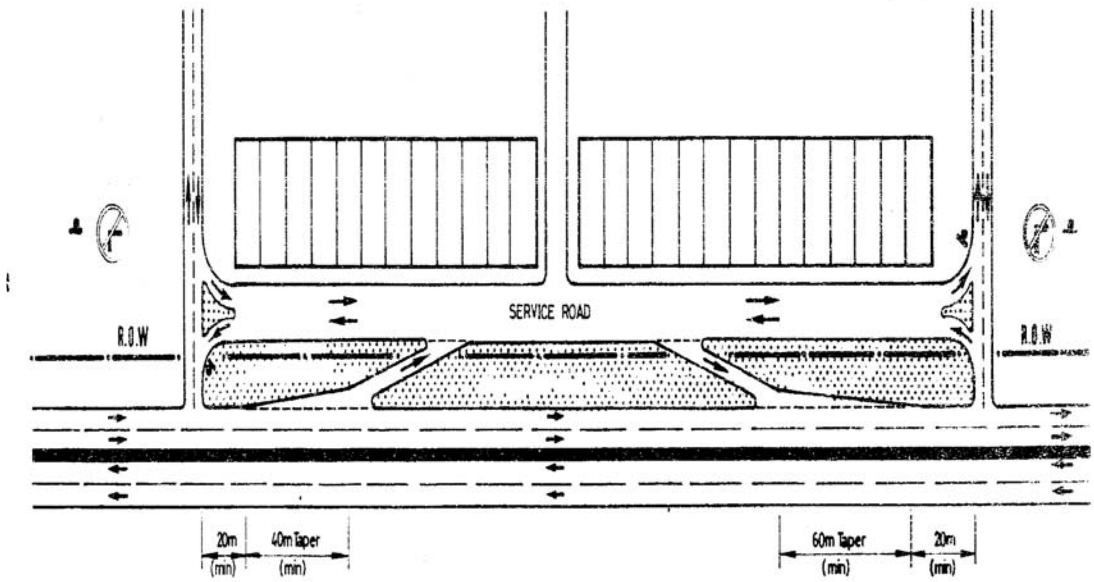
Pedestrian crossings (whether level, overpass or underpass) should be provided where pedestrian volumes, traffic volumes, intersection capacity and other conditions favour their use. They may be warranted in areas of heavy peak pedestrian movements such as factories, schools, athletic fields or control 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 does 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	Traffic Volume (1 way) at peak hour	Type of Crossing
< 50	< 1000	Ordinary level crossing
50 - 100	100 - 2000	Signalised level srossing
> 100	> 2000	Overhead crossing / underpass

FIGURE: 5-3 TWO-WAY SERVICE ROAD



For roads with dual 3-lanes or more, an overhead crossing or underpass hcali aI.wz!ys he considered if a pedestrian crossing is justified so as not to impede the smooth flow of traffic unduly.

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 <u>School Level Crossing</u>

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, an unsignalised crossing must always be provided.

5.10 <u>U-TURNS</u>

5.10.1 <u>General Considerations</u>

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

a. Locations beyond intersections to accomodate minor turuing movenente 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.

b. Locations just ahead of an intersection to accomodate U-turn movements that would interfere with through and other turning move ments at the intersection. Where a fairly wide median on the approach roadway has few openings, 1U-turning is necessary to reach roadside areas. Advance separate openings to accomodate them outside the intersection proper will reduce interference.

c. 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 volurne traffic, a shopping area, or other traffic generator that requires a median opening nearby and additional median openings would not be practical.

d 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.

e. 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.

(a) Minimum Design four 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 5-4 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 5-5A and B is suggested Where direct U-Turns will not be feasible because of median restrictions, indirect U-Turns can be used.

(b) Indirect U-Turns

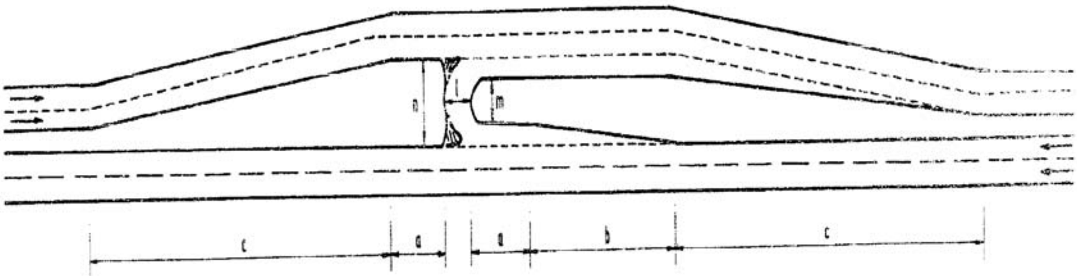
Indirect U--Turns can take many forms, the simpliest being to allow traffic to use existing local streets or to go around the block for their turning move-ments. Where medians are narrow, the special indirect U-turn as shown in Figure 5-6 can be used. Indirect U-Turns are only to be used for Road Standards R4, U4 and U3 only.

TABLE 4-s P ~t Widening On Open Road Curves

PKNW Win (m)	7				5.5				5-5				REU8iE0 WMENING	
DESM SPEED (Km/h)	80	50	50	I0	60	50	40	30	50	40	30	70	(M)	
	P--470	8-340	R> 180	R-1- 230	R'-' 1100	R 2 880	Rar 680	R2510	R R'100	R' 68	R> 52	M39 39	MORE	
	4M- ~8U	340- R-Ir	20- 11-- 1513	2309 R 130	1100>n R=340	880- R -2800	670} R ~ Z%	520'*- R ICU		68~ It- 4u	52=- R al.i	39> If -°	a	
		180~°' R~t150	150s- R~-100	130- 8286	340- R ~180	280- R~150	230y R'130	190- R'110				J7~'° R'`24	1	
				86' R264	180' R:!'150	150- R y100	130=- R286	110=- 8274				24 4'-16	, ·"	y
Note:				64~ 8260			86a R265	74~ R251				16~° R' 15	1.25	
							65' R260	57 8245					154	
								45 R 238					1.75	
								35> R 2- 35					2, 4 e	

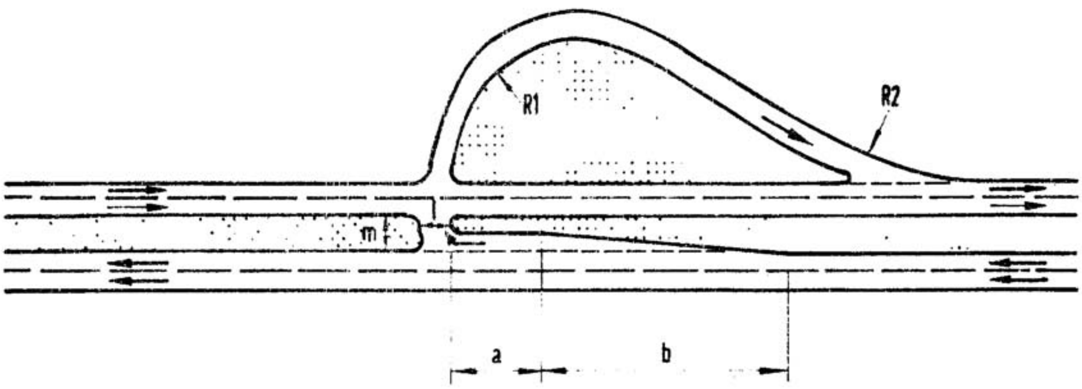
R rahm in meth s

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



DIMENSION	MINIMUM WIDTH / DISTANCE (m)
a	20
b	60
c	120
l	7·5
m	10·0
n	17·0

FIGURE: 5 - 6 SPECIAL INDIRECT U-TURN



DIMENSION	MINIMUM DISTANCE (m)	DIMENSION	MINIMUM DISTANCE (m)
m	4	R1	50
1	8	R2	100
a	20	b	60

(c) 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 5-8 can be used to determine the minimum distances between U-turns.

TABLE 5-8 : 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	1 km
		U3	0.5 km

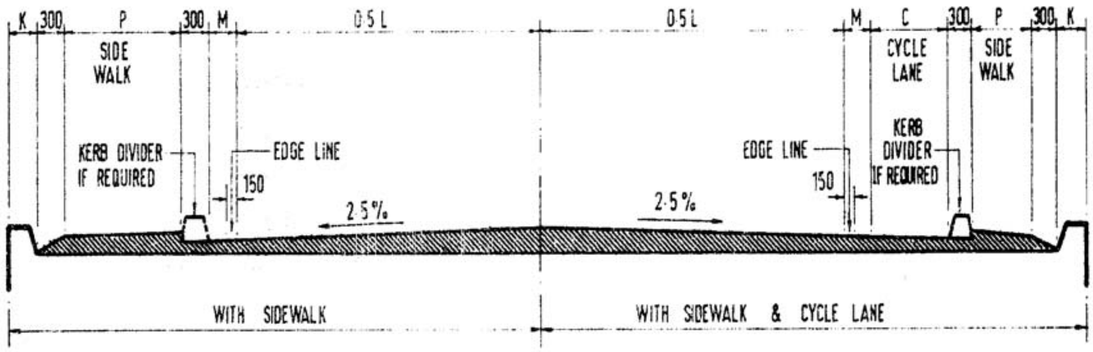
5.11 BRIDGE AND STRUCTURE CROSS-SECTIONS

5.11.1 Width of Shoulders

For bridges that are less than 100m long, the width of the usable shoulders should follow that as indicated in Table 5-4A and 5-4B. Where bridges are longer than 100m, the values as shown in Table 5-9 should be used. Figure 5-7 shows the typical bridge cross-section to be used.

TABLE 5-9 : USABLE SHOULDER WIDTH FOR BRIDGES > 100m

Design Standard	Usable Shoulder Width (m)
R6 / U6	1.5
R5 / U5	1.5
R4 / U4	1.0
R3 / U3	1.0
R2 / U2	0.5
R1 / R1a	0.5
R1a / U1a	0.5



DESIGN STANDARD	CARRIAGEWAY (L)	SHOULDER (S)		MARGINAL STRIP (M)
		SPAN < 100m	SAPN > 100m	
R5 / U5	7000	3000	1500	500
R4 / U4	6500	3000	1000	250
R3 / U3	6000	2500	1000	250
R2 / U2	6000	2000	500	0
R1 / U1	6000	1500	500	0
R1a / U1a	6000	1500	500	0

NOTE:

1) ALL DIMENSIONS ARE. IN MILLIMETRES UNLESS STATED OTHERWISE.

2) DESIRABLE WIDTH Of SIDEWALK I" is 2- 0 m. A MINIMUM WIDTH TO BE USED IS 1.25 m.

3) WIDTH OF CYCLE LANE "C" 15 AS INDICATED IN TABLE 5 - 12.

4) MINIMUM WIDTH Of 'K' TO HE USED IS 300mm. FOR PROVISION OF SERVICE DUCTS THIS VALUE IS TO BE INCREASED.

5) KERBS FOR BRIDGES ARE LEFT TO DISCRETION.

5.11.2 Required Warance

(a) The clear vertical height of all structures should be at least 5.0m over the entire width of traffic lanes, auxiliamy lanes and lateral clearance areas to kerbs, walls on piers including shoulders. However, for structures over railway lines, the minimum vertical clearance above the rail level should be at least 6.5m to meet the requirements of Keretapi Tanah Melayu. For underpasses, an additional clearance of 300mm should be provided initially to allow for one to two cycles of future resurfacings.

(b) For minor roads and local streets or where alternative routes with clearances above 5.Om within a reasonable distance are available, the clear vertical clearance may be reduced to a minimum of 4.6m.

(c) Figure 5-8 indicates the clearance required at underpasses for various types of cross-sections.

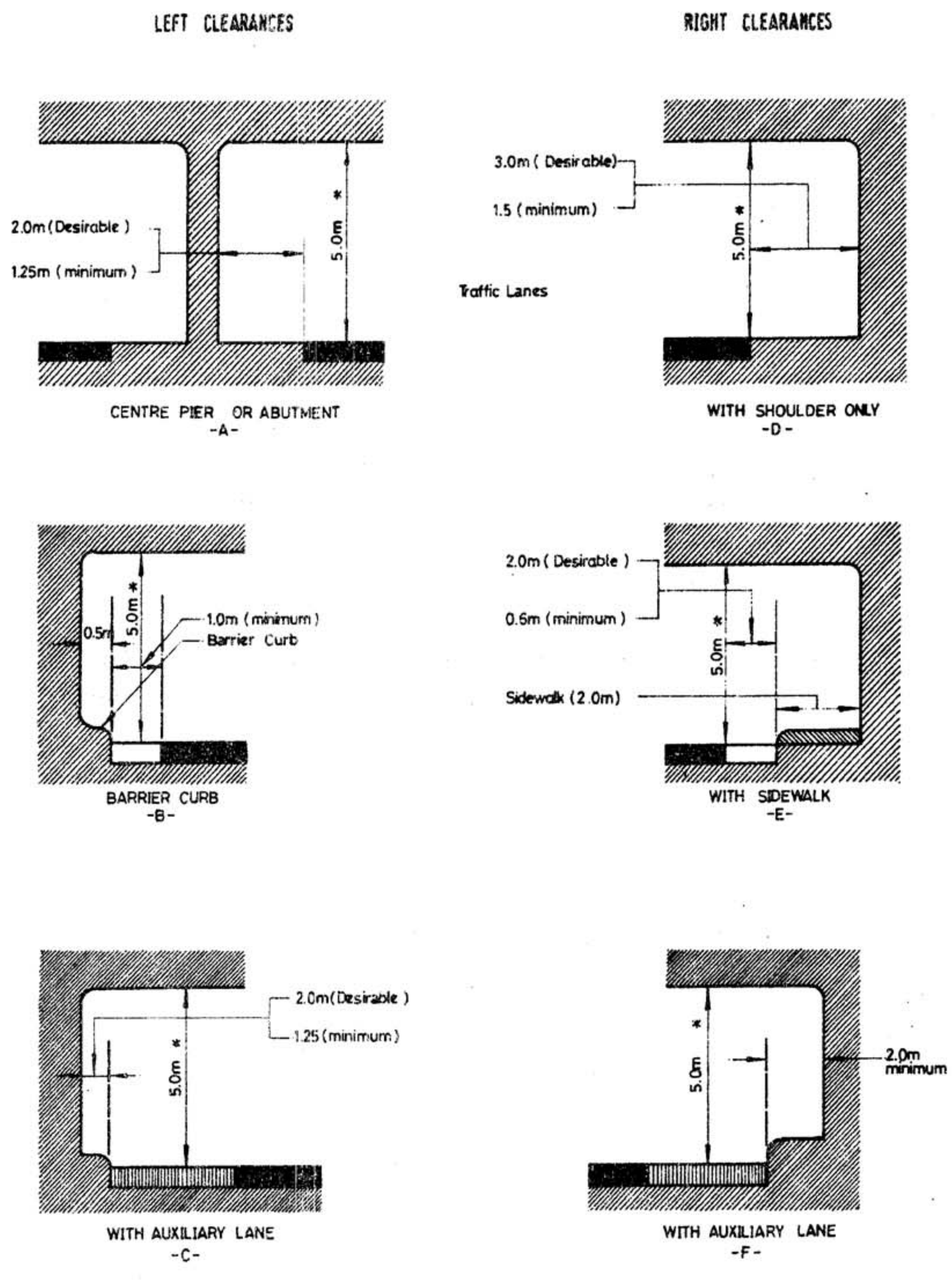
5.12 BUS LAYBYES

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

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

FIGURE 5-8 :CLEARANCES AT UNDERPASSES



* ADD 100mm for future resurfacing

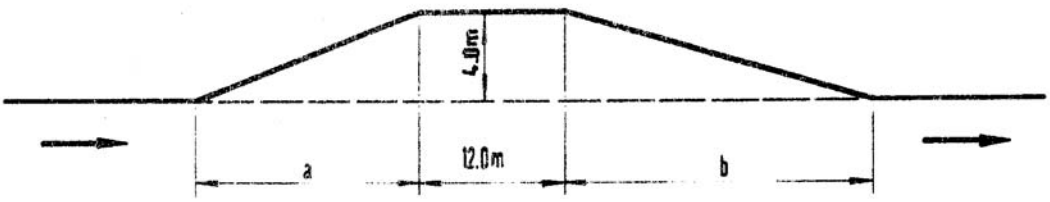
F1gure 5-9 show the typical layout and dimensions of bus laybyes that are to be use in both rural and urban areas. If possible dividing area between the outer edge of the roadway shoulder rind the edge of the bus laybye lane should be provided. This dividing area should be as wide as possible but not less than 0.6m.

The pavement areas of the laybyes Should be of concrete for contrast in ce)lour and tex-ture with the through traffic lanes to discourage through traffic from encroaching on or entering the bus stop.

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. Figure 5-2 OA and 5-108 show examples of the typical cross-sections for vari-ous road standard in rural and urban areas together with the minimum reserve required, while Table 5-10 lists the mini-mum reserve widths for the various road standards. The values as shown in Table 5-10 are for road standards in flat areas and will need to be increased accordingly for areas involving deep cuts or fills.

FIGURE 5-9: LAYOUT OF BUS LAYBYES



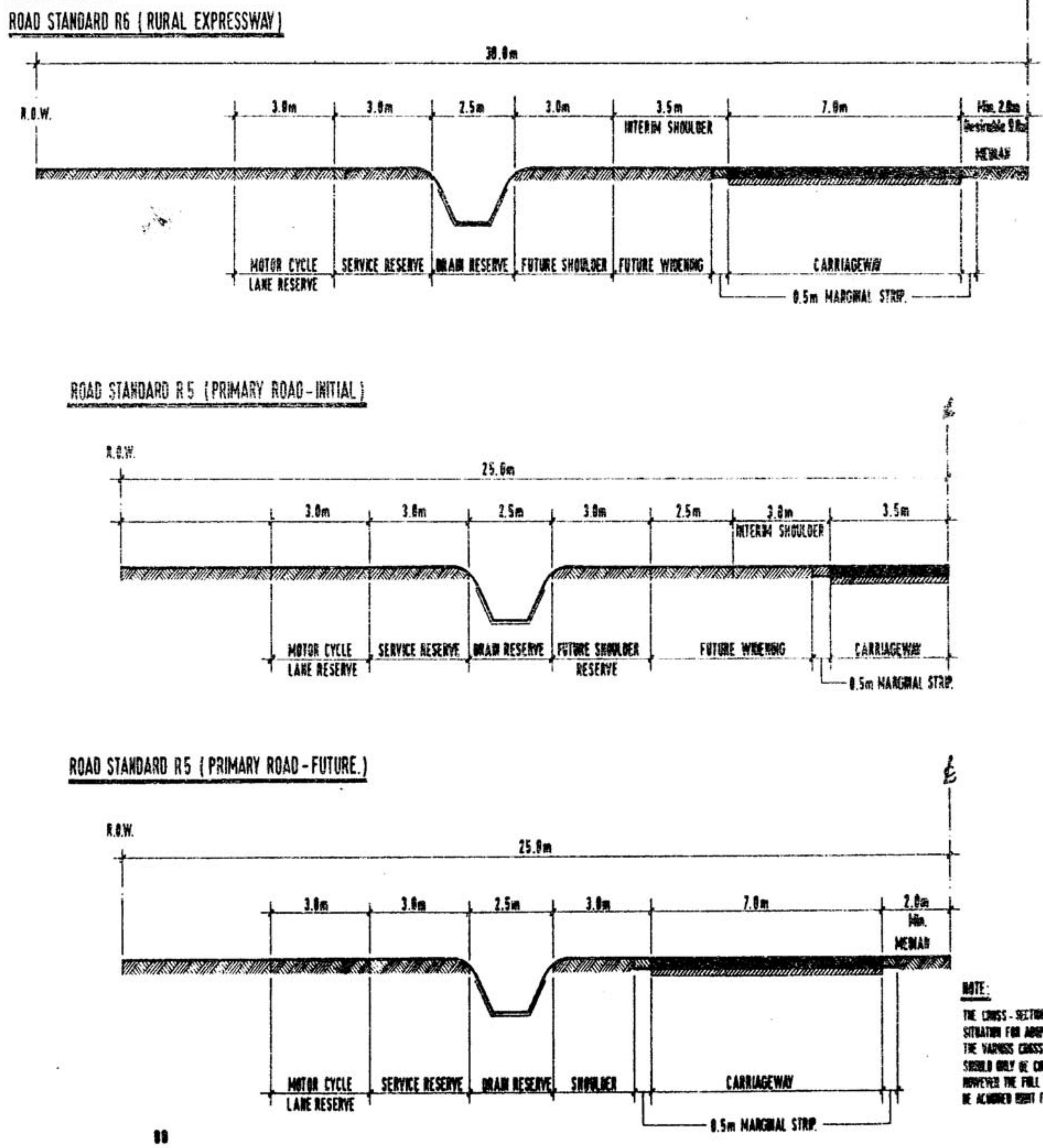
DESIGN STANDARD	DIMENSION (m)		DESIGN STANDARD	DIMENSION (m)	
	b	a		b	a
R5	70	35	U5	50	35
R4	50	35	U4	35	25
R3	35	25	U3	25	25
R2	25	25	U2	20	10
R1	20	10	U1	20	10
			Ula	15	10

TABLE 5-10 : 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 20
	Minor Road	R2 R1 R1a	20 12 12
URBAN	Expressway	U6	60
	Arterials	U5 U4	50 40
	Collector	U4 U3	40 30
	Local Street	U3 U2 U1 U1a	20 20 12 12

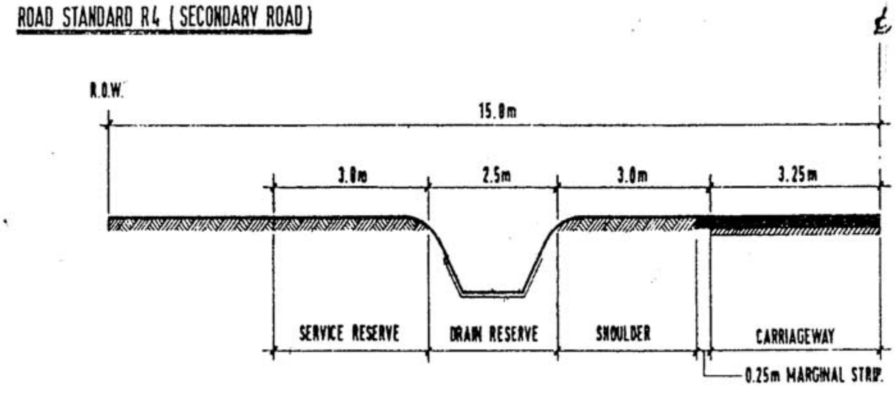
TYPICAL CROSS SECTIONS. (RURAL AREAS)

FIGURE 5 - 10A

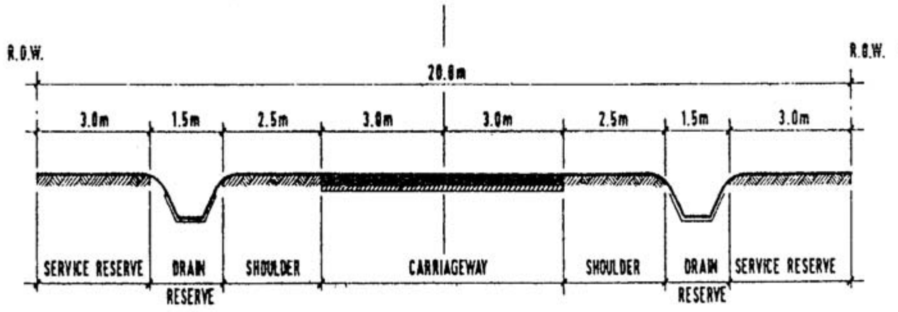


NOTE:
THE CROSS - SECTION REPRESENTS THE IDEAL SITUATION FOR ADOPTION WHEREVER POSSIBLE. THE VARIOUS CROSS - SECTIONAL ELEMENTS SHOULD ONLY BE CONSTRUCTED IF WARRANTED. HOWEVER THE FULL WIDTH OF RESERVE SHOULD BE ACQUIRED RIGHT FROM THE BEGINNING.

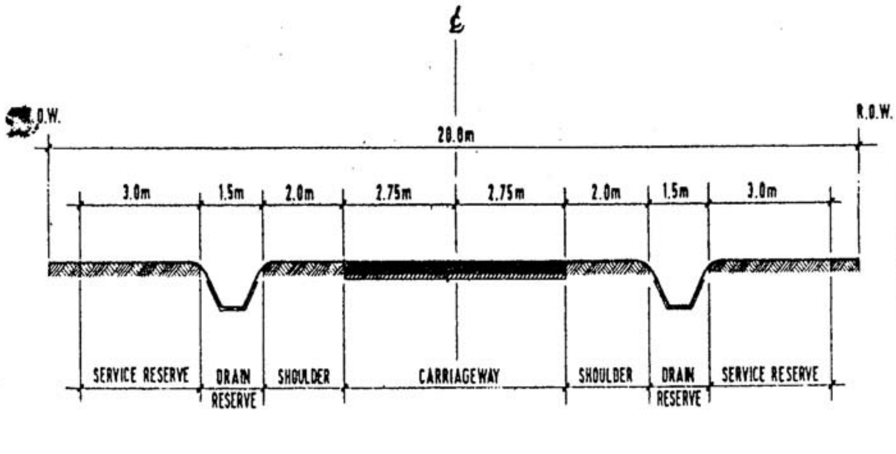
ROAD STANDARD R4 (SECONDARY ROAD)



ROAD STANDARD R3 (SECONDARY ROAD)



ROAD STANDARD R2 (MINOR ROAD)

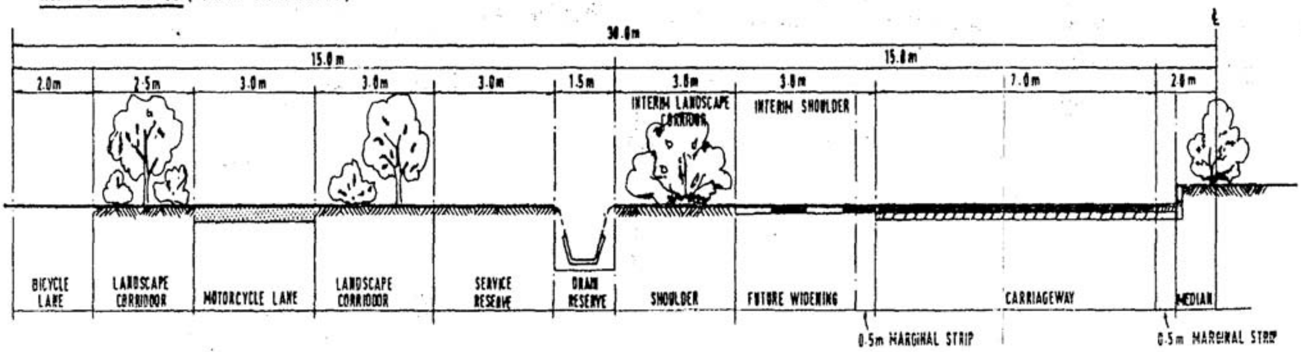


NOTE:
THE CROSS-SECTION REPRESENTS THE IDEAL SITUATION FOR ADOPTION WHEREVER POSSIBLE THE VARIOUS CROSS-SECTIONAL ELEMENTS SHOULD ONLY BE CONSTRUCTED IF WARRANTED. HOWEVER THE FULL WIDTH OF RESERVE SHOULD BE ACQUIRED RIGHT FROM THE BEGINNING

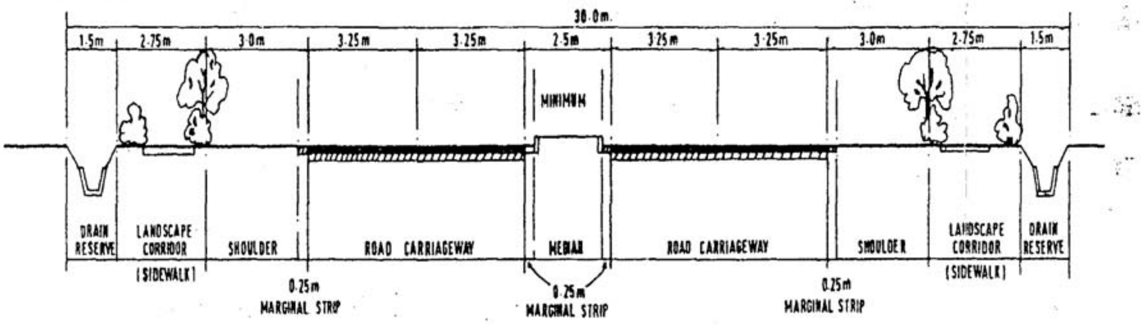
90A

FIGURE 5-10B TYPICAL CROSS-SECTION (URBAN AREA)

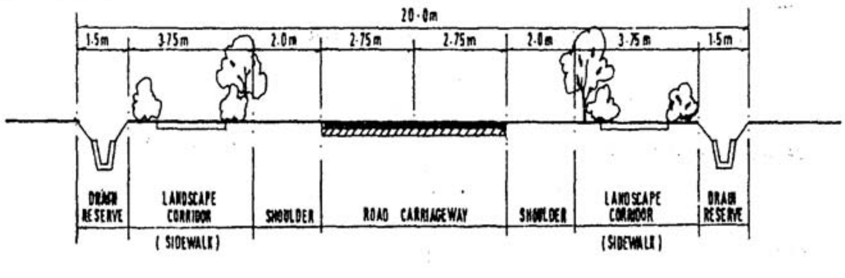
ROAD STANDARD U6 (URBAN EXPRESSWAY)



ROAD STANDARD U4 (MAJOR COLLECTOR)



ROAD STANDARD U2 (LOCAL STREET)



5.14 EXCLUSIVE CYCLE LANES

5.14.1 General Considerations

In areas where there is usually a high proportion of motor cyclists, the volume may be so substantial as to affect the smooth flow of traffic. In such instances, the provision of separate and exclusive cycle lanes should be considered. Figure 5-11 shows the various types of cycle tracks used. The general warrant for determining the need for an exclusive cycle lane are:

(i) the total volume of traf fic exceeds the provided lane cspacity

and

(ii) the volume of motorcycles exceeds 20% of the total volume of traffic.

5.14.2 Elements of Design

(a) Design Speed

The design speed to be used is 60 km/hr. However, where there are physical con straints, this may be lowered.

(b) Sight Distance

The priciples as established in Section 4.1 also applies.

(c) Horizontal Aligriment

(i) Minimum Radius

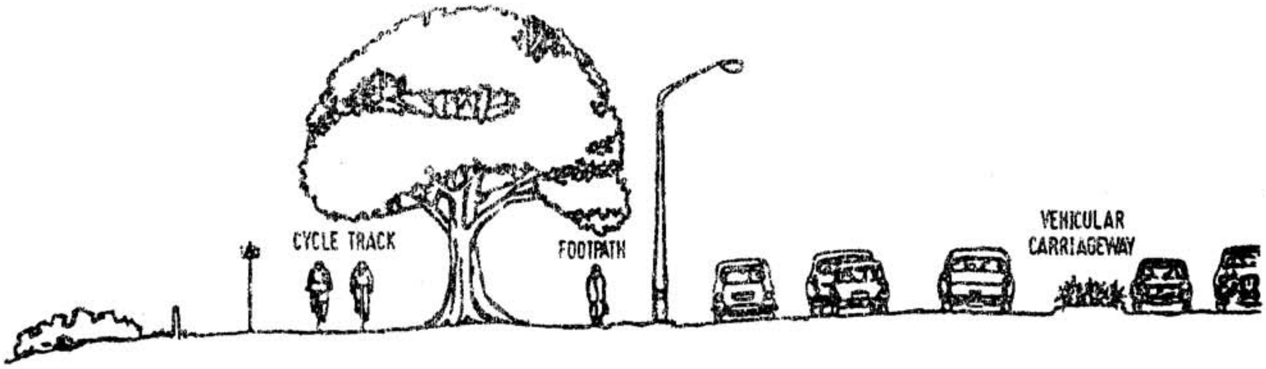
The following formula is used in determining the required minimum radius for the curves.

$$R = 0.24V + 0.43$$

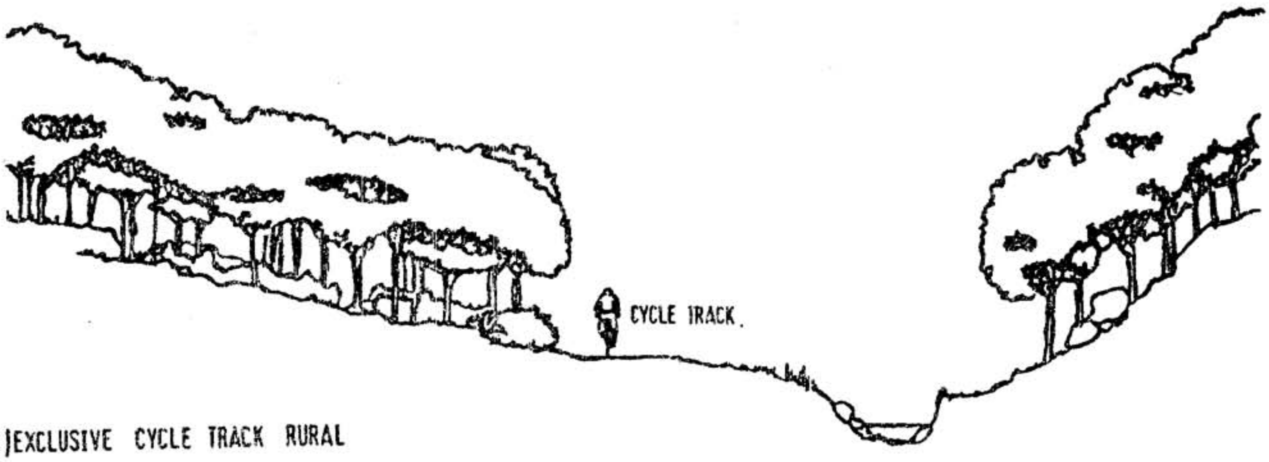
where R = radius of curve in (m)

V = speed in km/hr

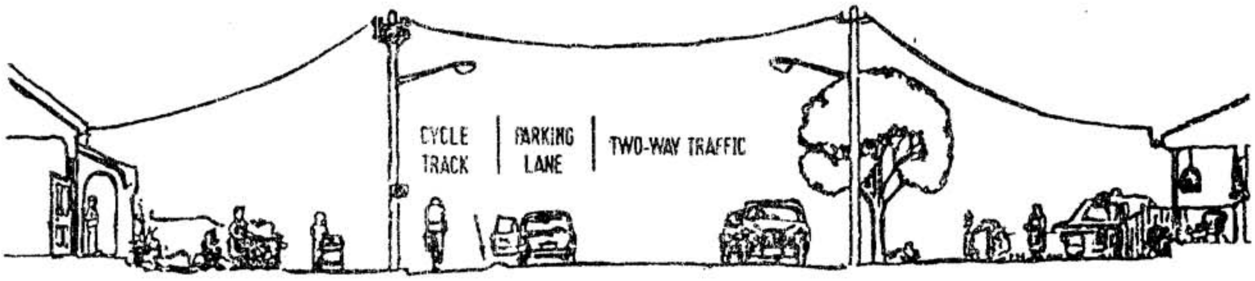
Table 5-11 gives the minimum radius that are to be used in design. Flatter curves should always be used wherever possible. There is no necessity to provide any transition (spiral) curves.



(A) EXCLUSIVE CYCLE TRACK URBAN



(B) EXCLUSIVE CYCLE TRACK RURAL



(C) RESTRICTED CYCLE TRACK.

FIGURE 5-11 : TYPES OF CYCLE TRACKS

TABLE 5-11 : MINIMUM RADIUS FOR DESIGN

Design Speed (km / hr)	Minimum Radius (m)
30	8
40	10
50	13
60	15
70	18
80	20

(ii) Curve Treatment

To allow for the motorcyclist to lean on curves, there is a need to increase the width through the curve. A maximum widening of 1.2m is to be allowed. Figure 5-12 shows the curve treatment that is to be used.

(iii) Superelevation

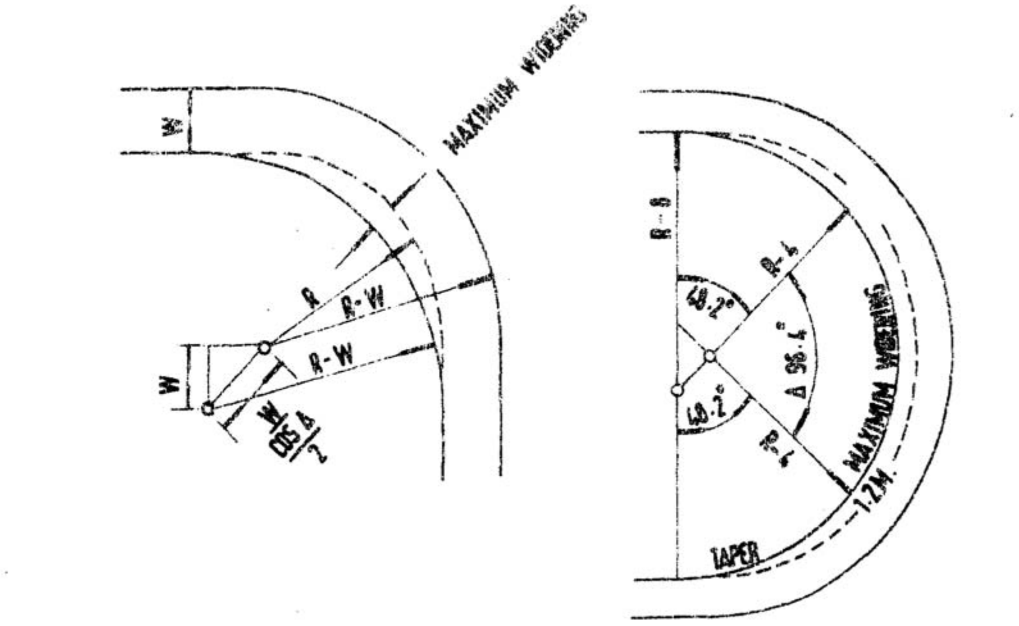
The maximum superele-vation to be used is 0.06. Superelevation should be applied from the tangent point to its required value at the point of maximum widening. This is as shown in Figure 5-13.

(d) Vertical Alignment

(i) Grades

The maximuin grade allot,7ed is 10% with a minimum grade Df 0.5%. At very flat areas, the minimum grade can be reduced further but with prior approval.

FIGURE 5-12: CURVE TREATMENT

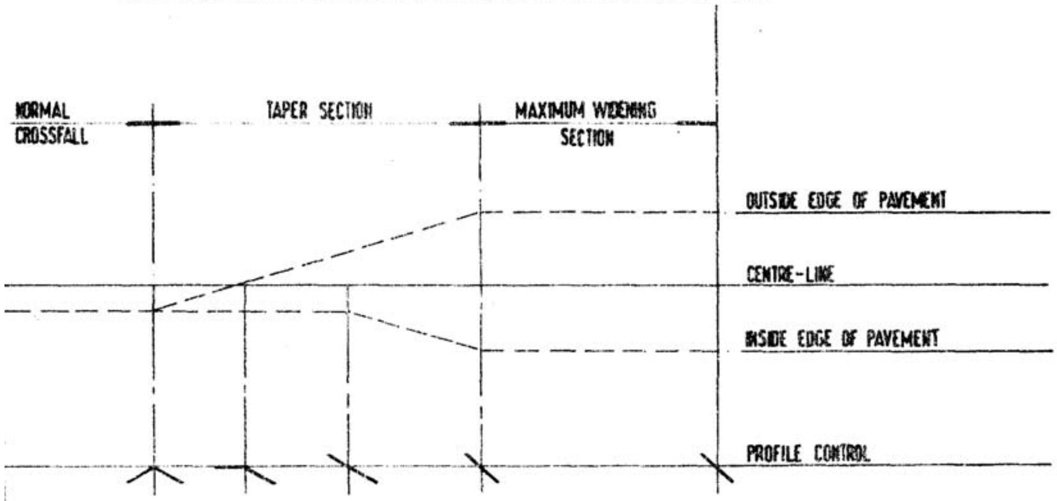


R - RADIUS OF CURVE
W - WIDTH
A - CENTRAL ANGLE OF IHE CURVE OF THE DEFLECrION BETWEEN TAN-GENTS,

MAXIMUM WIDEWING SHALL BE LIMITED TO 1.2M.

WHEN WKNK WANES I- 2M. (A 2' 95 - 4.) THAI WITH SHALL BE CAN G ON A M IS OF R-4 THROUGH THE CENTRAL PORITION OF THE CURVE (A>96.4) AS SHOWN

FIGURE 5-13 : METHOD OF ATTAINING SUPERELEVATION



(ii) Vertical Curves

The minimum vertical curve length to be used is 15m. On sag vert.ical curves, the design should be such as to avoid any ponding of surface water.

5.14.3 Cross-Section Elements

(a) Lane Width

The required widths of the cycle lane is as shown in Table 5-12, The cycle lane must be separated from any pedestrian sidewalk and the width of separation must be at Least 10-in.

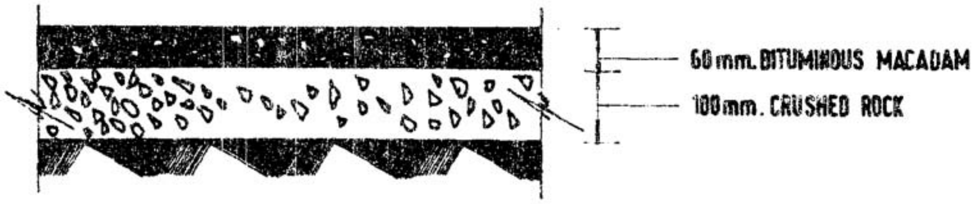
TABLE 5-12 : WIDTH OF CYCLE LANE

Volume of motorcycle / hr	Width of Cycle Lane (m)	
	Minimum	Desirable
1000 - 1500	2.0	2.5
1500 - 2000	2.5	3.0
> 2000	3.0	3.5

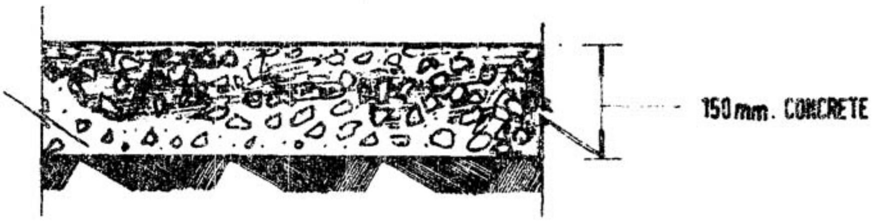
(b) Pavement

The pavement structure will vary according to ground and sub-surface conditions, volume of motorcycles and and type of use. Two types of standard design have been selected and are to be used. They are as indicated in Figure 5-14. The normal crossfall of the pavement is 2 %.

FIGURE 5-14 : CYCLE LANE PAVEMENT STRUCTURE



ALTERNATIVE 1



ALTERNATIVE 2

5.14.4 Intersection Treatment

At intersection or inter changex, some form of channelisation with specific roaces for rhe motor should be provided to minimise conflicts that will arise. Possible intersection treatment types include (i) at-grade or (ii) gzade-separated. The type of intersection treatment will depend on the volume of the traffic and the volume of motorcycles.

(a) An-Grade Intersection Treatment

An at-grade intersection treatment is sufficient where the volume of motorcycles dces not exceed 30 percent of the total volume of traffic at the particular intersection at. the peak hour.
An at-grade intersection treatment is sufficient with propey Agning, speed limits and lane markings.

(b) Grade Separated Intersection Treatment

When the volume of motorcycles exceeds 30 percent of the total volume of traffic at the particular intersection at the peak hour or when an at-grade intersecricn treatment does not provide a smooth flow or adequate safety to the motorcyclists grace separated intersection treatment should be considered.

These can be easily incorporated at roundabout and other intersections with the provision of underpasses (box culvert type).

The type of the underpass must have a 2.5m clearance with a maximum slope of 10 percent. They should be lighted to give better visibility and safety during the night Lime.

CHAPTER 6

OTHER ELEMENTS AFFECTING

6.1 Road Safety

Road safety have been a much neglected area in the design of roads. With an increasing number, of accidents each year, attention to road safety should be emphasised. While the road element is only one of the three groups 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 maljior changes in design speeds, tarisitions in roadway crass-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 inaintain. consistency in the roadway design. The highway should offer, no surprises to the driver, in terms of either geomet rics or traffic controls.
The tendency to base road designs upon minimum, standards rather than to adopt, an optimum design is unfortunate and must be avoided. A more liberal and optimi-mi design must always be considered, and although it would cost a little more in terms of capital cost of the project it would increase the road's level of safety substantially.

All too often, minimum design standards attempt to rely, on the use of warning signs or other added roadside appurtenances for the safe operation that could have been built in by the use of higher design standards, Often the safety deficiencies generated by minimum design are impossible to correct by any known traffic device or appurtenance. A warning sign is a poor substitute for adequate geometri design.

6.2 Drainage

Road drainage Utilities provide for carrying water across the right of way and for removal of storm water from the road itself. These facilities includs bridges, culverts, channels, gutters and various types cf drains. Drainage design considerations are an integral part of geometric design and flood plain encroachments frequently affect the highway alignment and profile.

The cost of drainage is neither incidental nor minor an most reads, Careful attention to requirements for adequate drainage and protectAn of the highway from floods in all phase of location axed design will prove to be effective in reducing costs in both construction and maintenance.

Reference should be made to the relevant JKR manual on this subject.

6.3 Lighting

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

Where lighting is being considered for future installation, considerable savings can be effected 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 the relevant JKR manual on this subject.

6.4 Utilities

All road improvement, whether upgrade 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 read, Full consideration should be given to measures necessary to preserve and protect the integrity and visual quality of the road, its maintenance efficiency arid the safety of traffic.

Utilities relocation shoold form part of design and the designer should Wase closely with the relevant service authorities in determining the existing utilities and their proposed relocation.

6.5 Signing and Markings

Signing and marking are directly related to the design of the road and are features of traffic control and operation that the engineer must, consider in the georretric layout of such a facility, The signing and marking should be designed concurrently with the geometrics as an integral part, arid this will reduce significantly tne possibility of future operational problems. The signing and marking should follow the standards that have been established. Feference should be made to Arahap Teknik (Jalan) 2A, 2B, 2C and 2D on the design, usage and application of signs an markings.

6.6 Traffic Signal

Traffic control signals are device 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.

6.7 Erosion Control Landscape Development and Environmental Impacts

Erosion prevention is one of the major factors in design, construction and maintenance of highways. It should be considered early in the location and design stages. Some degree of erosion control can be incorporated into the geometric design, particulatly in the cross-section elements.

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

(a) preservation of existing vegetation

(b) transplanting of existing vegetation where feasible

(c) planting of new vegetation

(d) selective clearing and thinning

(e) 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 on this subject.

The highway can and should be located and designed to complement its environment and serve as a catalyst to environmental improvement. The area surrounding a proposed highway is an interrelated system of natural, man-made and sociologic variables. Changes in one variable within this system cannot be made without some effect on other variable. Some of these consequences may be negligible, but others may have strong and lasting impact on the environment including the sustenance and quality of human life. Because highway location and design decisions have. an effect on adjacent area development, it is important that environmental variables be given full con sideration. Environmental impacts include social, economic and physical impacts. In geometric design, only physical impacts are assessed, while the social and economic impacts would have been taken care of during the planning stage.