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THE KINGDOM OF THAILAND

BANGKOK METROPOLITAN ADMINISTRATION

STUDY ON ROAD IMPROVEMENT, REHABILITATION AND TRAFFIC SAFETY IN BANGKOK

FINAL REPORT

VOLUME VI TECHNICAL GUIDELINE

MARCH 1987

JAPAN INTERNATIONAL COOPERATION AGENCY

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CONTENTS

Page

LIST OF FIGURE

Page

Page

Technical Guideline for Traffic Safety Devices

Page

LIST OF TABLES

Page

LIST OF APPENDICES

LIST OF ABBREVIATIONS

BMA	Bangkok Metropolitan Administration
CPD	City Planning Division, BMA
CMD	Construction and Maintenance Division, BMA
DD	Design Division, BMA
PPD	Policy and Planning Division, BMA
PPSd	Public Works Planning Sub-division, BMA
DPW	Department of Public Works, BMA
DDS	Department of Drainage and Sewerage, BMA
TED	Traffic Engineering Division, BMA
MOI	Ministry of Interior
OARD	Office of Accelerated Rural Development, MOI
OCMRT	Office of the Committee for the Management Road Traffic, MOI
OPP	Office of Policy and Planning, MOI
PWD	Public Works Department, MOI
TCPD	Town and Country Planning Department, MOI
TPD	Traffic Police Division, MOI
LDPD	License Division of Police Department, MOI
MOC	Ministry of Communications
DOH	Department of Highways, MOC
DLT	Department of Land Transport, MOC
ETA	Expressway and Rapid Transit Authority of Thailand
NESDB	National Economic and Social Development Board
SRT	State Railway of Thailand
MEA	Metropolitan Electricity Authority
AIT	Asia Institute of Technology
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
BS	British Standards
CAB	Cable Box
CBD	Central Business District
HCM	Highway Capacity Manual
MCI	Maintenance Control Index
MSL	Mean Sea Level
MTS	Mass Transit System
NECO	National Executive Council Order
PCU	Passenger Car Unit
PSI	Present Serviceablility Index
RAL	Richtlinien für die Anlage von Landstraßen
SSES	Second Stage Expressway System
STTR	Short Term Urban Transport Review

TECHNICAL GUIDELINE
FOR
ROAD PLANNING

1. TECHNICAL GUIDELINE FOR ROAD PLANNING

1.1 Introduction

Efficient and economical road construction, improvement and maintenance are attained by methods including applying reasonable technical standards developed based on well conceived techniques and ample experience gained through actual road management and operation. This has led to establishment of many technical standards on various aspects of highway engineering , based on research and studies carried out over many years. In contrast, in a number of developing countries, there appears to be no authorized standards of road engineering; thus, road planning and designing are done by the methods and technical criteria selected by individual engineers on a project by project basis.

Although BMA has accumulated knowledge and information on highway engineering, it is yet to develop a body of comprehensive technical standards applicable to all aspects of road planning and designing.

With the primary objective of applying them to a case study of planning for road improvement, pavement rehabilitation and traffic safety measures, some essential technical standards are proposed.

Any technical standard should be determined taking into account many matters such as the results of technical experience and research, the social environment, economic viability and characteristics of the drivers in the relevant country. As such the technical standards, to be proposed by this Study which has been performed under a number of constraints including limited time and information, could be safely called "technical guidelines". We do not rule out the possibility that BMA will make the proposed guidelines its technical standards, however, if they are found reasonable.

The technical guidelines are to be proposed for three categories : 1) road planning, 2) pavement rehabilitation planning, 3) traffic safety planning. They are to be worked out with reference and due considerations to the following:

- relevant standards prevailing in Thailand,
- relevant standards prevailing in other countries, and
- current BMA practices on highway engineering.

1.1.1 Collection and Review of Relevant Reference Materials in Thailand

The technical guidelines should maintain consistency with prevailing engineering practices in Bangkok. Relevant references were collected not only from BMA, but also from other concerned agencies including DOH, ETA, OCMRT and the Public Works Department of MOI.

Any technical guidelines or standards in any country should be worked out using the knowledge accumulated through studies, research and actual experience in the country. However, practices in road engineering projects in Bangkok have been based on the standards used in other countries such as U.S.A., U.K. and Japan.

(1) BMA

Examples of prevailing engineering practices in BMA were collected. In order to collect data, work sheets shown in Appendix 1.1.1 were prepared, and data of the construction work carried out in 1984 were provided by BMA's counterparts. The following is the summary of the result of this survey.

- BMA has its own standard for traffic signs and pavement marking, although it has no own authorized standards for other items.
- The design and planning of roads, bridges, pavement etc. are mainly based on AASHTO standards.

(2) Other agencies in Thailand

In order to know standards used by other agencies, the questionnaire on the design standard shown in Appendix 1.1.2 was prepared and distributed to the relevant agencies. Following is the summary of this survey.

In Thailand, there is no authorized standard which is used by all concerned agencies in common. DOH and OCMRT have their own standards. DOH's standard is based on AASHTO's standard. As for traffic safety facilities, there are two different standards in Thailand. One is used by the Ministry of Interior (MOI) mainly for municipal roads, the other is used by DOH for it's roads.

ETA and PWD (MOI) do not have their own standards. These agencies have been designing structures (bridges) based on AASHTO standards. In PWD (MOI), British Standards (BS) have been used for determining the live loads.

1.2 Road Planning

Descriptions enclosed in boxes in the following pages are the articles of the Guideline. The following statements are supplementary explanations of the articles, such as basis, reason, and application of them.

1.2.1 Classification of Roads

In Thailand the roads are classified according to the National Executive Council Order (NECO) No. 295 (1972) issued as an announcement of the Revolutionary Party. The Expressways under the jurisdiction of the Expressway and Rapid Transit Authority of Thailand (ETA) were prescribed by NECO 290 (1972).

In NECO 295 (1972), the classes of roads, their functions and administrators are prescribed. In NECO 295, roads are classified into the following seven categories (see Table 1.2.1).

- Special Highways
- National Highways
- Provincial Highways
- Rural Highways
- Municipal Highways
- Sanitation Highways
- Concessioned Highways

The roads under the jurisdiction of the BMA fall into the category of Municipal Highways.

The Municipalities are responsible for the construction, expansion, improvement and administration of the roads registered as Municipal Highways.

In a few cases, PWD of MOI carries out planning and construction of the roads not necessarily related to bridges.

Besides the road classification by NECO, there are several other classifications used by the respective authorities.

The classifications of roads in Japan, AASHTO, and STTR are shown in Appendices 1.2.1, 1.2.2 and 1.2.3.

Table 1.2.1 Road Classification and Relevant Authority by NECO 295 (1972)

Classification	Responsible Authority
(1) Special Highway	DOH (MOC)
(2) National Highway	DOH
(3) Provincial Highway	DOH
(4) Rural Highway	Changwat Administrative Organization
(5) Municipal Highway	Municipality, MOI
(6) Sanitation Highway	Sanitation area, MOI
(7) Concessioned Highway	Specified by the Government

Notes 1) The relevant authorities specified above are the general cases. In Thailand, there are many special cases that the authorities other than the above are responsible, e.g. RRDO, PWD, ETA.

2) The PWD of the MOI is in charge of planning, construction and road administration of the bridges and approaches over large rivers. In some rare cases, the PWD of the MOI carries out planning and construction of the roads not related to bridges.

(1) Classification of Roads in BMA

1. The roads of BMA are classified as shown in Table 1.2.2.

Table 1.2.2 Classification of BMA Road

Class	Design Speed (km/h) Urban	Design Speed (km/h) Suburban Rural	Number of lane Urban	Number of lane Suburban Rural	Access Control	Remarks
1. Major Trunk Road	60～80	80～100	≧4	≧4	* F.P.N	
2. Major Road	40～60	60～ 80	≧4	≧2	N.	
3. Minor Road	30～50	40～ 60	≧2	≧2	N.	
4. Access Road	20	30～ 50	≧1	≧1	N.	

F : Full control of access
P : Partial control of access
N : No control of access
*Full access control is included here for the future construction of high standard road and may be used only in very limited cases at present. Hence, other parts of this guideline do not cover designs for fully access-controlled road.

The general flowchart for the method of the road classification is shown in Figure 2.7.1, Volume I. Roads have several functions as such. In this method, the road classification is done taking into account both the road network characteristics and traffic functions. The network characteristics are composed of road classes prescribed by administrators and the network density. Traffic functions, however, are determined considering various factors including traffic volume, large commercial vehicle volume, trip length and ratio of daily traffic to daytime traffic.

In many countries, roads are classified according to their traffic function. The traffic function of roads is illustrated conceptually in Figure 1.2.1. The following classifications of urban roads are used in Japan.

- Urban Expressway
- Major Trunk Roads
- Major Roads
- Minor Roads
- Access Roads

In many other countries, similar classifications are used.

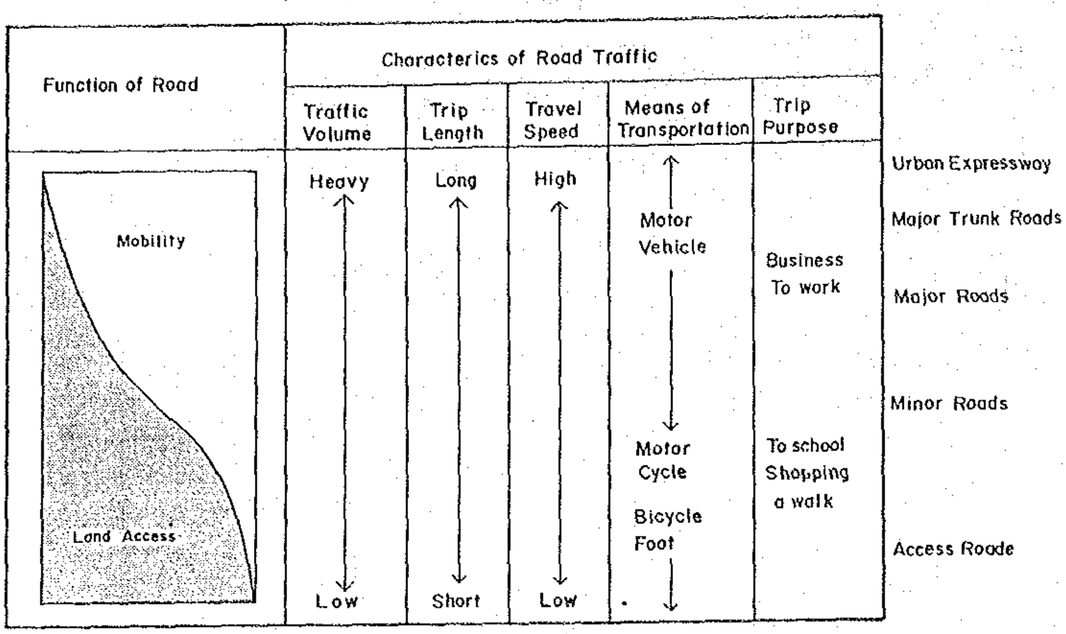


Figure 1.2.1 Relation of the Function of Road to the Characteristics of Road Traffic

Traffic functions of the roads in Bangkok were studied in this study as described in Chapter 2, Volume I. Some pictures for road classification are obtained from the result. Finally, the roads of BMA should be classified taking the road network characteristics and the traffic functions into consideration.

It is proposed here that the roads of BMA be classified into the following four categories according to the functional classification.

- Major Trunk Roads
- Major Roads
- Minor Roads
- Access Roads

The functions of these roads are described briefly in the following.

1) Major Trunk Roads

Major Trunk Roads compose the main frame of the road network. On the Major Trunk Roads, comparatively large number of trips with long trip lengths are made, such as in intercity traffic and through traffic. It is necessary to plan the road network so that it is simple, in order to manage these traffic flows effectively.

Major Trunk Roads should be designed by advanced standards. Major Trunk Roads have four or more lanes. Their main intersections should be separated in grades, and at-grade intersections should be adequately designed. As a result, such roads can carry heavy traffic volume. It is important for Major Trunk Roads to secure smooth and comfortable traffic flow. Travel speed of the roads should be kept at a high level. However, consideration of land access should not be ignored.

Among the categorized groups of roads shown in Figure 2.7.6, Volume I, the group 'I' and group 'II' roads are considered to be classified in the Major Trunk Roads.

2) Major Roads

Major Roads contribute to the traffic flow connecting the main traffic generation points in the area surrounded by Major Trunk Roads. Major Roads connect the long trip traffic to the Major Trunk Roads in a rational manner. Major Road network composes the frame of whole city in addition to Major Trunk Roads network.

Major Roads have four or more lanes to allow smooth flow of traffic in urban areas. Their intersections may be either grade separated or at-grade. In case of an at-grade intersection, left-turn and right-turn lanes should be provided. The stopping lanes for access service are designated on most of the road sections except near intersections.

Among the groups in Figure 2.7.6, Volume I, the group 'III' roads are considered to be classified in the Major Roads.

3) Minor Roads

Minor Roads connect the Major Roads to the Access Roads. Roads structural design is to be of simple standard. Their intersections should be at-grade intersections. They generally have two lanes and stopping lanes.

From Figure 2.7.6, Volume I, the group 'IV' roads are considered to be classified in the Minor Roads.

4) Access Roads

Access Roads primarily permit direct access to abutting lands. These roads connect the higher-class roads to each other. They offer the lowest level of mobility. They usually contain no bus routes. Service to through-traffic movement usually is deliberately discouraged. For example, the narrow one-lane roads, such as the dead end 'Soi', are considered to be classified in the Access Roads.

Described above is the fundamental discussion of road classification. More detailed discussion would be necessary for such cases as,

- Major Trunk Road with very low traffic volume
  (can be constructed as 2-lane road) and
- Major Road with extremely heavy traffic volume
  (may be treated as Major Road)

Since the objective of this guideline is to lay a fundamental basis for road planning, these detailed discussions are not included. It is hoped, hence, that further development of this guideline be done by BMA staff based on their philosophy and experience.

1.2.2 Design Speed

(1) Design speed

1. The design speeds should be the values shown in Table 1.2.3. They should be selected according to the classification of road and the area.

Table 1.2.3 Design Speed

Class	Type (Area)	
	Urban	Suburban, Rural
1. Major Trunk Roads	60～80	80 ～ 100
2. Major Roads	40～60	60 ～ 80
3. Minor Roads	30～50	40 ～ 60
4. Access Roads	20	30 ～ 50

1) Meaning of design speed

The design speed can be defined as the following from the viewpoint of not only the structural aspect of roads but also the running condition of vehicle.

- Design speed is the speed determined in order to design the physical elements of the road which affect the running of vehicle and to correlate these elements mutually.
- Design speed is the speed at which ordinary drivers can keep comfortable, on the condition that the function of design elements of road works efficiently.

The elements of alignment, such as radius of curvature, superelevation, and sight distance, have direct bearing upon the design speed. It is difficult to relate the width of a lane, or shoulder, with the design speed directly, but these elements have an influence on the running speed.

The design speed is the speed at which ordinary drivers can have safety and comfort in the following cases;

- Weather is fine
- Traffic density is low
- Running condition of vehicle is controlled by only structural condition of road.

For example, when traffic density is low, ordinary drivers can drive vehicles safely and comfortably at least at the speed of 80 km/h on the road with design speed of 80 km/h. Since the values of elements of geometric design were decided with enough margin for safety, the average motorists can safely drive their vehicles at speeds higher than the design speed, if other conditions are not unfavorable.

2) Value of design speed

The maximum value for the design speed of ordinary roads is decided to be 80 km/h on the assumption of no control of access. However, for the road with a design speed exceeding 80 Km/h, access control should be taken into consideration. As for design speed of not more than 60 Km/h, elements of geometric design give so great effects on actual running speed of vehicles that it is important to select the suitable design speed, considering the design section and roads function.

In practice, major roads of BMA have been designed with design speeds of 50 - 100 km/h.

In AASHTO Standards, the design speed of urban arterials generally ranges from 40 to 60 mph (64.4 - 96.6 km/h) and occasionally may be as low as 30 mph (48.3 km/h). The lower (40 mph and below) speeds are applied in built-up areas or under particularly restricted conditions in suburban areas. A high design speed (50 mph or above) is appropriate in outlying sections approaching rural conditions.

In Japan, the design speed of urban roads is not more than 60 km/h except for urban expressways because there are limits imposed by land use and topography.

In other road agencies of Thailand, the values of design speed used are shown in Appendices 1.2.4 and 1.2.5.

The maximum design speed of 100 km/h is proposed based on the above-mentioned consideration, prevailing practice in BMA and by referring to the values of other agencies and other countries.

(2) Design section

1. The design section is a stretch of a road where the same design standard and same road classification are used.

2. The same design standard and road classification should be applied on a stretch of road where the role and importance of the road, traffic volume, topography along the road and social condition can be regarded as identical.

3. The design section must be long enough for the vehicle to run safely and comfortably.

It is not desirable to change the design section in a short distance and at the site where drivers cannot expect it. If design speed is changed at an inappropriate location, drivers are confused, and comfort of drivers is spoiled. It is suitable to continue the geometric design of the route as long as possible. In urban areas, it is desirable not to change the design section at least between main intersections.

It is not desirable to connect the different design sections in the direction of through traffic, where the difference of design speed is not less than 20 km/h, because the geometric design of a road is changed greatly by design speed. It is a general rule that the point dividing design sections is set at such a location as where drivers are led to notice the change of design standard naturally. Such points can be found at points where such factors as traffic volume, topography and social condition change.

1.2.3 Design Vehicle

1. Major Trunk Roads and Major Roads should be designed for semi-trailer and small-sized motor vehicles. Whereas, other roads should be designed for large-sized motor vehicles and small- sized motor vehicles.

2. The dimensions of design vehicles are shown in Table 1.2.4 and Figure 1.2.2.

Table 1.2.4 Design Vehicle

Type of vehicles	Length	Width	Height	Front over-hang	Wheel base	Rear over-hang	Minimum turning radius
Small-sized motor vehicle	4.7	1.7	2.0	0.8	2.7	1.2	6
Large-sized motor vehicle	12.0	2.5	3.8	1.5	6.5	4.0	12
Semi-trailer	16.5	2.5	3.8	1.3	front 4 rear 9	2.2	12

Source : Japan - the Road Structure Ordinance
(enacted in 1970 according to the Road Law)

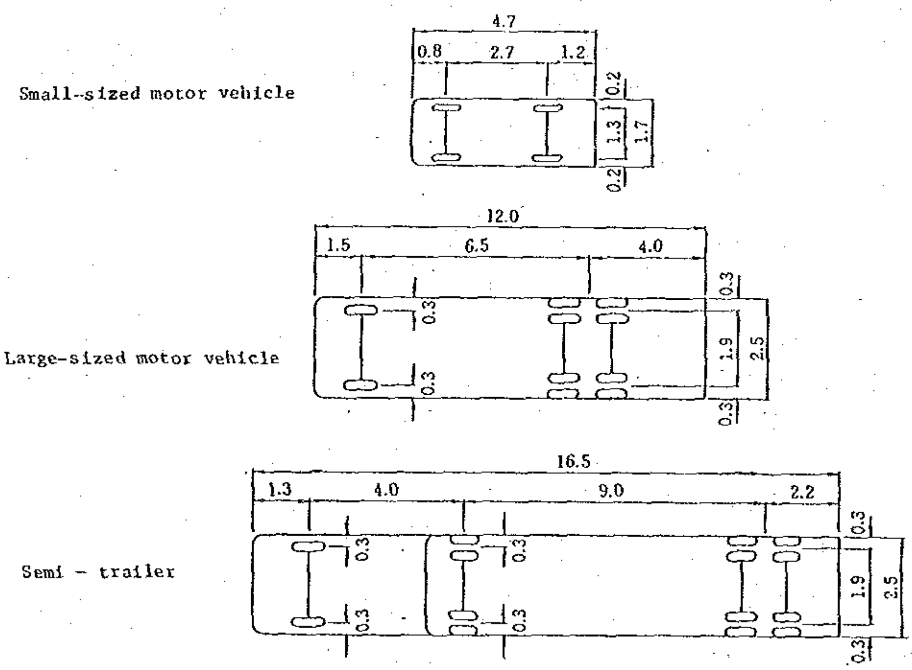


Figure 1.2.2 Design Vehicle

The physical characteristics of vehicles and the proportions of variously sized vehicles using the roads are positive controls in geometric design. Therefore, it is necessary to examine all vehicle types, select general class groupings, and establish representatively sized vehicles within each class for design use. Design vehicles are selected as the motor vehicles with the weight, dimensions, and operating characteristics suitable to establish road design controls for accommodating vehicles of designated classes. For the purposes of geometric design, each design vehicle has larger physical dimensions and larger minimum turning radii than these of almost all vehicles in its class.

In Thailand, the size of vehicles are prescribed by the Ministerial Regulation No. 5 (1981), issued under the Automobiles Act 1979. In the Ministerial Regulation, the maximum sizes of vehicles are 2.5 m in width and 12 m in length, except a tractor's width may be 3.0 m. The sizes of other types of vehicles are also prescribed in the Regulation. However, detailed dimensions of vehicles, such as wheelbase, are not prescribed.

The majority of the vehicles in operation in Thailand is made either in Japan or in Europe and the number of American cars are limited. Compared to European and Japanese cars, American cars' dimensions are larger. Dimensions of the bus used in AASHTO Standards (width: 2.6 m, length: 12.2 m) slightly exceed the values in the Ministerial Regulation No. 5.

Hence using vehicles dimensions similar to that used in Japan seems to be appropriate.

Size and performance of vehicle have a great effect on width of lane, widening of lane width at curve, design of intersection, grade, and sight distance etc.

In the Road Structure Ordinance of Japan, the following three types are used as design vehicles; Small-sized motor vehicle (passenger car), Large-sized motor vehicle (truck and bus), and Semi-trailer (trailer of four axles).

The criteria for width of lane and sight distance etc. are decided on the basis of the Small-sized motor vehicle(passenger car), and width of lane, wedening of lane width at curve, design of intersection, and grade, etc. are decided by Large-sized motor vehicle (truck and bus) and Semi-trailer (trailer of four axis).
The design vehicle dimensions and minimum turning radii of design vehicles as per the AASHTO Standards are shown in Appendices 1.2.6 and 1.2.7.

1.2.4 Cross Section

(1) General

When the cross section of a road is designed, the following should be taken into consideration.

- Adopt the cross section in accordance with the function of the planned road. The roads of higher design speed and more heavy design traffic volume need a cross section of higher standard.
- Provide enough traffic capacity to cope with the traffic demand the level of service in the target year.
- Examine the design from viewpoints of both safety and comfort.
- Separate the bicycles and pedestrians from the motor vehicles when necessary.
- Examine in correlation such factors as the means of access control, the capacity, and the means of traffic management at ramp terminals and/or intersections.
- Preserve the good living environment. The actual condition of plans for land use should be taken into account.
- Standardize the cross section of road. Minimize the effort required for maintenance and administration of roads. Secure the good landscape.

(2) Elements of a cross section and their combination

1. The elements of cross section are as follows.

   a) Carriageway
   b) Median
   c) Shoulder
   d) Stopping Lane
   e) Cycle Track
   f) Sidewalk
   g) Green Belt
   h) Frontage Road

2. Standard widths of the cross section elements are shown in Table 1.2.5. The values of the median, shoulder, sidewalk and cycle track shown in Table 1.2.5, are minimum widths, and the value for the greenbelt is a standard width.

Table 1.2.5 Cross Section Elements

Class	Lane	Median		Shoulder*	Sidewalk	Cycle track	Green belt	Stopping lane
		Median	Marginal strip					
1. Major Trunk Roads	3.5	over 1.5(1.0)	0.30(0.25)	over 0.75	over 5.00(3.50)	over [2.00(1.00)]	over [1.50]	[2.50(2.00)]
2. Major Roads	3.25	1.5(1.0)	0.30(0.25)	0.50	5.00(3.50)	[2.00(1.00)]	[1.50]	[2.50(1.75)]
3. Minor Roads	3.00	[1.5(1.0)]	[0.30(0.25)]	0.50	2.75(2.0)	[2.00(1.00)]	[1.50]	[2.00(1.75)]
4. Access Roads	4.00 or 3.00 roadway			0.50	1.50(1.0)	[2.00(1.00)]	[1.50]	[2.00(1.50)]

Note : [ ] : Only when necessary
( ) : Applied in the case of bridge, etc.
* : Desirable value is 2.0 m

When the design speed of Major Roads in a suburban or rural area is 80 Km/h, the value for each cross section element should be that of Major Trunk Roads.

An example of cross section elements and their combination is shown in Figure 1.2.3. Items of element are defined as follows.

a) Carriageway
The portion of the road which carries vehicles.

b) Median
The portion of a divided road separating the traveled way for traffic in opposing directions.

c) Shoulder
The portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles, for emergency use and for lateral support of subbase, base, and surface courses.

d) Stopping lane
The long and narrow portion of the carriageway for vehicles stopping or parking.

e) Cycle track
The portion of the road reserved for bicycles.

f) Sidewalk
The portion of the road reserved for pedestrians.

g) Greenbelt
The portion of the road for planting trees and shrubs.

h) Frontage road
The roads which segregate local traffic from the higher speed through traffic and intercept driveways of residences and commercial establishments along the highway.

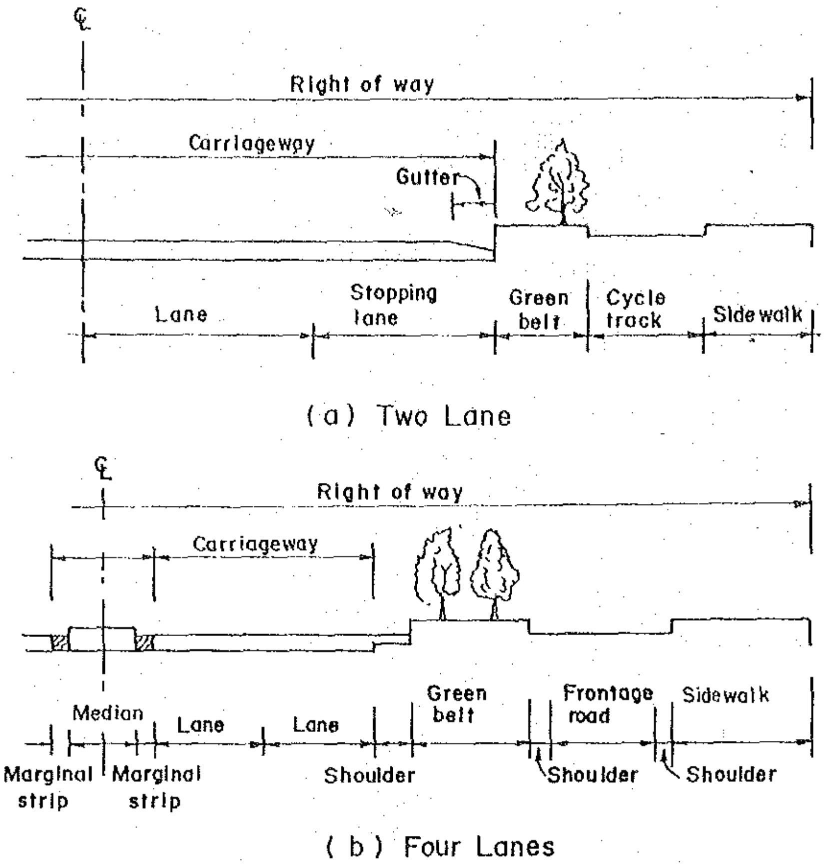


Figure 1.2.3 Example of Cross Section Elements and Their Combination

(3) Lane width

1. Lane widths should be the value shown in Table 1.2.5. They should be selected according to the classification of the road.
2. When the design speed is 80 Km/h in a suburban or rural area and/or high percentage of large-size vehicles is expected on the Major Roads (class 2), 0.25 m should be added to the value shown in Table 1.2.5, i.e. the lane width is 3.5 m.

Lane widths must have enough room for keeping lateral clearance of vehicles passing each other and/or overtaking other vehicles. It is known fact, however, that vehicles more than the number of lanes may run side by side, when an extremely wide lane width is used. Also, if lane widths are too wide, motorists feel some difficulty to drive their cars, resulting in increased possibility of accident. Therefore, excessively wide lane width is not desirable.

AASHTO Standard states the following.
"Although lane widths of 12 ft (3.66 m) are desirable on both rural and urban facilities, there are circumstance that necessitate the use of lanes less than 12 ft wide. In urban area where right-of-way and existing development become stringent controls, the use of 11 ft (3.35 m) is acceptable. Ten-ft (3.05 m) lanes are acceptable on low-speed facilities."

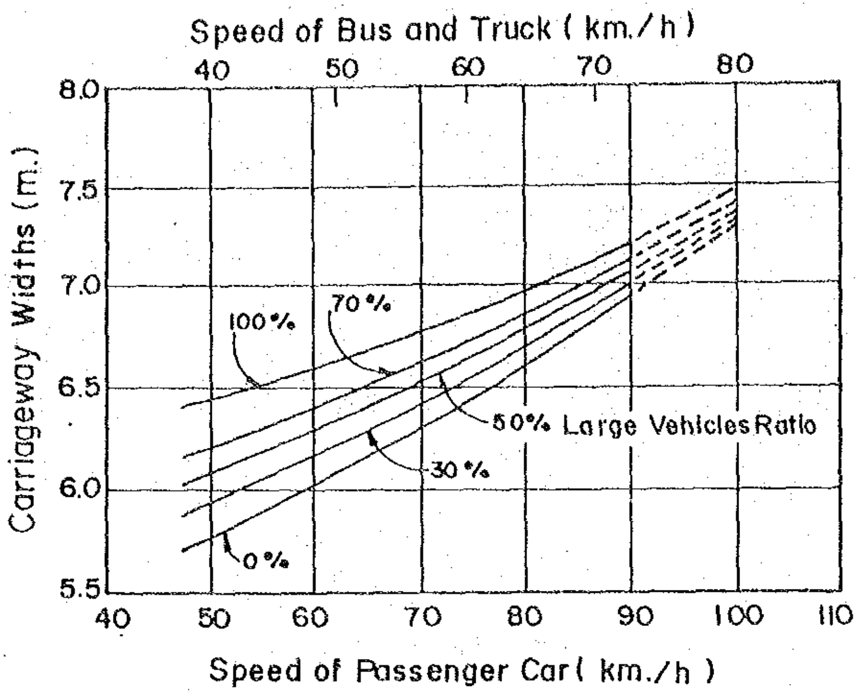
Figure 1.2.4 shows the relation of the running speed and carriageway widths which was obtained from the experiment. The result of the experiment done in Japan shows that there is a clear relation between the running speed and the carriageway widths. It is reasonable that the carriageway width of two-lane road is changed according to the running speed. For example, as for passenger cars, approximately 6.0 m is required for running speed of 40 Km/h (lane width 3.0 m), and from 7.0 to 7.5 m for 100 Km/h (lane width 3.5 m).

Lane widths, among the cross section elements, have an effect over the widest range of aspects, such as the running speed and comfort. Hence, it is reasonable to determine the lane width according to the design speed and the traffic volume. The standard lane widths are shown in Table 1.2.6 in relation to the design speed.

The carriageway width for Access Roads is proposed to be 4.0 m in consideration of passenger cars' passing each other, and fire fighting. When the design traffic volume is low (approximately less than 100 veh/day) and an unfavorable topographical condition exists, the width may be reduced to 3.0 m (the minimum total road width, 4.0 m, including shoulders)

Table 1.2.6 Standard Lane Width

Design Speed (km/h)	Standard Lane Width (m)	Appropriate Classification
80 over	3.50	Major Trunk Roads
60	3.25	Major Roads
60, 50 and 40	3.00	Minor Roads



Source : Explanation and Application of the Road Structure Ordinance (Japan Road Association) 1983.

Figure 1.2.4 Carriageway which of Two-lane Road Obtained from the Experiment

(4) Median

1. Median is a highly desirable element on all major roads having four or more lanes.

2. Width of median should be more than the value shown in Table 1.2.5. However, in unfavorable place, such as bridges, viaducts (including flyovers) and underpasses, the widths may be reduced to the values in parentheses.

3. Width of the marginal strip should be 0.30 m. In an unfavorable place, such as bridges and viaducts, the width may be reduced to 0.25 m.

In other countries, such as U.S.A. and Japan, width of median is expressed by the distance between the edges of through lanes in each direction and, thus, includes the right marginal strips, if they are to be provided. In other words, the median is composed of a medial separator (raised portion) and marginal strip.

However, "medial separator" (raised portion) has been called "median" in prevailing practices of the BMA. In this guideline, the raised portion is defined as median. When the literature of other countries is used, attention should be paid to the above-mentioned circumstances.

Median barriers should be set on the median, or the surface of median should be raised to secure directional separation. Marginal strips should be set on both sides of the median.

The functions of the median are as follows.
- Provide the desired freedom from the interference of opposing traffic.
- Provide for speed changes and storage of right- turning and U-turning vehicles.
- Minimize headlight glare.
- Provide width for future lanes.

As mentioned above, in many countries width of the median includes marginal strips on both sides of median. In this case, the following functions can be expected.
- Provide a recovery area for out-of-control vehicles.
- Provide a stopping area in case of emergencies.
- Be highly visible both night and day and in definite contrast to the through-traffic lanes.

The functions of the marginal strip are as follows
- Secure lateral clearance
- Clearly show the edge of carriageway and facilitate visual guidance.

Widths of each components of median are shown in Figure 1.2.5. Widths of each component on urban road are shown in Table 1.2.7.

Table 1.2.7 Width of Median

median	1.50 (1.00) (minimum)
marginal strip	0.30 (0.25)
c *	0.50 (0.25)
lateral clearance	0.80 (0.50)
appurtenances of road	0.50 (minimum)

( ) : applied in the case of bridge etc.

* see Figure 1.2.5.

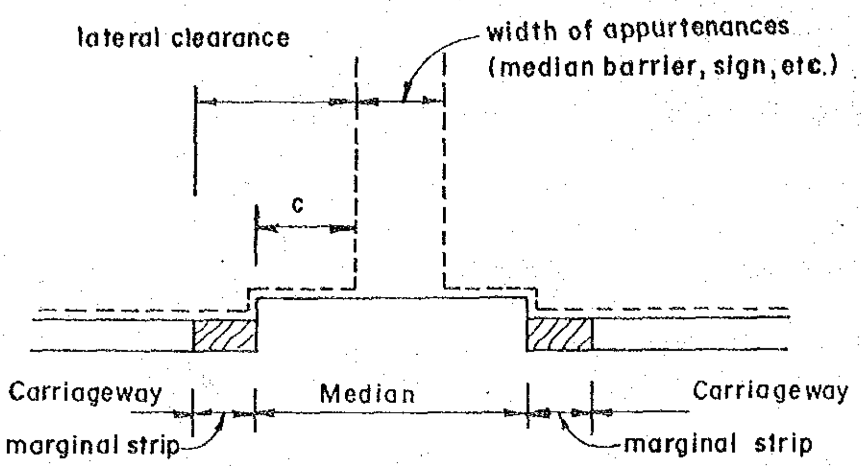


Figure 1.2.5 The Relation between Median Width, Lateral Clearance and Width of Appurtenances

(5) Shoulders

1. Shoulders should be provided beside the carriageway. If a median or stopping lane is provided, shoulders may be omitted.

2. Widths of the shoulders provided on the left side of the carriageway should not be less than the values shown in Table 1.2.5.

3. Shoulders provided on the right side of the carriageway should not be less than 0.5 m.

4. Shoulder width of 2.0 m is desirable.

5. Shoulders may be omitted or reduced for roads having sidewalks or equivalents thereof, when the main structure of the road is well supported and the function of road is kept up.

6. When the appurtenances, such as roadside barrier, are to be provided, the necessary width for setting the appurtenances should be added to the width of shoulders shown in Table 1.2.5.

7. The shoulder of a bridge, viaduct or underpass without sidewalk space, can be combined with the curb, taking into consideration economical efficiency.

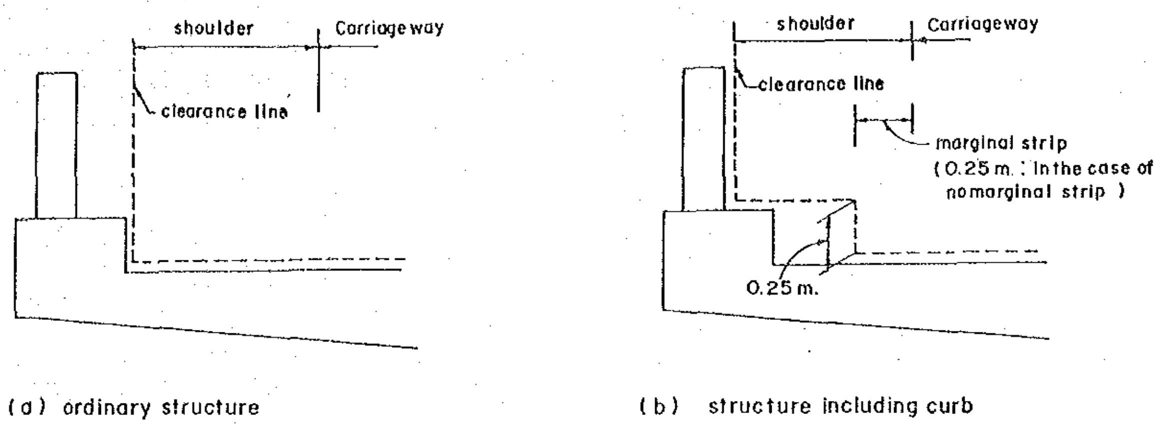


Figure 1.2.6 Structure of Shoulder for Bridge and Viaduct

The functions of shoulders are as follows.

- Structural support is given to the pavement.
- Space is provided for stopping free of the traffic lane when necessitated by mechanical difficulty, a flat tire, or other emergency.
- The sense of openness created by shoulders of adequate width contributes much to driving ease and freedom from strain.
- Space is provided to escape from potential accidents or reduce their severity.
- Lateral clearance is provided for signs and guardfences.
- Space is provided for maintenance operations and buried facilities.
- Sight distance is improved in cut sections, thereby improving safety.
- Storm water can be discharged farther from the pavement, and seepage adjacent to the pavement can be minimized. This may directly reduce pavement breakup.
- Some types of shoulders enhance the aesthetics of the road.
- Space is provided for pedestrian and bicycle use on a road having no sidewalk.

The right side shoulders are defined as the shoulders provided on the right side in the direction of traffic, on the road separated by methods other than a median. Width of the right side shoulder is determined to secure the necessary lateral clearance of median.

When a sidewalk is provided beside the carriageway, gutters are necessary for surface drainage between the carriageway and the sidewalk. Gutters should be stable enough to support occasional vehicle loads. They should be provided in the shoulder.

(6) Stopping lanes

1. Stopping lanes should be provided on the left side of the carriageway, if necessary, in order that the smooth flow of traffic is not disturbed by stopping of vehicles.

2. Widths of stopping lanes should be the values shown in Table 1.2.5. They should be selected according to the classification of road. However, when the percentage of large vehicles using the road is acceptably low, the widths may be reduced to the values in parentheses.

On urban roads, there is a great demand for stopping and parking. Their objective is stopping for land access. On urban roads of two-lanes, through-traffic is disturbed greatly by stopping, so providing stopping lanes is very desirable.

Maximum standard width of 2.5 m is determined in consideration of stopping of large sized vehicles. The minimum reduced value of 1.5 m is for passenger cars. Structure of stopping lanes is a identical plane with road surface as shown in Figure 1.2.7. Gutters may be provided in stopping lanes.

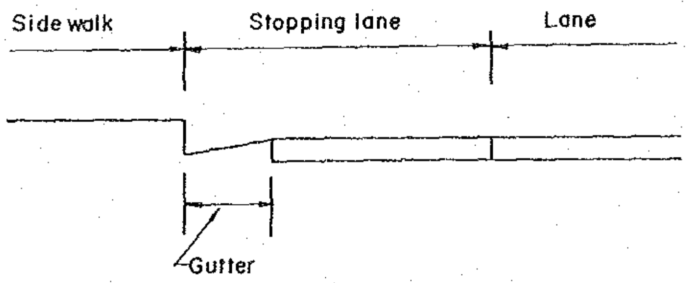


Figure 1.2.7 Structure of Stopping Lane

It is natural that stopping lanes are used for stopping for short times. However several other practical uses are seen, for example, motorcycles and/or bicycles can use this lane, if long-time parking is not allowed on this lane. In the neighborhood of an intersection, it is not desirable to allow stopping and parking. In such cases, stopping lanes can be converted to an additional traffic lane at entrance and exit of intersection.

(7) Bus lane and bus bay

1) Bus lane

1. The width of an exclusive bus lane should be at least 4.0 m, whenever it is provided. In unavoidable cases, the width of bus lane including shoulder should be 4.0 m.

When the width of a bus lane is 4.0 m, the width of shoulder may be reduced to 0.30 m, i.e., the total width of bus lane and shoulder is 4.30 m. When the width of the bus lane including the shoulder is 4.0 m, the width of the shoulder can be omitted. In this case, lateral clearance of 0.5 m on the sidewalk must be secured as shown in Figure 1.2.12. The width was determined in consideration of width of bus, lateral clearance as well as prevailing practices in Bangkok.

2) Bus bay

1. A bus bay should be provided, if necessary, when the stopping of a bus would disturb the through traffic flow, and/or the traffic capacity of route is not enough for design traffic capacity.

The length of the bus bay is determined referring to Table 1.2.8 in consideration of through traffic volume, bus users and land-use pattern.

Table 1.2.8 Lengths of Bus Bay

Design Speed (km/h)	80	60	50	40
Deceleration Lane Length (m) ℓ_1	30	20	15	12
Bus Stopping Lane Length (m) ℓ_2	15	15	15	15
Acceleration Lane Length (m) ℓ_3	35	25	20	13
Bus Bay Length (m) ℓ	80	60	50	40
Weaving Length (m)	80	50	40	30

When a bus stop is located near an intersection, the bus stop should be kept away more than the distance of necessary weaving length.

An example of a bus bay is shown in Figure 1.2.8. The width of the speed change lane and stopping lane should be 4.0 m, as a general rule. Even in a very unfavorable situation,the width of the bus lane including shoulder must be at minimum 4.0 m.

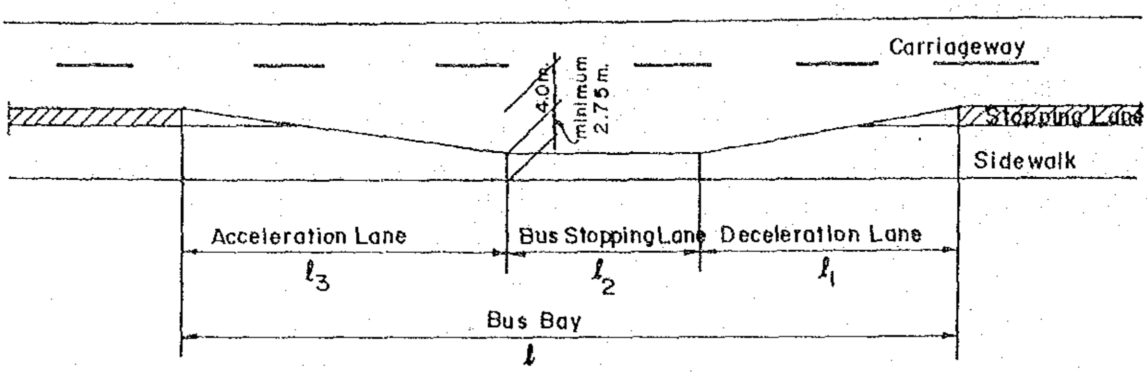


Figure 1.2.8 Elements of Bus Bay

The sidewalk width along the bus stop should have 2.75 m as a minimum value.

In Japan, the width of the speed change lane and stopping lane is 3.5 m, and reduced value is 3.0 m.

(8) Sidewalk

> 1. Roads except Access Roads should be provided with a sidewalk on the both sides of carriageway.
>
> 2. Widths of sidewalk should not be less than the values shown in Table 1.2.5. However, on bridges, viaducts, and/or where pedestrian traffic is low, the widths may be reduced to the values in the parentheses.

Assuming the width needed for movement by a pedestrian as 0.6 m, the unit width of a row of pedestrians (that may be called a "lane") shall be 0.75 m including marginal spaces (refer to Figure 1.2.9 as for occupied width). Width of sidewalk is to be determined by number of pedestrians as well as their moving patterns as illustrated in Figure 1.2.10. The minimum width of sidewalk is determined by adding a lateral clearance of 0.5 m to the required width for pedestrians multiplied by the number of pedestrian lanes. This clearance can be used as the space for road appurtenances where required width of shoulder is secured between sidewalk and carriageway.

In sub-urban areas, a sidewalk may be constructed at sections where vehicle traffic on outer lanes of both directions per day is 3,000 or more and pedestrian traffic is 250 or more. The section 2.5 "Sidewalk" provides the warrants in detail.

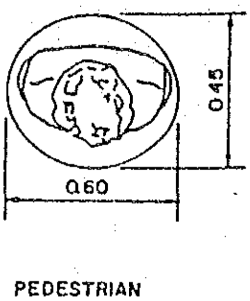


Figure 1.2.9 Dimension of Pedestrian

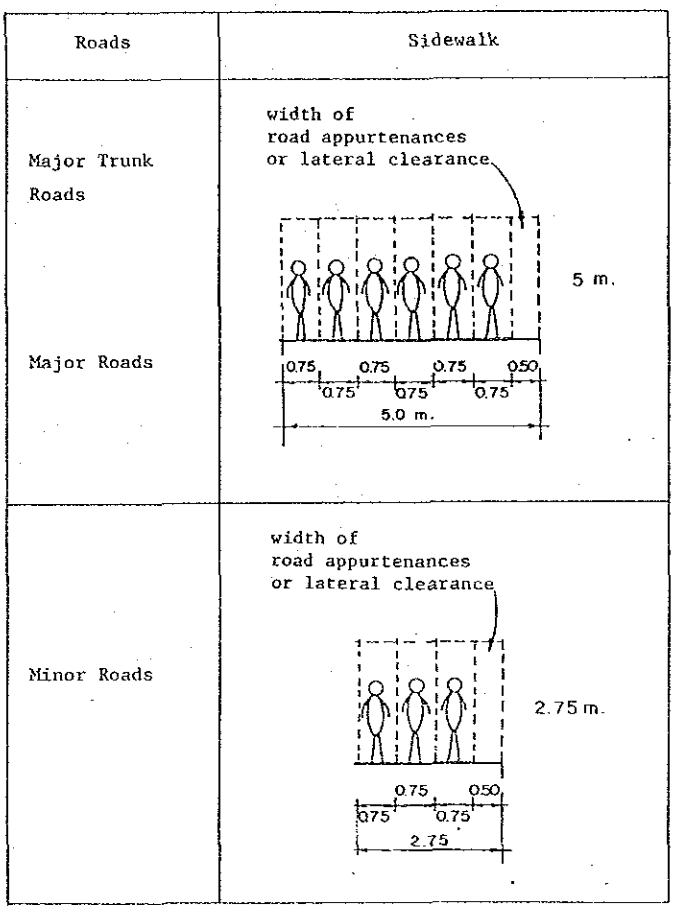


Figure 1.2.10 Minimum Width of Sidewalk

(9) <u>Standard cross section</u>

1. When widths and the combination of elements of road cross section are designed, they should be determined in consideration of the standard width. However, in unavoidable cases, such as where the conditions of the area and the traffic are unfavorable, they may not be done based on standard width.

Examples of actual combinations of cross section elements are shown in Figure 1.2.11.

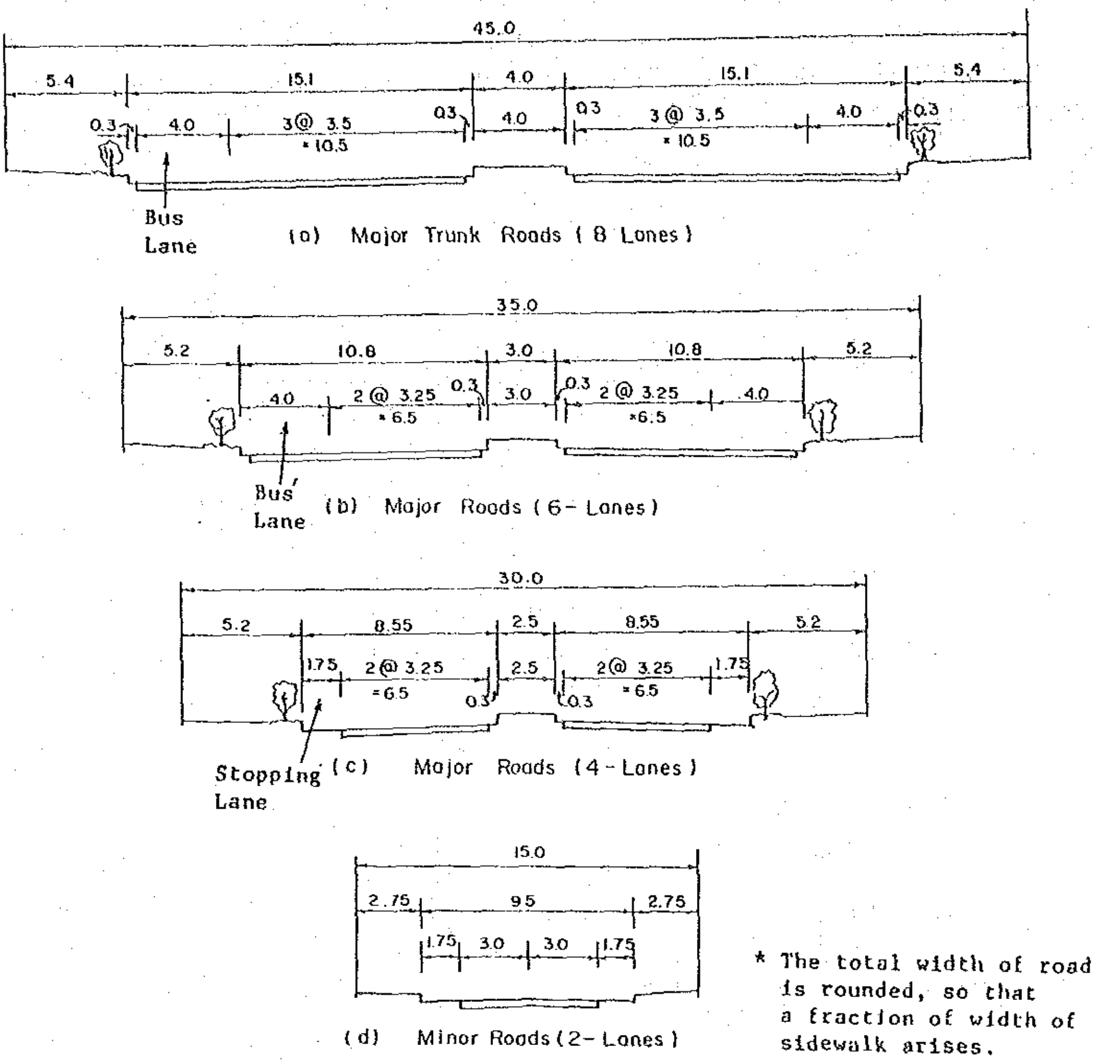


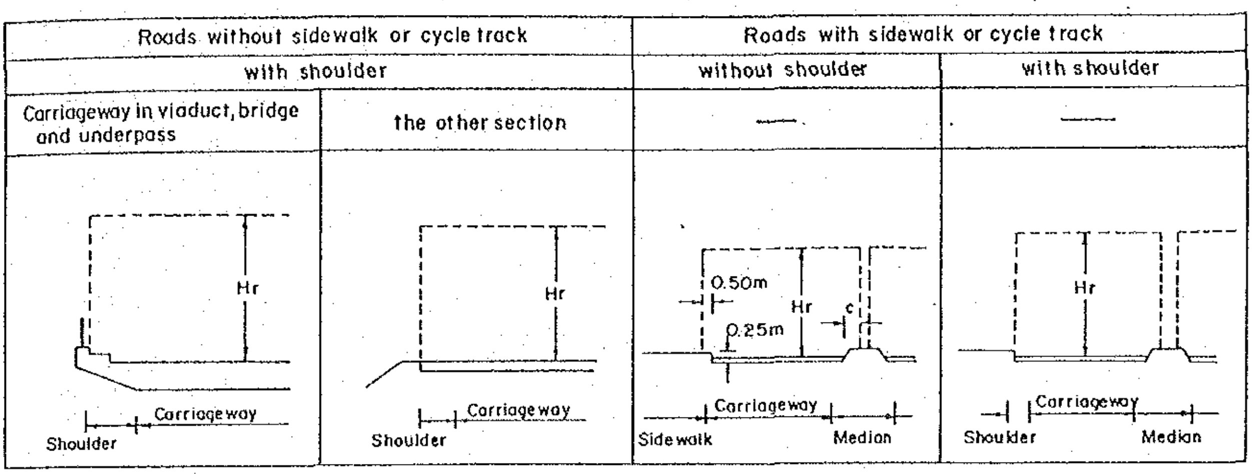
Figure 1.2.11 Standard Cross Section

(10) Clearance

1. Clearances on roads are shown in Figure 1.2.12 for the carriageway and in Figure 1.2.13 for sidewalk.

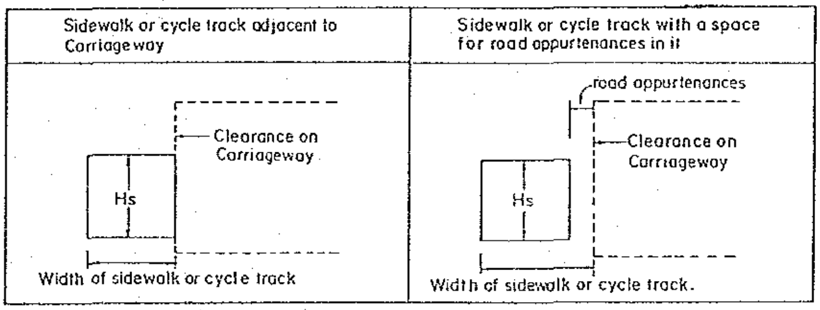
2. Vertical clearance on carriageway is 5.0 m. In an unavoidable case, such as for a topographical reason, the value may be reduced to 4.5 m.

3. Vertical clearance on sidewalk is 2.5 m. However, in unavoidable cases, the value may be reduced to 2.4 m.



Hr : 5.0m. In case it is necessary because of the topographical condition or for any other reasons, the value can be reduced to 4.5 m.

Figure 1.2.12 Clearance on Carriageway



Hs : 2.5 In unavoidable case the value can be reduced to 2.4 m

Figure 1.2.13 Clearance on Sidewalk

AASHTO Standards, contain the following: "Vertical clearance on State trunk highways and interstate systems in rural areas shall be at least 16 ft (4.88 m) over the entire roadway width with an allowance for resurfacing. On State trunk highways and interstate routes through urban areas, a 16 ft clearance shall be provided except in highly developed areas. A 16 ft clearance should be provided in both rural and urban areas where such clearance is not unreasonably costly and where needed for defense requirements. On all other highways, vertical clearance shall be at least 14 ft (4.27 m) over the entire roadway width with an allowance for resurfacing."

In Japan, vertical clearance is 4.5 m, which is determined by adding the allowance to the height of design vehicles of 3.8 m. When an overlay of pavement is expected in the future, vertical clearance of 4.7 m is recommended.

In this Guideline, the value of vertical clearance is proposed to be 5.0 m, in consideration of prevailing engineering practices in Bangkok. The reduced value of 4.5 m is proposed referring to the Japanese standard. The width of clearance is sum of the widths of carriageway and shoulder. When the traffic volume of large-sized vehicles is extremely low and there is another road where large-sized vehicles can make a detour near the road concerned, the value may be reduced to a minimum of 4.0m.

Details of clearance at the edge of shoulder are determined as shown in the following.

1) Road with shoulder

Roads should be provided with shoulders on both sides. If road appurtenances, such as traffic signs are set along the shoulder, the necessary width for setting the appurtenances should be added to the width of shoulder. Therefore, they should be set on the outside of clearance, although the appurtenances can be provided in the shoulder.

2) Road without shoulder

In cases where stopping lanes are provided and shoulders are omitted, clearance shown in Figure 1.2.11 should be adopted. On a road having no shoulder, lateral clearance of 0.50 m is secured on the outside of carriageway as shown in Figure 1.2.12.

3) Road with median and traffic island

Explanation of details of the clearance line at the median and traffic island is shown in Figure 1.2.12.

1.2.5 Alignment and Sight Distance

(1) General

1) General items of design of alignment

Road alignment plays an important role in insuring safe and smooth traffic flow. Sharp curves and improperly designed connecting curves are likely to cause accidents. Also, they reduce traffic capacity. They do not harmonize with the visual beauty and the roadside view. Furthermore, they may result in unreasonable increase of construction cost.

The following are taken into consideration in order to avoid these situations.

- harmony with topography and land-use.
- continuity of alignments.
- harmony of horizontal and vertical alignments, and configuration of cross section elements.
- visual examination of alignment.
- safety and comfort of traffic.
- restricted condition of geology and topography.
- economical efficiency of construction, maintenance and administration cost.

These items are related to each other. It is necessary to examine horizontal and vertical alignments in combination in order to plan better road alignment so as to the above requirements. Therefore horizontal and vertical alignments should not be designed individually.

2) Design of alignment of roads in urban area

In urban areas, when the route location and/or the geometric design of major roads are carried out, the following should be taken into consideration in addition to the above-mentioned items.

- Take roadside land-use into consideration.
  When the geometric design of urban major roads is carried out, it is necessary to consider the zone of daily life and any school district through which the route passes. When a major road is constructed dividing an existing residential zone, the following may occur.
  * convenience of the inhabitants' life is spoiled
  * traffic safety can not be fully secured
  * essential function of major roads may not be fully attained.

- Take the relation with the existing road network into consideration, and select the alignment which is neither a multileg intersection, nor a deformed intersection. When the intersection is a multileg or deformed intersection, the existing cross road has to be removed or improved.

- Avoid planning at-grade intersections close to the approach of a grade-separated intersection, railway crossing, or urban expressway ramp, from the viewpoint of securing safety and traffic capacity.

(2) Combination of horizontal and vertical alignments

1) General consideration

Horizontal and vertical alignments are permanent design elements for which thorough study is warranted. It is extremely difficult and costly to correct alignment deficiencies after the highway is constructed. On freeways there are numerous controls such as multilevel structures and costly right-of-way. On most arterial streets heavy development takes place along the property lines, which makes it impractical to change the alignment in the future. Thus, compromises in the alignment designs must be weighed carefully, because any initial savings may be more than offset by the economic loss to the public in the form of accidents and delays.

Horizontal and vertical alignments should not be designed independently. They complement each other, and poorly designed combinations can spoil the good points and aggravate the deficiencies of each. Horizontal alignment and profile are among the more important of the permanent design elements of the highway, for which thorough study is warranted. Excellence in their design and in the design of their combination increase usefulness and safety, encourage uniform speed, and improve appearance, almost always without additional cost.

(3) Radius of horizontal curvature

1) Minimum radius of horizontal curvature

1. Radii of curvature of the centerline of the carriageway should not be less than the values shown in the left column of Table 1.2.9, according to the design speed. However, in an unavoidable case, such as caused by a topographical reason, radii of curvature may be reduced to the values shown in the right column of Table 1.2.9.

Table 1.2.9 Minimum Radius of Horizontal Curvature

Design Speed (Km/h)	Radius of Curvature	
100	460	380
80	280	230
60	150	120
50	100	80
40	60	50
30	30	
20	15	

For balance in highway design all geometric elements should, as far as economically feasible, be determined to provide safe, continuous operation at a speed likely under the general conditions for that highway or street. For the most part this is done through the use of design speed as the overall control. In the design of highway curves it is necessary to establish the proper relation between design speed and curvature and also their joint relations with superelevation and side friction. Although these relations stem from the laws of mechanics, the actual values for use in design depend on practical limits and factors determined more or less empirically over the range of variables involved. These limits and factors are explained below with determination of logical controls for highway curve design.

a) Computation of radius of curvature

When a vehicle moves in a circular path, it is forced radially outward by centrifugal force. The centrifugal force is counterbalanced by the vehicle weight component related to the roadway superelevation or the side friction developed between tires and road surface or by a combination of the two.

- Z = centrifugal force (kg)
- v = vehicle speed (m/sec)
- g = acceleration of gravity (= 9.81 m/sec^2)
- G = weight of vehicle (kg)
- f = side friction factor
- e = rate of roadway superelevation, (tan α ,m/m)
- R = radius of curvature (m)

According to the relation between radius of curvature and vehicle speed, centrifugal force can be calculated as follows.

$$Z = G/g \times v^2/R \qquad .....(1)$$

i) Condition of no skid

In Figure 1.2.14, a center of curvature is in the opposite direction of centrifugal force (Z). The superelevation angle isositive in the case of Figure 1.2.14, and negative in the case of the reverse to Figure 1.2.14.

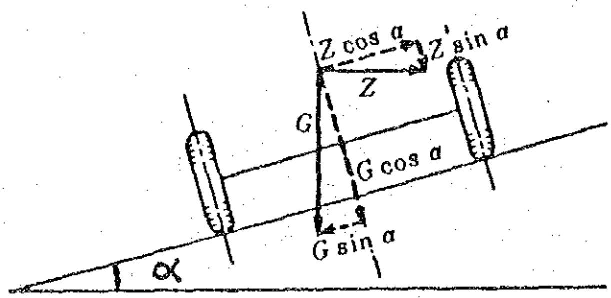


Figure 1.2.14 Geometry for Running on the Curve

In order to prevent a vehicle from skidding, the conditions in the following formula must be met.

$$Z \cos \alpha - G\sin \alpha \leqq f(Z \sin \alpha + G \cos\alpha) \quad ...(2)$$

Dividing cos α into Equation (2), and substituting $Z = G/g \times v^2/R$, tan α = e into Equation (2), we get

$$\frac{v^2}{gR} - e \leqq f\left(\frac{v^2}{gR} e + 1\right) \qquad .....(3)$$

$$v^2 \leqq gR \frac{e + f}{i - ef} \quad \text{or} \quad R \geqq \frac{v^2}{g} \cdot \frac{1 - ef}{e + f} \qquad .....(4)$$

The value of the product ef in the formula (4) is always small. As a result, the (l-ef) term is normally omitted in highway and street designs, thus providing slightly more conservative values.

$$v^2 \leqq gR\ (e + f) \quad \text{or} \quad R \geqq \frac{v^2}{g} \cdot \frac{1}{(e + f)} \qquad .....(5)$$

thus, using V as design speed (Km/h), we obtain

$$V^2 / \frac{v^2}{g} = 3.6^2 \times 9.81 = 127$$

$$V^2 \leqq 127\ R\ (e+f) \quad \text{or} \quad R \geqq \frac{V^2}{127\ (e+f)} \qquad \text{.....(6)}$$

The minimum safe radius, Rmin, can be calculated directly from the following simplified curve formula

$$Rmin = \frac{V^2}{127\ (e+f)} \qquad \text{.....(7)}$$

The term (e + f) in the equation (7) has a direct effect on safety and comfort when a vehicle is running on the curve section. The term (e + f) is a important factor to determine the value of minimum radius Rmin, and has to be given much consideration.

In AASHTO standards, the minimum safe radius Rmin is determined as follows.

$$Rmin = \frac{V^2}{15\ (e+f)} \qquad \text{.... (8)}$$

where Rmin = minimum radius of curvature (ft)
V = vehicle speed (mpH)
f = side friction factor
e = rate of roadway superelevation

The Equation (8) is equivalent to the equation (7) in the Japanese Standard except for a difference in units ('meter' and 'feet').

b) Superelevation and side friction factor

i) General Considerations

From accumulated research and experience, limiting values have been established for e and f. Using the e max value with a conservative f value in the basic formula permits determination of minimum curve radii for various design speeds. For a given design speed, use of curves with radii longer than the minimum calls for balance in the factors involved to determine the desirable superelevation rates below the maximum.

ii) Superelevation

As mentioned above, if values of superelevation and side friction factor are determined according to a certain design speed, the minimum radius of curvature can be calculated. When the maximum value of superelevation is determined, the following factors are taken into account as the conditions.

- comfort of running
- separation of bicycle traffic
- weather conditions
- area - urban or rural
- roadside conditions

It is clear that the maximum superelevation rate cannot be determined uniformly for all roads of different classes.

The maximum values of superelevation, taking several conditions into consideration, are mentioned below. The minimum values of radius of curvature are calculated by using these values of superelevation.

iii) Side Friction Factor

The coefficient of friction, f, is the friction force divided by the weight perpendicular to the pavement and is expressed as the following simplified curve formula:

$$f = \frac{V^2}{127\ R} - e$$

The upper limit of this factor is that at which the tire is skidding or at the point of impending skid. Because highway curves are designed to avoid skidding conditions with a margin of safety, the f values should be substantially less than the coefficient of friction of the impending skid.

The side friction factor at which side skidding is imminent depends on a number of factors, among which the most important are the speed of the vehicle, the type and condition of the roadway surface, and the type and condition of the tires. Different observers have recorded different maximum rates at the same speeds for similar composition pavements, and logically so, because of the inherent differences in pavement texture, weather conditions, and tire condition.

Curves should not be designed directly on the basis of the maximum available side friction factor. The portion of the side friction factor that can be used with comfort and safety by the vast majority of drivers should be the maximum allowable value for design. Values that relate to pavements that are glazed, bleeding, or otherwise lacking in reasonable skid resistant properties should not control design, because these conditions are avoidable and geometric design should be based on acceptable surface conditions attainable at reasonable cost.

Figure 1.2.15 summarizes the findings of the cited tests relating to side friction factors recommended for curve design in AASHTO standards. Although some variation is noted, all are in agreement that the side friction factor should be lower for high-speed design than for low-speed design.

Figure 1.2.15 shows the recommended values of the side friction factor for all rural highways and high-speed urban streets as a solid line.

In Japanese standards, the values of the side friction factor are determined as shown in Table 1.2.10. The values take into account the study data and comfort. Figure 1.2.16 shows the f values as compared with AASHTO and Reichtlinien für die Anlage von Landstraßen (RAL, West Germany).

Table 1.2.10 gives the radii for each of the maximum superelevation rates for design speeds from 20 Km/h to 120 Km/h in Japan, and Table 1.2.11 in AASHTO.

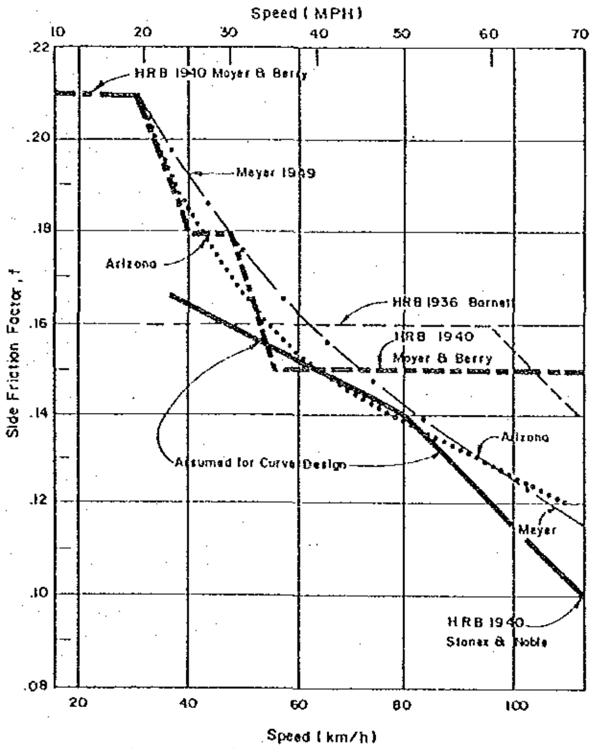


Figure 1.2.15 Side Friction Factors for Rural Highways and High-Speed Urban Streets

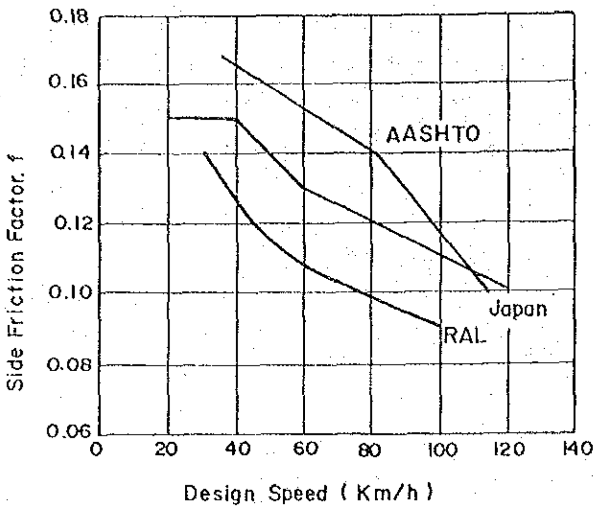


Figure 1.2.16 Comparison of Side Friction Factors and Design Speed in Various Standards

Table 1.2.10 Minimum Radius of Curvature (Japan) (the Values of Calculation)

Design speed Km/h (mph)	f	Minimum Radius (m)				
		(e = -2%)	e = 4%	e = 6%	e = 8%	e = 10%
120 (74.6)	0.10		810	709	630	567
100 (62.1)	0.11		525	463	414	375
80 (49.7)	0.12		315	280	252	229
60 (37.3)	0.13	(0.15) 218	167	149	135	123
50 (31.1)	0.14	(0.15) 151	109	98	89	82
40 (24.9)	0.15	97	66	60	55	50
30 (18.6)	0.15	55	37	34	31	28
20 (12.4)	0.15	24	17	15	14	13

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

(Japan)

R : Minimum Radius (m)
V : Design speed (km/h)
e : Superelevation
f : Side Friction Factor

Table 1.2.11 Minimum Radius of Curvature (AASHTO)
(the Values of Calculation)

Design Speed Km/h (mph)	f	Minimum Radius (m)				
		(e =-2%)	e = 4%	e = 6%	e = 8%	e = 10%
120 (74.6)	0.10		810	709	630	567
100 (62.1)	0.12		492	437	394	358
80 (49.7)	0.14		280	252	229	210
60 (37.3)	0.15	218	149	135	123	113
50 (31.1)	0.16	141	98	89	82	76
40 (24.9)	0.17	84	60	55	50	47
30 (18.6)	0.17	47	34	31	28	26
20 (12.4)	0.17	21	15	14	13	12

In case that a superelevation is not provided on the urban street due to reasons such as topography, the minimum radii are calculated using the standard cross slope (e = -2%). In this case, the f value of side friction factor, f=0.15, is adopted in all cases in Japanese Standards.

c) Prescribed values of the minimum radius of curvature

The values corresponding to the superelevation of 6% are adopted as the prescribed values of the minimum radii as shown in Table 1.2.9. The value of the superelevation of 6% was used taking into account traffic safety and comfort, with a certain margin. The values corresponding to the superelevation of 10% are adopted as the reduced values of the minimum radii as shown in Table 1.2.9.

On urban roads, in unavoidable cases, such as when constraints of topography and/or land use exist, the superelevation may not be provided, in such cases, the minimum radii may be the values shown in Table 1.2.12.

Table 1.2.12 Design Value of Minimum Radius of Curvature

Design Speed (km/h)	f	Minimum Radius Rounded Without Superelevation
60	0.13	220
50	0.14	150
40	0.15	100
30	0.15	55
20	0.15	25

2) Desirable value of minimum radius of curvature

1. The desirable values of minimum radii of curvature are values shown in Table 1.2.13 according to the design speed.

Table 1.2.13 Desirable Value of Minimum Radius of Curvature

Design Speed (km/h)	Desirable Value of Radius of Curvature (m)
100	700
80	400
60	200
50	150
40	100
30	65
20	30

In geometric design, designers tend to too easily use the radius of curvature near the prescribed minimum value, without paying attention to whether the geometric design has enough room topographically. Though the prescribed values of minimum radii of curvature guarantee safety and comfort when vehicle is running at the design speed, it is desirable to design the horizontal alignment which has sufficient leeway on condition that the harmony and good combination of the whole alignment is maintained. It is necessary for this purpose to provide the desirable value of minimum radius of curvature in consideration of the following;

- to guarantee comfort fully.
- be a useful value, that is, the value can be used on the whole route without great trouble.

Table 1.2.14 shows centrifugal acceleration, superelevation, and side friction factor for each design speed.

Table 1.2.14 Side Friction Factors for the Desirable Value of Minimum Radius of Curvature

Design Speed (km/h)	Radius of Curvature (m)	$\frac{V^2}{127 R}$	Superelevation e	Side Friction Factor f
100	700	0.11	0.06	0.05
80	400	0.13	0.07	0.06
60	200	0.14	0.08	0.06
50	150	0.13	0.08	0.05
40	100	0.13	0.07	0.06
30	65	0.11	0.06	0.05
20	30	0.11	0.06	0.05

3) Note for application of the minimum radius of curvature

The prescribed values of the minimum radii were determined to secure safety and comfort. These values are the necessary minimum values and are not the desirable design values, i.e., that those have a satisfactory margin for safety.

Therefore, it is appropriate to avoid the usage of the minimum radius and encourage usage of the desirable minimum radius.

(4) Length of curve

1) Minimum length of curve

1. The length of curve at the center line of a carriageway should not be less than two times the length of the transition (spiral) curve prescribed in the section (7), in the case that the intersecting angle is not less than 7°. When the intersecting angle is less than 7°, the length of curve should not be less than the left side value shown in Table 1.2.15. However, in an unavoidable case, such as when required by a topographical reason, the value may be reduced to the right side value regardless the intersecting angle.

Table 1.2.15 Minimum Length of Curve

Design speed (unit:km/h)	Curve length (unit: m)	
100	1,200/θ	170
80	1,000/θ	140
60	700/θ	100
50	600/θ	80
40	500/θ	70
30	350/θ	50
20	280/θ	40

θ = intersecting angle (°)

When the value of θ is less than 2° , the value of θ is taken at 2°.

(5) Superelevation

1. When the curve section of roadway is designed, the superelevation should be provided taking into consideration such as factors as design speed, radius of curvature, and terrain condition. The superelevation rates should not be more than the values in Table 1.2.16. Superelevation may be omitted on low speed urban streets subject to severe constraints.

Table 1.2.16 Maximum Superelevation Rate

	maximum superelevation
urban area	6%
others	10%

1) Maximum superelevation rate

For a given speed the maximum superelevation rate and the assumed value for maximum side friction factor in combination determine the maximum curvature. The maximum rates of superelevation are controlled by several factors; climate, terrain conditions, land use and frequency of slow-moving vehicles. Consideration of these factors jointly leads to the conclusion that no single maximum superelevation rate is universally applicable and that a range of values must be used.

Table 1.2.17 and Table 1.2.18 show the maximum values prescribed in the Japanese standards and AASHTO standards. Where traffic congestion or extensive marginal development acts to curb top speeds in an urban area, it is common practice to utilize a low maximum rate of superelevation, usually 0.040 to 0.060. Similarly, either a low maximum rate of superelevation or no superelevation is employed within important intersection areas or where there is a tendency to drive slowly because of turning and crossing movements, warning devices, and signals. In these areas it is difficult to warp crossing pavements for drainage without negative superelevation for some turning movements.

Table 1.2.17 Maximum Superelevation Rate in Japan

	Snow and Ice not Likely	Snow and Ice Slightly Likely	Snow and Ice very Likely
Type 1, Type 2 and Type 3 Road with cycle track, etc.	10	8	6
Other roads		6	

(JAPAN)

Table 1.2.18 Maximum Superelevation Rate in AASHTO

Maximum superrelevation	Application
12%	Exceptional value for low volume local roads
10%	Without snow and ice
8%	Regardless of snow or ice conditions
6%	Urban Area

2) Minimum radius of curvature for normal crown section

1. In case that the radius of curvature is more than the value shown in Table 1.2.19 according to the design speed and cross slope of straight section, superelevation may be omitted.

Table 1.2.19 Minimum Curvature for Normal Crown Section (Design Value)

(unit:m)

Cross Slope (%)	Design Speed (km/h)						
	100	80	60	50	40	30	20
2.0	5,000	3,500	2,000	1,300	800	500	200
1.5	4,000	2,500	1,500	1,000	600	350	150

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

where Re = Minimum radius of curvature for normal cross section(m)
V = Design Speed (Km/h)
e = -0.02 or -0.015
f = 0.035

Table 1.2.20 Minimum Curvature for Normal Crown (Calculated Value; AASHTO)

(unit:m)

Cross Slope (%)	Design Speed (km/h)						
	100	80	60	50	40	30	20
2.0	5,249	3,360	1,889	1,312	839	472	210
1.5	3,937	2,520	1,417	984	630	354	154

Table 1.2.21 Minimum Curvature for Normal Crown Section (Calculated Value; AASHTO)

V (km/h)	100	80	60	50	40	30	20
f	0.029	0.028	0.027	0.026	0.025	0.024	0.024
e							
- 2.0%	8,749	6,299	4,049	3,221	2,520	1,772	787
- 1.5%	5,624	3,876	2,362	1,790	1,260	787	350

3) Radius of curvature and superelevation

1. On the curve section of a roadway, the values of superelevation shown in Table 1.2.22 should be provided according to the radius of curvature and the design speed. However, in an urban area, superelevation may be omitted on low speed urban streets subjected to severe constraints.

Table 1.2.22 Design Superelevation Rates According to the Radius of Curvature and Design Speed

Radius of curvature (m)							Superelevation (%)
100	80	60	50	40	30	20 (km/h)	
Not less 350 Less 430	230 280	120 150	80 100	50 65	- -	- -	10
430 480	280 330	150 190	100 130	65 80	- -	- -	9
480 550	330 380	190 230	130 160	80 100	30 40	15 20	8
550 640	380 450	230 270	160 200	100 130	40 60	20 30	7
640 760	450 540	270 330	200 240	130 160	60 80	30 40	6
760 930	540 670	330 420	240 310	160 210	80 110	40 50	5
930 1,210	670 870	420 560	310 410	210 280	110 150	50 70	4
1,210 1,700	870 1,240	560 800	410 590	280 400	150 220	70 100	3
1,700 5,000	1,240 3,500	800 2,000	590 1,300	400 800	220 500	100 200	2

Standard Cross Slope = 1.5%

not less 1,700 less 2,130	1,240 2,100	800 1,370	590 1,000	400 600	220 350	100 150	2
2,130 4,000	2,100 2,500	1,370 1,500	-	-	-	-	1.5

* *See Section 1.2.5 (12) Cross Slope*

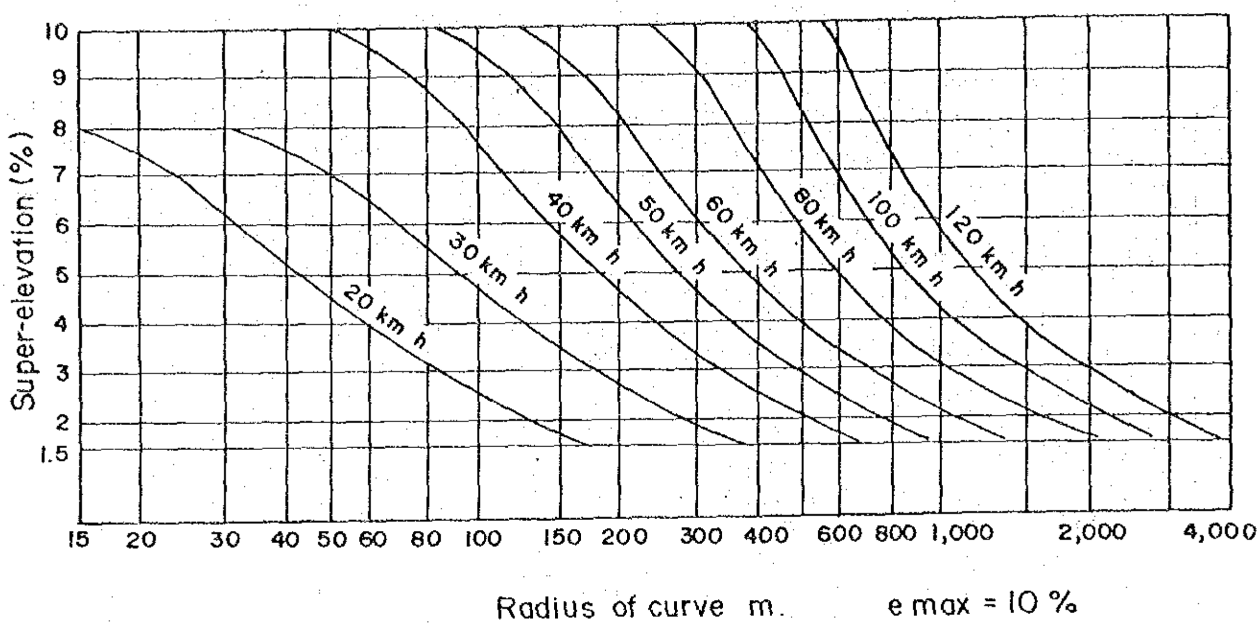


Figure 1.2.17 Values of Superelevation (e max = 10%)

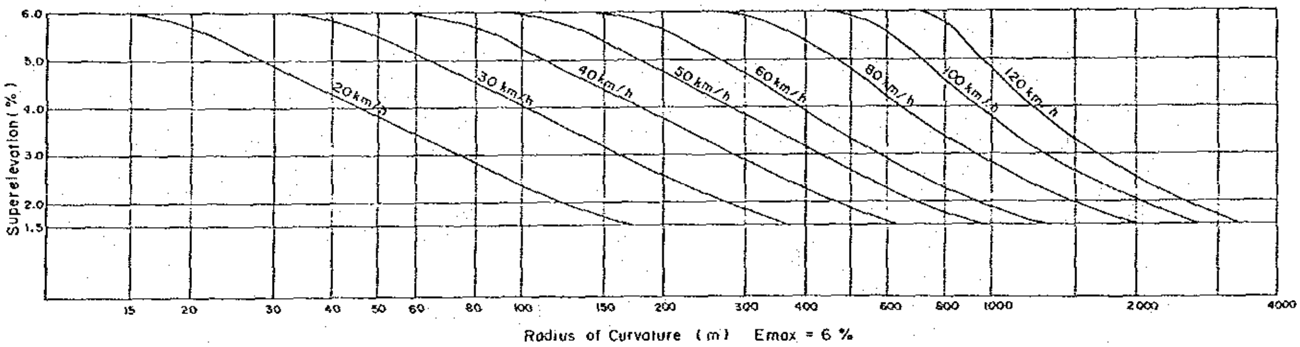


Figure 1.2.18 Values of Superelevation (e max = 6%)

a) <u>Note for application</u>

On urban roads, there is a tendency that no superelevation is employed because of the roadside condition, the relation of intersection and street, and the problem of drainage. But, it is highly desirable that superelevation is employed on the road from the viewpoints of the safety and comfort. In a highly urbanized area, it is appropriate to design the radius of curvature so that superelevation may be omitted, but with the exceptional value shown in Table 1.2.23.

Therefore no superelevation is permitted on urban roads, but the value of side friction factor should be not more than 0.15. When the radius of curvature is more than the radius shown in Table 1.2.12, superelevation may be omitted. When the radius of curvature is less than that value, the rate of superelevation shown in Table 1.2.23 should be employed. When the rate of superelevation has to be decreased, the relation shown in Table 1.2.23 should be used.

Table 1.2.23 Exceptional Value of Superelevation and Radius of Curvature in Urban Area

Radius of curvature					Superelevation (%)
60 km/h	50	40	30	20	
-	-	60 63	30 35	15 16	6
-	100 105	63 65	35 37	16 17	5
Not less than 150 Less than 160	105 110	65 70	37 40	17 18	4
160 165	110 115	70 74	40 42	18 19	3
165 220	115 150	74 100	42 55	19 25	2

(unit:m)

Standard Cross Slope = 1.5%*

Not less than 165 Less than 170	115 120	74 76	42 43	19 20	2
170 220	120 150	76 100	43 55	20 25	1.5

* See Section 1.2.5 (12) Cross Slope

(6) Widening on curves

1. Carriageway widths on curves sometimes are widened according to the design vehicle and the radius of curvature. However, on urban roads, this rule may be waived in extraordinary cases.

1) Widening on curves

Carriageway widths on curves sometimes are widened to make operating conditions on curves comparable to those on tangents. Widening of carriageway width is needed on certain curves for one or a combination of the following reasons:

- The vehicle occupies a greater width because rear wheels generally track inside front wheels in rounding.
- The drivers experience difficulty in steering their vehicles in the center of the lane.

2) Amount of widening on curves

1. On a curve section, each lane should be widened by the value shown in Table 1.2.24, according to the road classification and the radius of curvature. On urban roads, this rule may be waived in extraordinary situations.

Table 1.2.24 Widening of One Lane at Curve

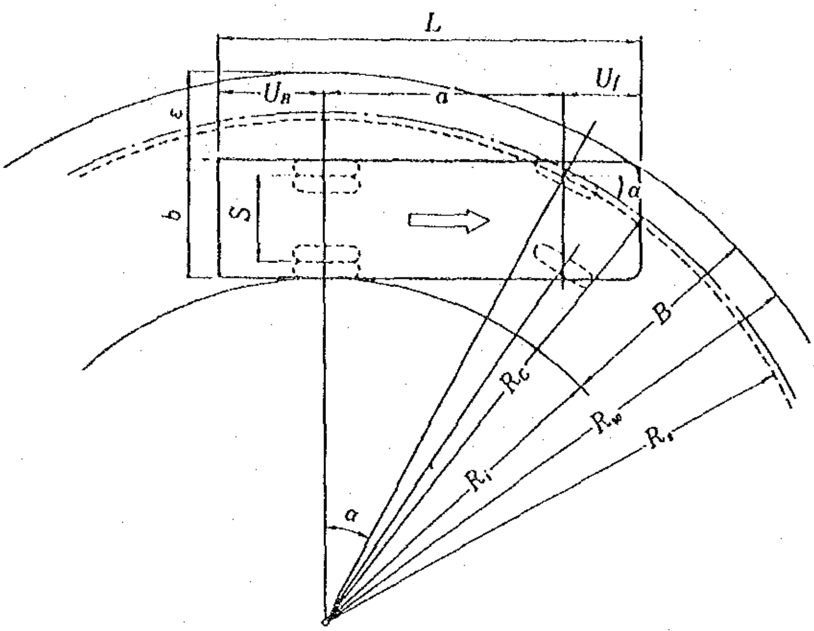
Radius of Curve R(m)		Widening (m) (one lane)
Major Trunk Roads Major Roads	Other Roads	
150 ≦ R < 280	90 ≦ R < 160	0.25
100 150	60 90	0.50
70 100	45 60	0.75
50 70	32 45	1.00
	26 32	1.25
	21 26	1.50
	19 21	1.75
	16 19	2.00
	15 16	2.25

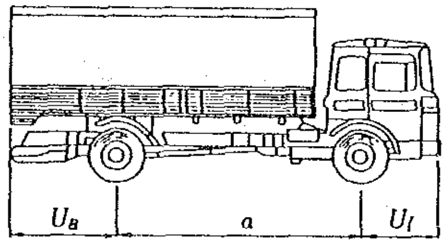
a) Derivation of design value

Lane widths are determined by the value adding the maximum width of vehicle, 2.5m, and a certain margin according to the design speed. Widening is needed on certain curves for the reason that the vehicle occupies a greater width because rear wheels generally track inside front wheels in rounding curves. The extent of widening is calculated from the relation with the center line of carriageway.

i) Widening for large-sized motor vehicle

Figure 1.2.19(a) shows the turning path of large-size motor vehicles on curves.





where B = required width, m;

Rw = radius of outer curvature, m;

Rc = radius of curvature of center line on the carriageway, m;

Rs = turning radius of outer front wheel, m;

Ri = radius of inner curvature, m;

l = length of vehicle, m;

a = wheelbase, m;

b = width of vechicle, m;

S = space of wheel, m;

Uf = front overhang, m;

U_B = rear overhang, m; and

α = conversion angle of outer front wheel

Figure 1.2.19 (a) Widening on Curves, for Large-sized Motor Vehicles

Required width at curve, B can be obtained by the following equation;

$$B = Rw + b - \sqrt{Rw^2 - (a+Uf)^2} \quad \ldots(9)$$

$$Rw = \sqrt{(\sqrt{Rc^2 - (a+Uf)^2} + \frac{b}{2})^2 + (a+Uf)^2} \quad \ldots(10)$$

By substituting, b=2.5m, a=6.5m, Uf=1.5m (dimensions of design vehicle) into the above equations, we get;

$$B = Rw + 1.25 - \sqrt{Rc^2 - 64} \quad \ldots(11)$$

where $Rw = \sqrt{(\sqrt{Rc^2 - 64} + 1.25)^2 + 64}$

Thus necessary widening of lane, ε is

$$\varepsilon = B - b$$

The results of calculation are shown in Figure 1.2.20(a).

ii) Widening for semi-trailer

Figure 1.2.19(b) shows the turning path of a semi-trailer on curves.

The following equations are obtained from Figure 1.2.19(b).

$$B = Rw + \frac{b_2}{2} - \sqrt{(\sqrt{Rw^2 - (a+Uf)^2} - \frac{b}{2})^2 - a_2{}^2 + a_s{}^2} \quad \ldots.(12)$$

$$Rw = \sqrt{(\sqrt{Rc^2 - (a+Uf)^2} + \frac{b}{2})^2 + (a+Uf)^2} \quad \ldots(13)$$

By substituting, a=4.0m, $b=b_2$ = 2.5m, Uf=1.3m, a_2=9.0m, a_s=0 into the above equations, we get;

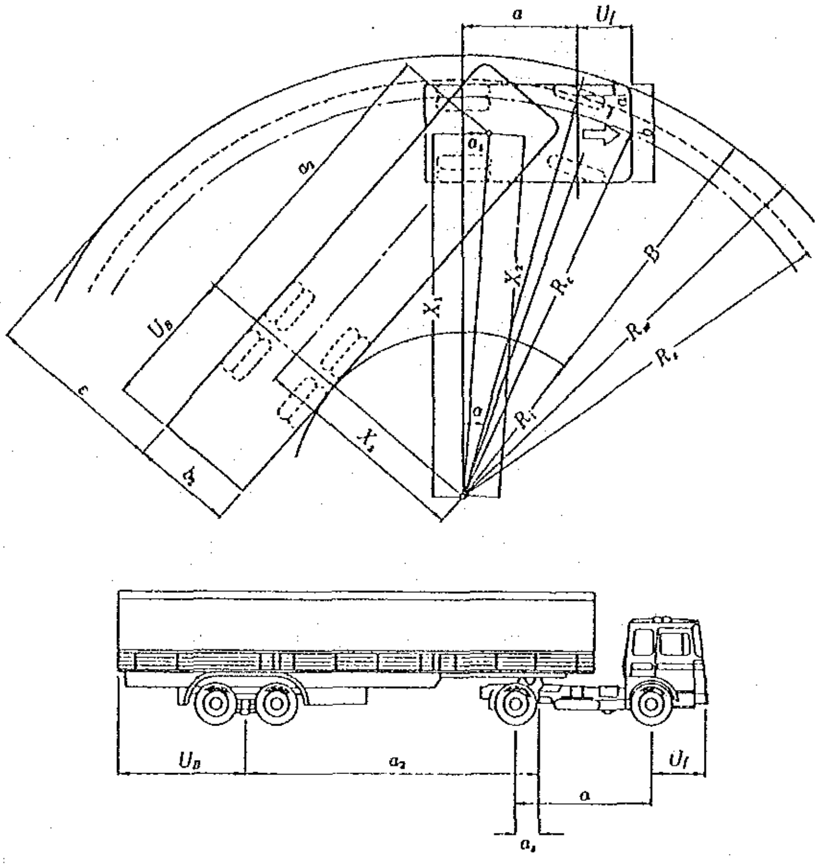
$$B = Rw + 1.25 - \sqrt{Rc^2 - 109.09} \quad \ldots.(14)$$

where, $Rw = \sqrt{(\sqrt{Rc^2 - 28.09} + 1.25)^2 + 28.09}$

Thus necessary widening of lane, is

$$\varepsilon = B - b$$

The results of calculation are shown in Figure 1.2.20(a).



where a = front wheelbase, m;

a_2 = rear wheelbase, m;

a_s = offset (distance from kingpin to rear wheel of tractor), m;

b_2 = width of trailer, m

Figure 1.2.19 (b) Widening on Curves, for Semi-trailer

iii) Calculated value and design value

Figure 1.2.20(a) shows the amount of widening obtained from the above-mentioned calculations.

The extent of widening for AASHTO's design vehicle is calculated using the same equations by substituting each dimension of AASHTO's design vehicle. The extent of widening for AASHTO's design vehicle is much wider than those of the Japanese standard. Therefore, if the widening of AASHTO is adopted on curve section, excessive widening results.

Figure 1.2.20(b) shows design values in units of 0.25m, based on the design vehicle, considering the convenience of design and execution of work.

The amounts of widening on Major Trunk Roads and Major Roads are determined by those for a semi-trailer, and the values on other roads are determined by those for a large-sized motor vehicle.

3) Note for application

a) Direction of widening

When a vehicle runs on the curve section, the rear wheels track inside the front wheels in rounding curves. Hence, in general, widening is done toward the center of the curve. In the case where more than two lanes are widened, widening should be done from the outermost lane and no lane should invade an other lane.

b) Roads of more than six lanes

Widening on the road of more than six lanes should be done considering the situation of large vehicle traffic. Generally it is not necessary to widen all lanes. In many cases, widening of the two lanes in one direction can meet the demands fully.

c) Urban roads

The proviso states that widening may not be applied in extraordinary situations. However, on a road where heavy traffic of large vehicles is expected, the lane width should not be less than the sum of the value obtained from Figure 1.2.20(a) and the width of a large-sized motor vehicle of 2.5m.

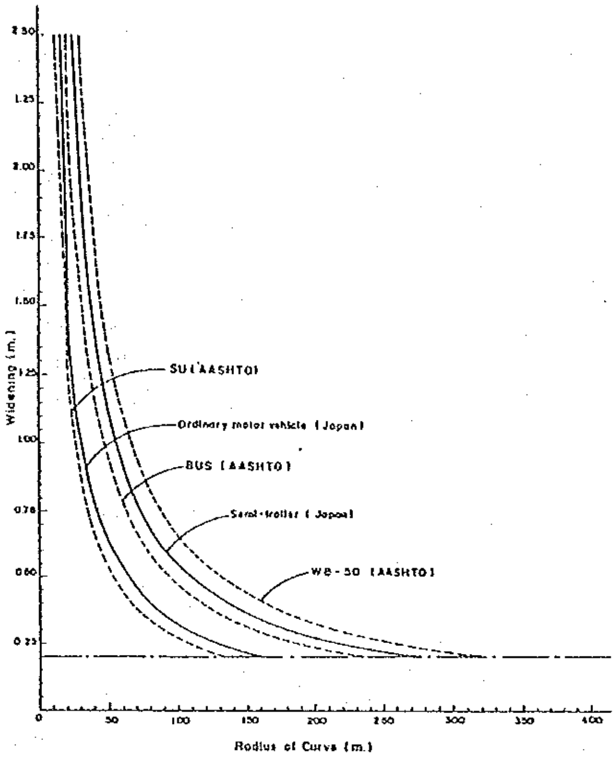


Figure 1.2.20(a) Widening of One Lane at Curve (Calculated)

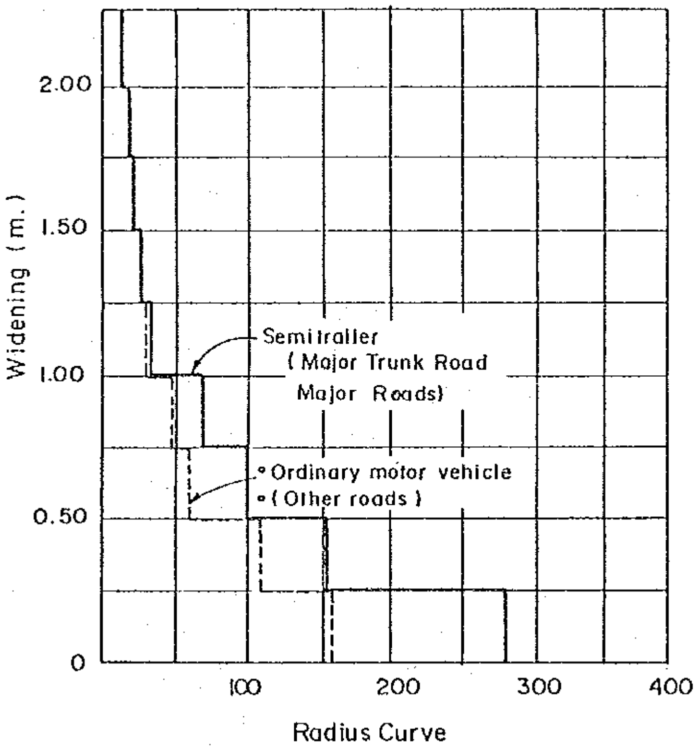


Figure 1.2.20(b) Widening of One Lane at Curve (Rounded)

(7) Transition section

1. As a general rule, a transition section should be provided between a tangent and a circular curve with small radius, and between circular curves of substantially different radii. However, this rule may be waived in special situations.

2. When a curved section is widened and/or provided with superelevation, runoff for this widening and/or superelevation should be done in the transition section.

3. The transition curve length should not be less than the right-side value shown in Table 1.2.25 according to the design speed of the road.

Table 1.2.25 Transition Curve Length

Design speed (km/h)	Transition curve length (m)
100	85
80	70
60	50
50	40
40	35
30	25
20	20

The principal advantages of transition curves in horizontal alignment are as follows;

- A properly designed transition curve provides a natural, easy-to-follow path for drivers, such that the centrifugal force increases and decreases gradually as a vehicle enters and leaves a circular curve.
- The transition between the normal cross slope and the fully superelevated section on the curve can be effected along the length of the transition curve in a manner closely fitting the speed-radius relation for the vehicle traversing it.
- The spiral facilitates the transition in width where the carriageway is to be widened around a circular curve. Use of spirals provides flexibility in the widening on sharp curves.
- The use of spirals avoids the noticeable breaks at the beginning and ending of circular curves.

(8) Superelevation runoff

1. When providing the superelevation and changing the value of superelevation, widening on curves, the superelevation runlff and transition of widening should be proveded.

2. It is desirable that the length of superelevation runoff shall not be less than the length which is derived from the ratio of superelevation runoff along the edge of the carriageway by design speed as shown in Table 1.2.26.

3. Superelevetion runoff and transition of widening should be done in the transition section.

Table 1.2.26 Rate of Superelevation Runoff

Design speed V (km/h)	Rate of superelevation runoff
100	1/175
80	1/150
60	1/125
50	1/115
40	1/100
30	1/75
20	1/50

1) Superelevation runoff

AASHTO prescribed that the minimum length of design superelevation runoff for a carriageway wider than two lanes should be as follows;

- Three-lane carriageway: 1.2 times the corresponding length for two-lane highways;
- Four-lane undivided carriageway: 1.5 times;
- Six-lane undivided carriageway: 2.0 times;

2) Transition runoff in addition or reduction of number of lane

On a mid-block section of road when the number of lanes is added or reduced, transition runoff should be done appropriately according to the design speed, horizontal alignment and type of area.

When the number of lanes is added or reduced on the mid-block section it is necessary to provide a transition section with an appropriate taper. (see

Figure 1.2.21).

The values shown in Table 1.2.27 may be used as the rate of transition.

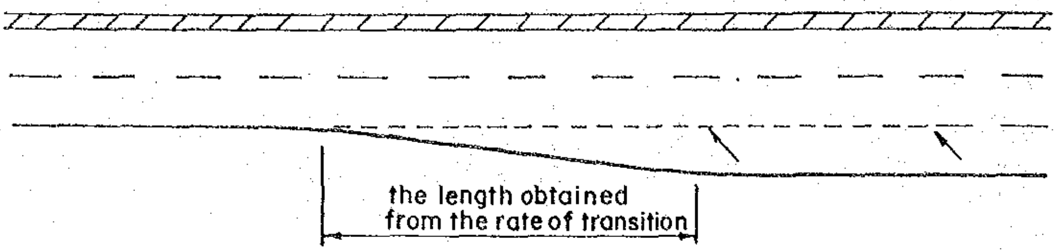


Figure 1.2.21 Transition in Addition or Reduction of Number of Lane

Table 1.2.27 Standard Value of Rate of Transition

Design speed (km/h)	Standard value of rate of transition	
	Rural area	Urban area
100	1/60	-
80	1/50	1/40
60	1/40	1/30
50	1/30	1/25
40	1/25	1/20
30	1/20	1/15
20	1/15	1/10

When the through lane is to be shifted in order to provide an added lane at an at-grade intersections, reference should be made to section 1.2.6 "At-grade intersection".

(9) Stopping sight distance and Passing sight distance

1. Stopping sight distance should not be less than the value shown in Table 1.2.28 according to the design speed.

2. On two-lane two-way roads, the section should have a length which secure sufficient sight distance for the passing vehicle, if necessary.

Table 1.2.28 Stopping Sight Distance

Design speed (km/h)	Stop Sight Distance (m)
100	160
80	110
60	75
50	55
40	40
30	30
20	20

1) General Consideration

The ability to see ahead is of the utmost importance in the safe and efficient operation of a vehicle on a highway. For safety on highways the designer must provide sight distance of sufficient length that drivers can control the operation of their vehicles to avoid striking an unexpected object.

Sight distance is the distance along a roadway that an object of specified height is continuously visible to the driver. This distance is dependent on the height of the driver's eye above the road surface, the specified object height above the road surface, and the height of sight obstructions within the line of sight.

For all sight distance calculations, the height of the driver's eye is assumed to be 1.2 m above the road surface. For stopping sight distance calculations, the height of object is assumed to be 0.10 m above the road surface. For passing sight distance calculation, the height of object is assumed to be 1.2m above the road surface.

These values are quoted from the Japanese standards, and Table 1.2.29 shows the standard values in various countries. In Japanese standards, the height of the driver's eye, 1.2 m, is determined using the average passenger car height.

Table 1.2.29 Height of Driver's Eye and Height of Object in Various Countries

	Height of Driver's Eye (m)	Height of Object (m)
Australia	1.14	0.23
West Germany	1.0	0
Finland	-	0
Finland (special case)	-	0.1
France	1.0	0.15
Italy	-	-
Netherlands	-	-
United Kingdom	1.05	1.05
U.S.A (stopping)	1.14	0.15
U.S.A (Passing)	1.14	1.37

2) Calculation of sight distance

a) Stopping sight distance

Sight distance is the length of roadway ahead visible to the driver. The minimum sight distance available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path.

Stopping sight distance is the sum of two distances; the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied and the distance required to stop the vehicle from the instant brake application begins. These are referred to as brake reaction distance and braking distance, respectively.

The stopping sight distance is computed by the use of the standard equation:

$$D = \frac{V}{3.6} t + \frac{V^2}{2gf(3.6)^2}$$

where: D = stopping sight distance, m;

V = initial speed, km/h;
f = coefficient of friction between tires and roadways
t = reaction time, sec.

In this formula, the first term is the brake reaction distance, and the second term is the braking distance. From various studies, a reaction time of 2.5 sec was found to be adequate in Japanese standards and also AASHTO standards. Substituting $t=2.5s$, $g=9.8m/s^2$, into the above equation stopping sight distance, D is obtained as follows.

$$D = 0.694V + 0.00394\frac{V^2}{f}$$

Table 1.2.30 shows the stopping sight distance under the wet pavements in the Japanese standard, and Table 1.2.31, in AASHTO. The difference of the values of stopping sight distance is caused by only the values of coefficient of friction, f.

When a highway is on a grade, the standard formula for braking distance is the following:

$$d = 0.00394\frac{V^2}{(f+G)}$$

Table 1.2.30 Stopping Sight Distance (Wet Pavement) in Japan

Design Speed (km/h)	Running Speed (km/h)	Coefficient of Friction f	Brake Reaction Distance 0.694 V	Braking Distance $0.00394\frac{V^2}{f}$	Stopping Sight Distance	
					Computed (m)	Rounded for Design
120	102	0.29	70.7	141.3	212.0	210
100	85	0.30	58.9	94.8	153.7	160
80	68	0.31	47.1	58.7	105.8	110
60	54	0.33	37.4	34.8	72.2	75
50	45	0.35	31.2	22.8	54.0	55
40	36	0.38	24.9	13.4	38.3	40
30	30	0.44	20.8	8.1	29.9	30
20	20	0.44	13.8	3.5	17.8	20

Table 1.2.31 Stopping Sight Distance (Wet Pavement) in AASHTO

Design Speed (km/h)	Running Speed (km/h)	Coefficient of Friction f	Brake Reaction Distance 0.649 V	Braking Distance $0.00394 \frac{V^2}{f}$	Stopping Sight Distance Computed (m)	Stopping Sight Distance Rounded for Design
120	102	0.28	70.7	140.4	217.1	220
100	85	0.29	58.9	98.2	157.1	160
80	68	0.30	47.1	60.7	107.8	110
60	54	0.32	37.4	35.9	73.3	75
50	45	0.34	31.2	23.5	54.7	55
40	36	0.38	24.9	13.4	38.3	40
30	30	0.40	20.8	8.9	29.7	30
20	20	0.40	13.8	3.9	17.7	20

in which G is the grade expressed as a decimal and the other terms are as previously stated. The safe stopping distances on upgrades are shorter; those on downgrades are longer. However, G is usually very small compared to f and thus, the effect of G is negligible. Also, sufficient margin is included in reaction time. Consequently, it is not necessary to take the effect of grade into consideration in calculation of stopping sight distance.

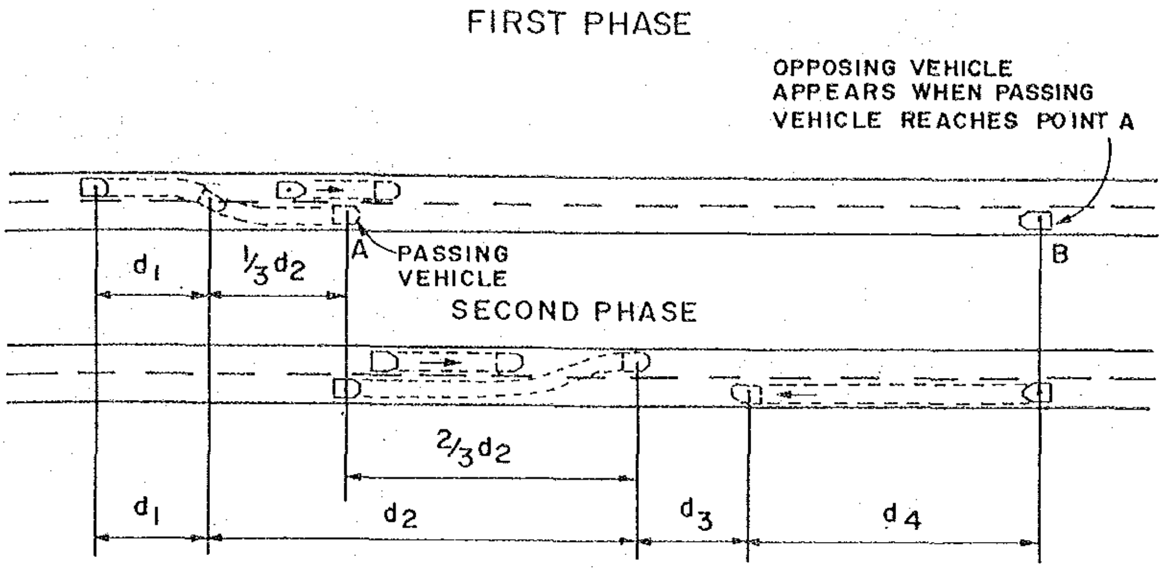
b) Passing sight distance

On most two-lane roads, vehicles frequently overtake slower moving vehicles. In this case the passing must be accomplished on lanes regularly used by opposing traffic.

Passing sight distance used in design should be determined on the basis of the length needed to safely complete normal passing maneuvers.

Calculation of passing sight distance

Figure 1.2.22 shows the passing sight distance for two-lane roads. The minimum passing sight distance for two-lane roads is determined as the sum of the four distances:



where :

d1 = Distance traversed during perception and reaction time and during the initial acceleration to the point of encroachment on the right lane.

d2 = Distance traveled while the passing vehicle occupies the right lane.

d3 = Distance between the passing vehicle at the end of its maneuver and the opposing vehicle.

d4 = Distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the right lane, or 2/3 of d2 above.

Figure 1.2.22 Elements of Passing Sight Distance - Two-Lane Highway

i) Initial maneuver distance (d1)

The distance, d1, traveled during the initial maneuver period is computed from the following formula:

$$d_1 = \frac{V_o}{3.6} t_1 + \frac{1}{2} \alpha t_1^2$$

where : t1 = time of initial maneuver, sec;
α = average acceleration, m/s^2; and
Vo = average speed of passed vehicle, Km/h

The time of initial maneuver obtained from the passing study data varied from 2.9 sec to 4.5 sec.

ii) Distance while passing vehicle occupies right lane (d2)

The distance, d2, traveled in the right lane by the passing vehicle is computed by the following formula:

$$d_2 = \frac{1}{3.6} Vt_2$$

where : t2 = time passing vehicle occupies the right lane, sec: and
V = average speed of passing vehicle, Km/h

Passing vehicles were found in the study to occupy the right lane from 9.3 to 10.4 sec.

iii) Clearance length (d3)

The clearance length between the opposing and passing vehicles at the end of the maneuvers found in the passing study varied from 30m to 100m.

iv) Distance traversed by an opposing vehicle (d4)

Passing sight distance includes the distance traversed by an opposing vehicle during the passing maneuver to minimize the chance of a passing vehicle meeting an opposing vehicle while in the right lane. Conservatively, this distance should be the distance traversed by an opposing vehicle during the entire time it takes to pass or during the time the passing vehicle is in the right lane, but such a distance is questionably long. During the first phase of the passing maneuver the passing vehicle has not yet pulled abreast of the vehicle being passed, and even though the passing vehicle occupies the right lane, its driver can return to the left lane if he sees an opposing vehicle. It is unnecessary to include this trailing time interval in computing the distance traversed by an opposing vehicle. This time interval, which can be computed from the relative positions of passing and passed vehicle, is about one-third the time the passing vehicle occupies the right lane, so that the passing sight distance element for the opposing vehicle is the distance it traverses during two-thirds of the time the passing vehicle occupies the right lane. The opposing vehicle is assumed to be traveling at the same speed as the passing vehicle, so

$$d_4 = \frac{2}{3} d_2 = \frac{2}{3} \cdot \frac{1}{3.6} Vt_2$$

Table 1.2.32 shows the computed value of passing sight distance.

Table 1.2.32 Passing Sight Distance (Computed)

Average speed of the passing vehicle (km/h) and the opposing vehicle		100	80	60	50	40	30	20
Average speed of passed vehicle (km/h)		80	65	45	37.5	30	20	15
d_1	Average acceleration a (m/s^2)	0.66	0.65	0.63	0.62	0.61	0.60	0.60
	Time of initial maneuver t_1 (s)	4.5	4.2	3.7	3.4	3.1	2.9	2.7
	Initial maneuver distance d_1 (m)	113	82	51	34	28	19	10
d_2	Time passing vehicles occupies the right lane t_2 (s)	11.4	10.4	9.5	9.0	8.5	8.0	7.6
	Distance while passing vehicle occupies right lane d_2 (m)	317	231	159	125	95	67	42
d_3	Clearance length d_3 (m)	80	60	40	30	25	20	15
d_4	Distance traversed by an opposing vehicle $d_4 = 2/3\ d_2$(m)	211	154	106	81	63	45	28
Total passing sight distance ($d_1+d_2+d_3+d_4$ (m))		700	550	350	250	200	150	100
Minimum passing sight distance ($2/3\ d_2+d_3+d_4$ (m))		500	350	250	200	150	100	70

Application of passing sight distance

Sight distance adequate for passing should be encountered frequently on two-lane roads. However, in practice, this may not be attained because of various constraints. It seems to be desirable that the passing sight distance should be secured one time while vehicle is running for a minimum of one minute, and in special cases, for three minutes. The ratio of passing sight distant to total length is kept to be more than 30% as shown in Table 1.2.33, in special cases, it is more than 10%. However, on a two-lane two-way road, passing is not possible when a opposing vehicle travels on the opposing lane. Hence, the above-mentioned ratio seems to be the minimum condition.

Table 1.2.33 Ratio of Passing Sight Distance to Total Length

Design Speed	Distance of running for one minute	Passing sight distance	One time / One minute	One time / Three minutes
80 km/h	1.33 km	550 m	38%	13%
60	1.00	350	35	12
50	0.83	250	30	10
40	0.67	200	30	10
30	0.50	150	30	10

3) Stopping Sight Distance on Horizontal Curves

Where there are sight obstructions (such as walls, cut slopes, buildings, or a guardrail under certain conditions) on the inside of curves, a design to provide adequate sight distance may require adjustment in the normal highway cross section or change in alignment if the obstruction cannot be removed. Using 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.

For general use in design of a horizontal curve, the sight line is a chord of the curve, and the applicable stopping sight distance is measured along the center line of the inside lane around the curve. Figure 1.2.23 is design chart showing the required middle ordinates for clear sight areas to satisfy the values of stopping sight distance required for curves of various radii. In this case, the middle ordinate, M, shown in Figure 1.2.23 is calculated from the following formula;

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

$$= R\left(1 - \cos\frac{S}{2R}\right)$$

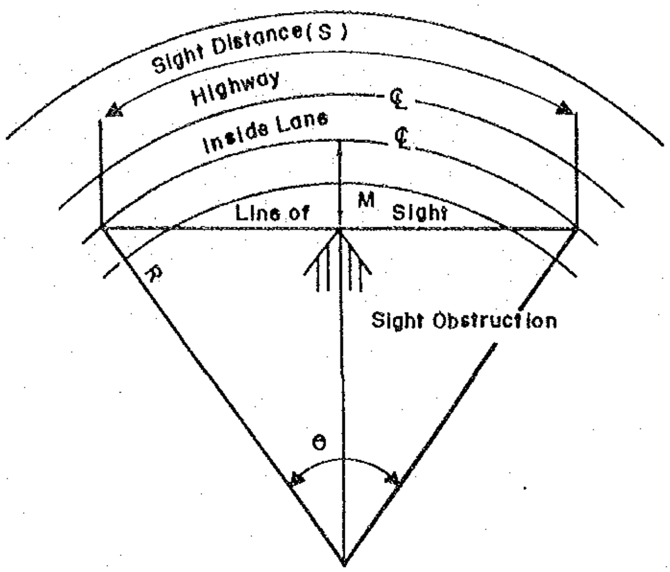
where : M = middle ordinate, m;
S = stopping sight distance, m; and
R = radius, m

Using Taylor's expansion in the right member,

$$M = \frac{S^2}{8R} - \frac{S^4}{384R^3} \cdots = \frac{S^2}{8R}\left(1 - \frac{S^2}{48R^2} \cdots\right)$$

$$\fallingdotseq \frac{S^2}{8R}$$

This formula is illustrated with Figure 1.2.24. For example, with 80km/h design speed and a 250m of radius of curvature, a clear sight distance with a middle ordinate of 6m, is needed for a stopping sight distance of 110m. However this formula applies only to circular curves longer than the sight distance for the pertinent design speed. When the sight distance is longer than circular curves, and is reaching the transition section, the value of the middle ordinate is a bit less than the ordinary value.



S = Stopping Sight Distance (m)
M = Middle Ordinate (m)
R = Radius (m)

Figure 1.2.23 Sight Distance on Horizontal Curve

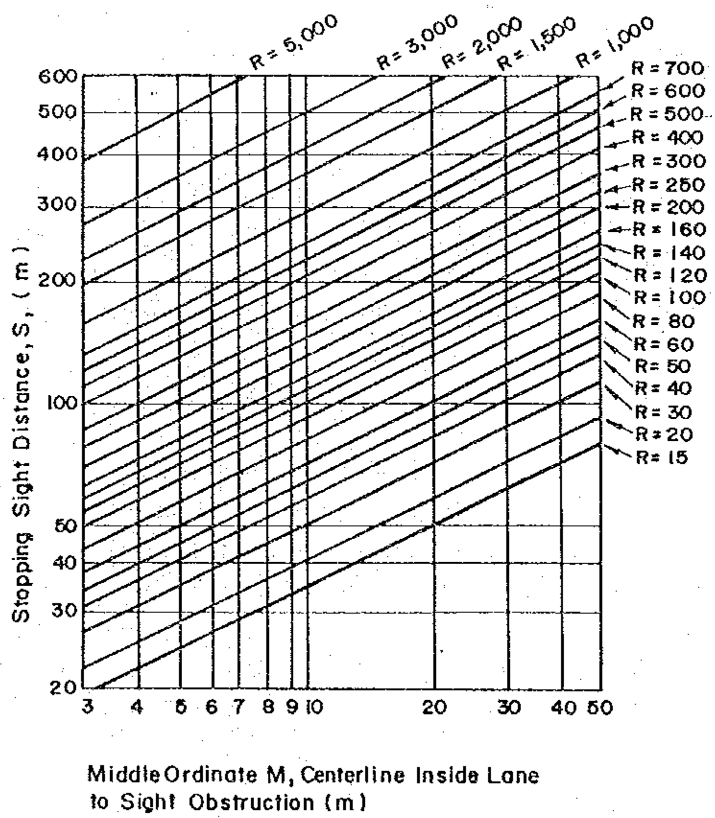


Figure 1.2.24 Relation Between Radius of Curvature and Value of Middle Ordinate Necessary to Provide Sight Distance

(10) Grades

1) Grades

1. The grades should not be more than the values shown in Table 1.2.34 according to the design speed. However, in special cases, 2% may be added to each value in the urban area, and 3% be added in the suburban or rural area.

Table 1.2.34 Grades

Design Speed (km/h)	Grade (Percent)
100	3
80	4
60	5
50	6
40	7
30	8
20	9

a) General

Roads and streets should be designed to encourage uniform operation throughout. Use of a selected design speed as previously discussed is a means towards this end by correlation of various geometric features of the road or street. Design values have been determined and agreed upon for many highway features, but few conclusions have been reached on roadway grades in relation to design speed.

Recently, the performance of vehicles has been greatly improved over past levels, and nearly all passenger cars can readily negotiate grades without appreciable loss in speed below that normally maintained on level highways. However, the effect of grades on truck speed is much more pronounced than on speeds of passenger cars. The decrease in speed on upgrades (1) hampers drivers in other vehicles from staying at high-speed, (2) confuses the traffic flow, and (3) results in a decrease of traffic capacity.

b) Policy of determining standard grades

In determining standard grades, the following conditions are considered.

- On upgrades of ordinary value, passenger cars are assumed to be able to climb at the average running speed. On the other hand, trucks are assumed to be able to climb at the speed which is approximately half of the design speed. In this case, the length of grade is not restricted.
- On upgrades of exceptional value of grade and length, the speed of passenger cars at the end of the grade is assumed to be approximately equal to the average running speed. In this case, at the beginning of the grade, the passenger car speed is the design speed. On the other hand, the speed of truck at the end of the grade is assumed to be approximately half of the design speed. The speed of a truck at the beginning of grade is assumed to be 80 Km/h when the design speed is 80 Km/h or higher.
- When an extra climbing lane is provided, the climbing lane is designed so that the speed of passenger cars is maintained at approximately two-thirds of the design speed, and the climbing speed of trucks is not taken into consideration.
- Even the case of a fully-loaded truck, the above-mentioned speeds should be maintained.
- The exceptional values of special cases for urban roads should be smaller than for suburban roads.

2) Exceptional value of grades

1. When the exceptional values of grades are adopted in an unavoidable case, such as for a topographical reason, the critical lengths of grades should be the proper value shown in Table 1.2.35.

Table 1.2.35 Exceptional Values and Critical Lengths of Grades

Design Speed (km/h)	Grade (%)	Critical Length (m)
100	4	700
	5	500
	6	400
80	5	600
	6	500
	7	400
60	6	600
	7	400
	8	300
50	7	500
	8	400
	9	300
40	8	400
	9	300
	10	200

To establish design value for critical lengths of grade for which gradeability of vehicles is the determining factor, data or assumptions are needed for the following:

- The speed used at the entrance to a critical length of grade can be the design speed, (when the design speed is not less than 80 km/h, speed at entrance is limited to 80 km/h). The critical length is obtained from the climbing distance over which the running speed is reduced to the allowable speed.

When a climbing lane is provided, however, grades may be provided over the critical lengths shown in Table 1.2.35.

Maximum grade in itself is not complete design control. It is also necessary to consider the length of a particular grade in relation to desirable vehicle operation. The term "critical length of grade" is used to indicate the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed.

By the time when the entering speed is reduced to the speed which is correspond to the grade, the vehicle can climb at respectable distance on the upgrade. If the length of grade is short, the vehicle can climb on the steeper upgrade without slow to the allowable speed. Therefore, the maximum length of the grade is obtained from the relation of the entrance speed' to the allowable speed. This maximum length is needed for the critical length of the grade. Figure 1.2.25 shows the critical length and the value of grade.

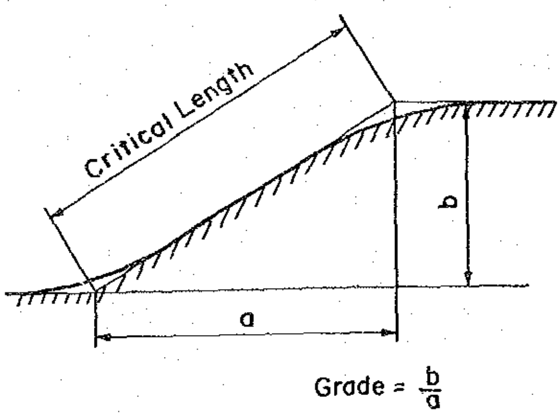


Figure 1.2.25 The Value of Grade and Critical Length

a) <u>Grades and lengths of grades</u>

In general, grades and lengths of grades have a great effect on running vehicles. Therefore, if the steeper grade is on the shorter section, the effect on traffic flow is little, but if the gentle grade is continuing on a long section, the effect on the vehicles increases.

However, the extremely steep grade breaks the proper combination of geometrical factors and decreases the safety and comfort. For that reason, the exceptional value of grades is limited to the value of 3% exceeding to the standard value. The standard for the exceptional value of grade is determined from the viewpoint of setting a limit to the length the constant grade in order to have a same effect on the vehicles as the ordinary value of grades. The critical length of grade is calculated by assuming for a truck of 10ps/t which has lower gradeability than a passenger car's. Table 1.2.36 shows the climbing distance according to the grade at which the vehicle can climb during the slowing to the allowable speed.

The critical lengths of grade are determined as follow with reference to Table 1.2.36.

Roads with design speed of 60 km/h or higher usually function as arterial roads. The grade and the critical length of grade have such a great effect on the traffic that the critical lengths are determined based on the calculated value shown in Table 1.2.36.

Table 1.2.36 Critical Lengths of Grades (Calculated)

Design Speed (km/h)		120	100	80	60	50	40
Entering Speed (km/h)		80	80	80	60	50	40
Allowable Speed (km/h)		60	50	40	30	30	25
Grade (%)	3	830 m					
	4	480	720				
	5	340	500	760			
	6		380	520	490		
	7			410	320	230	
	8				240	170	130
	9					130	100
	10						80