LAO PEOPLE’S DEMOCRATIC REPUBLIC

Peace Independence Democracy Unity Prosperity

Ministry of Public Works and Transport

DEPARTMENT OF ROADS

ROAD DESIGN MANUAL

FINAL –August 2018

CONTENTS

APPENDIX

PREFACE

A Road project is conducted following the procedure below and this manual is applied for Preliminary Design for Feasibility Study (F/S) and Detailed Design (D/D). Required activities and drawings of Preliminary design for Feasibility Study (F/S) and Detailed Design (D/D) are detailed in APPENDIX-C.

<u>Master Plan (M/P)</u>

Prioritized projects are planned based on socio-economic demands.

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<u>Preliminary Design for Feasibility Study (F/S)</u>

Preliminary design is conducted for providing cost estimation and economic analysis for the Feasibility Study (F/S).

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<u>Detailed Design (D/D)</u>

Detailed Design is conducted for project budget and bid preparation.

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<u>Procurement (Tender/Bidding)</u>

Announcement of Bids, Bids, Evaluation of Bids, Contract Negotiation and Contractor Selection

↓

<u>Construction</u>

↓

<u>Operation & Maintenance</u>

For each design, reference is made to Road Design (Chapter 1 - 3), Pavement Design (Chapter 4), Hydraulic Design (Chapter 5) and River Bridge Design (Chapter 6) in the Road Design Manual.

Chapter 1 Road Classification

This chapter provides definition of roads in the Lao PDR, national roads designated as Asian Highway and statistic road length data.

Chapter 2 Design Considerations

In this chapter, the concept of a set of key design considerations is introduced, i.e. 1) Road function and control of access, 2) Economic and financial consideration, 3) Road safety consideration, 4) Environmental consideration, 5) Climate change adaptation and 6) Topography and Land use and physical features.

Chapter 3 Road Geometric Design

This chapter is the main part of road design. In this chapter, firstly road design process is introduced. In addition, detailed explanation of each design requirements and criteria under road design process are described such as road classification, design speed, cross section, curve radius, grade and etc. information for design at grade intersection is introduced.

Chapter 4 Road Pavement Design

In this chapter, pavement material and earthwork material including slope failure and measures introduced. This chapter provides mainly the procedure of pavement design for flexible pavement and rigid pavement by using AASHTO and Overseas Road Note method.

Chapter 5 Hydraulic Design

In this chapter, two main steps in the design of drainage structures for roads are introduced. The first step is to perform a hydrologic analysis of the particular drainage area to determine the quantity of water to be discharged. The second step is to design the drainage structure by performing hydraulic design calculations using the discharge values obtained from the hydrologic analysis in the first step.

Chapter 6 River Bridge Design

This chapter provides an explication on the requirements for planning of the bridges such as span length, abutment/pier location, type of bridge and protection work and an overview of the design methodology adopted including design process, design philosophies and design loads according to AASHTO.

INTRODUCTION

The Road Design Manual was updated in 2018 by the Ministry of Public Works and Transport in cooperation with ADB financed Road Sector Governance and Maintenance Project so as to form a basis for surveying, design and planning of road construction works in Lao PDR. The manual was first issued in 1981 and revised in 1994 and 1996.

The main objectives of the Road Design Manual are as follows:

- To use same standards for the road construction works in Lao PDR, assure more accuracy in techniques, traffic safety on roads, be efficient on economy and be useful in society and culture and environmental protection.
- To coincide with international standards but be practical to use in Lao PDR
- To provide a basis for design for Government and domestic and foreign non-Governmental Organizations who are undertaking researching, surveying, designing and planning, and technical control for road construction works in Lao PDR.

The Road Design Manual sets forth design standards and recommendations to be adopted for the design of roads in the Lao PDR. The Manual consist of followings.

- Chapter 1 Road Classification
- Chapter 2 Design Considerations
- Chapter 3 Geometric Design
- Chapter 4 Pavement Design
- Chapter 5 Hydraulic Design
- Chapter 6 River Bridge Design

In some cases however local conditions, which include weather, topography, geology condition and etc., may demand modifications to these standards.

It is therefore the responsibility of the designer, in such cases, to put forward to the Lao Road authorities, any proposals for modifications to the standards which he considers will result in a better and more economical design.

In case of readers will have found that some standards which are missing in the manual, it is suggested to refer to the latest AASHTO standards and Overseas Road Note.

However the manual is a basic design standard for roads in Lao PDR and it shall be updated frequently in accordance with revised laws, new technology and real experiences.

CHAPTER 1 ROAD CLASSIFICATION

1.1 DEFINITION OF ROAD AND ITS NETWORK

‘Road’ means the strip of land over which the road is constructed for public use of traffic.

Public Road is the road system that is property to the national patrimony constructed for public interest and for communication inside the county and with neighboring counties.

The roads in the Lao PDR is divided into 6 classes namely (see Figure 1.1.1):

1. National Road;
2. Provincial Road;
3. District Road;
4. Urban Road;
5. Rural Road;
6. Special Road;

National Road

The National Road means a road of strategic importance for the overall interest of national and international economy and includes:

1) Road connecting the national capital to the provincial capital;

2) Road to international borders;

3) Road of importance with regard to socio-economic and national defense security purpose.

Provincial Road

Provincial Road means a road of importance for the economic, political, socio-cultural development and for the national defense and security purposes at the provincial level including:

1) Inter-provincial road;

2) Road connecting a provincial capital to district centers, river ports, tourist and importance historic sites of the province.

District Road

District Road mean a road of importance for the economic political socio-economic development and for the national defense and security purposes at the district level including:

1) Inter-district road;

2) Road connecting the district center to the village, river ports tourist and historic sites.

Urban Road

Urban Road means a system of roads used for the traffic within the urban area.

Rural Road

Rural Road means a road connecting villages and to various production and service centers of the village.

Special Road

Specific Road means a road used specifically for the production and service of a sector of activities for the national defense and security and the forest preservation zone.

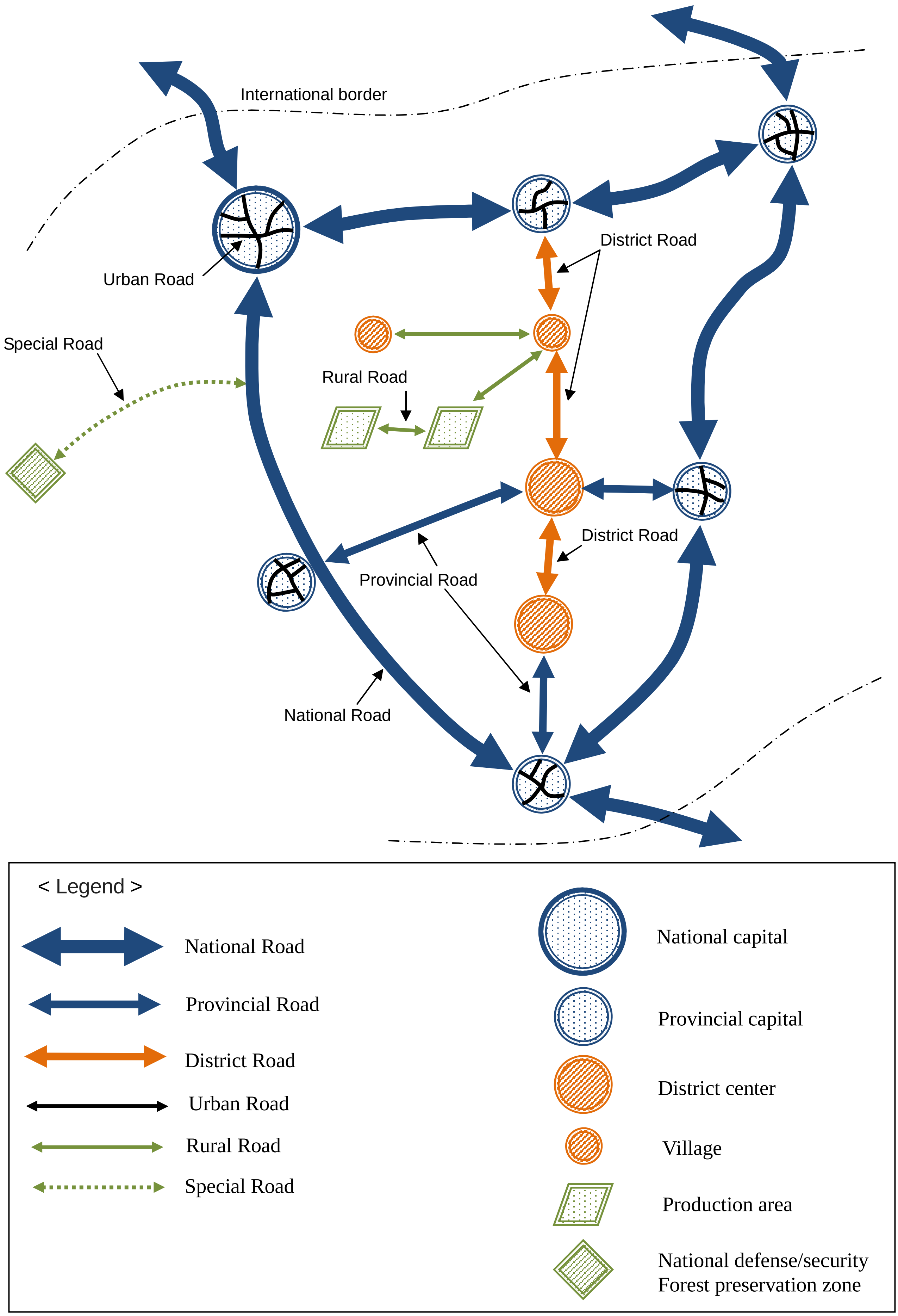


Figure 1.1.1 Image of Road Class and Road Network

1.2 INTERNATIONAL ROAD AND NETWORK

Currently the folowing 8 National Roads are designated as Asian Highways.

No.	National Road	Asian Highway
1	National Road 2	Asian Highway 13
2	National Road 3	Asian Highway 3
3	National Road 8	Asian Highway 15
4	National Road 9	Asian Highway 16
5	National Road 12	Asian Highway 131
6	National Road 13N	Asian Highway 12
7	National Road 13S	Asian Highway 11
8	National Road 18	Asian Highway 132

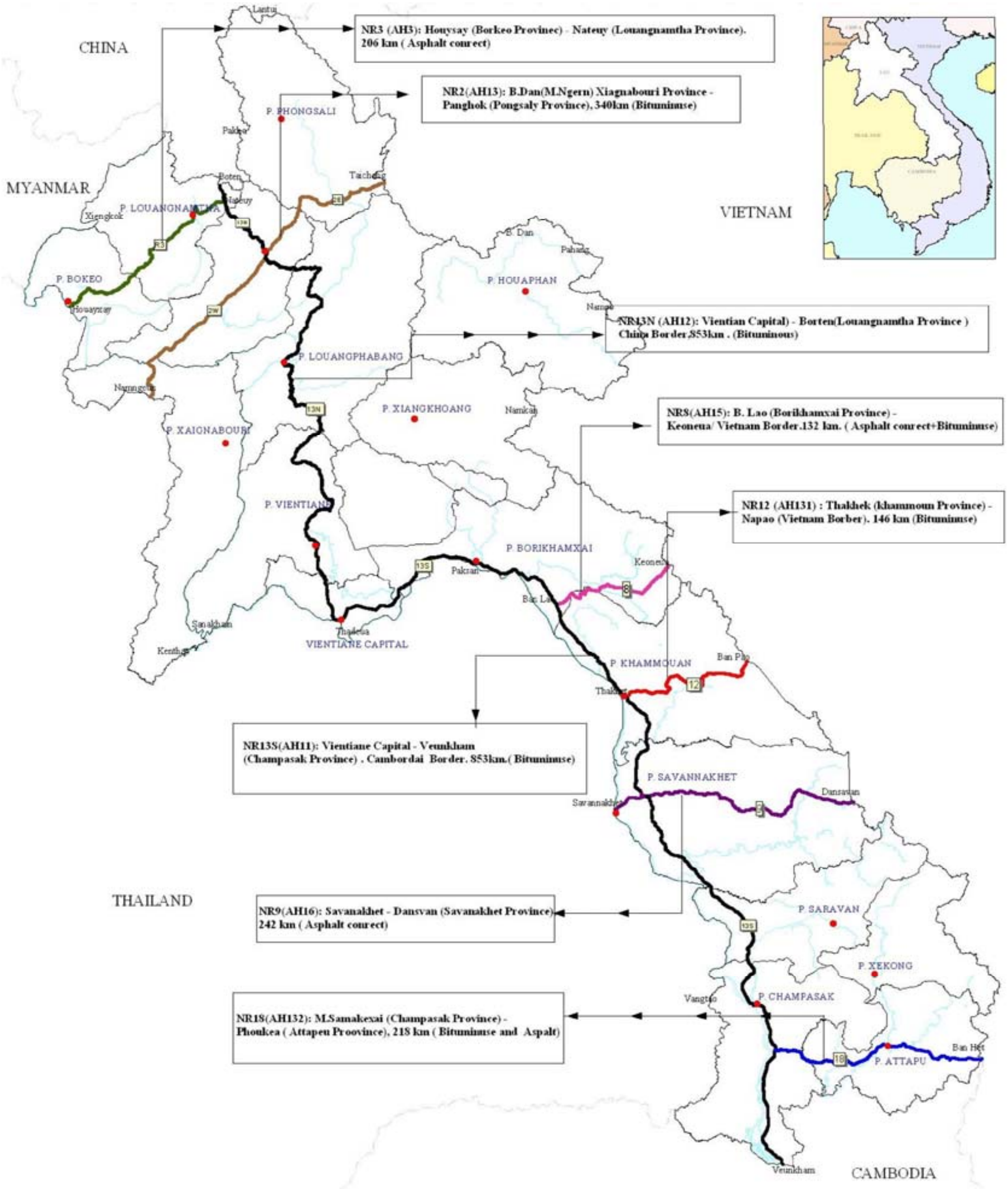


Figure 1.2.1 Location of International Road

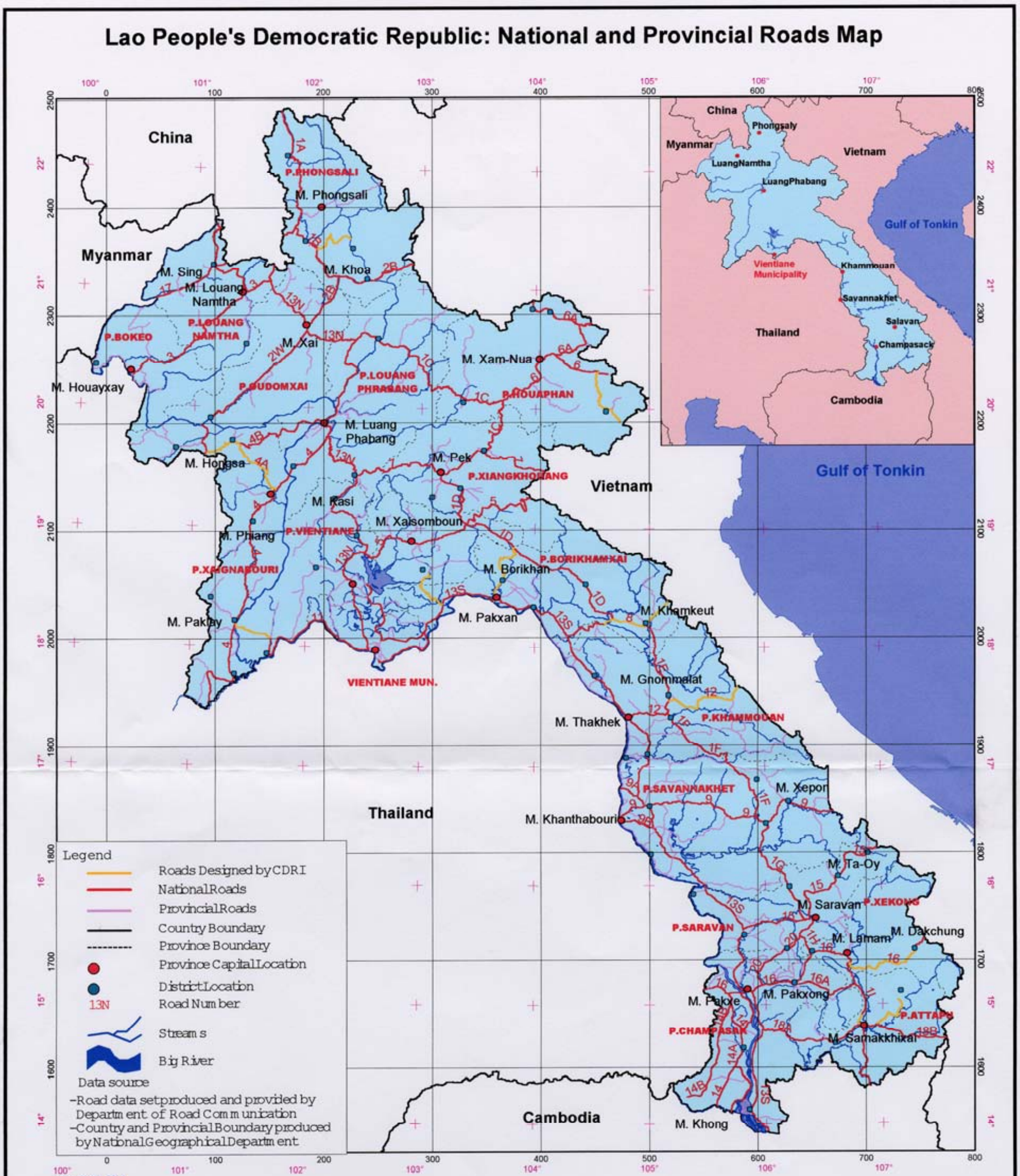
1.3 CURRENT STATUS OF ROAD AND NETWORK

The total length of the road network reported in 2015 is 51,597 km, of which 7,448 km is national roads, and 38,617 km is local roads. Urban roads amount to 2,720 km and special roads are 2,810 km. The list and map of the road and network are shown in Table 1.3.1 and Figure 1.3.1.

Table 1.3.1 Road Length by Road Class and Provice

Pro. Code	Province's name	Road Length (Km) by Road Classes						Total (km)
		National	Provincial	District	Urban	Rural	Special	
1	Vientiane Capital.	246.05	283.70	481.13	664.47	618.31	103.86	2,397.52
2	Phongsali	474.00	205.10	204.50	53.80	1,786.16	111.15	2,834.71
3	Louangnamtha	299.68	493.08	111.00	105.05	876.46	156.74	2,042.00
4	Oudomxai	326.50	285.65	545.65	125.45	1,399.59	46.40	2,729.24
5	Bokeo	175.72	273.35	150.30	84.22	568.80	30.43	1,282.82
6	Louangphabang	605.20	565.05	354.630	143.840	2,057.69	334.17	4,060.58
7	Xaignabouli	549.00	878.50	603.26	233.56	751.74	193.23	3,209.29
8	Houaphan	445.00	585.90	627.15	73.15	2,244.41	347.55	4,323.16
9	Xiengkhouang	436.00	428.47	65.84	71.34	2,064.56	197.29	3,263.50
10	Vientiane	398.00	673.99	570.98	300.88	823.84	129.03	2,896.71
11	Bolikhamxai	510.00	614.10	325.80	67.11	517.42	45.00	2,079.43
12	Khammouan	594.16	315.63	412.65	161.08	1,915.03	57.94	3,456.48
13	Savannakhet	604.00	860.11	426.10	128.14	3,615.68	57.00	5,691.03
14	Salavan	534.50	192.00	342.34	130.59	1,849.30	50.10	3,098.83
15	Champasak	460.83	625.93	474.04	160.02	1,841.22	654.85	4,216.88
16	Xekong	208.62	286.40	396.46	55.36	238.04	115.30	1,300.18
17	Attapeu	359.90	192.83	58.10	124.51	534.91	60.20	1,330.45
18	Xaysomboun	221.30	616.90	253.10	37.76	134.95	120.20	1,384.21
Total		**7,448.46**	**8,376.69**	**6,403.03**	**2,720.33**	**23,838.11**	**2,810.44**	**51,597.02**

Source: Annual Road Asset Report 2015 issued by MPWT, PTI



Vientiane Mun

% LENGTH OF THE ROAD BETWEEN EACH PROVINCE (km)

Phonsali	815.6																				
Louangnamtha	698.6	347.0																			
Oudomxai	583.6	232.0	115.0																		
Bokeo	893.6	542.0	195.0	310.0																	
Louangphabang	389.6	426.0	309.0	194.0	504.0																
Houaphan	639.6	665.0	548.0	433.0	743.0	512.0															
Xaignabouri	458.6	543.0	426.0	311.0	621.0	117.0	581.0														
Xeingkhouang	389.6	688.0	571.0	456.0	766.0	262.0	250.0	331.0													
Vientiane	70.6	745.0	628.0	513.0	823.0	319.0	569.0	388.0	319.0												
Borkhamxai	150.0	965.6	848.6	733.6	1043.6	539.6	789.6	558.6	539.6	220.6											
Khammouan	353.0	1168.6	1051.6	936.6	1246.6	742.6	992.6	761.6	742.6	423.6	203.0										
Savannakhet	469.0	1284.6	1167.6	1052.6	1362.6	858.6	1108.6	877.6	858.6	539.6	319.0	116.0									
Saravan	682.7	1614.3	1497.4	1382.3	1692.4	1072.3	1322.3	1091.3	1072.3	753.3	532.7	329.7	255.2								
Xekong	814.0	1629.6	1512.6	1397.6	1707.6	1203.6	1453.6	1222.6	1203.6	884.6	664.0	461.0	386.5	86.0							
Champasak	675.0	1490.6	1373.6	1258.6	1568.6	1064.6	1314.6	1083.6	1064.6	745.6	525.0	322.0	247.5	144.3	139.0						
Attapu	838.0	1653.6	1536.6	1421.6	1731.6	1227.6	1477.6	1246.6	1227.6	908.6	688.0	485.0	410.5	164.0	78.0	163.0					
Xaisomboun	254.6	801.0	684.0	569.0	879.0	375.0	381.0	444.0	131.0	184.0	404.6	503.0	619.0	832.7	964.0	825.0	988.0				
Chinese Boder (Lantui)	920.6	185.0	452.05	337.0	647.0	531.0	770.0	648.0	793.0	850.0	1070.6	1273.6	1389.6	1719.3	1734.6	1595.6	1758.6	906.0			
Vietnam Boer (Nameo)	723.6	749.0	632.0	517.0	827.0	596.0	84.0	665.0	334.0	653.0	873.6	1076.6	1192.6	1406.3	1537.6	1398.6	1561.6	465.0	854.0		
Vietnam Boder (Namkan)	726.6	752.0	635.0	520.0	830.0	393.0	277.0	462.0	131.0	450.0	670.6	873.6	989.6	1203.3	1334.6	1195.6	1358.6	262.0	857.0	361.0	
Cambodia Boder	835.0	1650.6	1533.6	1418.6	1728.6	1224.6	1474.6	1234.6	1224.6	905.6	685.0	482.0	407.5	304.3	299.0	160.0	192.0	985.0	1755.6	1558.6	1355.6

Coordinate system:
Projection: UTM
Spheroid: Everest
Vertical Datum: Indian 1954
Grid Distance: 1 km

Scale 1:3,500,000

50 0 50 100 Km

Mapped and printed by CDRI

Figure 1.3.1 Road Network in Lao PDR

CHAPTER 2 DESIGN CONSIDERATIONS

2.1 DESIGN CONCEPT

Design should always be justified based on the following:

- Road function and control of access
- Economic and financial considerations
- Road safety considerations
- Environmental considerations
- Climate change adaptation
- Topography, land use and physical features

The optimum choice will vary with costs for road construction inclusive of road maintenance costs and road user costs. The construction costs will be related to terrain type and selection of pavement construction, whereas road user costs will be related to level and composition of traffic, journey time, vehicle operation and road accident costs. The most economic design will often not involve the use of minimum design standards.

2.2 ROAD FUNCTION AND LEVEL OF ACCESS CONTROL

The function of a particular road as defined by the majority of the road users (long distance traffic, through traffic, local traffic, etc.) has to be taken into account in the determination of design standards for the project and in particular in the selection of the design speed and cross-section.

In particular, careful consideration must be given to the choice of design standards for roads whose major function is to cater for long-distance regional traffic (generally I and II Design Class roads). Because of the long distances involved, traffic tends to move at high speeds on some of these roads, and it may therefore be necessary to adopt higher standards than are warranted by traffic volumes in order to provide an acceptable level of road safety.

Guidelines for the selection of design standards in relation to road function are given in subsequent chapters for a number of design elements.

Depending on the function of a road, various levels of access control should be imposed. All points of access should be carefully considered and planned at the design stage. Access should not be allowed at locations where entering and leaving vehicles will create a hazard, particularly where sight distances are restricted or at points too close to other junctions. The proper location and design of access points may in some cases necessitate adjustments to the initial alignment.

2.3 ECONOMIC CONSIDERATIONS

The relationship between costs and benefits is a measure of justification and often a deciding factor in determining the geometric features of design. The costs include right-of–way (road reserve) construction, maintenance and vehicle operation. The general value resulting from an improved road link includes services and benefits to the country, community and road users. A high geometric design standard is warranted where there are sufficient benefits to road users to justify the additional costs above that of a low design standard. Particularly in the stages of planning the preliminary design when decisions regarding location and general design parameters are made, the relationship between costs and benefits for different alternatives is of great importance.

Preliminary or approximate cost estimates should be made for each alternative plan. All major items should be included.

- Road reserve acquisition
- Site clearance
- Earthworks
- Drainage
- Pavement
- Structures and where significant.
- Relocation of utility services and.
- Cost of maintaining traffic during construction
- Estimated annual costs of maintaining and operating the roadways, roadsides (ditches) and roadway structures.

To complete a financial analysis of alternative plans the total road user costs should be determined for each alternative. Road user costs are the drivers' vehicular operating costs and where relevant, the value of (gained) time. The total of the road user costs for each alternative plan can be a good factor for comparison as they reflect speed, distance (directness) and operating conditions.

2.4 ROAD SAFETY CONSIDERATIONS

Designing safety into roads is one of the main objectives of geometric design. Safety features applicable to a given type of road should be built into the road during its initial construction.

Safety considerations in road design have two different objectives:

(i) To provide design features aimed at preventing accidents, and

(ii) To provide design features aimed at reducing seriousness of accidents when they occur

For the prevention of accidents the following points are of particular importance:

(1) Provision of physical separation between motor vehicles and non-motorized traffic (pedestrians, cyclists, animals), and separate facilities for these two road user types.

(2) Provision of a balanced design, i.e. compatibility between the various design elements.

(3) Avoidance of surprise elements for the drivers, i.e. no abrupt changes in standard, adequate visibility and sight distance conditions and proper phasing of horizontal and vertical alignment.

(4) Avoidance of situations where drivers must make more than one decision at a time.

(5) Provision of a design features that reduce speed differentials between vehicles, e.g. flat grades and speed change lanes.

(6) Proper location and design of junctions with particular emphasis on sufficient sight distances, a minimum of conflict points, and clearly defined and controlled traffic movements.

(7) Proper design, application and location of traffic signs, road markings and other traffic control devices.

(8) Provision of design elements compatible with traffic volumes and type of traffic (long-distance, through, local, etc.)

(9) Provision of proper drainage of the road surface.

Because of the human element involved, some accidents will happen even on roads designed to high safety standards. Therefore, a basic consideration in road design is to minimize injuries and damage when accidents do occur.

Important points in this respect are as follows:

(a) Roadside slopes should be made as flat as feasible, desirably 1:4 or flatter, and the roadside area should be well rounded where slope planes intersect.

(b) Road sign and lighting supports and other utility poles should be located far enough from the carriageway to make them unlikely to be struck by an out-of-control vehicle, or they should have breakaway capability.

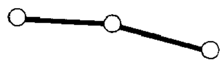
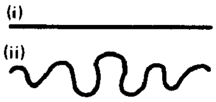
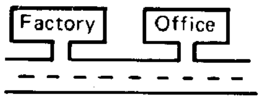
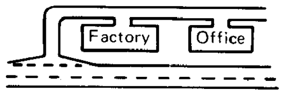
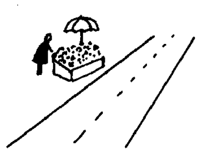
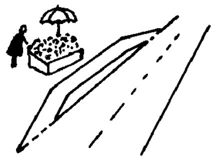
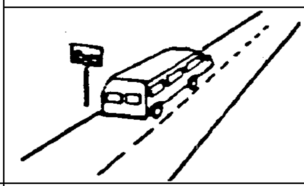
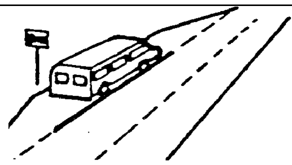
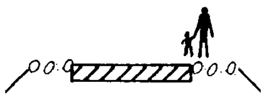
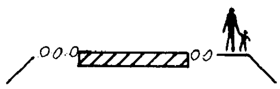
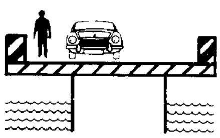
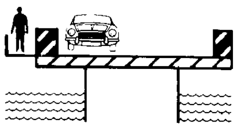
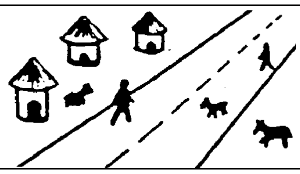
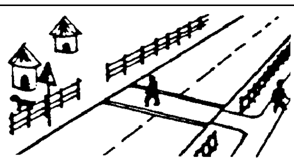
(c) All drainage structures should be designed so that out-of-control vehicle can either pass safely over them or be safely deflected.

(d) Guard rails should be considered only when fill slopes of 1:4 or flatter are not feasible, and the damage caused by hitting the guard rail, would be less serious than damage from leaving the carriageway.

(e) Guard rails should be provided at dangerous obstacles which cannot be removed, and which would cause serious damage if hit by an out-of-control vehicle (e.g. bridge piers and abutments).

Road safety considerations and features are built into the principles, criteria and values for the various design elements given in this Road Design Manual. However, this does not necessarily ensure that the completed road will be of a safe design unless the design engineer is fully aware of, and takes into account, the road safety aspects throughout all phases of the design work.

Table 2.4.1 Example of effect of engineering designing on road safety

	Undesirable	Desirable	Principle Applied
Route Location			Major routes should by-pass towns and villages
Road geometry			Gently curving roads have lowest accident rates
Roadside access			Prohibiting direct frontal access has lowest accident rates
			Use lay-bys or widened shoulders to allow villagers to sell local produce
			Use lay-bys for bus and taxisto avoid restriction and improve visibility
Segregate motorized and non-motorized vehicles, pedestrians and animals			Seal shoulder and provide rumble divider when pedestrian and animal traffic is significant
			Construct protected footways for pedestrians and animals on bridge
			Fence through villages and provide pedestrian crossings

2.5 ENVIRONMENTAL CONSIDERATIONS

No road project is without both positive and negative effects on the environment. The location and design of a road should aim at maximizing the favorable traffic from environmentally sensitive areas, while at the same time minimizing the adverse effects of the project as much as possible.

The following factors, related to the road as a physical feature in the environment, have to be considered in the location and design of a road project.

(1) The preservation of the natural beauty of the countryside.

(2) The preservation of areas and land use of particular value, including:

- National parks and other recreational areas;
- Wildlife and bird sanctuaries;
- Forests and other important natural resources;
- Land of high agricultural value or potential;
- Other land use of great economic or employment importance; and
- Historical sites and other man-made features of outstanding value.

(3) The prevention of soil erosion and sedimentation.

(4) The prevention of health hazards by ponding of water leading to the formation of swamps.

(5) The avoidance or reduction of visual intrusion.

(6) The prevention of undesirable roadside development

Other considerations are mainly related to the operation for the road as a facility for moving traffic and include the following detrimental effects:

- Noise pollution
- Air pollution
- Vibration
- Severance of areas (barrier effect)

These operational effects are mainly a problem of urban roads and traffic, but in some cases also relevant to the design of roads in rural areas.

Some of the adverse environmental effects are fairly easy to quantify (e.g. noise levels and air quality) whilst others are more difficult (e.g. visual impact). In many cases it is necessary to seek the advice and services of other profession to reach a proper evaluation of the problems and establish adequate remedial measures.

2.6 CLIMATE CHANGE ADAPTATION

Disasters resulting from natural hazards in Lao PDR are accompanied by high economic cost and social consequences, particularly for poor and vulnerable rural and urban groups. These hazards include climatic related tropical storms, floods, droughts and landslides as well as earthquakes in the Northern provinces. The continued development of Lao PDR may be considered therefore to be at significant risk from current disaster impact. The increased volatility of and intensity of climate events due to forecasted climate changes can only add to this risk.

The Ministry of Planning and Investment (MPI) executed a project titled Mainstreaming Disaster and Climate Risk Management into Investment Decisions (MDCID) with support from the World Bank. The main goal of the Government of Lao PDR under the MDCID is to mainstream disaster risk management and climate change adaptation into public infrastructure investments, thereby potentially decreasing the vulnerability of the population and national economy to the effects of climate change and natural hazards. The report is recommended for infrastructure development considering climate change adaptation.

The report targets the Ministry of Planning and Investment (MPI) and three key growth sectors: urban planning, road transport and irrigation, which are the main sectors suffering most from natural disaster. The following six distinct but closely interlinked components were implemented during 2013-2015:.

Task 1: Project Management

Task 2: Risk Assessment

Task 3: Mainstreaming Disaster and Climate Risk Management into Investments

Task 4: Institutional Strengthening and Capacity Building

Task 5: Pilot Sub-Projects in two Disaster-prone Provinces

Task 6: Project Monitoring and Evaluation

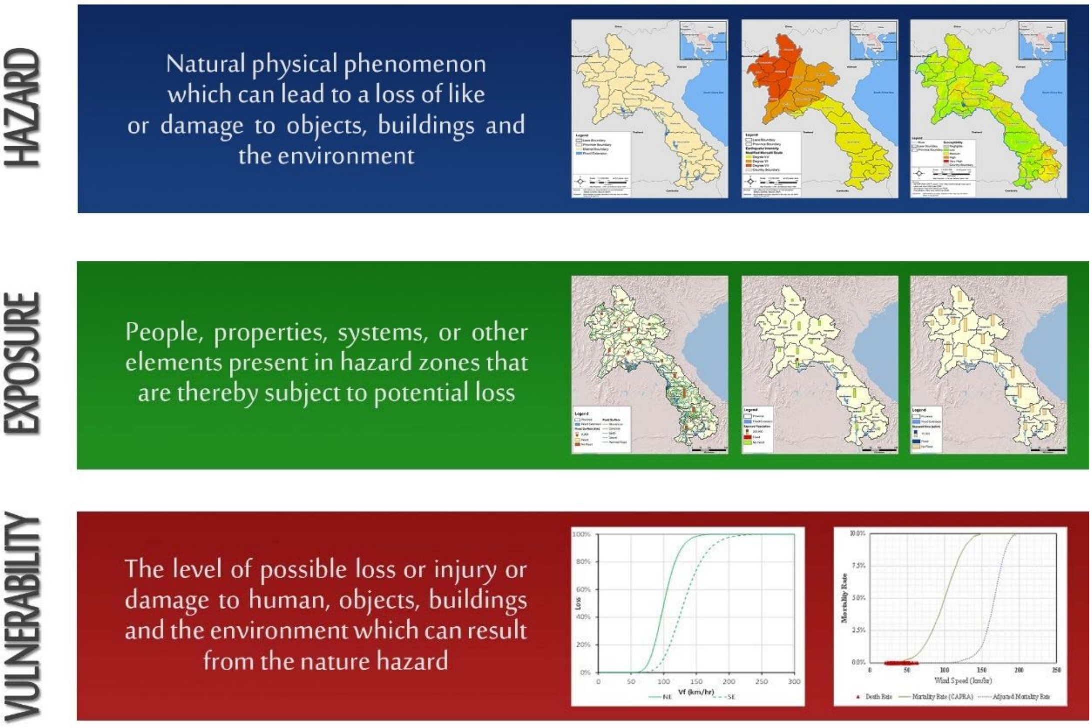
The key products are as follows;

- Technical Report-Sectoral Risk Assessment
- National and Provincial Level Risk Assessment Report of Lao PDR: Volume I: Hazard Assessment and Element at Risk
- National and Provincial Level Risk Assessment Report of Lao PDR: Volume II: Exposure, Vulnerability and Risk Assessment
- District Level Risk Assessment of Lao PDR: Risk Profile of Beng and Bolikhan Districts of Lao PDR
- Risk Assessment Guidelines for the Irrigation Sector
- Risk Assessment Guidelines for the Rural Housing
- Risk Assessment Guidelines for the Transportation Sector
- Landslide Inventory Framework for Critical National and Recommendations for Government of Lao PDR
- Technical Report-Mainstreaming Disaster and Climate Risk Management into Investments
- Lesson Learnt from Good Practices on Resilient Infrastructure and its Relevance to Lao

PDR

- Policy Guidelines
  - Mainstreaming Disaster and Climate Risks in the 8th National Socio-Economic Development Plan (NSEDP) of Lao PDR, 2016-2020
  - Mainstreaming Disaster and Climate Risks Accreditation into the Public Investment Project Process in Lao PDR
  - Mainstreaming Disaster and Climate Risks Accreditation for Monitoring and Evaluation Process in Lao PDR
- Strategic Guidelines
  - Mainstreaming Disaster and Climate Risks in the Rural Housing Sector in Lao PDR
  - Mainstreaming Disaster and Climate Risks in the Road Sector in Lao PDR
  - Mainstreaming Disaster and Climate Risks in the Irrigation Sector in Lao PDR
- Technical Guidelines
  - Resilient Rural Housing Construction in Lao PDR
  - **<u>Resilient Road Infrastructure in Lao PDR</u>**
  - Resilient Irrigation Infrastructure in Lao PDR
  - Irrigation Standards and Specifications in Lao PDR
- Recommendations for Reducing Structural Vulnerability in Housing Sector in Lao PDR
- Training and Capacity Buildings Needs Assessment in Lao PDR
- Training Modules and Delivery
  - Mainstreaming Disaster and Climate Risk Management into the Curriculum of Irrigation Engineering Program of the Faculty of Water Resources Engineering, -NUoL
  - Mainstreaming Disaster and Climate Risk Management into the Curriculum of the Department of Building Technology, Faculty of Architecture, -NUoL
  - Mainstreaming Disaster and Climate Risk Management into the Curriculum of the Department of Road-Bridge and Transport, Faculty of Engineering, -NUoL
  - Training Courses for Training Center for Economic Planning and Management, MPI
  - Training Courses Curriculum for Public Works and Transport Training Center (PTTC), Ministry of Public Works and Transport
  - Training Courses Curriculum for Irrigation Survey and Design Center (ISDC), Department of Irrigation, Ministry of Agriculture and Forestry
- Technical Report-Institutional Strengthening and Capacity Building
- Report of Lesson Learnt from the Pilot Studies
- Technical Report on the Detail of Pilot Studies
- Monitoring and Evaluation Report
- Final Report-Mainstreaming Disaster and Climate Risk Management into Investment Decisions in Lao PDR

The report mentions defining procedures for the identification of natural hazards and risks at national, provincial and local level and initiating their use in the development policies and planning.



Source: RISK ASSESSMENT Procedure & Guideline TRANSPORT SECTOR

Figure 2.6.1 Three Ingredients of Risk Assessment

Major products of the risk assessment are;

- National and Provincial Level Risk Assessment Report of Lao PDR: Volume I: Hazard Assessment and Element at Risk
- National and Provincial Level Risk Assessment Report of Lao PDR: Volume II: Exposure, Vulnerability and Risk Assessment
- Hazard Maps and Risk Profiles of Beng and Bolikhan Districts
- Risk Assessment Guidelines for the Irrigation Sector
- Risk Assessment Guidelines for the Rural Housing
- **Risk Assessment Guidelines for Transportation Sector**
- **Landslide Inventory Framework for Critical National and Provincial Roads**
- Geographical Information System Needs Assessment and Recommendations
- Technical Report on Risk Assessment

Above "Risk Assessment Guidelines for Transportation Sector" is attached in APPENDIX -D.

2.7 TOPOGRAPHY, LAND USE AND PHYSICAL FEATURES

Road design is an exercise in three-dimensional planning whose success will be measured by the efficiency of the road by its appearance and impact upon the adjoining area.

A fundamental consideration in route location and final design is to fit the road harmoniously in to the landscape, with a broad awareness of the character and features of the area through which it passes. This is required to obtain an aesthetically pleasing alignment and necessary in order to obtain the most economical solution and the best possible service to the traversed area with the least damaging effects.

Topography is a major factor in determining the physical location, alignment, gradients, sight distances, cross-section and other design elements of a rural road. In flat terrain the topography may have little influence on location, but it may cause difficulties in some design elements, e.g. drainage. Furthermore, it may encourage monotonous straight alignments with abrupt changes in direction which may be surprising and difficult to recognize by drivers because the topography gives no indication of what to expect. In mountainous terrain the route location and certain design features may be almost entirely governed by the topography.

Geological, soil, climatic and drainage conditions also affect the location and geometrics of a road. Of particular importance is the prevention of soil erosion.

Man-made features such as agricultural, industrial, commercial, residential and recreational developments are important controls for the route location and final design. Care should be taken to avoid unnecessary destruction, demolition or severance of valuable properties.

Information regarding topography, land use and physical features are essential and should be obtained in the early stages of planning and design. In this respect it is necessary to consult with the physical planning authorities in order to co-ordinate the project with existing and proposed land uses to protect the selected route from conflicting development.

CHAPTER 3 ROAD GEOMETRIC DESIGN

3.1 ROAD GEOMETRIC DESIGN PROCESS

The road geometry is a key component for initiating road design, and the process is shown in Figure 3.1.1 with the main features detailed below.

(1) Basic Design Requirements

A road construction project must be consistent with an overall development strategy of the road transport network in the country. A transport plan is normally updated every 25 years. Depending on the future regional development plan or the future road network assessment within the project area, a road should be assigned for its functional classification in the road network, as to whether it is an arterial, collector or local road and as a part of the primary or the secondary road network.

At first, traffic analysis should be done in accordance with the planned development of the area served. An appropriate traffic analysis including trip generation, trip distribution and trip assignment needs to be conducted.

On the basis of the traffic volume, the Road Design Class shall be determined, and the Design Speed shall be determined by terrain and road design class. The road design class and design speed indicated in Table 3.2.1 and Table 3.2.2 respectively are selected.

(2) Design Conditions

As a next step, based on road design class and design speed, the Design Conditions, which are cross section, horizontal alignment and vertical alignment mentioned following sections and summarized in Table 3.3.30, are identified.

In case of design for low traffic volume road, it is required to consider adaptation of "Low Volume Rural Road Environmentally Optimised Design Manual[1]". On the other hand, for development of International roads, which are designated as 'Asian Highway' road, it is required to consider application of ASEAN Highway Standards (Table

[1] Low Volume Rural Road Environmentally Optimised Design Manual (LVRREO design manual) was developed in 2009. This manual is applicable for low traffic volume road, which traffic volume is none of "large truck and large bus" and less than 150 vehicles categorized "car, pick up, mini bus and medium bus". Detail of LVRREO design manual is attached in APPENDIX-E.

3.3.31) and Asian Highway standards (Table 3.3.32).

(3) Design Drawing

Finally the design drawings shall be developed in accordance with the design conditions such as road cross section, horizontal alignments and vertical alignments, which accomplish an optimum road function. Horizontal and vertical alignments have close relation to construction cost. Especially, the structure length (bridges, tunnels) and the scale of earth works are closely referred to its vertical alignment. Therefore, setting of vertical alignment is important. At the design stage, it shall be iterated to check i) the alignment smoothness, ii) the co-ordination between horizontal and vertical alignment, iii) the adequacy of sight distances and iv) to ensure an optimum work volume(cut and fill balance) and workability.

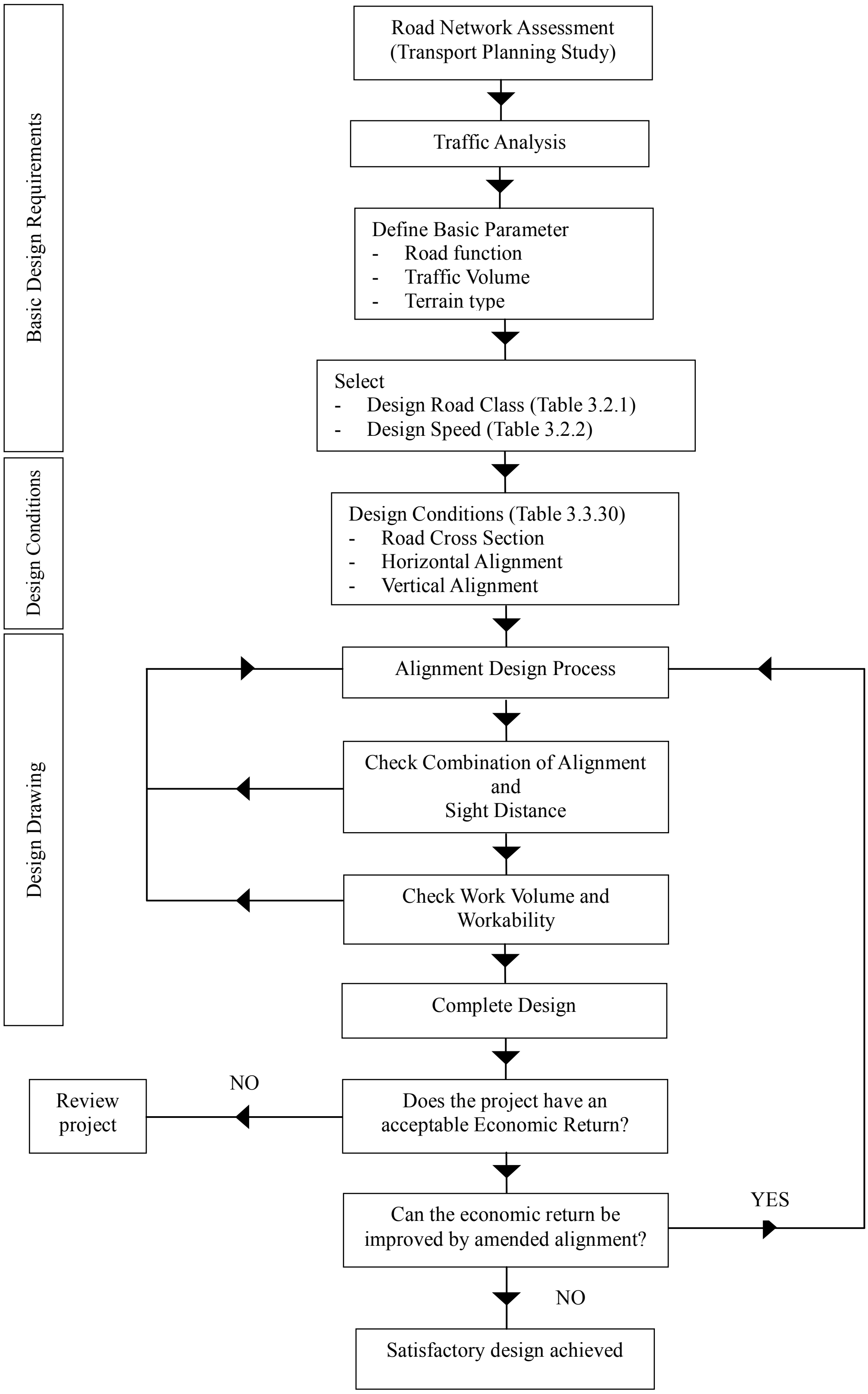


Figure 3.1.1 Road Design Process

3.2 BASIC DESIGN REQUIRMENTS

3.2.1 Road Design Class

The geometric feature is designed dividing into Road Design Class Primary through VII based on Traffic volume as listed in the following table.

Table 3.2.1 Road Design Class

Road Design Class	Primary & I	II	III	IV	V	VI	VII
Traffic Volume (pcu/day)	>8,000	3,000-8,000	1,000-3,000	300-1,000	100-300	50-100	<50

The design of a road should be based upon factual data on the traffic volumes which the road will have to accommodate. The usual design control is Design Volume, which is the estimated traffic volume at a certain future year, the “Design Year” usually 10 years after the year of opening of the new road.

The general measures of vehicular traffic on a road are:

(1) Average Annual Daily Traffic (AADT) the total traffic volume for the year divided by 365.

(2) Average Daily Traffic (ADT) the total traffic volume during a given time period in whole days greater than one day and less than one year divided by the number of days in that time period.

The most adequate design control for low-volume roads is AADT in year 10 after opening, estimated from historical AADT data and the envisaged socio-economic development pattern. For routes with large seasonal variations but still moderate traffic volumes, it may be sufficient to determine the Design Volume in year 10 after opening as ADT during the peak months of the year.

On major roads carrying relatively heavy traffic volumes throughout the year (current AADT>1000), hourly traffic shall be used for determination of the Design Volume. However, it would obviously be wasteful to design the road for the maximum peak hour traffic in the design year since this traffic volume would occur only during one or a very few hours of the year. As a general rule, heavily trafficked rural road should be designed to accommodate the 30th to 50th highest hourly volume in year 10 after opening (DHV=Design Hourly Volume), depending on economic considerations.

When the design volume exceeds 8,000 Passenger Car Unit (PCU) a dual carriageway may be considered, particularly if the road traverses through a typical rural area. Close to major towns a single carriageway road may carry a Design Volume of up to 15,000 pcu.

Although most new roads in Laos will carry traffic volumes far below capacity the designer must be aware of the Basic and Design Capacities of the new road.

3.2.2 Terrain Classification

For design purposes terrain is classified into flat, rolling and mountainous using the following definitions.

Flat: 0-10 meter ground contours per kilometer

Level or gently rolling terrain with largely unrestricted horizontal and vertical alignment

Rolling: 11-25 meter ground contours per kilometer

Rolling terrain with low hills introducing moderate levels of rise and fall with some restrictions on the vertical alignment

Mountainous: Greater than 25 meter ground contours per kilometer

Rugged, hilly and mountainous terrain with substantial restrictions in both horizontal and vertical alignment

Urban Type-I: Urbanized area and relatively free in road location with little problem as regards land acquisition, affected building or other socially sensitive areas

Urban Type-II: Urbanized area and restrictive in road location with problem as regards land acquisition, affected building or other socially sensitive areas

3.2.3 Design Speed

The Design Speed is a speed determined for the design and correlation of the physical features of a road that influence vehicle operation. It is the maximum safe speed that can be maintained over a specified section of road when conditions are so favorable that the design features of the road govern.

Some design elements e.g. curvature super elevation, sight distance and gradient are directly related to and vary appreciably with design speed. Other features like carriageway and shoulder widths are less directly related to design speed, but because they can affect vehicle speeds, higher standards for them should be sued on roads with higher design speeds. Thus nearly all the geometric design elements of the road are affected by the selected design speed.

Economic and environmental considerations should determine the selection of design speed, which is influenced by the following factors:

(a) The classification and function of the road

(b) The nature of the terrain

(c) The density and character of the adjoining land use

(d) The traffic volumes expected to use the road

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

Guiding values for the selections of design speed are given in Table 3.2.2. The following general guidelines should also be considered:

(i) Drivers on long-distance journeys are apt to travel at higher speeds than local traffic; this should be taken into account when selecting design speed for major trunk roads.

(ii) On local roads whose major function is to provide access high speeds are undesirable and the design speed should be selected accordingly.

(iii) Drivers do not usually adjust their speeds to the importance of a road but to the physical limitations and prevailing traffic conditions. Where a difficult location is obvious to the driver, he is more apt to accept a lower speed of operation than where there is no apparent reason for it. A low design speed should not be assumed for a road where the topography is such that drivers are apt to travel at high speeds.

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

(v) Changes in design speed, if required should not be effected abruptly but over sufficient distances to enable drivers to change speed gradually. The change in design speed should not be greater than 25% and the section with the lower geometric standards should be long enough to be clearly recognizable by drivers (not, for example, just one single curve).

It is important to note that the design of a road in accordance with a chosen design speed does not necessarily ensure a safe design. The various design elements have to be combined in a balanced way, avoiding the application of minimum values for one or a way of the elements at a particular location when the other elements are considerably above the minimum requirements. As a general rule speeds selected by the road users are determined more by the horizontal than by the vertical alignment.

Table 3.2.2 General guide values for selection of design speed (km/h)

Terrain	Road classification				
	Primary	I & II	III & IV	V & VI	VII
Level	120	100	80	60	40
Rolling	100	80	60	40	30
Mountainous	80	60	40	Minimum 20	Minimum 20
Urban Type-I	100	60	60	-	-
Urban Type-II	-	40	40	-	-

*The actual operational speed shall be determined in consultation with Lao Road authorities considering local condition. Recommended operational speed is mentioned in Table 3.3.30.

In difficult terrain special conditions will often dictate much lower design speed. Such situations will apply where for example the alignment runs through series of hairpin curves in mountainous terrain and where design speed will probably need to be low as 15 km/h.

3.2.4 Design Vehicles

The physical characteristics of vehicles and the proportions of the various sizes of vehicles using the road system are positive controls in geometric design. The principal vehicle dimensions affecting design are:

- The minimum turning radius
- The path of the inner rear tire, the tread width and the wheel base.

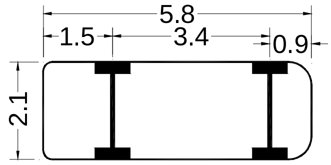
The principal design elements affected are the cross-section of the road, road widening in horizontal curves and junction lay-out. Unit more detailed information on the different vehicle types using the roads in Laos becomes available; the design vehicles shown in subsequent figures can be used as controls in the geometric design.

Table 3.2.3 Design Vehicle Dimentions

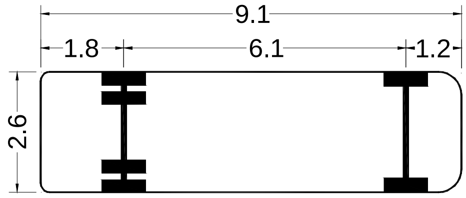
Road Design Class	Length	Width	Height	Front Overhang	WB	Rear Overhang	Min. Turning Radius
Passenger car	5.8	2.1	1.3	0.9	3.4	1.5	7.3
Single unit truck Or bus	9.1	2.6	3.4 - 4.1	1.2	6.1	1.8	12.8
Large bus (B12)	12.2	2.6	3.7	1.8	7.3 +1.1	1.9	12.0
Trailer truck combination (WB12)	15.2	2.6	4.1	1.2	12.2	1.8	12.2
Trailer truck combination (WB15)	16.7	2.6	4.1	0.9	15.2	0.6	13.7
Semi Trailer (WB19)	21.0	2.6	4.1	1.2	5.9	1.4	13.7

Design Vehicles

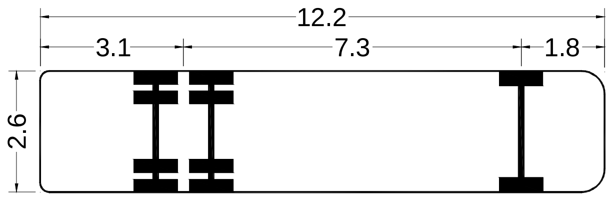
P
Passenger car
Or pickup truck



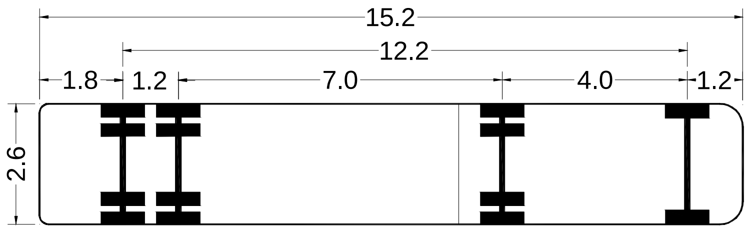
SU
Single unit truck
Or bus



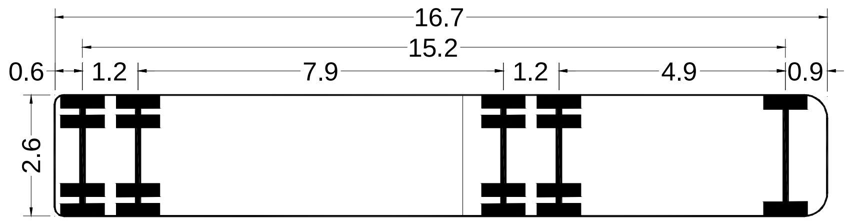
B12
Large bus



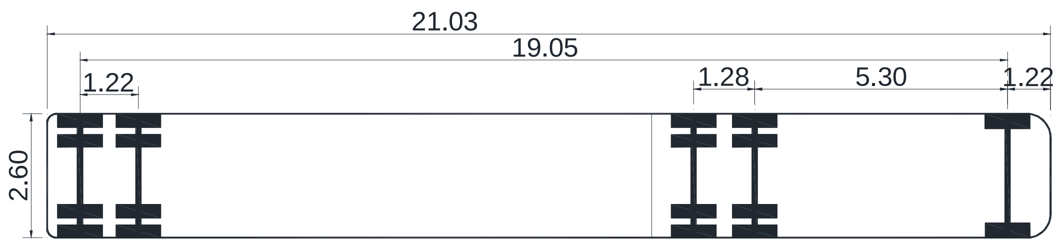
WB12
Trailer-truck
Combination



WB15
Trailer-truck
Combination

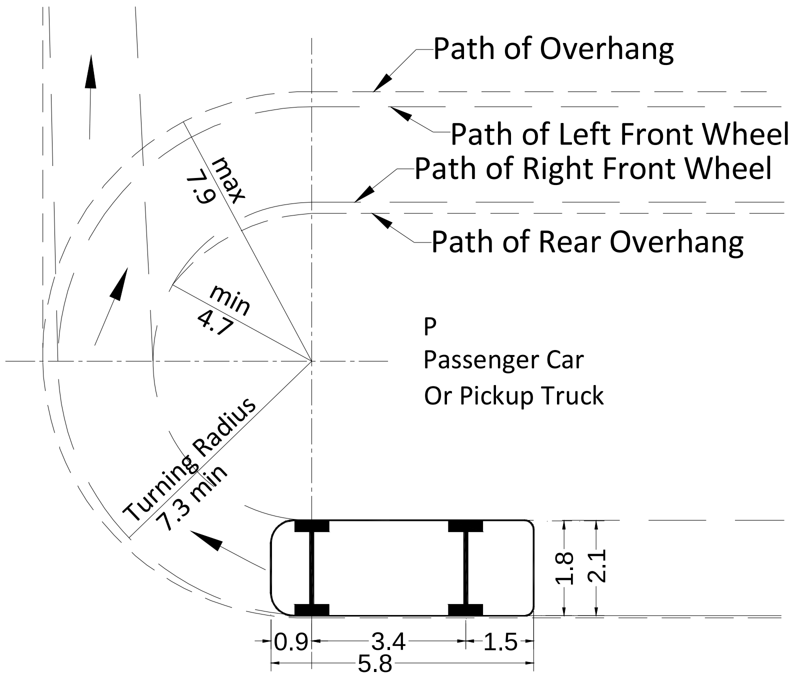


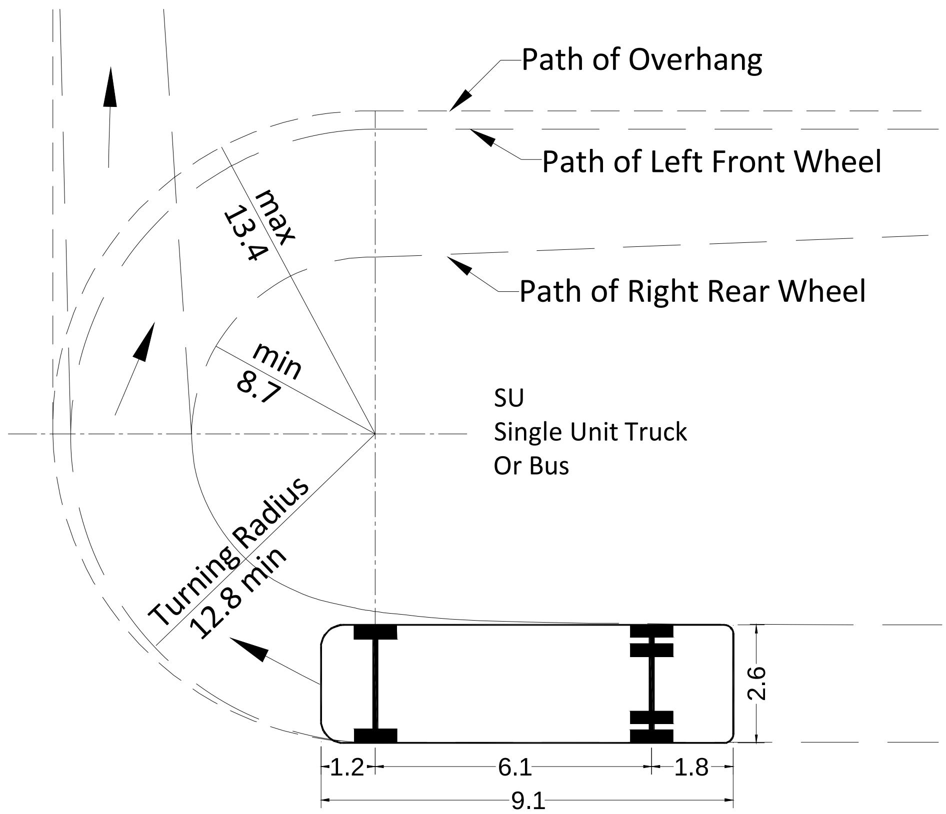
WB19
Semitrailer

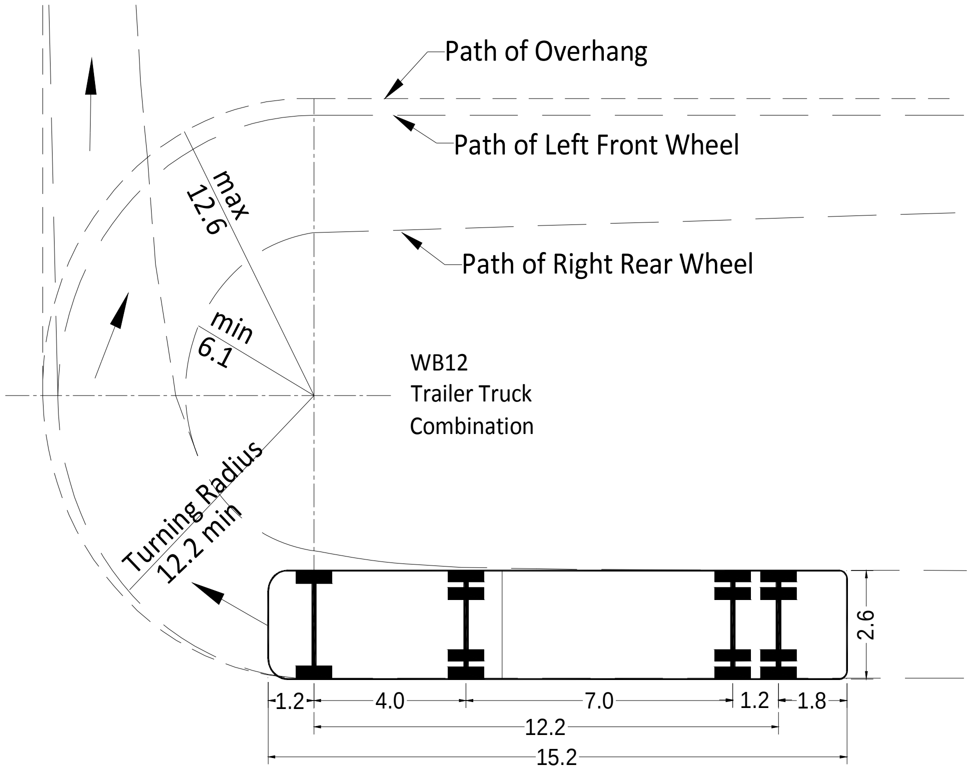


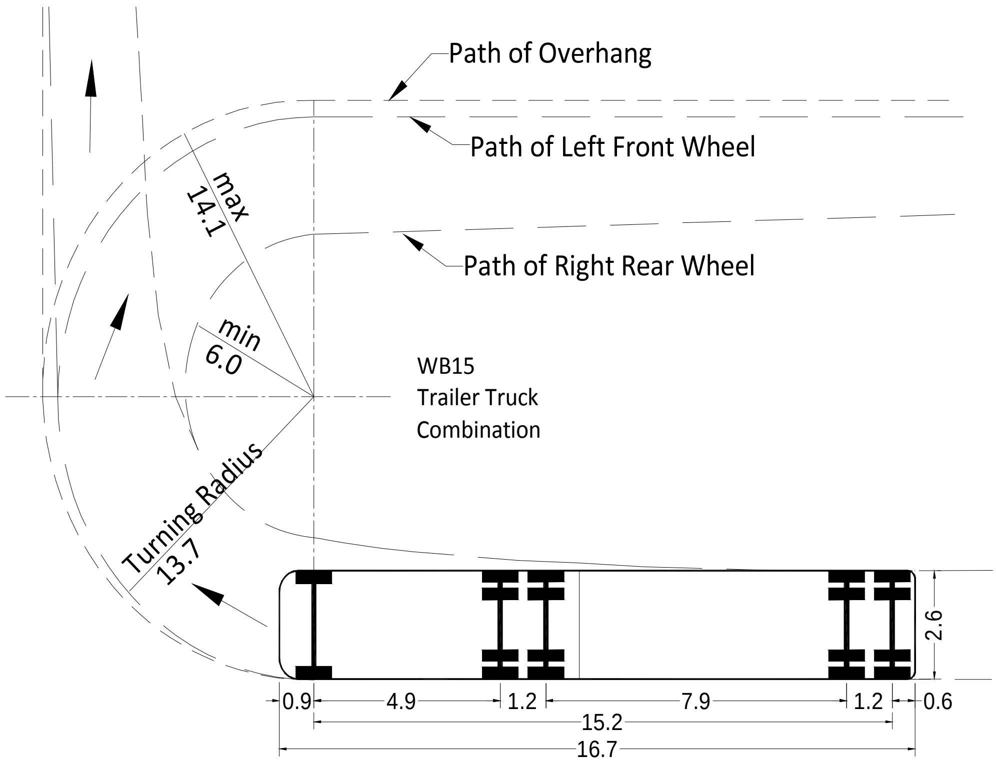
All dimensions in meters

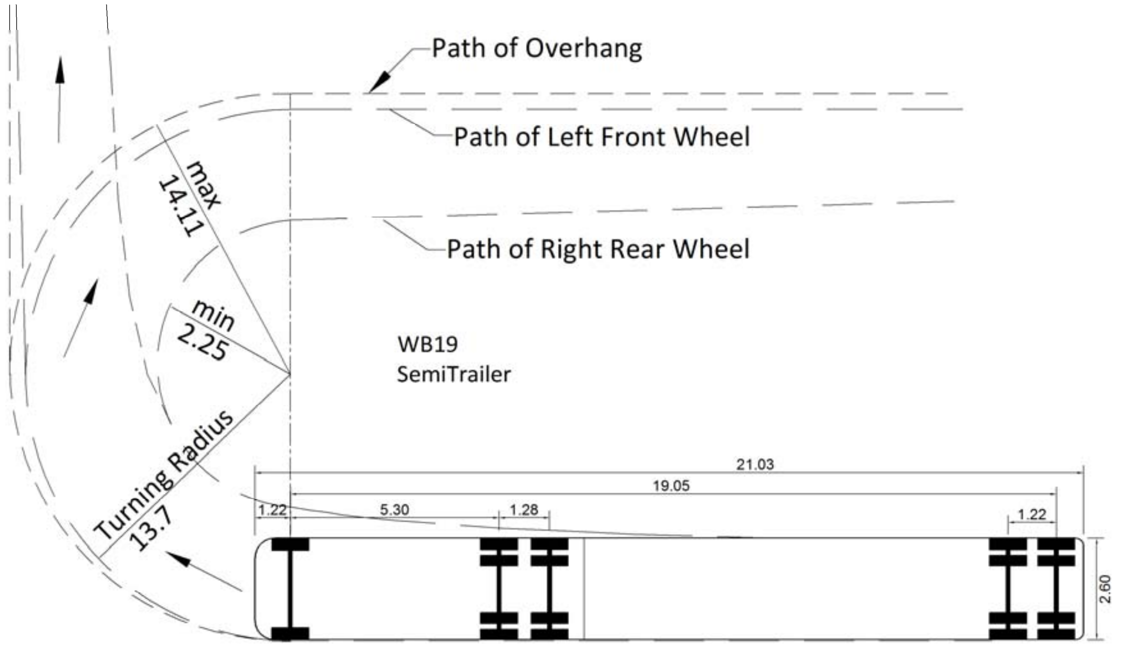
Minimum Vehicle Turning Radius and Turning Path






Path of Overhang
Path of Left Front Wheel
Path of Right Rear Wheel
max
12.6
min
6.1
WB12
Trailer Truck
Combination
Turning Radius
12.2 min
2.6
1.2
4.0
7.0
1.2
1.8
12.2
15.2


Path of Overhang
Path of Left Front Wheel
Path of Right Rear Wheel
max
14.1
min
6.0
WB15
Trailer Truck
Combination
Turning Radius
13.7
2.6
0.9
4.9
1.2
7.9
1.2
0.6
15.2
16.7


Path of Overhang
Path of Left Front Wheel
Path of Right Rear Wheel
max
14.11
min
2.25
WB19
SemiTrailer
Turning Radius
13.7
21.03
19.05
1.22
5.30
1.28
1.22
2.60

3.2.5 Axle Loading

According to Department of Transport (DOT) and Department of Roads (DOR), at present these maximum axle loadings are as follows:

On NR3, NR9, NR4

Single axle, 4-wheel	11 tons
Tandem axle (per axle)	10 tons
Triple axle (per axle)	8.2 tons
Single axle, 2-wheel	7 tons

On other National Roads

Single axle, 4-wheel	9.1 tons
Tandem axle (per axle)	8.2 tons
Triple axle (per axle)	6.8 tons
Single axle, 2-wheel	6.8 tons

In the wet season (1 June to 30 November) the axle weight limit is reduced by 20% on unpaved roads.

3.3 DESIGN CONDITIONS

3.3.1 Cross Section

(1) General

The major geometric design elements constituting the cross-section are:

- The carriageway
- The shoulders and the ditches and
- For dual carriageway roads, the central reserve.

The carriageway includes the travelled way, any auxiliary lanes such as acceleration and deceleration lanes, climbing lanes, passing and bus bays and lay-bys.

Also related to the cross-section are cycle tracks and footpaths. Many roads particularly those providing access as their major function carry a considerable number of pedestrians and cyclists who make use of the shoulders and carriageway edges because separate facilities for them are not provided. From a traffic safety point of view this is an undesirable situation and cycle tracks and/or footpaths should be included in the cross-section where appropriate.

The selection of standards for the cross-section is dependent on Road Design Class and Design Speed described in Section 3.2 and 3.2.3.

Lane and shoulder widths (ditch) slopes etc. should be adjusted to traffic requirements (traffic volume, traffic composition, vehicle speeds) and characteristics of the terrain.

This means that the cross-section may vary over a particular route because the controlling factors are varying. The basic requirements are, however, that changes in cross-section standards be uniform within each sub-section of the route and that any changes of the cross-section shall be effected gradually and logically over a transition length. Abrupt or isolated changes in cross-section standard lead to increased hazards and reduced traffic capacity and complicate construction operations.

The width of bridges and culverts shall be sufficient to carry the full carriageway width that is the traffic lanes and preferably plus the shoulders. Economic considerations and special bridges in low traffic user areas may however warrant reduction of the bridge cross section to the carriageway width only plus a minimum clearance distance. Section 3.3.1(11) specifies the cross-section standards for bridges for the Road Design Classes and for Special Bridges.

In specific cases it may be economic to select a stage-construction i.e. to construct a road to a gravel standard in a first stage and improve the road to a bitumen standard when warranted by increased traffic. The conversion from gravel to bitumen has to be taken into account in the selection of the cross section.

(2) The Choice of the Cross-Section

The choice of the cross-section elements depends on a number of factors, the most important of which are:

1. The traffic volume which the road will have to accommodate
2. The selected design speed
3. The road function, i.e. the predominant type of traffic that the road serves.

For the selected cross-section, the most appropriate type of side ditches and cut-off ditches are to be chosen in accordance with the guidelines given in Chap. V.

Elements complementary to the normal cross-section include bus bays.

Pavement widening required in horizontal curve is covered in section 3.3.2(3)8). Climbing lanes are dealt with in section 3.3.2(6). The design of speed change (deceleration and acceleration) lanes is included in section 3.4.5.

Bus bays are provided in order to prevent vehicles from stopping and standing on the carriageway. The sitting of bus bays and parking bays will depend greatly upon local conditions. The long established habits of public service and other vehicle drivers and their passengers shall not be disregarded. Bus bays shall not be sited where visibility is restricted.

(3) Road Reserve

The boundaries of the land reserved for various categories of road in the LAO PDR areas are defined as follows in the Road Law, Article 22:

A. National road: 25 m measure on each side from center line of the road.

B. Provincial road: 15 m measure on each side from center line of the road.

C. District road: 10 m measured on each side from center line of the road.

D. Rural road: 5 m measured on each side from center line of the road.

E. Urban road: compliance with urban master plan. For specific road shall follow its individual special of each road.

For the reservation width of new national road to be constructed in the future shall have at least 40 m to 60 m measured on each side from center line of the road.

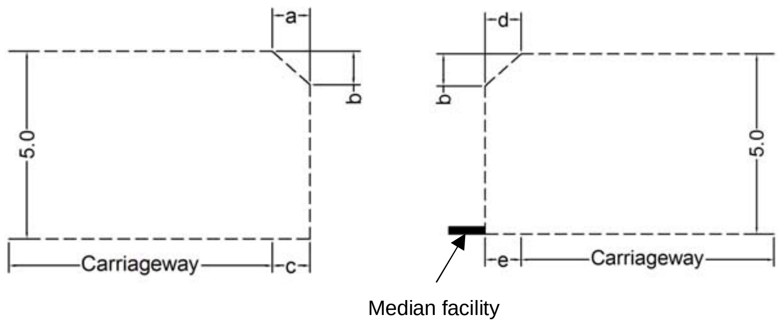
Reservation width for each type of road in mountainous area, steep area, the Ministry of Public Works and Transport will be specified based on the situation of the terrain.

In case of intersection design, it is required to check adequate reservation area considering plan and profile.

(4) Road Clearances

The minimum headroom or clearance under bridges and overhead clearance for cables for national rural roads shall be 5.0m for All Road Class, which is mentioned in the Road Law, Article 23.

The minimum horizontal (lateral) clearances between roadside objects and the edge of the shoulder shall normally be 1.5 meters. This standard can in exceptional cases be reduced to 0.5 meter.



Note:

a and c: Width of shoulder connected with carriageway(for shoulder where on-street facilities are provided, shoulder width minus values required for on street facilities), provided that the value exceeds 1m, "a" and "c" shall be 1m.

b: Value after subtracting height of design vehicle from 5m.

d: Value same as lateral margin

(Design speed 120, 100km/h: 0.75m, Design speed 80 – 20km/h: 0.5m)

(5) Cross Fall

Undivided pavements with two or more lanes on tangents or flat curves shall have a high point, or crown, along the centerline with uniform downward slopes towards each edge in order to facilitate surface water run-off and to prevent mud from the verge from spreading over the carriageway. This downward slope is termed cross fall.

For paved roads, the minimum Cross fall shall be 2.0 – 3.0% and for unpaved roads shall be 3.0 – 4.0%. Auxiliary lanes shall have a cross fall of the same direction and rate as the adjacent lane. The cross fall on paved shoulders shall in all cases be minimum 3.0% and the cross fall on unpaved shoulders shall in all cases be minimum 4.0%.

In curves the carriageway shall be super elevated in order to counter-balance part of the centrifugal acceleration. The relationships between design speed, curvature and super elevation are described in section 3.3.2(3)4)

(6) Lane Widths

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

Table 3.3.1 indicates the lane widths that are to be used for the various road classifications.

Table 3.3.1 Lane Width

Road Design Class	Primary				I					II				
Terrain	F	R	M	U Type -I	F	R	M	U Type -I	U Type -II	F	R	M	U Type -I	U Type -II
Design Speed	120	100	80	80	100	80	60	60	40	100	80	60	60	40
Lane Width (m)	3.5 [3.75]	3.5 [3.75]	3.5	3.5	3.5 [3.75]	3.5 [3.75]	3.5	3.5	3.5	3.5 [3.75]	3.5 [3.75]	3.5	3.5	3.5

Road Design Class	III					IV					V		
Terrain	F	R	M	U Type -I	U Type -II	F	R	M	U Type -I	U Type -II	F	R	M
Design Speed	80	60	40	60	40	80	60	40	60	40	60	40	20
Lane Width (m)	3.5	3.5	3.0	3.0	3.0	3.0					2.75	2.75	2.5

Road Design Class	VI			VII		
Terrain	F	R	M	F	R	M
Design Speed	60	40	20	40	30	20
Lane Width (m)	3.5			3.5		

[ ] : Desirable value

(7) Shoulder Widths

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

Table 3.3.2 indicates the shoulder widths that are to be used for the various road classifications.

Table 3.3.2 Shoulder Width

Road Design Class	Primary				I					II				
Terrain	F	R	M	U Type -I	F	R	M	U Type -I	U Type -II	F	R	M	U Type -I	U Type -II
Design Speed	120	100	80	80	100	80	60	60	40	100	80	60	60	40
Shoulder (m)	3.0	3.0	2.5	2.0	3.0 (0.75)*	3.0 (0.5)*	2.5	1.5 (0.5)*	1.5 (0.5)*	2.5 (0.75)*	2.5 (0.5)*	2.0	1.5 (0.5)*	1.5 (0.5)*

Road Design Class	III					IV					V		
Terrain	F	R	M	U Type -I	U Type -II	F	R	M	U Type -I	U Type -II	F	R	M
Design Speed	80	60	40	60	40	80	60	40	60	40	60	40	20
Shoulder (m)	2.0	2.0	1.0	1.0 (0.5)	1.0 (0.5)	1.0	1.0	0.5	0.5	0.5	0.75	0.75	0.5

Road Design Class	VI			VII		
Terrain	F	R	M	F	R	M
Design Speed	60	40	20	40	30	20
Shoulder (m)	1.5 **	1.5 **	1.25 **	1.25 **	1.25 **	1.0 **

( ) : Minimum value

* In case of installation of island for service road, width can be reduced to 0.75, 0.5m

** Unpaved shoulder

(8) Median Width

A median is a highly desirable element on all roads carrying four or more lanes and should be provided wherever possible. The principal functions of a median are to provide the desired freedom from the interference of opposing traffic, to provide a recovery area for out-of-control vehicles, to provide for speed changes and storage of right-turning and U-turning vehicles and to provide for future lanes. Table 3.3.3 indicates the median widths.

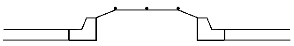
Table 3.3.3 Median Width

Road Design Class	Primary				I				
Terrain	F	R	M	U Type -I	F	R	M	U Type -I	U Type -II
Design Speed	120	100	80	100	100	80	60	60	40
Median Width at road center (m)	4.0 (2.0)	4.0 (2.0)	3.0 (1.5)	3.0 (2.0)	3.0 (2.0)	3.0 (1.5)	2.5 (1.5)	2.5 (1.5)	2.5 (1.5)
Lateral Margin (m)	0.75	0.75	0.5	0.5	0.5	0.5	0.5	0.5	0.5

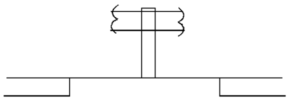
Road Design Class	II					III				
Terrain	F	R	M	U Type -I	U Type -II	F	R	M	U Type -I	U Type -II
Design Speed	100	80	60	60	40	80	60	40	60	40
Median Width at road center (m)	- [0.75]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]
Lateral Margin (m)	0.75	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5

Road Design Class	IV				
Terrain	F	R	M	U Type -I	U Type -II
Design Speed	80	60	40	60	40
Median Width at road center (m)	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]
Lateral Margin (m)	0.5	0.5	0.5	0.5	0.5

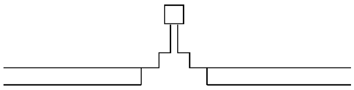
( ) : Minimum value
[ ] : Desirable value
Note: Median width of Minimum value for 120, 100km/h is set based on concrete barrier width 0.5m and lateral margin 0.75m.
Median width of Minimum value for 80, 60, 40km/h is set based on concrete barrier width 0.5m and lateral margin 0.5m.
For 2 lane road(Class II, III, IV), paved stripe median is applied as a desirable.



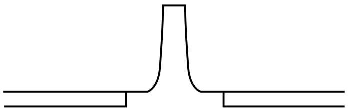
(a) Curb



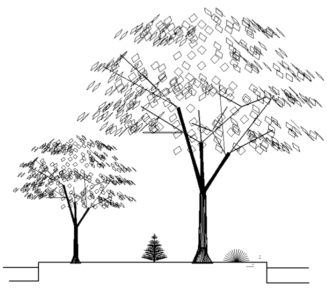
(b) Guardrail



(c) Barrier



(d) Concrete Barrier



(e) Green Stripe

(f) Stripe Median

Figure 3.3.1 Type of Median Facility

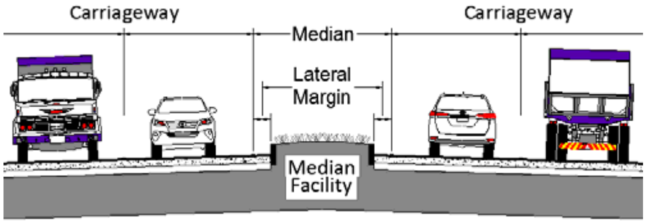


Figure 3.3.2 Median and Lateral Margin

In case of continuously divided road, it is assumed difficulty of traffic due to traffic accident or construction works. In such a case, it is effective to utilize the opposite line in order to promptly handle accidents and rescue activities. In order to utilize the opposite lane in emergency, it is necessary to provide an opening of median to enable movement between the lanes.

Since it is possible to move to the opposite lane side from at grade intersection, the openings of median are not provided as much as possible. In case of unavoidably install of opening of median, it is standard to install about every 500m, but it is necessary to consider traffic safety.

For installation of left turn lane, geometric feature is referred to 3.4.5(3).

(9) Service Road Widths and Island Divided between Paved Shoulder and Service Road

Service road is important to divide long distance vehicles and short distance vehicles in order to keep safety and avoid reduction of capacity.

Table 3.3.4 indicates the service road widths and island width to divide main road and service road that are to be used for Road Design Class I and II.

Table 3.3.4 Service Road Width and Island Width

Road Design Class	I					II				
Terrain	F	R	M	U Type -I	U Type -II	F	R	M	U Type -I	U Type -II
Design Speed	100	80	60	60	40	100	80	60	60	40
Island divided between paved shoulder and service road	3.0	3.0	-	3.0	3.0	3.0	3.0	-	3.0	3.0
Service road for low traffic (m)	3.0	3.0	-	3.0	3.0	3.0	3.0	-	3.0	3.0

(10) Sidewalk

Sidewalks shall be provided on urban area (excluding Road Design Class Primary) where it is required for safe and smooth traffic, except where topographic conditions or any other reasons do not permit such provision. The sidewalk width shall be wider than 2m for urban area.

(11) Standard Bridge Cross Section

1) **Criteria**

1. The minimum cross section width, curb face to curb face or guard rail to guard rail, in these standards, shall be 6.0 meters for minimum dual lane roads for Roads Design Classes IV and V and Mountain region for Road Design Class III.

2. The minimum net height for guard rail shall be 1.0 meter in mountainous region.

3. The minimum cross section width, curb face to curb face or guard rail to guard rail, for Road Design Classes primary, I and II, shall be 1.0 meter greater than the carriage way width.

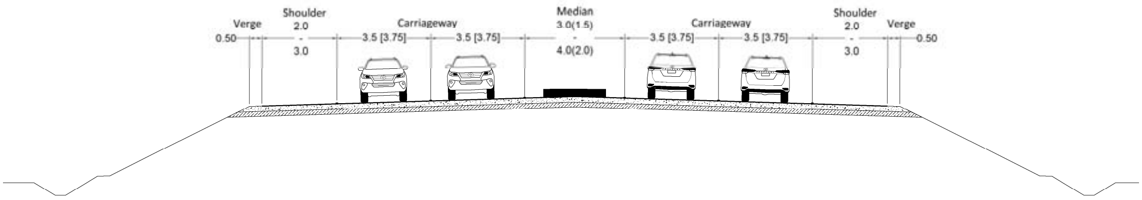
4. Sidewalks or footways for pedestrians, shall be separated away from the bridge carriage way by either temporary or fixed curb stone, fixed rail or railings, and shall be placed where determined needed onto the structure or attached outside the structure or the structure shall be prepared for future installation of sidewalks on either side of the structure. The minimum net width for a footway for pedestrians shall be 1.0 meter in rural areas and 1.5 meters in urban or sub-urban areas.

2) **Special Bridge type (single lane)**

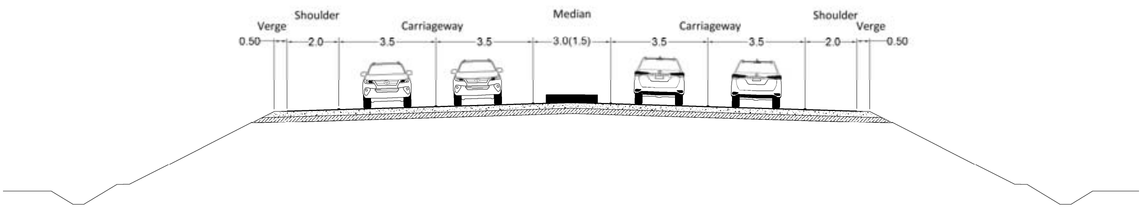
The minimum net cross section width, curb face to curb face, for a single lane bridge shall be 3.5 meters and 4.5 meters between guard rail to guard rail or railing whichever is closest to the carriage way.

(12) Standard Cross-Section

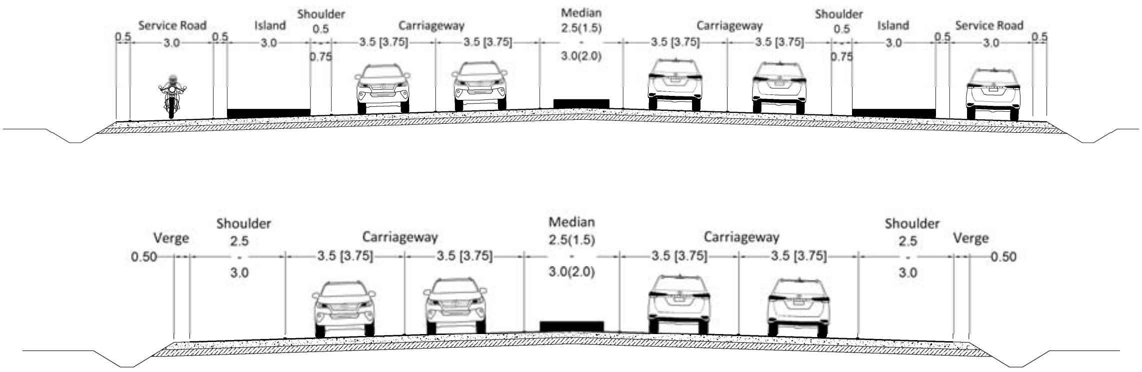
Primary



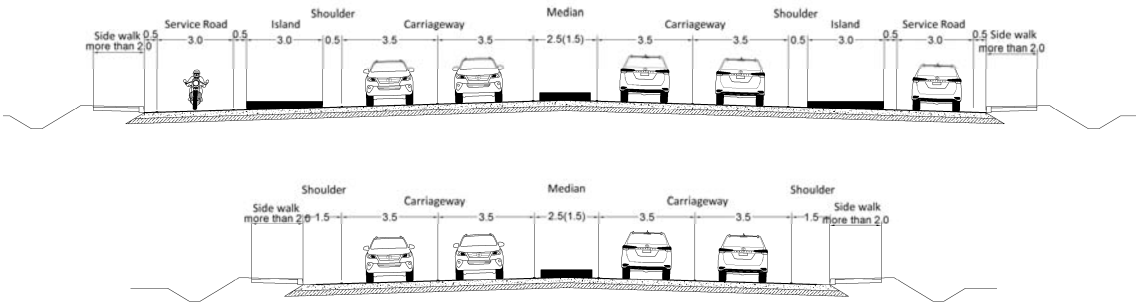
Primary - Urban



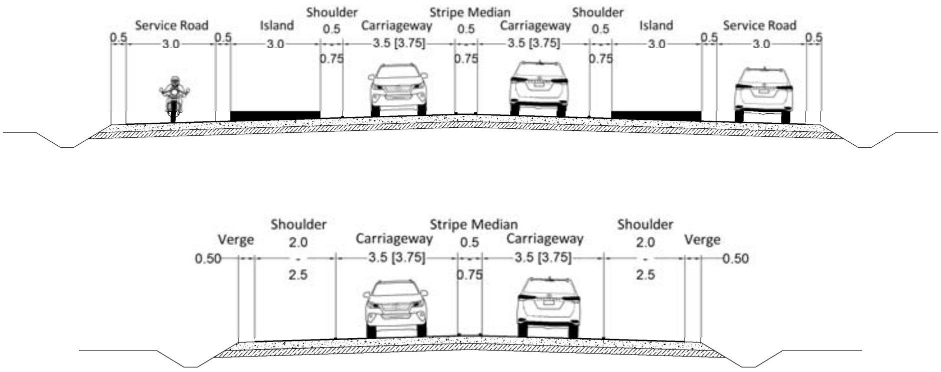
Class I



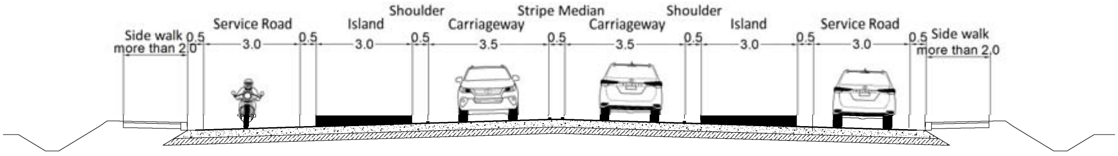
Class I - Urban

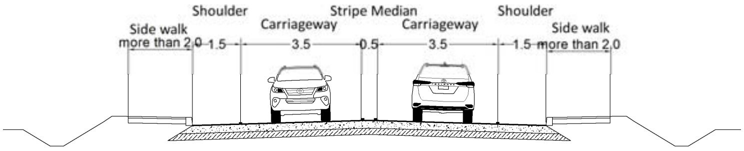


Class II

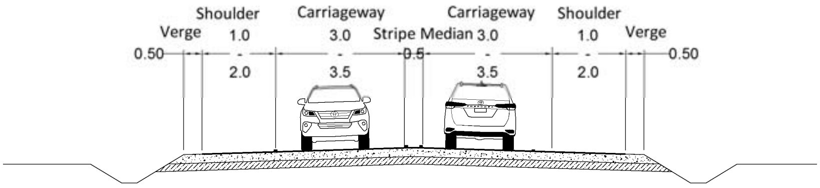


Class II - Urban

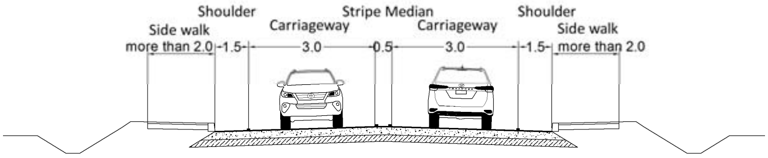




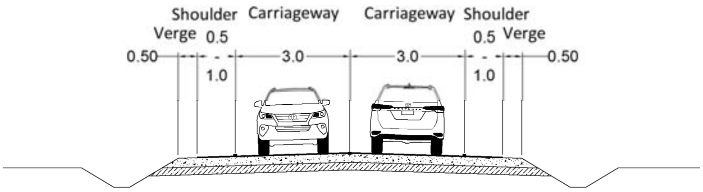
Class III



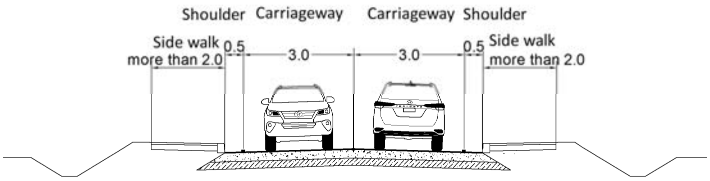
Class III - Urban



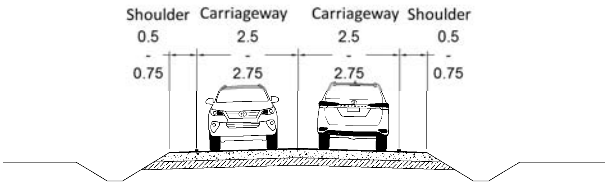
Class IV



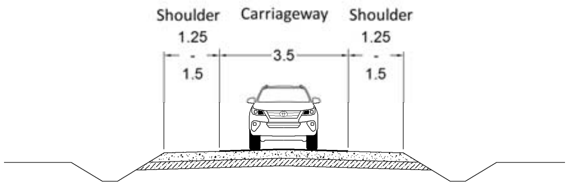
Class IV - Urban



Class V



Class VI



Class VII

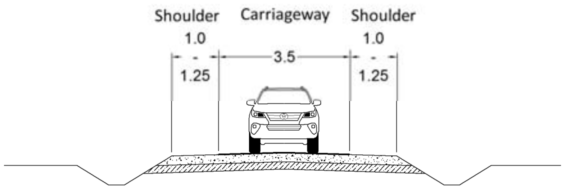


Figure 3.3.3 Typical Cross Section

3.3.2 Alignment

(1) General

The geometric form of a road is a three dimensional alignment which is presented in two projections, the horizontal and the vertical alignment. The horizontal alignment consists of three elements, the straight, the circular curve and the transition curve. The vertical alignment consists of two elements, the straight and the vertical curve (circular or parabolic). Other elements of the alignment are sight distance and super elevation.

The standards and values to be chosen for these alignment elements are dependent on the controls and criteria, all of which have to be considered in the design of the alignment. Furthermore, the horizontal and vertical alignments have to be combined in such a way that a safe and aesthetically pleasing design results. A road alignment is judged by its appearance in three dimensions, and a good three-dimensional design will increase utility and safety and encourage a uniform speed and improve appearance, almost always without additional costs.

Good optical guidance is important for safety and traffic operation. This is achieved when the road alignment ahead has a continuous and flowing appearance, and the course of the carriageway is clear and distinct and can be readily perceived. The guidance is further assisted by proper application of road makings and traffic signs, which should always be treated as integral parts of road design.

It is important to note that the use of the minimum criteria and guide values given in this chapter will not ensure a proper alignment design. It is the task of the design engineer to make the right choice, with due consideration of controls and criteria and of the proper combination of the horizontal and vertical alignments, and to put forward a line that is technically, economically and aesthetically sound.

(2) Sight Distance

Throughout the length of any road, sight distance must be provided that is sufficient to enable drivers to absorb all relevant features of the road and the traffic conditions ahead and take the necessary actions to avoid hazards and proceed in a safe, efficient and orderly way. The following sight distance concepts are applicable to geometric design.

1. Stopping Sight Distance
2. Meeting Sight Distance
3. Passing Sight Distance
4. Visibility Splays

Stopping Sight Distance is applicable to all types of roads. Meeting Sight Distance is applicable to two-way single carriageway roads with insufficient width for passing. Passing Sight Distance is applicable to two-way, 2-lane single carriageway roads and visibility splays should be clearly detailed for all junctions.

The minimum values for sight distances are generally determined by the design speed. However, on road sections where it has to be expected that actual vehicle speeds will be considerably above the design speed, sight distances should be determined be this expected speed rather than the design speed in order to ensure safe operation of vehicles.

1) **Control of Sight Distance**

Available sight distance should be checked throughout the road length in the early stages of the design of the alignment, and any necessary adjustments to the line should be made to meet the minimum requirements for sight distance. The following guide-lines for control of sight distances apply;

1. Available sight distance should be checked separately for each type of sight distance and for each direction of travel.
2. The following values should be used for determination of sight lines;

   <u>Drivers eye height:</u>

   - Passenger car-1.15 m
   - Commercial vehicle-1.80 m

   <u>Object cut off height above road surface:</u>

   - Approaching vehicle-1.15 m
   - Stationary object on road -200 mm
   - Vehicle tail light & stop light -600 mm

3. In horizontal curves it may be necessary to remove obstructions or widen cuttings on the insides of the curves to obtain required sight distance. Required sight areas for various radii and sight distances are given in Figure 3.3.5. Within the sight area the terrain should be the same level as the carriageway, and other obstructions should be removed. In cases where the provision of the sight area requires extensive earthworks or costly removal of obstructions, it may be necessary to adjust the alignment.

4. Sudden reductions of available sight distance should be avoided. Where reductions are necessary; they should be logical in relation to the physical surroundings.

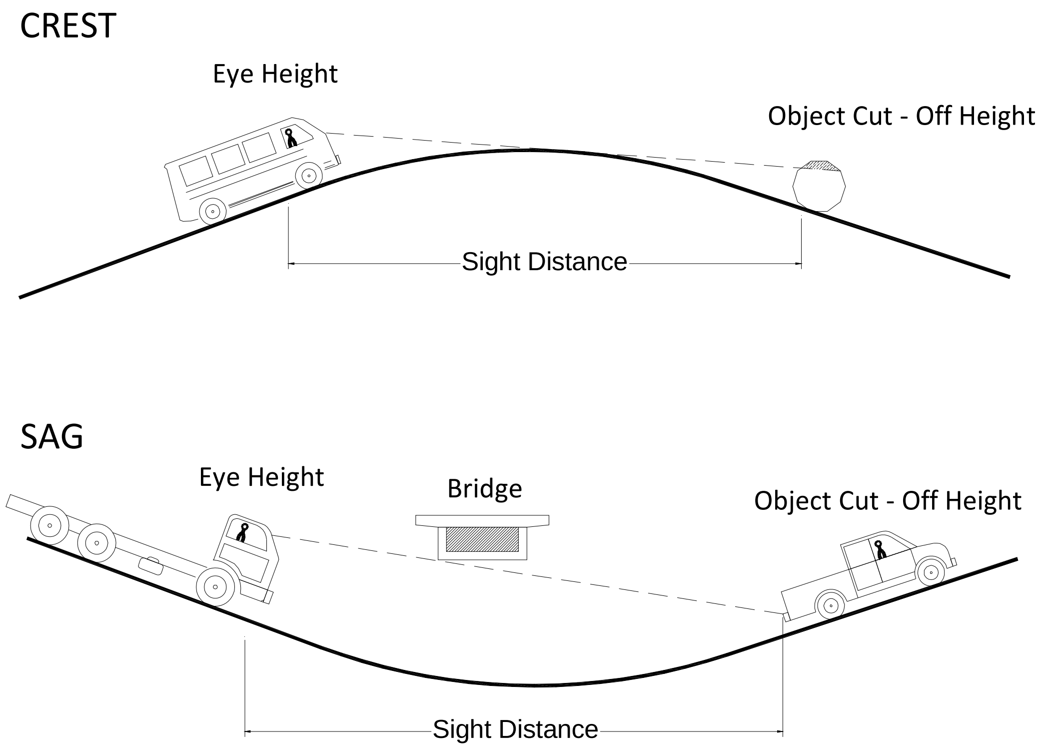


Figure 3.3.4 Assumed Geometrics of Sight Line

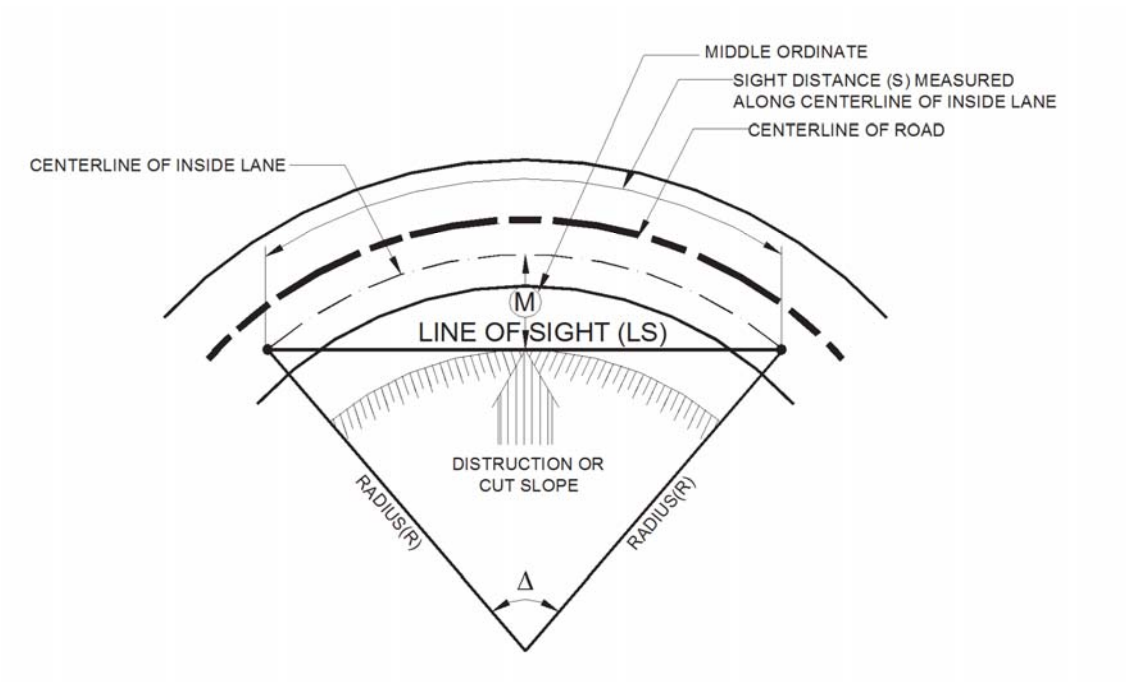


Figure 3.3.5 Sight Distance for Horizontal Curves

2) **Stopping sight distance**

Stopping Sight Distance (SSD) is the distance required by a driver of a vehicle travelling at a given speed to bring his vehicle safely to a stop before reaching an object that becomes visible on the carriageway ahead. It includes the distance travelled during the perception and reaction times and the vehicle braking distance.

Stopping Sight Distance is the minimum sight distance requirement for all types of road and must be provided at every point along the road.

Stopping Sight Distance is the sum of the distance the vehicle travels during the total reaction time of 2.5sec, and the braking distance.

Brake Reaction Distance

$$L = 0.278Vt$$

Braking Distance on Level

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

Stopping Sight Distance

$$SSD = 0.278Vt + 0.039\frac{V^2}{a}$$

Where:

SSD = stopping sight distance, m

V = design speed, km/h

t = brake reaction time, 2.5s

a = deceleration rate, 3.4m/s^2

Note: approximately 90 percent of all drivers decelerate at rates greater than 3.4 m/s^2. Such decelerations are within the driver's capability to stay with in his or her lane and maintain steering control during the braking maneuver on wet surfaces. Therefore, 3.4 m/s^2 is recommended as the deceleration threshold for determining stopping sight distance.

Table 3.3.5 Stopping Sight Distance on Level Roadways

Design speed (km/h)	Brake Reaction Distance (m)	Braking Distance on Level (m)	Stopping Sight Distance	
			Calculated (m)	Design (m)
20	13.9	4.6	18.5	20
30	20.9	10.3	31.2	35
40	27.8	18.4	46.2	50
50	34.8	28.7	63.5	65
60	41.7	41.3	83.0	85
70	48.7	56.2	104.9	105
80	55.6	73.4	129.0	130
90	62.6	92.9	155.5	160
100	69.5	114.7	184.2	185
120	83.4	165.2	248.6	250

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

The distance a vehicle travels while applying brakes to a halt is longer on downhill grades and shorter in uphill. Following equation amended to take into account the effect of grade is:

$$SSD = 0.278Vt + \frac{V^2}{254\left[\left(\frac{a}{9.81}\right) \pm G\right]}$$

Where:

SSD = stopping sight distance, m

V = design speed, km/h

t = brake reaction time, 2.5s

a = deceleration rate, 3.4m/s^2

G = grade, rise/run, m/m

Table 3.3.6 Stopping Sight Distance on Grades

Design speed (km/h)	Stopping Sight Distance					
	Downgrades			Upgrades		
	3%	6%	9%	3%	6%	9%
20	20	20	20	19	18	18
30	32	35	35	31	30	29
40	50	50	53	45	44	43
50	66	70	74	61	59	58
60	87	92	97	80	77	75
70	110	116	124	100	97	93
80	136	144	154	1123	118	114
90	164	174	187	148	141	136
100	194	207	223	174	167	160
120	263	281	304	234	223	214

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

3) **Meeting sight distance**

Meeting Sight Distance is the distance required to enable the drivers of two vehicles travelling in opposite directions on a two-way road with insufficient width for passing to bring their vehicles to a safe stop after becoming visible to each other. It is the sum of the stopping sight distances for the two vehicles as given in Table 3.3.5 plus 10m safety distance.

4) **Passing sight distance**

Passing Sight Distance is the minimum sight distance on two-way single carriageway roads that must be available to enable the driver of one vehicle to pass another vehicle safely without interfering with the speed of an oncoming vehicle travelling at the design speed, should it come into view after the overtaking man oeuvre is started.

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

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

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

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

d_4 = distance traversed by the opposing vehicle

The formula for these components are as indicated below:

$$d_1 = \frac{V_0}{3.6} t_1 + \frac{1}{2} \alpha t_1^{\ 2}$$

Where:

t_1 = time of initial maneuver in sec

a = average acceleration, km/h/s

V_0 = speed of passed vehicle, km/h

$$d_2 = \frac{1}{3.6} V t_2$$

Where:

t_2 = time the passing vehicle occupies left lane, s

V = speed of passing vehicle, km/h

d_3 = safe clearance distance between vehicles at the end of the maneuver, is dependent on ambient speeds

d_4 = distance traversed by the opposing vehicle, which is approximately equal to d_2 less the portion of d_2 whereby the passing vehicle is entering the left lane, estimated at: $2d_2/3$

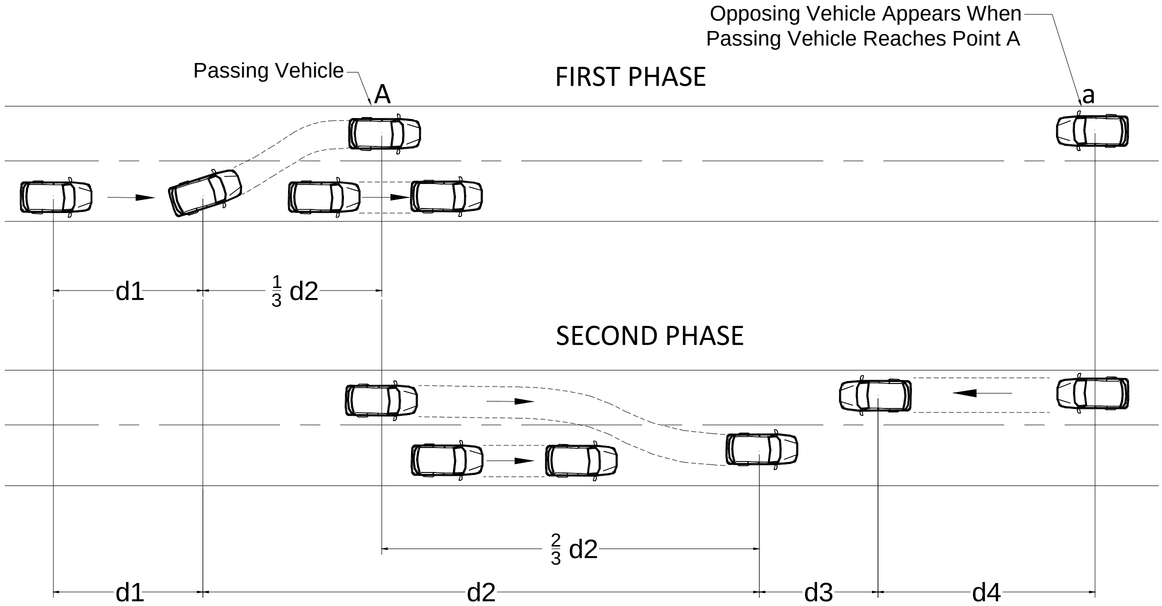
The minimum Passing Sight Distance (PSD) for design is therefore:

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

Table 3.3.7 Passing Sight Distance

Speed of passing vehicle (km/h)	120	100	90	80	70	60	50	40	30	20
Speed of passed vehicle (km/h)	100	80	72.5	65	55	45	37.5	30	20	15
Average acceleration α (m/s2)	0.67	0.66	0.66	0.65	0.64	0.63	0.62	0.61	0.6	0.6
Time of initial maneuver (sec)	4.8	4.5	4.4	4.2	4.0	3.7	3.4	3.1	2.9	2.7
d1	141	107	94	82	65	51	39	29	19	13
Time the passing vehicle occupies left lane (sec)	12.4	11.4	10.9	10.4	10.0	9.5	9.0	8.5	8.0	7.6
d2	413	317	273	231	193	158	125	94	67	42
d3	100	80	70	60	50	40	30	25	20	15
d4	276	211	182	154	129	106	83	63	44	28
Normal Passing Sight Distance (=d1+d2+d3+d4)	930	720	620	530	440	360	280	220	150	100
Reduced Passing Sight Distance (=2/3d2+d3+d4)	660	510	440	370	310	260	200	160	110	80

Note: values of average acceleration, time of initial maneuver, time the passing during which passing vehicles at the end of the maneuver and clerance distance between vehicles at the end of the maneuver are reffered to Road Structure Ordinance in Japan and calculated based on Road Structure Ordinance in Japan.



5) **Sight distance at junctions**

Sight distance requirements at junctions are described in section 3.3.3. These requirements may influence the location of junctions and the road alignment through the junction area.

(3) Horizontal Alignment

The design elements of the horizontal alignment are the straight, the circular curve, the transition curve and the super elevation.

1) **The straight**

From an aesthetic point of view, the straight may often be beneficial in flat country but rarely in rolling or mountainous terrain. However, long straights increase the danger from headlight glare and usually lead to excessive speeding. But overtaking opportunities must be provided at reasonable intervals and straights are often the most appropriate solution.

Short straights between curves in the same sense should be avoided ("broken back" effect). If such straights have to be used, the unfavorable appearance may be improved by the introduction of a sag curve.

The following guide-lines apply for the lengths of straights;

1. Straights should not have lengths greater than (20 * V_D) meters (V_D in km/h)
2. Straights between circular curves following the same direction should have lengths greater than (6 * V_D) meter (V_D in km/h)

2) **Maximum Superelevation**

Vehicles passing around circular curves are forced out of the curves by centrifugal forces. Superelevation is the raising of the edges of a road towards the center of a horizontal curve in order to counteract centrifugal forces.

The normal maximum permissible super elevation rate, e max., is 10%. And in order to reduce influence to residence along the road, the maximum permissible elevation rate is 6% for urban area.

Table 3.3.8 Maximum Superelevetion

Design Speed (km/h)	Maximum Superelevation (%)
Urban area	6
Others	10

Note: Limiting superelevation rate is 8% for Design speed 20km/h and 30km/h.

3) **Minimum Curve Radius**

For calculation of the minimum horizontal radius, R min, for a particular design speed, the following equation shall be used:

$$R\min = \frac{{V_D}^2}{127(e + f_k)}$$

V_D = Design Speed (km/h)

e = Cross fall of road or the maximum super elevation

f_k = Side friction coefficient

According to AASHTO, side friction coefficient is used from 0.09 at design speed 120km/h at 0.35 on design speed 20km/h. side friction coefficient on each design speed are shown in Table 3.3.9.

Table 3.3.9 Side Friction Coefficient

Design Speed V_D (Km/h)	20	30	40	50	60	70	80	90	100	120
Side Friction Coefficient	0.35	0.28	0.23	0.19	0.17	0.15	0.14	0.13	0.12	0.09

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Compared between AASHTO, Asian Highway Standard and Road Structure Ordinance in Japan, for minimum radii of high speed range, the value of AASHTO is relatively within safety side. On the other hand, for low speed range, the value of AASHTO is relatively small. Therefore, in case of the design applied low speed, the engineer should carefully decide application of minimum radii.

Table 3.3.10 Comparison of Minimum Radii

Design speed	20	30	40	50	60	70	80	90	100	120
AASHTO(2011)	7	19	38	68	105	154	210	277	358	597
Asian Highway Standards	-	30	50	80	115	-	210	-	350	520
Road Structure Ordinance in Japan	15	30	50	80	120	-	230	-	380	570

The values of minimum Radii which shall be applied to curves in geometric design are shown in Table 3.3.11.

Table 3.3.11 Minimum Radius of Horizontal Curves with Maximum e (10%)

Design Speed (km/h)	Maximum e (%)	Maximum f	Total (e/100+f)	Calculated Radius (m)	Rounded Radius (m)
20	10.0%	0.35	0.45	7.0	7
30	10.0%	0.28	0.38	18.6	19
40	10.0%	0.23	0.33	38.2	38
50	10.0%	0.19	0.29	67.9	68
60	10.0%	0.17	0.27	105.0	105
70	10.0%	0.15	0.25	154.3	154
80	10.0%	0.14	0.24	210.0	210
90	10.0%	0.13	0.23	277.3	277
100	10.0%	0.12	0.22	357.9	358
120	10.0%	0.09	0.19	596.8	597

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Short curves between long straights have an optical appearance for a kink and should be avoided. Where the change of direction between two straights (intersection angle) is 8 degrees or less, the length of the horizontal curve should be at least 200m to avoid the impression of a broken line.

The horizontal curvature over a particular road section should be as consistent as possible. The safety of a winding alignment is usually not seriously affected by a smaller curve, while isolated sharp curves on an otherwise flowing alignment are dangerous. When a reduction in the standard of horizontal curvature is necessary, it should be affected gradually and it is recommended that the radii of consecutive curves should be within the limits given in Figure 3.3.6. Particular care should be taken to avoid sharp curves at the ends of long straights. The following guide-lines shall be applied:

If $L < 500$ m, then $R < L(m)$

If $L > 500$m, then $R > 500$ m

Where: L = length of straight, and

R = horizontal radius

Very gentle curvature should applied on high, long fills as in the absence of cut slopes, shrubs. Tree, etc. above the roadway, it is difficult for drivers to perceive the extent of curvature and adjust to the conditions. The same principle applies to the absence of physical features in very flat terrain, where small radii and short curves should be avoided.

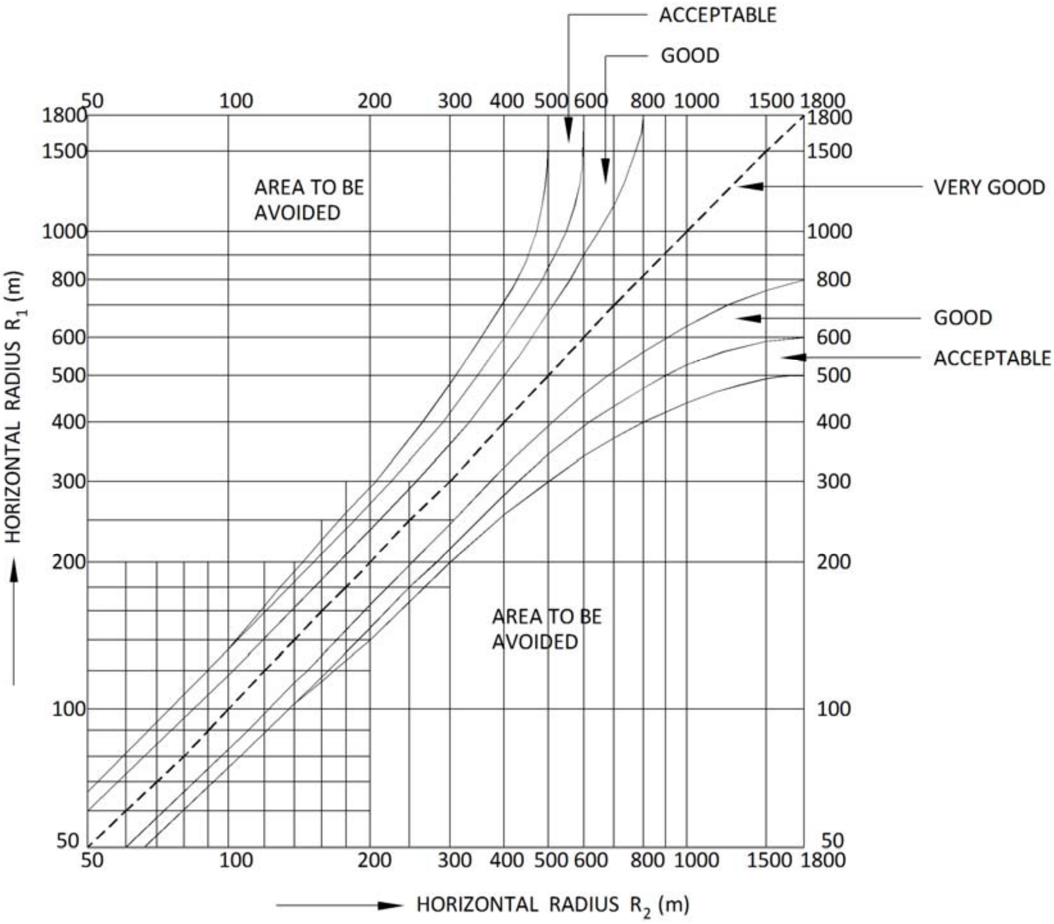


Figure 3.3.6 Acceptable combinations of Radius for consecutive curves

4) **Design of Superelevation**

Vehicle passing around circular curves are forced out of the curves by centrifugal forces. Superelevation is the raising of the edges of a road towards the center of a horizontal curve in order to counteract centrifugal forces. Super elevation rates for radii larger than R min are given in Table 3.3.12 - Table 3.3.15 for different design speeds.

Table 3.3.12 Minimum Radius for Design Superelavation Rates, Design Speed and emax=4%

e(%)	Vd=20 km/h	Vd=30 km/h	Vd=40 km/h	Vd=50 km/h	Vd=60 km/h	Vd=70 km/h	Vd=80 km/h	Vd=90 km/h	Vd=100 km/h
	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)
NC	163	371	679	951	1310	1740	2170	2640	3250
RC	102	237	411	632	877	1180	1490	1830	2260
2.2	75	187	363	534	749	1020	1290	1590	1980
2.4	51	132	273	435	626	865	1110	1390	1730
2.6	38	99	209	345	508	720	944	1200	1510
2.8	30	79	167	283	422	605	802	1030	1320
3	24	64	137	236	356	516	690	893	1150
3.2	20	54	114	199	303	443	597	779	1010
3.4	17	45	96	170	260	382	518	680	879
3.6	14	38	81	144	222	329	448	591	767
3.8	12	31	6	121	187	278	381	505	658
4	8	22	47	86	135	203	280	375	492

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Notes:

e = Super elevation rate m/m

V = Design speed, km/h

RC = Remove crown and super elevate at normal crown rate

NC = Normal crown super elevation not required (1.5%)

Table 3.3.13 Minimum Radius for Design Superelavation Rates, Design Speed and emax=6%

e(%)	Vd=20 km/h	Vd=30 km/h	Vd=40 km/h	Vd=50 km/h	Vd=60 km/h	Vd=70 km/h	Vd=80 km/h	Vd=90 km/h	Vd=100 km/h	Vd=120 km/h
	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)
NC	194	421	738	1050	1440	1910	2360	2880	3510	4770
RC	138	299	525	750	1030	1380	1710	2090	2560	3510
2.2	22	265	465	668	919	1230	1530	1880	2300	3160
2.4	109	236	415	599	825	1110	1380	1700	2080	2870
2.6	97	212	32	540	746	1000	1260	1540	1890	2630
2.8	87	190	334	488	676	910	1150	1410	1730	2420
3	78	170	300	443	615	813	1050	1290	1590	2240
3.2	70	152	269	402	561	761	959	1190	1470	2080
3.4	61	133	239	364	511	697	822	110	1360	1940
3.6	51	113	206	329	465	640	813	1020	1260	1810
3.8	42	96	117	294	422	586	749	939	1170	1700
4	36	82	155	261	380	535	690	870	1090	1590
4.2	31	72	136	234	343	488	635	806	1010	1500
4.4	27	63	121	210	311	446	584	746	938	1410
4.6	24	56	108	190	283	408	538	692	873	1330
4.8	21	50	97	172	258	347	496	641	812	1260
5	19	45	88	156	235	343	457	594	755	1190
5.2	17	40	79	142	214	315	421	549	701	1120
5.4	15	36	71	128	195	287	386	506	648	1060
5.6	13	32	63	115	176	260	35	463	594	980
5.8	11	28	56	102	156	232	315	416	537	900
6	8	21	43	79	123	184	252	336	437	756

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Table 3.3.14 Minimum Radius for Design Superelavation Rates, Design Speed and emax=8%

e(%)	Vd=20 km/h	Vd=30 km/h	Vd=40 km/h	Vd=50 km/h	Vd=60 km/h	Vd=70 km/h	Vd=80 km/h	Vd=90 km/h	Vd=100 km/h	Vd=120 km/h
	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)
NC	184	443	784	1090	1490	1970	2440	2970	3630	4900
RC	133	322	571	791	1090	1450	1790	290	2680	3640
2.2	119	288	512	711	976	1300	1620	1980	2420	3290
2.4	107	261	463	644	885	1190	1470	1800	2200	3010
2.6	97	237	421	587	808	1080	1350	1650	2020	2760
2.8	88	216	385	539	742	992	1240	1520	1860	2550
3	81	199	354	496	684	916	1150	1410	1730	2370
3.2	74	183	236	458	633	849	1060	1310	1610	2220
3.4	68	169	302	425	588	790	988	1220	1500	2080
3.6	62	156	279	395	548	738	924	1140	1410	1950
3.8	57	144	259	368	512	690	866	1070	1320	1840
4	52	134	241	344	479	648	813	1010	1240	1740
4.2	48	124	224	32	449	608	766	948	1180	1650
4.4	43	115	208	301	421	573	722	985	1110	1570
4.6	38	106	192	281	395	540	682	847	1050	1490
4.8	33	96	178	263	371	509	645	803	996	1420
5	30	87	163	246	349	480	611	762	947	1360
5.2	27	78	148	229	328	454	579	724	901	1300
5.4	24	71	136	213	307	429	59	689	859	1250
5.6	22	65	125	198	288	405	521	656	819	1200
5.8	20	59	115	185	270	382	494	625	781	1150
6	19	55	106	172	253	360	469	595	746	1100
6.2	17	50	98	161	238	340	445	567	713	1060
6.4	16	46	91	151	224	322	422	540	681	1020
6.6	15	43	85	141	210	304	400	514	651	982
6.8	14	40	79	132	198	287	379	489	620	948
7	13	37	73	123	185	270	358	464	591	914
7.2	12	34	68	115	174	254	338	440	561	879
7.4	11	31	62	107	162	237	318	451	531	842
7.6	10	29	57	99	150	211	296	389	499	803
7.8	9	26	52	90	137	202	273	359	462	757
8	7	20	41	73	113	168	299	304	394	667

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Table 3.3.15 Minimum Radius for Design Superelavation Rates, Design Speed and emax=10%

e(%)	V_d=20 km/h	V_d=30 km/h	V_d=40 km/h	V_d=50 km/h	V_d=60 km/h	V_d=70 km/h	V_d=80 km/h	V_d=90 km/h	V_d=100 km/h	V_d=120 km/h
NC	197	454	790	1110	1520	2000	2480	3010	3690	4960
RC	145	333	580	815	1120	1480	1840	2230	2740	3700
2.2	130	300	522	735	1020	1340	1660	2020	2480	3360
2.4	118	272	474	669	920	1220	1520	1840	2260	3070
2.6	108	249	434	612	844	1120	1390	1700	2080	2830
2.8	99	229	399	564	778	1030	1290	1570	1920	2620
3	91	211	368	522	720	952	1190	1460	1790	2440
3.2	85	196	342	485	670	887	1110	1360	1670	2280
3.4	79	182	318	453	626	829	1040	1270	1560	2140
3.6	73	170	297	424	586	777	974	1200	1470	2020
3.8	68	159	278	398	551	731	917	1130	1390	1910
4	64	149	261	374	519	690	866	1060	1310	1810
4.2	60	140	245	353	490	652	820	1010	1240	1720
4.4	56	132	231	333	464	617	777	953	1180	1640
4.6	53	124	218	315	439	586	738	907	1120	1560
4.8	50	117	206	299	417	557	703	864	1070	1430
5	47	111	194	283	396	530	670	824	1020	1430
5.2	44	104	184	269	377	505	640	788	975	1370
5.4	41	98	174	256	359	482	611	75	934	1320
5.6	39	93	164	243	343	461	585	723	896	1270
5.8	36	88	155	232	327	441	561	693	860	1220
6	33	82	146	221	312	422	538	666	827	1180
6.2	31	77	138	210	298	404	516	640	795	114
6.4	28	72	130	200	285	387	496	616	766	1100
6.6	26	67	121	191	273	372	476	593	738	1060
6.8	24	62	114	181	261	357	458	57	712	1030
7	22	58	107	172	249	342	441	551	688	993
7.2	21	55	101	164	238	329	425	532	664	963
7.4	20	51	95	156	228	315	409	513	642	934
7.6	18	48	90	148	218	303	394	496	621	906
7.8	17	45	85	141	208	291	380	479	601	882
8	16	43	80	135	199	279	366	463	582	857
8.2	15	40	76	128	190	268	353	448	564	834
8.4	14	38	72	122	182	257	339	432	546	812
8.6	4	36	68	116	174	246	326	417	528	790
8.8	13	34	64	110	166	236	313	402	509	770
9	12	32	61	105	158	225	300	386	491	751
9.2	11	30	57	99	150	215	287	371	472	731
9.4	11	28	54	94	142	204	274	354	453	709
9.6	10	26	50	88	133	192	259	337	432	685
9.8	9	24	46	81	124	179	242	316	407	656
10	7	19	38	68	105	154	210	277	358	597

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

5) **Minimum Radius for Normal Crown**

The values of minimum radius for super elevation which is not required mentioned in Table 3.3.12 - Table 3.3.15 are based on 1.5% of typical superelevation. The values of minimum radius for super elevation which is not required for 2.0%, 2.5%, and 3.0% are calculated based on side friction factor applied in AASHTO and shown in Table 3.3.16 - Table 3.3.18.

Table 3.3.16 Minimum Radius for Normal Crown(2.0%)

Design Speed (km/h)	Typical Superelevation e (%)	Side Friction f	Total (e/100+f)	Calculated Radius (m)	Rounded Radius (m)
20	-2.0%	0.031	0.01	286	290
30	-2.0%	0.031	0.01	644	640
40	-2.0%	0.031	0.01	1,145	1,200
50	-2.0%	0.033	0.01	1,514	1,600
60	-2.0%	0.034	0.01	2,025	2,100
70	-2.0%	0.034	0.01	2,756	2,800
80	-2.0%	0.035	0.02	3,360	3,400
90	-2.0%	0.036	0.02	3,986	4,000
100	-2.0%	0.036	0.02	4,921	5,000
110	-2.0%	0.037	0.02	5,604	5,700
120	-2.0%	0.038	0.02	6,299	6,300
130	-2.0%	0.040	0.02	6,654	6,700

Table 3.3.17 Minimum Radius for Normal Crown(2.5%)

Design Speed (km/h)	Typical Superelevation e (%)	Side Friction f	Total (e/100+f)	Calculated Radius (m)	Rounded Radius (m)
20	-2.5%	0.031	0.01	525	520
30	-2.5%	0.031	0.01	1,181	1,200
40	-2.5%	0.031	0.01	2,100	2,100
50	-2.5%	0.033	0.01	2,461	2,500
60	-2.5%	0.034	0.01	3,150	3,200
70	-2.5%	0.034	0.01	4,287	4,300
80	-2.5%	0.035	0.01	5,039	5,100
90	-2.5%	0.036	0.01	5,798	5,800
100	-2.5%	0.036	0.01	7,158	7,200
110	-2.5%	0.037	0.01	7,940	8,000
120	-2.5%	0.038	0.01	8,722	8,800
130	-2.5%	0.040	0.02	8,871	8,900

Table 3.3.18 Minimum Radius for Normal Crown(3.0%)

Design Speed (km/h)	Typical Superelevation e (%)	Side Friction f	Total (e/100+f)	Calculated Radius (m)	Rounded Radius (m)
20	-3.0%	0.031	0.00	3,150	3,200
30	-3.0%	0.031	0.00	7,087	7,100
40	-3.0%	0.031	0.00	12,598	12,600
50	-3.0%	0.033	0.00	6,562	6,600
60	-3.0%	0.034	0.00	7,087	7,100
70	-3.0%	0.034	0.00	9,646	9,700
80	-3.0%	0.035	0.01	10,079	10,100
90	-3.0%	0.036	0.01	10,630	10,700
100	-3.0%	0.036	0.01	13,123	13,200
110	-3.0%	0.037	0.01	13,611	13,700
120	-3.0%	0.038	0.01	14,173	14,200
130	-3.0%	0.040	0.01	13,307	13,400

6) **Development of Superelevation**

Superelevation within a curve is applied by rotating the road lanes from the normal camber until the road attains full superelevation. The rotation is divided in two stages which are called Tangent Runoff and Superelevation runoff. This rotation can be done around the road centerline, the edge of the inner lane or the edge of the outer lane.

Tangent Runoff (TR) is the length of the road whereby the normal camber on the outside of the curve is rotated until adverse crown is removed from the outer half of the road carriageway.

Superelevation Runoff (SR) is the length of the road from the point with adverse crown removed up to the point where the road has attained the maximum superelevation.

The length of superelevation run-off is given by the formula:

$$L_r = \frac{(wn_1)e_d}{\Delta}(b_w)$$

Where:

L_r = minimum length of superelevation runoff, m

w = width of one traffic lane, m

n_1 = number of lanes rotated

e_d = design superelevation rate, percent

b_w = adjustment factor for number of lanes rotated(Table 3.3.20)

Δ = maximum relative gradient, percent

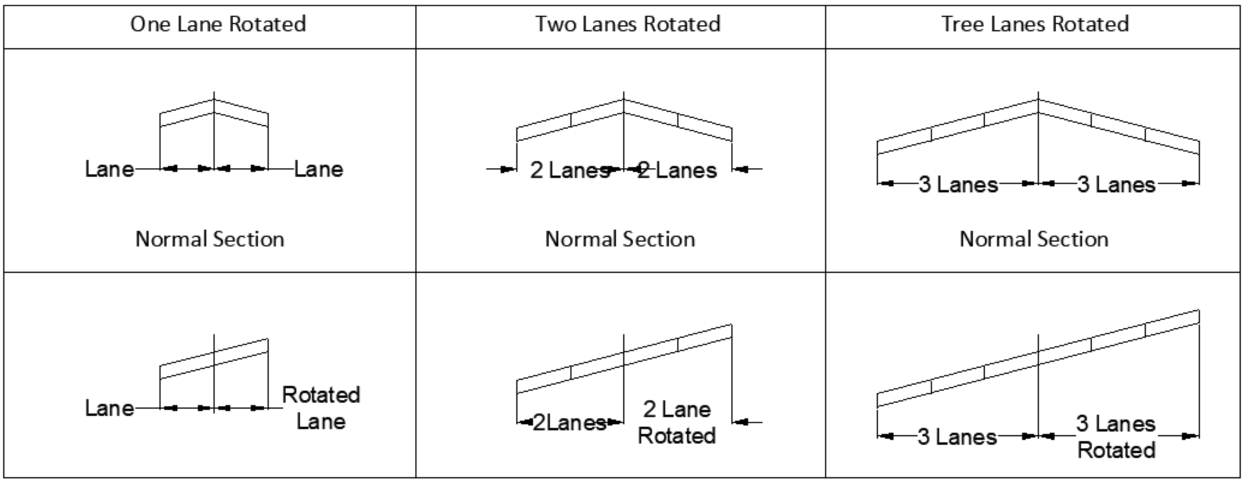
Table 3.3.19 Maximum Relative Gradients

Design Speed (km/h)	Maximum Relative Gradient (%)	Equivalent Maximum Relative Slope
20	0.80	1:125
30	0.75	1:133
40	0.70	1:143
50	0.65	1:154
60	0.60	1:167
70	0.55	1:182
80	0.50	1:200
90	0.47	1:213
100	0.44	1:227
120	0.38	1:263

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Table 3.3.20 Adjustment Factor for Number of Lanes Rotated

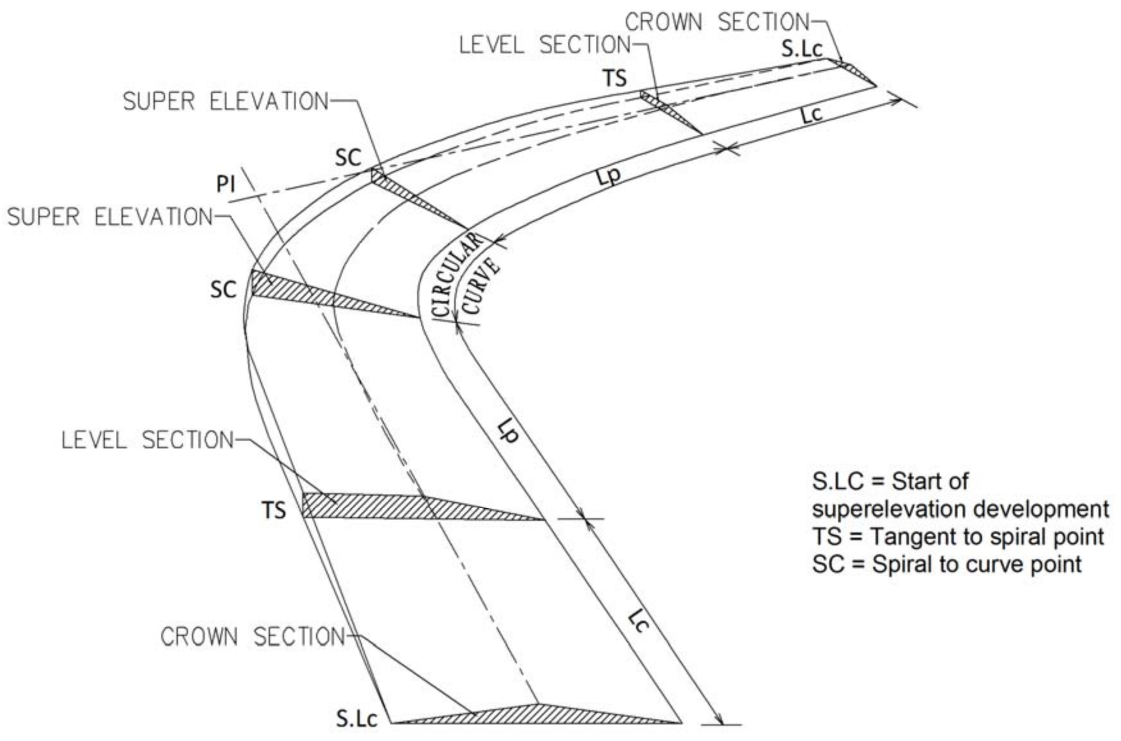
Number of Lanes Rotated, n_1	Adjustment Factor, b_w*	Length Increase Relative to One Lane Rotated, $(=n_1 b_w)$
1	1.00	1.00
1.5	0.83	1.25
2	0.75	1.50
2.5	0.70	1.75
3	0.67	2.00
3.5	0.64	2.25



* $b_w = [1 + 0.5(n_1 - 1)]/n_1$

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

A. Perspective



B. Elevation

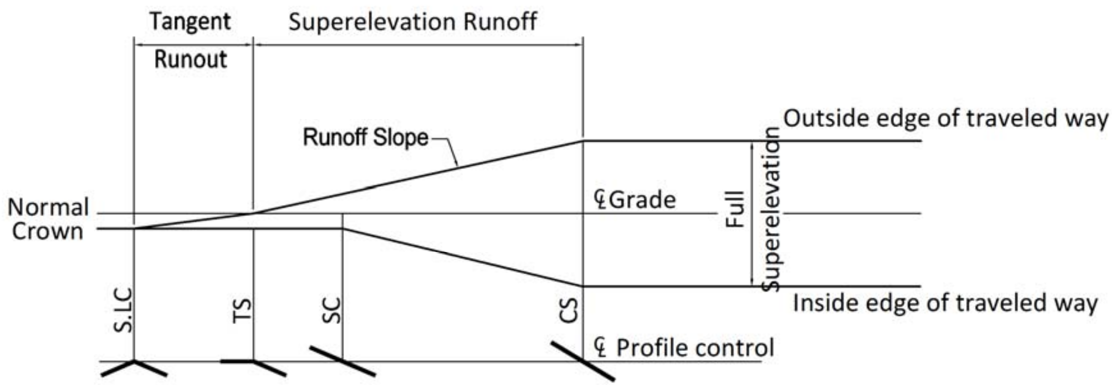


Figure 3.3.7 Development Superelevation

7) **The Transition Curve Length**

The transition curve is a spiral curve applied to effect the transition between two circular curves or between a circular curve and a straight. The advantages of transition curves are to enable smooth turning movement of vehicles between tangents and circular curves, provide basis for application of superelevation and for aesthetic purposes.

Minimum length of spiral

Criteria based on driver comfort are intended to provide as spiral length that allows for a comfortable increase in lateral acceleration as a vehicle enters a curve. The criteria based on lateral shift are intended to provide a spiral curve that is sufficiently long to result in a shift in a vehicle's lateral position within its lane that is consistent with that produced by the vehicle's natural spiral path. It is recommended that these two criteria be used together to determine the minimum length of spiral. Thus, the minimum spiral length can be computed as;

$$L_{s,\min} = \sqrt{24(p_{\min})R}$$

or

$$L_{s,\min} = 0.0214\frac{V^3}{RC}$$

Where:

$L_{s,\min}$ = minimum length of spiral, m

$p_{\min}$ = minimum lateral offset between the tangent and circular curve (0.20m)

R = radius of circular curve, m

V = design speed, km/h

C = maximum rate of change in lateral acceleration(1.2 m/s^3)

Maximum length of spiral

Spiral should not be so long that drivers are misled about the sharpness of approaching curve. A conservative maximum length of spiral that should minimize the likelihood of such concerns can be computed as:

$$L_{s,\max} = \sqrt{24(p_{\max})R}$$

Where:

$L_{s,\max}$ = maximum length of spiral, m

$p_{\max}$ = maximum lateral offset between the tangent and circular curve (1.0m)

R = radius of circular curve, m

Desirable length of spiral

AASHTO standard suggest that (2) seconds is required for a movement from tangent section to full curve section or from full curve section to tangent section as a desirable length of spiral. The length is calculated in following equation.

$$L = \frac{V}{3.6} t$$

Where:

L = length of transition curve, m

V = design speed, km/h

t = travel time for a movement of rom tangent section to full curve section, s

Table 3.3.21 Desirable Length of Transition Curve

Design Speed (km/h)	Spiral Length (m)
20	11
30	17
40	22
50	28
60	33
70	39
80	44
90	50
100	56
120	67

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

In transition design with a spiral curve, it is recommended that the superelevation runoff be accomplished over the length of spiral. The spiral curve length shall not be less than the length required for runoff when length required for runoff as specified in

Section above exceeds values as listed in Table 3.3.19. The change in cross slope begins by introducing a tangent runout section just in advance of the spiral curve. Full attainment of superelevation is then accomplished over the length of the spiral. In such a design, the whole of the circular curve has full superelevation.

Length of tangent runout

The tangent runout length for a spiral curve transition design is based on the same approach used for the tangent-to-curve transition. Specifically, a smooth edge of pavement profile is desired so that a common edge slope gradient is maintained throughout the superelevation runout and runoff sections. Based on this rationale, the following equation can be used to compute the tangent runout length:

$$L_t = \frac{e_{NC}}{e_d} L_S$$

Where:

L_t = length of tangent runout, m

L_S = length of spiral, m

e_d = design superelevation rate, percent

e_{NC} = normal cross slope rate, percent

8) **Pavement Widening**

Pavement on horizontal curves may require widening to ensure that operating conditions are compatible with those on tangents; such widening in curves is required for two reasons.

(i) Vehicle in curves occupy a greater width of carriageway since the rear wheels will track inside the front wheels, and

(ii) Drivers generally experience some difficulty in holding their vehicles in the center of the lane as they tend to shy away from the carriageway edge.

Pavement widening is costly but very little is gained from a small amount of widening. A minimum amount of 0.6m has therefore been adopted.

The amounts of pavement widening for 2-lane (one-way or two-way) pavements with widths on tangents of 7.0m, 6.5m are given in Table 3.3.22. For single-lane pavements, one half of the amount of widening quoted to Table 3.3.22 for a 7.00m wide pavement shall be used.

One half of the amount of widening required shall be applied to each side of the pavement and should be introduced at a uniform rate along the transition length from normal camber to full super elevation.

Table 3.3.22 Caluculated and Design Values for Traveled Way Widening on Curves

Radius of Curve (m)	Roadway width =7.0 m						Roadway width =6.5 m						Roadway width =6.0 m					
	Design Speed (km/h)						Design Speed (km/h)						Design Speed (km/h)					
	50	60	70	80	90	100	50	60	70	80	90	100	50	60	70	80	90	100
3000	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.4	0.4	0.4	0.4	0.4	0.5	0.6	0.6	0.6	0.6	0.6
2500	0.2	0.2	0.2	0.2	0.2	0.3	0.4	0.4	0.4	0.4	0.4	0.5	0.6	0.6	0.6	0.6	0.6	0.7
2000	0.2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7
1500	0.2	0.3	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8
1000	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.9	0.9
900	0.4	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9
800	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	1.0
700	0.5	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	1.0	1.0	1.0
600	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	1.0	1.0	1.0	1.1	1.1
500	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	1.0	1.0	1.0	1.0	1.1	1.1	1.2	1.2
400	0.7	0.8	0.8	0.9	0.9	1.0	0.9	1.0	1.0	1.1	1.1	1.2	1.1	1.2	1.2	1.3	1.3	1.4
300	0.9	1.0	1.0	1.1	1.2	1.2	1.1	1.2	1.2	1.3	1.4	1.4	1.3	1.4	1.4	1.5	1.6	1.6
250	1.1	1.2	1.2	1.3	1.3		1.3	1.4	1.4	1.5	1.5		1.5	1.6	1.6	1.7	1.7	
200	1.3	1.4	1.5	1.5			1.5	1.6	1.7	1.7			1.7	1.8	1.9	1.9		
150	1.7	1.8	1.9				1.9	2.0	2.1	2.2			2.1	2.2	2.3	2.4		
140	1.8	1.9					2.0	2.1					2.2	2.3				
130	2.0	2.0					2.2	2.2					2.4	2.4				
120	2.1	2.2					2.3	2.4					2.5	2.6				
110	2.3	2.4					2.5	2.6					2.7	2.8				
100	2.5	2.6					2.7	2.8					2.9	3.0				
90	2.7						2.9	0.1					3.1					
80	3.0						3.2						3.4					
70	3.4						3.6						3.8					

Source: The values are calculated based on AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Note: The values are shown for WB-19 design vehicle. Values less than 0.6m may be disregarded.

The values are shown for road width 7.2m, 6.6m and 6.0m respectively in AASHTO.

The above value are developed considering the gap between 0.2m (=7.2m -7.0m) and 0.1m (=6.6m -6.5m) for widening width.

(4) Vertical Alignment

Vertical alignment is the combination of parabolic vertical curves and tangent sections of a particular slope. The selection of rates of grade and lengths of vertical curves is based on assumptions about characteristics of the driver, the vehicle and the roadway. Vertical curvatures may impose limitations on sight distance, particularly when combined with horizontal curvature. The slope of tangent sections introduces forces which affect vehicle speed, drive comfort and the ability to accelerate and decelerate.

With the whole-life economy of the road in mind, vertical alignment should always be designed to as high a standard as is consistent with the topography.

The vertical alignment should also be designed to be aesthetically pleasing. In the regard due recognition should be given to the interrelationship between horizontal and vertical curvature As a general guide a vertical curve that coincides with a horizontal curve should, if possible be contained within the horizontal curve and should have approximately the same length.

A smooth grade line with gradual changes appropriate to the class of road and the character of the topography is preferable to an alignment with numerous short lengths of grade and vertical curve. The “roller coaster” or “hidden dip” type of profile should be avoided. A broken-back alignment is not desirable on aesthetic grounds in sags where a full view of the profile is possible. On crests the broken back adversely affects passing opportunity.

Short grades between two successive vertical sag curves appear as optical humps and should be avoided. In particular this applies when the vertical alignment includes a bridge. Further, short grades between two successive crest curves give an optical oblation (flatness) and should be avoided.

As long as the driver’s line of sight is contained within the width of the roadway, the sup elevation generated by horizontal curvature improves the availability of sight distance, even though the edge profiles may have a curvature sharper than the minimum suggested below. When the line of sight goes beyond the roadway edge, the effect on sight distance of lateral obstructions such as cut faces or high vegetation must be checked.

1) Maximum Gradients

Maximum gradients in relation to design speed and terrain are given in Table 3.3.23. These values should be adhered to for trunk roads and other roads where a large portion of the traffic volume is heavy vehicles.

On secondary and minor roads and other roads with little traffic (Class VI and VII) the values in Table 3.3.23 can be exceeded by 2%.

Table 3.3.23 Maximum Gradients

Design Speed (km/h)	Maximum gradient, (%)
20	10
30	9
40	8
60	7
80	6
100	5
120	4

Through at-grade junction the gradient should not exceed 2.5%.

To avoid standing water in side ditches, the minimum gradient for roads in cutting is 0.5%.

Critical length of grade is the maximum length of the designated up-grade on which a loaded truck can operate without unreasonable reduction in speed. As a reference, the values of critical length of grades in Road Structure Ordinance in Japan are shown in Table 3.3.24.

Table 3.3.24 Critical Length of Grades at Different Gradients

Design Speed (km/h)	Gradient (%)	Length of Grade (m)
120	3	800
	4	500
	5	400
100	4	700
	5	500
	6	400
80	5	600
	6	500
	7	400
60	6	500
	7	400
	8	300
50	7	500
	8	400
	9	300
40	8	400
	9	300
	10	200

Source: Road Structure Ordinance in Japan

2) **Combined Gradient**

There are cross fall or superelevation and gradient on the road surface. Then, maximum gradient is larger than cross sectional gradient and longitudinal gradient. The resultant combined gradient is calculated in following equation.

$g_r = g^2 + e^2$ should be within the following limits:

0.5% < gr < 10.5

where: g = longitudinal gradient

e = super elevation

3) **Length of Vertical Curves**

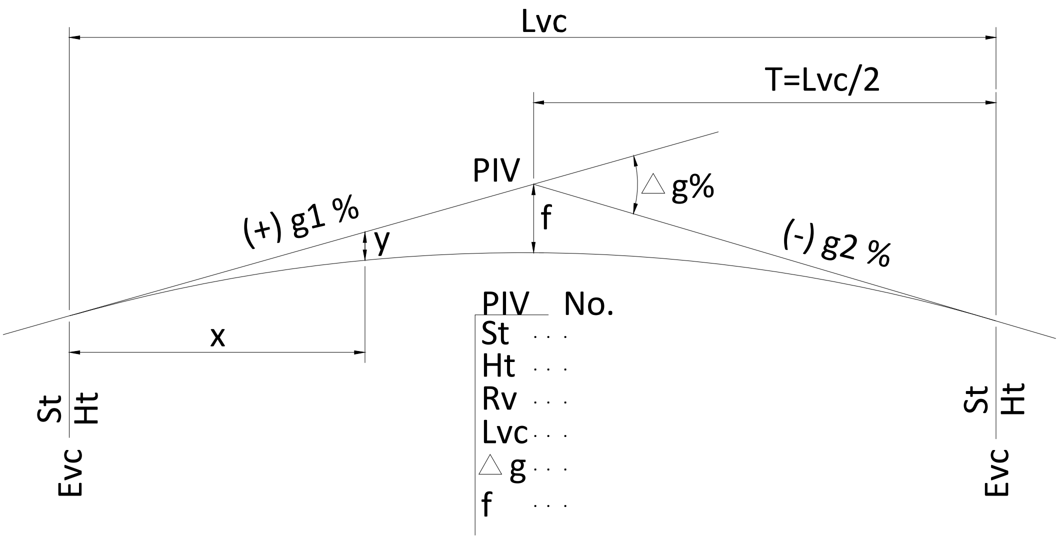
Vertical curves are applied to effect the transition between straight. There are two types of vertical curves, namely crest vertical curves and sag vertical curves. The major control for the design of vertical curves is the minimum sight distance required, but also aesthetics, drainage requirements and comfort of vehicle occupants shall be considered.

The form generally adopted for vertical curves is the circle or the simple parabola.

The equations for the circular vertical curve are:

$$T = \frac{R_V}{2} \times \frac{g1 + g2}{100} \quad \text{and} \quad Y = \frac{X^2}{2R_V}$$ (see Figure 3.3.8)

VERTICAL CURVE



SYMBOL	DESCRIPTION	SYMBOL	DESCRIPTION
G	Gradient (%)	Bvc	Beginning of vertical curve
PIV	Point of intersection of vertical curve	Evc	End of vertical curve
St	Station chaionage	T	Tangent length of vertical curve
Ht	Height above sea level (m)	y	Tangent offset (vertical)
Rv	Equivalent radius of vertical curve (m)	x	Horizontal length
Lv	Length of vertical	f	Center correction
Lvs	Length of vertical sag curve	Ls	Stopping sight distance
Lvc	Length of vertical crest curve	Lm	Meeting sight distance
△g	Algebraic difference in gradients (%)	Lp	Passing sight distance

Figure 3.3.8 Standard Symbol and Abbreviations for Vertical Curves

Minimum length of vertical curve is mainly decided based on followings.

- Comfort criterion
- Sight distance requirements
- Appearance criterion

- Length of Vertical Curve for Comfort Criterion

Discomfort is felt by a human being subject to rapid changes in vertical acceleration. To minimize such discomfort when passing from one grade to another, it is required to install vertical curve. According to Road Structure Ordinance in Japan, following equation is known empirically as length of vertical curve for comfort criterion.

$$L_v = \frac{V^2 A}{360}$$

Where:

V: Speed of the vehicle, m/sec

A: Algebraic difference in gradients, %

L_v: Length of vertical curve, m

Table 3.3.25 Minimum Vertical Curve Length based on Comfort Criterion

Design Speed (km/h)	Rate of Vertical Curvature, K_a
20	1.1
30	2.5
40	4.4
50	6.9
60	10.0
70	13.6
80	17.8
90	22.5
100	27.8
120	40.0

Note: Rate of vertical curvature, K is the length of curve per percent algebraic difference in intersecting grades (A), K=L/A

Source: Road Structure Ordinance in Japan

- <u>Length of Vertical Curve for Sight Distance Requirements (Crest Curve)</u>

On crest curves, the driver's sight line is obstructed by the vertical geometry of the road. In that, sight distance is measured from an assumed driver's eye point to an object point with a certain height behind the obstruction as shown in Figure 3.3.9.

Case 1 : $S < L_v$

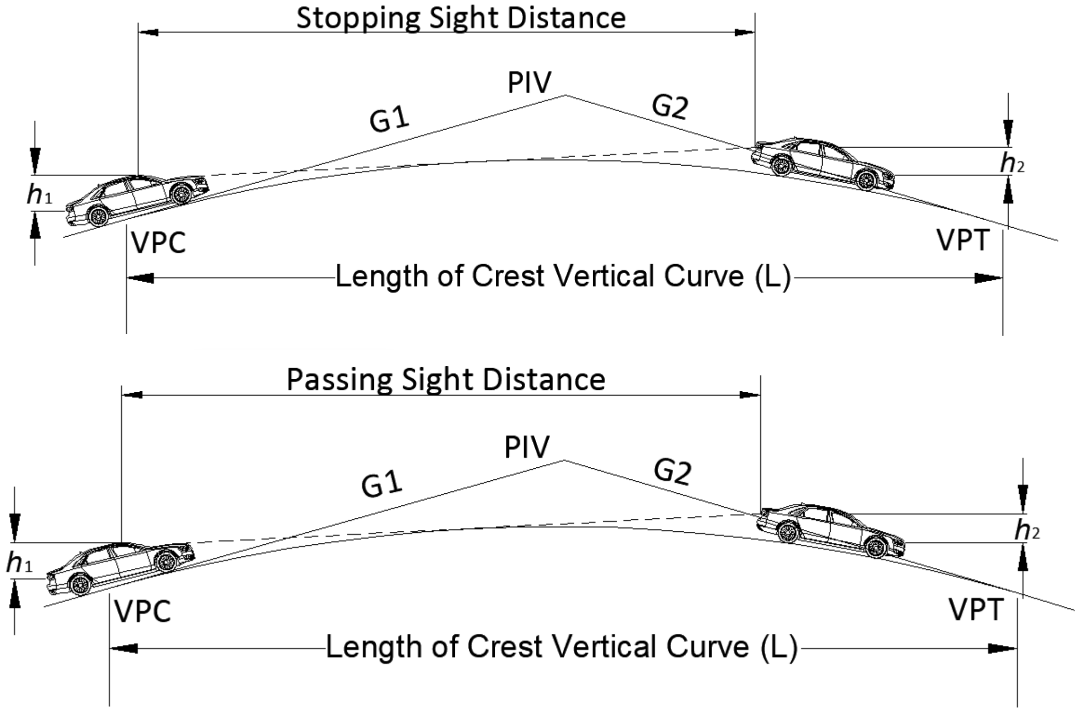


Figure 3.3.9 Length of Vertical Curve for Sight Distance Requiremdnts

Working on the parabolic of vertical curves, it can be shown that for crest curves,

$$L_v = \frac{AS^2}{200(\sqrt{h_1} + \sqrt{h_2})^2}$$

Where;

L_v: Length of vertical curve, m

S: Sight distance, m

h_1: Driver's eye height, m

h_2: Object height, m

A: algebraic difference in gradients, %

Above equation shows that the standard value of vertical curve length for a design speed is only a function of A value, since the other parameters involved (i.e. S, h_1, and h_2) are all fixed as the standard values. It is then convenient to express this equation as a linear function as L_v =KA, where K (in m per %) is a constant for each design speed.

The calculated length of vertical curve is usually rounded up to the next highest multiple of 10m. Table 3.3.26 gives the minimum length of crest vertical curves (K values) for various design speeds based on the stopping sight distance, and passing sight distance criteria.

Table 3.3.26 Minimum Crest Vertical Curve Length based on Sight Distance Criteria

Design speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature, K_a		Passing Sight Distance (m)	Rate of Vertical Curvature, K_a	
		Calculated	Design		Calculated	Design
20	20	0.6	1	100	14.7	15
30	35	1.8	2	150	33.0	33
40	50	3.7	4	220	70.9	71
50	65	6.2	7	280	114.9	115
60	85	10.6	11	360	190.0	190
70	105	16.2	17	440	283.8	284
80	130	24.8	25	530	411.7	412
90	160	37.5	38	620	563.4	564
100	185	50.2	51	720	759.8	760
120	250	91.6	92	930	1267.7	1268

Note: Rate of vertical curvature, K is the length of curve per percent algebraic difference in intersecting grades (A), K=L/A

The height of eye and the height of object are 1.15m and 0.6m, respectively.

Case 2: $S > L_v$

Working on the parabolic properties of vertical curves, it can be shown that for crest curves:

$$L_v = 2S - \frac{200(\sqrt{h_1} + \sqrt{h_2})^2}{A}$$

Where;

L_v: Length of vertical curve, m

S: Sight distance, m

h_1: Driver's eye height, m

h_2: Object height, m

A: algebraic difference in gradients, %

The length of vertical curve resulting in case 2 ($S > L_v$) is insignificantly less than those resulting in case 1 ($S < L_v$). However, since curves with small A value and high design speed are frequently found in flat terrain, longer vertical curves may easily be accommodated.

- <u>Length of Vertical Curve for Sight Distance Requirements (Sag Curve)</u>

During day light hours, it is assumed that adequate sight distance is available on sag curves. At night, however, visibility is limited by the distance illuminated by the head light beam as illustrated in Figure 3.3.10.

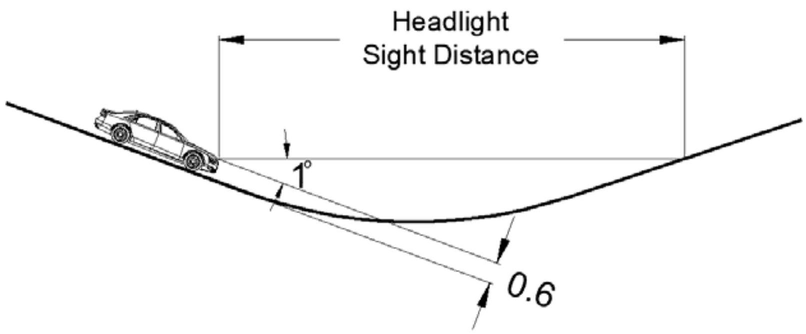


Figure 3.3.10 Length of Sag Vertical Curve for Head Light Criterion

Case 1 : $S < L_v$

Working on the parabolic propertities of vertical curves, it can be shown that for sag curves,

$$L_v = \frac{AS^2}{120 + 3.5S}$$

Where;

L_v: Length of vertical curve, m

S: Sight distance, m

A: algebraic difference in gradients, %

Given the fixed value of S in equation above, L_v is again a linear function of A value. Hence, the length of vertical curve can be represented by the K value.

Table 3.3.27 gives the minimum length of vertical sag curves for various design speeds based on the head light criterion.

Table 3.3.27 Minimum Sag Vertical Curve Length based on Head Light Sight Distance Criterion

Design speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature, Ka	
		Calculated	Design
20	20	2.1	3
30	35	5.1	6
40	50	8.5	9
50	65	12.2	13
60	85	17.3	18
70	105	22.6	23
80	130	29.4	30
90	160	37.6	38
100	185	44.6	45
120	250	62.8	63

Case 2: $S > L_v$

Working on the parabolic properties of vertical curves, it can be shown that for sag curves:

$$L_v = 2S - \frac{120 + 3.5S}{A}$$

Where;

L_v: Length of vertical curve, m

S: Sight distance, m

A: algebraic difference in gradients, %

The length of vertical curve resulting in case 2 ($S > L_v$) is insignificantly less than those resulting in case 1 ($S < L_v$). However, since curves with small A value and high design speed are frequently found in flat terrain, longer vertical curves may easily be accommodated.

➢ <u>Over Head Obstructions</u>

Overhead obstructions such as road or rail overpasses or even overhanging trees may limit the sight distance available on sag vertical curves. With the minimum overhead clearances normally specified for roads, these obstructions would not interfere with minimum stopping sight distance. They may, however, need to be considered with the upper limit of stopping distance and passing provision.

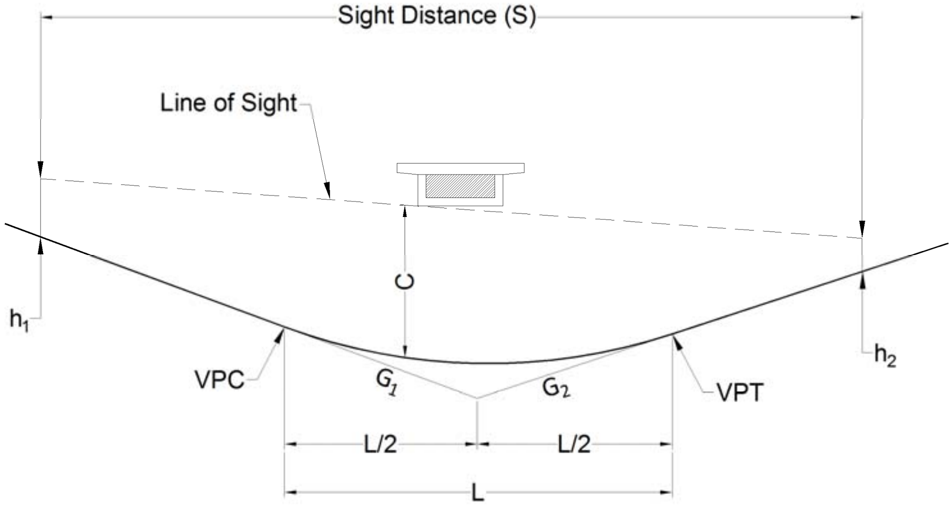


Figure 3.3.11 Sight Distance at Undercrossings

Case 1 : $S < L_v$

Length of vertical curve over sags with overhead structure based on sight distance criteria is given by

$$L_v = \frac{S^2 A}{800\left[C - \left(\frac{h_1 + h_2}{2}\right)\right]}$$

Where;

L_v: Length of vertical curve, m

S: Sight distance, m

A: algebraic difference in gradients, %

H: height of obstruction, m

C: Vertical clearance, m

h_1: Driver's eye height, m

h_2: Object height, m

Case 2 : $S > L_v$

Length of vertical curve over sags with overhead structure based on sight distance criteria is given by

$$L_v = 2S - \frac{800\left[C - \left(\frac{h_1 + h_2}{2}\right)\right]}{A}$$

Where;

L_v: Length of vertical curve, m

S: Sight distance, m

A: algebraic difference in gradients, %

H: height of obstruction, m

C: Vertical clearance, m

h_1: Driver's eye height, m

h_2: Object height, m

In addition to above the length of vertical curve L should be checked for comfort and appearance criteria. The same values which were derived for crest curves can be sued as K values.

- Length of Vertical Curve for Appearance Criterion

At very small change of grade, a vertical curve has little influence other than appearance of the profile and may be omitted. At any significant change of grade, short vertical curves detract from the appearance. At least more than limit length is required to set. The required length is decided based on the basis of appearance, then, it is considered to be proportional to driving speed.

$$L_v = \frac{V}{3.6} \times 3$$

$$= \frac{V}{1.2}$$

Where:

L_v: Length of Vertical for Satisfactory Appearance, m

V: Design Speed

According to Road Design Ordinance in Japan, 3 seconds travel distance is used as minimum length of vertical.

Table 3.3.28 Length of Vertical Curve

Design Speed (km/h)	Length of Vertical Curve for Satisfactory Appearance (m)	
	Calculated (m)	Design (m)
20	16.7	20
30	25.0	25
40	33.3	35
50	41.7	45
60	50.0	50
70	58.3	60
80	66.7	70
90	75.0	75
100	83.3	85
120	100	100

(5) Resting bays (plateau)

In mountainous terrain where the alignment often runs in steep gradients for relatively long lengths, it is advisable to provide Resting Bays. The frequency of placing these speed relief areas varies with the length and type of gradients. Present concept used in Laos is to install Resting Bays of 50-100m lengths every 2km where the road alignment traverses through mountainous terrain.

Table 3.3.29 Maximum Grade Length

Length of Resting bays (m)	80	75	70	65	60	55	50
Gradient (%)	6	7	8	9	10	11	12
Maximum Length (m)	400	350	300	260	200	150	100
Design speed (Km/h)	80	70	60	50	40	30	20

(6) Climbing Lanes

Where longitudinal gradients are long enough and/or steep enough to cause significant increases in the speed differences between cars and heavy vehicles, both traffic safety and road capacity may be adversely affected.

The following three criteria, reflecting economic considerations, should be satisfied to justify a climbing lane:

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

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

(iii) The following condition exists:

A 15-km/h or greater speed reduction is expected for a typical heavy truck.

In addition, high crash frequencies may justify the additional of a climbing lane regardless of grade or traffic volumes.

Consideration must always be given to the balance between the benefits to traffic and the initial construction cost. For example, in sections requiring heavy side cut, the provision of climbing lanes may be unreasonably high in relation to the benefits and hence climbing lanes may be omitted leading to reduced “levels of service” over such sections.

For design purposes it may be assumed that the highest obtainable speed on a level or downhill section of road for a typical heavy vehicle will be 80% of design speed or 80km/h whichever is the lower.

The climbing lane shall be terminated when the speed of a typical heavy vehicle reaches the value at which the climbing lane was introduced. However, it must be ensured that a typical heavy vehicle will regain this speed without creating a traffic hazard i.e. passing sight distance must be adequate. This latter requirement may lead to an extension of the climbing lane beyond the point determined from speed considerations alone.

(7) Turnout

Turnout shall be provided on Road Design Class VI and VII roads as specified below, except for on roads where smooth traffic can be ensured.

1. Distance between two turnouts shall be within 300m.

2. Roads between two turnouts shall be visible from one of these turnouts.

3. The length shall be more than 20m and the total width of the carriageway shall be more than 5m.

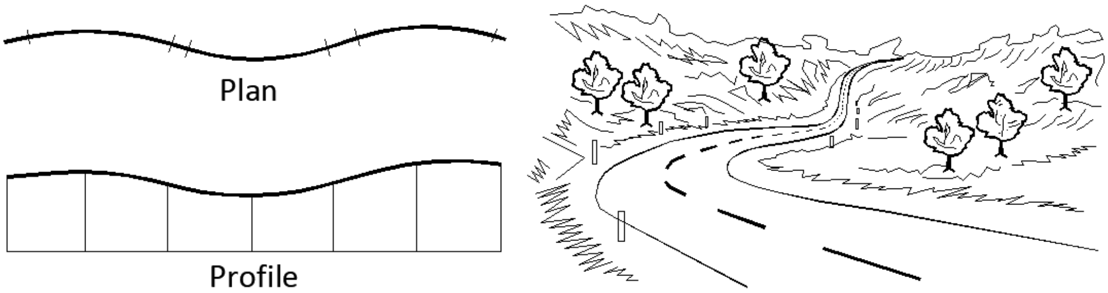
(8) The Co-ordination of Horizontal and Vertical Alignments

The horizontal and vertical alignment shall not be designed independently. The plan and profile must be coordinated to ensure a safe, aesthetic and economic design. The following guide line shall be applied.

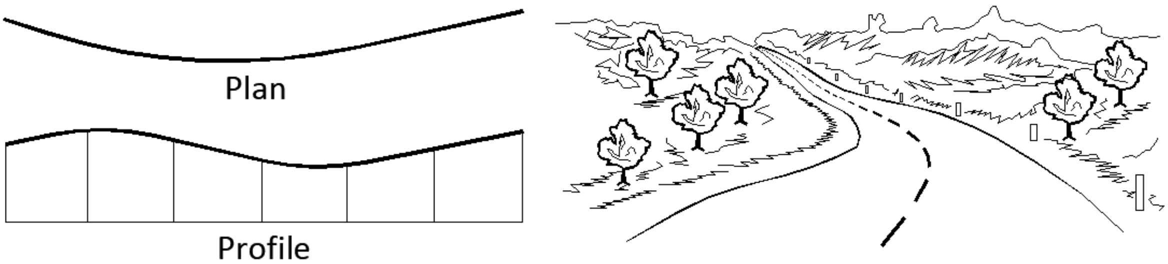
1. Generally a satisfactory alignment is obtained when the intersection points of vertical and horizontal curves nearly coincide (within about 10% of the horizontal curve length). The start of the horizontal curve is then clearly visible to the driver on crest sections, and surface water drainage in facilitated by the maximum longitudinal gradient corresponding to sections of changing super elevation and the minimum longitudinal gradient corresponding to sections of maximum super elevation.
2. The lengths of the vertical and horizontal curves should be almost equal. If however, the curves are not of equal length, then the horizontal curve should be longer than the vertical curve. The equivalent radius of the vertical curve should be at least 5 times the radius of the coincident horizontal curve.
3. A larger number of horizontal intersection points than vertical intersection points are undesirable particularly when the road is visible to the driver for a considerable distance ahead.

4. The beginning of a horizontal curve shall always fall within the available sight distance. Thus, a horizontal curve should never be introduced near the top or end of a sharp crest curve. The same applies for sharp horizontal curves at the bottom of steep grades.
5. Where the horizontal alignment is straight, a sequence of crest and sag curves which allow the driver to see the road appear and disappear must be avoided as this may cause dangerous overtaking.
6. On straight, crest curves in cuttings should be avoided as they will be visible over a long distance and often appear as scars on the landscape.
7. On dual carriageways, variations in the width of the central reserve and the use of separate horizontal and vertical alignments should be considered so as to derive the design and operational advantages of one-way roads.
8. At junction the horizontal and vertical curvature shall be such as to facilitate the required visibility splays (section 3.4.5(2)).

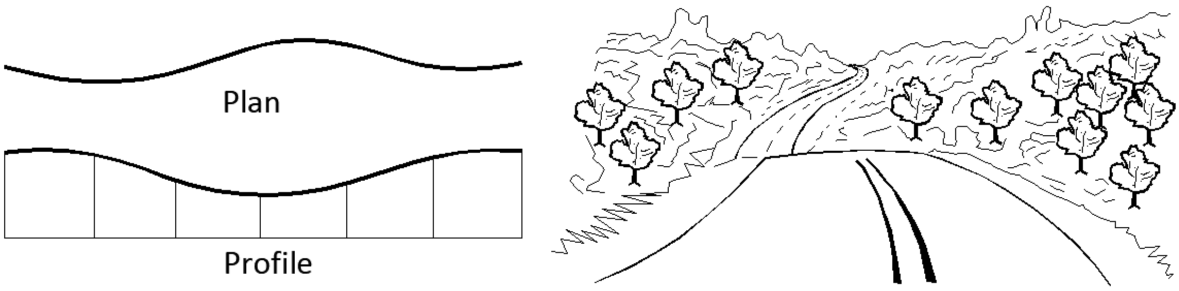
Where there is doubt about the three-dimensional appearance of any alignment, it should be more thoroughly investigated by preparing perspective drawings which will enable any visual discontinuities to be detected.



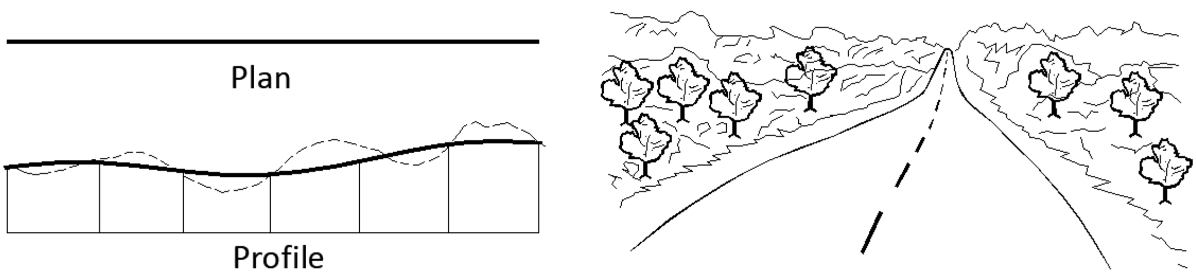
- The ideal combination. A smooth flowing appearance result when vertical and horizontal curves coincide. Ideally horizontal curves should slightly overlap the vertical



- If the horizontal scale is large and the vertical scale relatively small, it may be satisfactory to include two vertical movements on one long horizontal curves.

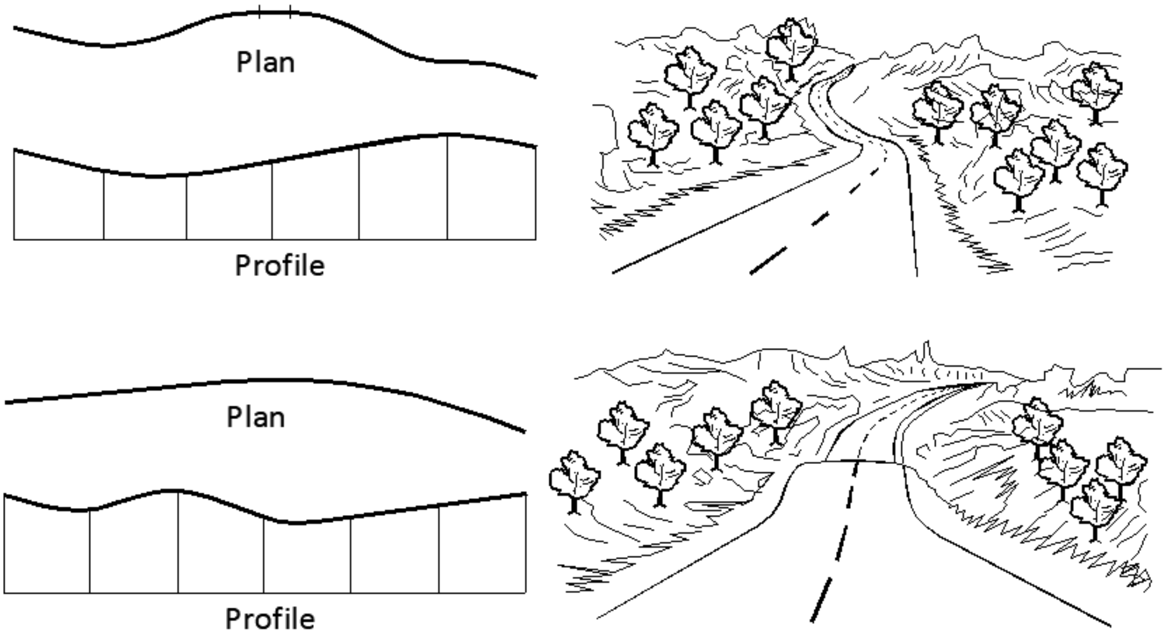


- The summit vertical curve restricts the driver's view of the start of the horizontal curve and can produce a dangerous situation.

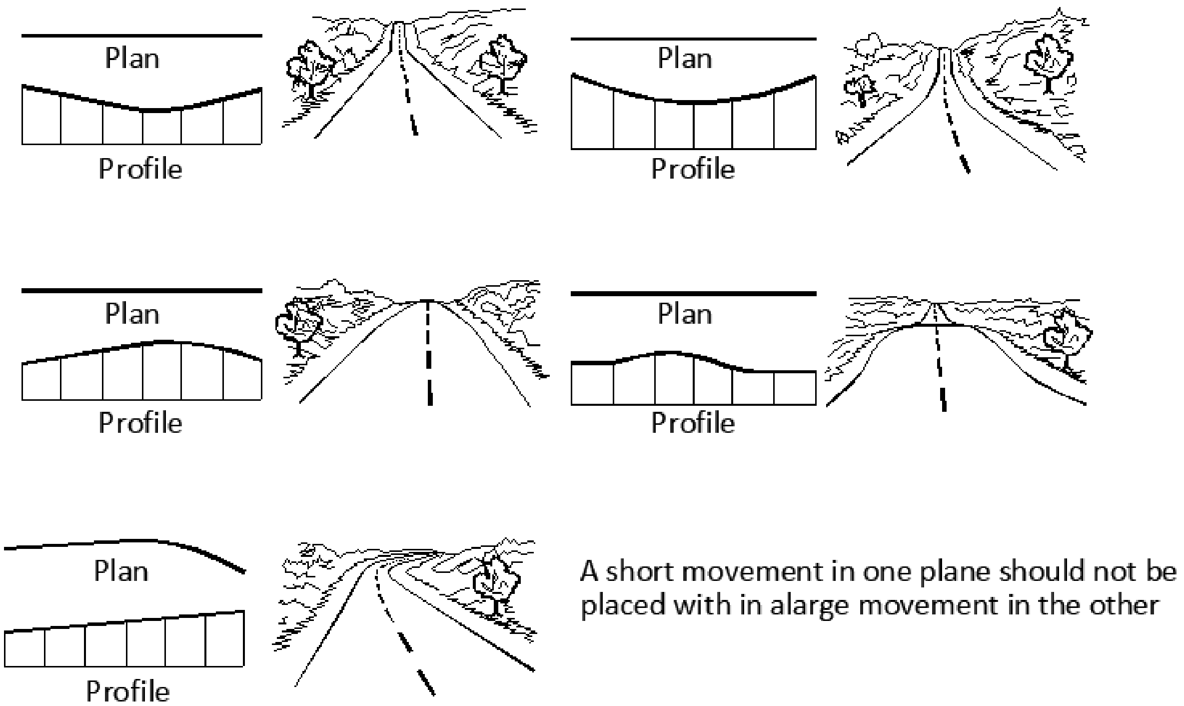


- Lack of integration with land form

Figure 3.3.12 Co-Ordination of Alignments and Terrain Fitting(1)



- Both examples have visually poor alignment with unrelated horizontal and vertical curves and broken backed horizontal curves.



- A short movement in one plane should not be placed with a large movement in the other

Figure 3.3.13 Co-Ordination of Alignments and Terrain Fitting(2)

3.3.3 Summary of Design Conditions

Design conditions mentioned in above section are summarized in Table 3.3.30.

Table 3.3.30 Design Conditions

Basic Design Requirements	I	Road Design Class	Primary (Access control)				I					II					III					IV					V			VI			VII		
	II	Traffic (pcu/day)*	>8,000				>8,000					3,000-8,000					1,000-3,000					300-1,000					100-300			50-100			<50		
	III	Terrain	F	R	M	U Type-I	F	R	M	U Type-I	U Type-II	F	R	M	U Type-I	U Type-II	F	R	M	U Type-I	U Type-II	F	R	M	U Type-I	U Type-II	F	R	M	F	R	M	F	R	M
	IV	Design Speed (km/h)	120	100	80	80	100	80	60	60	40	100	80	60	60	40	80	60	40	60	40	80	60	40	60	40	60	40	20	60	40	20	40	30	20
	IV	Recommended Operational Speed (km/h) Note: Maximum speed limit is 90 km/h in Lao PDR	90	90-80	80 - 60	80 - 60	90-80	80-60	60-40	60-40	40-30	90-80	80-60	60-40	60-40	40-30	80-60	60-40	40-30	60-40	40-30	80-60	60-40	40-30	60-40	40-30	60-40	40-30	20-15	60-40	40-30	20-15	40-30	30-20	20-15
Design Condition	1	Number of Lane	4 or more				4 or more					2					2					2					2			1			1		
	2	Lane Width (m)	3.5 [3.75]	3.5 [3.75]	3.5	3.5	3.5 [3.75]	3.5 [3.75]	3.5	3.5	3.5	3.5 [3.75]	3.5 [3.75]	3.5	3.5	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	2.75	2.75	2.5	3.5	3.5	3.5	3.5	3.5	3.5
	3	Median Width at road center (m)	4.0 (2.0)	4.0 (2.0)	3.0 (1.5)	3.0 (1.5)	3.0 (2.0)	3.0 (1.5)	2.5 (1.5)	2.5 (1.5)	2.5 (1.5)	- [0.75]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	- [0.5]	-	-	-	-	-	-	-	-	-
	4	Lateral Margin (m)	0.75	0.75	0.5	0.5	0.75	0.5	0.5	0.5	0.5	0.75	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5									
	5	Island divided between paved shoulder and service road	-	-	-	-	3.0	3.0	-	3.0	3.0	3.0	3.0	-	3.0	3.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	6	Outer shoulder (m)	3.0	3.0	2.5	2.0	3.0 (0.75)**	3.0 (0.5)**	2.5	1.5 (0.5)**	1.5 (0.5)**	2.5 (0.75)**	2.5 (0.5)**	2.0	1.5 (0.5)**	1.5 (0.5)**	2.0	2.0	1.0	1.0 (0.5)	1.0 (0.5)	1.0	1.0	0.5	0.5	0.5	0.75	0.75	0.5	1.5***	1.5***	1.25***	1.25***	1.25***	1.0***
	7	Service road for low traffic (m)	-	-	-	-	3.0	3.0	-	3.0	3.0	3.0	3.0	-	3.0	3.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	8	Max. Gradient (%)	4	5	6	6	5	6	7	7	8	5	6	7	7	8	6	7	8	7	8	6	7	8	7	8	7	8	9	7	8	9	8	9	10
	9	Min. Radius of Horizt. Curve (m)	597	358	210	210	358	210	105	105	38	358	210	105	105	38	210	105	38	105	38	210	105	38	105	38	105	38	7	105	38	7	38	19	7
	10	Min. Radius of Vertical Curves (K value):****																																	
	10-1	Crest (K value)	92	51	25	25	51	25	11	11	4	51	25	11	11	4	25	11	4	11	4	25	11	4	11	4	11	4	1	11	4	1	4	2	1
	10-2	Sag (K value)	63	45	30	30	45	30	18	18	9	45	30	18	18	9	30	18	9	18	9	30	18	9	18	9	18	9	3	18	9	3	9	6	3
	11	Max. Superelevation (%)	10%, 6% for Urban area																																
	12	Cross fall:																																	
	12-1	Paved (%)	2 - 3																																
	12-2	Unpaved (%)	3 - 4																																
	12-3	Paved shoulder (%)	> 3																																
	12-4	Unpaved shoulder (%)	> 4																																

*Annual Average Daily Traffic
**In case of installation of island for service road
***unpaved shoulder
**** Minimum vertical curve length based on Sight Distance Criteria. Other criteria are also required to be checked for the design.
[ ] : Desirable value
( ) : Minimum value

F: Flat area
R: Rolling area
M: Mountainous area
U: Urban area

*****The actual speed limit on the roads shall be those specified in Lao PDR applicable Law and regulation

For design of International roads, either ASEAN Highway standards or Asian Highway standards shall be applied instead of the design conditions (Table 3.3.30). Summary of these design standards are attached in Table 3.3.31 for ASEAN Highway Standards and Table 3.3.32 for Asian Highway standards. The details of each standard are attached in APPENDIX -G, H respectively.

Table 3.3.31 ASEAN Highway Standards

Highway classification		**Primary (4 or more lanes) (control access)**			**Class I (4 or more lanes)**		
Terrain classification		**L**	**R**	**M**	**L**	**R**	**M**
Design speed (km/h)		100-120	80-100	60-80	80-110	60-80	50-70
Width (m)	Right of way	(50-70) ((40-60))			(50-70) ((40-60))		
	Lane	3.75			3.5		
	Shoulder	3		2.5	3		2.5
Min. horizontal curve radius (m)		390	230	120	220	120	80
Type of pavement		Asphalt/cement concrete			Asphalt/cement concrete		
Max. superelevation (%)		(7) ((6))			(8) ((6))		
Max. vertical grade(%)		4	5	6	5	6	7
Min. vertical clearance (m)		4.50 [5.00]			4.50 [5.00]		
Structure loading (minimum)		HS20-44			HS20-44		
Highway classification		**Class II (2 lanes)**			**Class III (2 lanes)**		
Terrain classification		**L**	**R**	**M**	**L**	**R**	**M**
Design speed (km/h)		80-100	60-80	40-60	60-80	50-70	40-60
Width (m)	Right of way	(40-60) ((30-40))			30-40		
	Lane	3.5			3.00[3.25]		
	Shoulder	2.5		2	1.50[2]		1.0[1.5]
Min. horizontal curve radius (m)		200	110	50	110	75	50
Type of pavement		Asphalt/cement concrete			Double bituminous treatment		
Max. superelevation (%)		(10) ((6))			(10) ((6))		
Max. vertical grade(%)		6	7	8	6	7	8
Min. vertical clearance (m)		4.5			4.5		
Structure loading (minimum)		HS20-44			HS20-44		

Note: 1. Abbreviation: L = Level Terrain M = Mountainous Terrain R = Rolling Terrain

2. ( ) = Rural (( )) = Urban

3. [ ] = Desirable Values

4. The right of way width, lane width, shoulder width and max. superelevation rate in urban or metropolitan area can be varied if necessary to conform with the member countries design standards.

Table 3.3.32 Asian Highway Standards

Highway classification		Primary (4 or more lanes)				Class I (4 or more lanes)				Class II (2 lanes)				Class III (2 lanes)			
Terrain classification		L	R	M	S	L	R	M	S	L	R	M	S	L	R	M	S
Design Speed (km/h)		120	100	80	60	100	80	50		80	60	50	40	60	50	40	30
Width (m)	Right of way	(50)				(40)				(40)				(30)			
	Lane	3.50				3.50				3.50				3.00 (3.25)			
	Shoulder	3.00		2.50		3.00		2.50		2.50		2.00		1.5 (2.0)		0.75 (1.5)	
	Median strip	4.00		3.00		3.00		2.50		N/A		N/A		N/A		N/A	
Min. radii of horizontal curve (m)		520	350	210	115	350	210	80		210	115	80	50	115	80	50	30
Pavement slope (%)		2				2				2				2 - 5			
Soulder slope (%)		3 - 6				3 - 6				3 - 6				3 - 6			
Type of pevment		Asphalt / Cement concrete				Asphalt / Cement concrete				Asphalt / Cement concrete				Bituminous treatment			
Max. superelevation (%)		10				10				10				10			
Max. vertical grade (%)		4	5	6	7	4	5	6	7	4	5	6	7	4	5	6	7
Structure loading (minimum)		HS20-44				HS20-44				HS20-44				HS20-44			

Note: Figures in parentheses are desirable values.
Minimum radii of horizontal curve should be determined in conjunction with superelevation.
The recommended width of the median can be reduced with the proper type of guard fence.
The Parties should apply their national standards when constructing such as bridges, culverts and tunnels along the Asian Highway.

3.4 AT-GRADE INTERSECTION

3.4.1 General

A junction or intersection is the general area where two or more roads join or cross within which are included the carriageway(s) and roadside facilities for traffic movement in that area.

Most of the intersection-accidents occur at the very lightly trafficked at –grade intersections and from a traffic safety aspect these lightly trafficked intersections require as much attention as to those intersections where heavier conflicting traffic movements occur.

In this Chapter, Section 3.4.2-3.4.5 describes the design for all at-grade intersections.

3.4.2 Definitions

(1) A intersection

For the purposes of this Manual, an intersection shall be defined as the intersection of two or more classified roads and the design procedures and standards in this Manual shall be applied to such intersection.

(2) An access

An access shall be defined as the intersection of an unclassified road with a classified road and shall generally be provided within the road reserve boundary of classified road. An access shall have entry and exit radii of between 10 and 15 meters depending upon the turning characteristics of the expected traffic with no left or right turning lanes, left turn merging lane or traffic islands.

However in certain locations, the constant daily vehicular movement or heavy peak hour flows on an access may justify its design to intersection standards. This may occur for example at an entrance to an industrial development or factory site.

(3) Type of at grade intersection

At grade intersections can be classified into two main intersection categories based on the type of control used. For each category, there are a number of different intersection types.

1) **Priority Intersections**

Priority intersections will be adequate in most rural situations. Three types of T intersections are given below:

<u>Unchannelised T-intersection (Type A)</u>

The unchannelised design is suitable for intersections where there is a very small amount of turning traffic. It is the simplest design and has no traffic islands.

<u>Partly Channelised T-intersection (Type B)</u>

The partly channelised design is for intersections with a moderate volume of turning traffic. It has a traffic island in the minor road arm. In urban areas, the traffic island would normally be kerbed in order to provide a refuge for pedestrians crossing the road.

Channelised T-intersection (Type C)

The fully channelised design is for intersections with a high volume of turning traffic or high speeds. It has traffic islands in both the minor road and the main road.

Typical priority intersections in rural areas are shown in Figure 3.4.1.

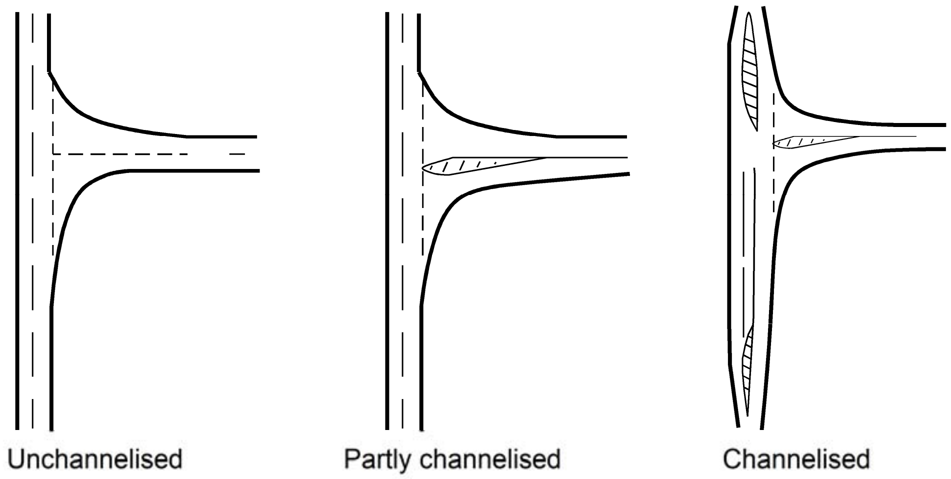


Figure 3.4.1 Typical T-Intersections

2) **Control Intersections**

Control intersections are mostly used in towns and trading centers. However, roundabouts can be used in rural areas in intersections between major roads or other intersections with high traffic volumes. A basic requirement for all controlled intersections is that drivers must see the control device soon enough to perform the action it indicates. There are two types of control intersections:

Roundabout (Type D)

Roundabouts are controlled by the rule that all entry traffic must give way to circulating traffic. The ratio of minor road incoming traffic to the total incoming traffic should preferably be at least 10 to 15%. Roundabouts can be of normal size, i.e. with central island radius 10m or more, or small size, i.e. with central island radius less than 10 m.

Signalised intersection (Type E)

Signalised intersections have conflicts separated by traffic signals. No conflicts are allowed between straight through traffic movements.

Typical layouts of control intersections are shown in Figure 3.4.2.

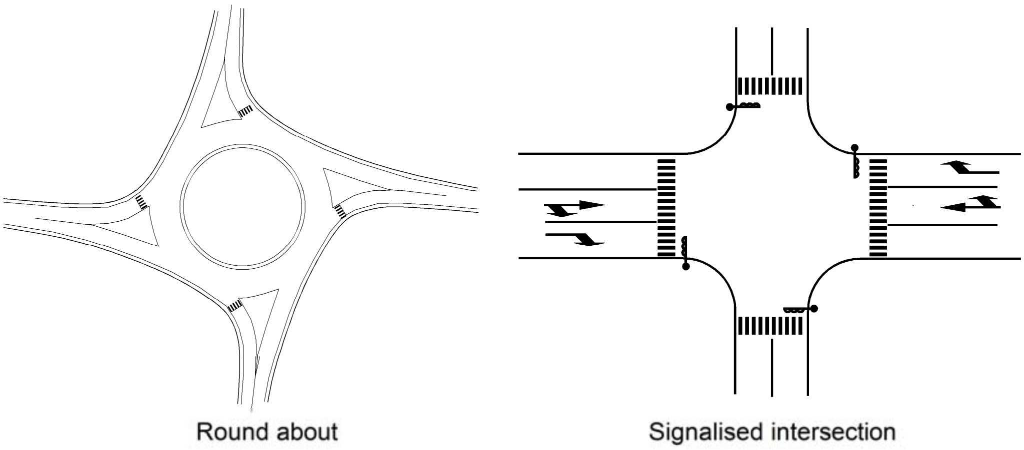


Figure 3.4.2 Typical Layouts for Control Intersections

(4) Intersection Maneuvers

Three basic movements or maneuvers occur at intersections namely merging, setting and cutting and merging. These maneuvers are illustrated below.

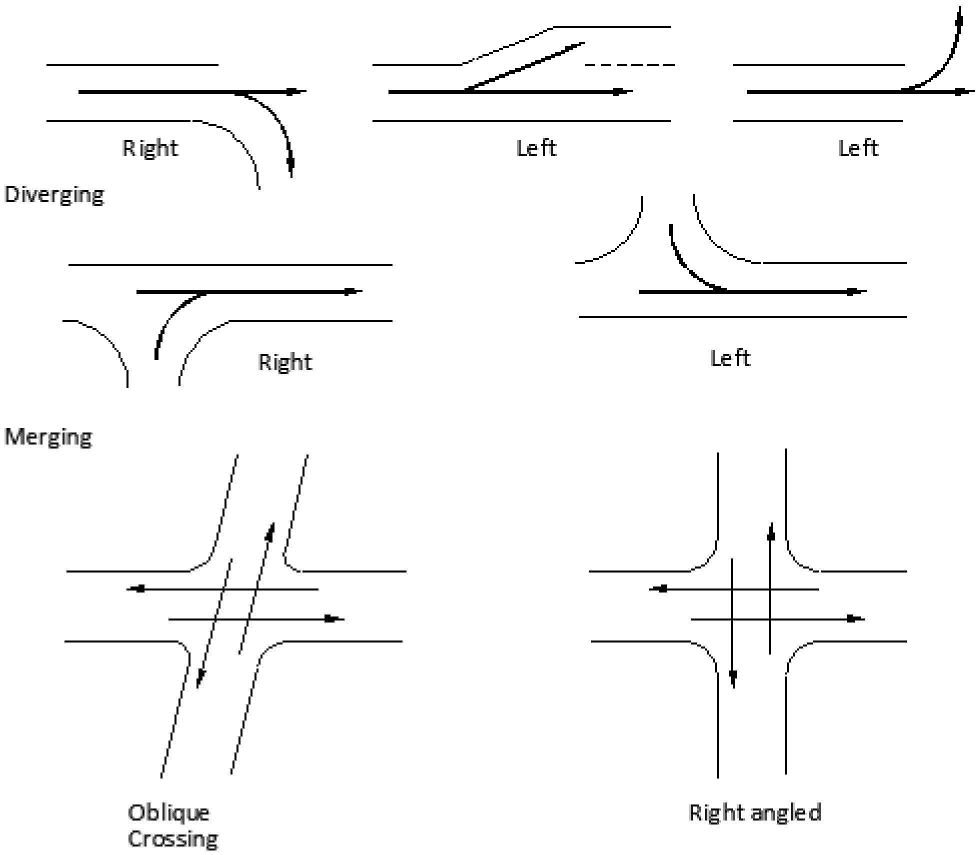


Figure 3.4.3 Intersection Maneuvers

(5) Intersection Design Speed

The Intersection Design Speed, which is the principal design parameter upon which the geometrical layout and capacity of an intersection is based, is the design speed of the major road in the vicinity of the intersection. This design speed will not necessarily be the same as the average major road design speed but may be higher or lower. Therefore the designer must give careful consideration to the selection of the appropriate Intersection Design Speed as this will greatly affect both the safety and efficiency of the intersection and construction cost.

For reasons of economy, the Intersection Design Speed shall not be more than 20 km/h higher than the average design speed of the major road.

For safety reasons, the Intersection Design Speed should never be less than 20 km/h lower than the average design speed for major road.

(6) The Major Road

At T-intersections, the through road, which will usually be carrying the higher traffic volume, shall be considered the major road.

At four, or more, leg intersections the major road shall be the road with the higher sectional characteristics as determined by:

(a) Right of way regulations on adjoining intersections

(b) Vehicle operating speeds

(c) Traffic volumes

The longer the section of road with continuous priority, higher vehicle operating speeds and/or traffic volumes, the higher rated the sectional characteristics shall be, regardless of the administrative classification of the road.

3.4.3 Design Requirement.

The design of at-grade intersection must take account of the following basic requirements

- Safety
- Operational comfort
- Capacity
- Economy

(1) Safety and Operational Comfort

A intersection is considered safe when it is perceptible, comprehensible and maneuverable. These three requirements can generally be met complying with the following guide-lines.

(a) Perception

(i) The intersection should be sited so that the major road approaches are readily visible.

(ii) The intersection should be sited on less than 2.5 grades.

(iii) Early widening of the intersection approaches.

(iv) The use of traffic islands in the minor road to emphasize a “yield” or “stop” requirement.

(v) The use of early and eye catching traffic signs.

(vi) Optical guidance by landscaping and the use of road furniture, especially where an intersection must be located on a crest curve.

(vii) The provision of visibility splays which ensure unobstructed sight lines to the left and right along the major road.

(viii) The angle of intersection of the major and minor road should be between 70 and 110 degrees.

(ix) The use of single lane approaches is preferred on the minor road in order to avoid mutual sight obstruction from two vehicles waiting next to each other to turn or cross the major road.

(b) Comprehension

(i) The right of way should follow naturally and logically from the intersection layout.

(ii) The types of intersection used throughout the whole road network should be similar.

(iii) The provision of optical guidance by the use of clearly visible kerbs, traffic islands, road markings, roads signs and other road furniture.

(c) Maneuverability

(i) All traffic lanes should be of adequate width for the appropriate vehicle turning characteristics.

(ii) The edges of traffic lanes should be clearly indicated by road markings.

(iii) Traffic islands and kerbs should not conflict with the natural vehicle paths.

(2) Capacity

The operation of uncontrolled intersections depends principally upon the frequency of gaps which naturally occur between vehicles in the main road flow.

These gaps should be of sufficient duration to permit vehicles from the minor road to merge with or cross, the minor road flow. In consequence intersections are limited in capacity but this capacity may be optimized by, for example, channelization or the separation of maneuvers.

(3) Economy

An economical intersection design generally results from a minimization of the construction, maintenance and operational costs.

Delay can be an important operational factor and the saving in time otherwise lost may justify a more expensive, even grade separated, intersection.

Loss of lives, personal injuries and damage to vehicles caused by intersection-accidents are considered as operational "costs" and should be taken into account.

The optimum economic return may often be obtained by a phased construction for example by constructing initially an at-grade intersection which may latter be grade separated.

3.4.4 Intersection Design Procedure

The procedure to be used for intersection design involves four basic steps which are as follows:

(i) Data collection (3.4.4(1))

(ii) Define the major road (3.4.2(6)) and determine the intersection design speed (3.4.2(5))

(iii) Select the basic intersection layout and check that it offers adequate capacity for the predicted traffic man oeuvres.

(iv) Refine and modify the basic intersection layout to meet the safety and operational requirements outlined in 3.4.3(1). This is done by applying the principles of intersection design which are described in detail in 3.4.5 under the following headings:

- (a) Distance between adjoining intersections
- (b) Visibility splays
- (c) Turning lanes
- (d) Turning roadways
- (e) Traffic islands and minor road widening
- (f) Alignment of the major road.

(1) Data collection

The following data will be required to ensure that a safe economic and geometrically satisfactory design is produced:

(a) A plan to scale of at least 1:500, showing all topographical details.

(b) Characteristics of the crossing or joining roads, i.e. horizontal and vertical alignments, distances to adjoining intersections, cross-sectional data, vehicle operating speeds, etc.

(c) Characteristics of the predicted volumes and compositions of the various traffic streams (traffic volume in each direction during peak period).

(d) Other factors affecting the design, such as topographical or geotechnical peculiarities, locations of public utilities, pedestrian movements, adjacent land usage etc.

(e) Traffic accident data, especially where the reconstruction of an existing intersection is involved.

(2) Basic intersection layout and capacity

The basic intersection layout for both single and dual carriageway roads is the T-intersection with the major road traffic having priority over the minor road traffic. Cross-roads, although not recommended, may also be used but only on single carriageway roads where traffic flows are very low and where site conditions will not permit the use of staggered T-intersections.

Where staggered T-intersections are used to replace a cross-road, the left-right stagger as indicated in Figure 3.4.4 is preferred to the right-left stagger and the minimum stagger should be 50m. On traffic grounds this because in the latter case opposing queues of left turning vehicles from the major road will have to wait side by side with the consequent possibility the whole intersection locking.

In a right/left staggered intersection the minimum distance should be at least 100 meters and should be longer in order to allow the provision of left turn lanes in the major road. The length of left turn lane depends on the junction design speed and traffic turning left in pcu/h.

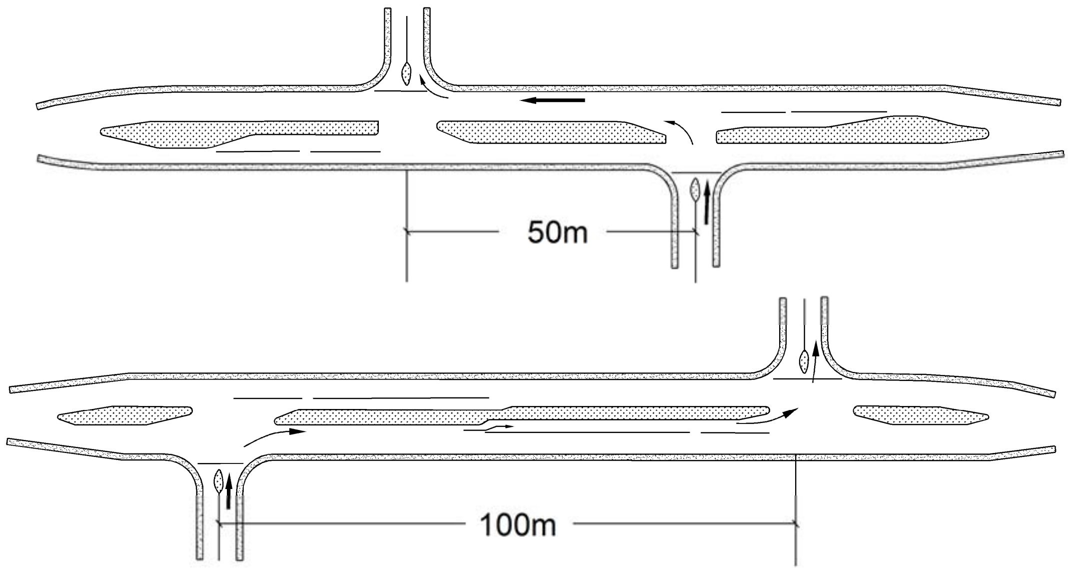


Figure 3.4.4 Left/Right and Right/Left staggered Intersections

Where more complex intersection layouts involving the intersection of four or more roads are encountered, these should be simplified by realigning the approaches, to safer, more comprehensible and maneuverable layouts. Examples of such simplifications are given in Figure 3.4.5.

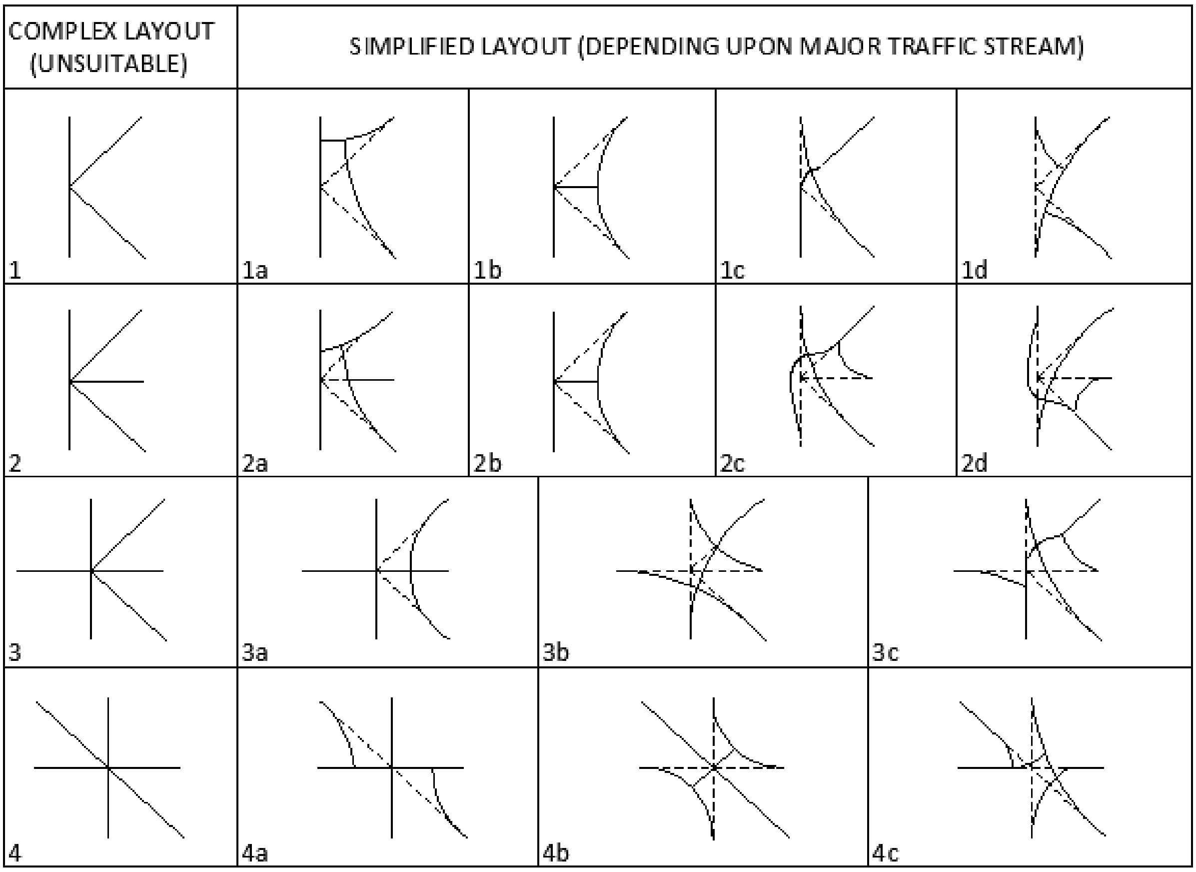


Figure 3.4.5 Example of the Simplification of Complex Intersection

Having selected the basic intersection layout, it is necessary to check that it offers sufficient capacity for both the major and minor road turning maneuvers.

The capacity of a major/minor priority intersection depends upon the traffic flow and turning proportions from the different approaches. Because the basic of control means that traffic on the major road is not delayed, capacity is expressed in terms of the number of minor road pcu that can enter the intersection and particular level of major road flow.

3.4.5 Principles of Intersection Design

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

(1) Distance between adjoining intersections

The minimum distance between consecutive intersections shall preferably be equal to (10 x VD) meters where VD is the major road design speed in km/h.

Where it is impossible to provide this minimum spacing, then the design shall incorporate either, or both, of the following:-

(i) A distance between minor road centerlines equal to the passing sight distance appropriate for the Intersection Design Speed plus half the length of the widened major road sections at each intersection, or

(ii) A grouping of minor road intersections into pairs to form staggered T-Intersections and a distance between pairs as in (i) above.

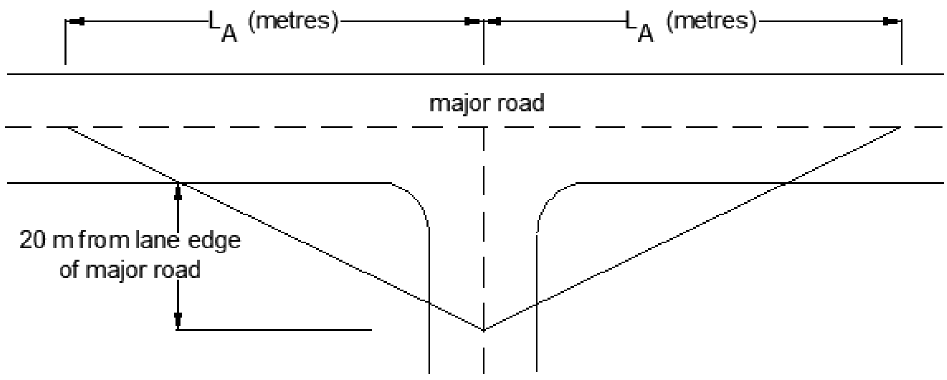
(2) Visibility splays

At major/minor priority intersections visibility splays to the standards described below should be provided at all new intersections and aimed at for existing intersections.

The visibility splays for both the "Approach Conditions" (Figure 3.4.6) and the "Stop Conditions"(Figure 3.4.7) should be provided.

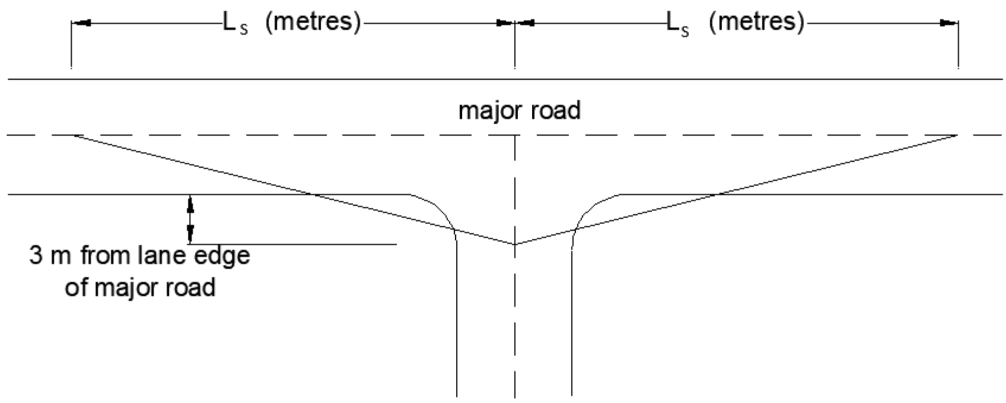
On the minor road, particularly where the approach to the intersection is on a horizontal curve, the visibility of traffic signs is essential and a visibility splay in accordance with Figure 3.4.8 must be provided.

Where site conditions make it impossible to improve an existing intersection to these standards, at least the visibility splays for the "Stop Condition" must always be provided.



Intersection Design Speed (km/h)	40	50	60	70	80	90	100	120
Length L_A (meters)	110	135	160	185	210	235	260	310

Figure 3.4.6 Visibility splays for "Approach condition"



Intersection Design Speed (km/h)	40	50	60	70	80	90	100	120
Length L_S (meters)	120	145	185	230	280	340	400	500

Figure 3.4.7 Visibility splays for "Stop condition"

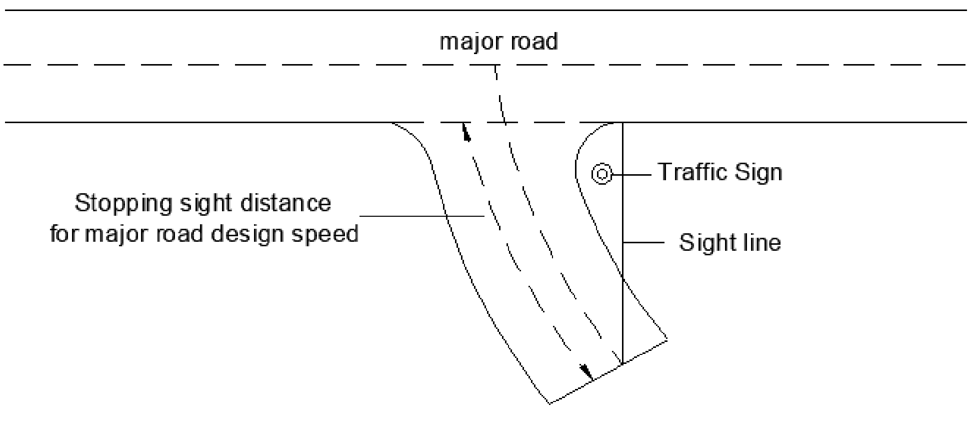


Figure 3.4.8 Minor Road Approach Visibility Requirements

(3) Turning lanes

Left and right turning lanes are of particular value on the higher speed roads when a vehicle slowing down to turn and leave the major road may impede following vehicles.

(i) Right turn lanes

Right turn lanes, comprising diverging sections and deceleration sections, shall be provided under any of the following conditions:

(a) On dual carriageway roads.

(b) When the Intersection Design Speed is 100 km/h or greater and the AADT, on the major road in Design Year 10 is greater than 2000 pcu.

(c) When the AADT, of the right turning traffic in Design Year 10 is greater than 800pcu.

(d) Where intersections are sited on right-hand bends and perception of the intersection for Major road traffic would be greatly improved by its inclusion.

(e) On four or more lane undivided highways.

The minimum lengths for diverging sections are given in Table 3.4.1 and shall be formed by direct tapers.

The minimum lengths for deceleration sections are dependent upon the Intersection Design Speed, the exit radius from the major road into the minor road and the approach gradient of the major road. Where right turn lanes are required, the exit radius shall be 25 meters and the minimum length of deceleration sections shown in Table 3.4.2 apply.

The width of the deceleration lane shall be the same as the major road approach lane.

(ii) Left turn lanes

A separate lane for left turning traffic (i.e. traffic turning right from the major road into the minor road) shall be provided under any of the following conditions:

(a) On dual carriageway roads.

(b) When the Intersection Design Speed is 100 km/h or greater and the. A.A.D.T. on the major road in Design Year 10 is greater than 1500 pcu.

(c) When the ratio of the major road flow being cut to the left turning flow exceeds the values given on Table 3.4.3.

(d) On four, or more lane undivided highways. A left turn lane will consist of a diverging section, a deceleration section and a storage section.

The minimum lengths for diverging sections are as for left turn lanes and are given in Table 3.4.1 and shall be formed by direct tapers.

The minimum lengths for deceleration sections are given in Table 3.4.2

.

Table 3.4.1 Minimum lengths for diverging section

Intersection design speed(km/h)	120	100	80	70 or less
Length of diverging section(m)	60	50	40	30

Table 3.4.2 Minimum lengths for left turn deceleration sections

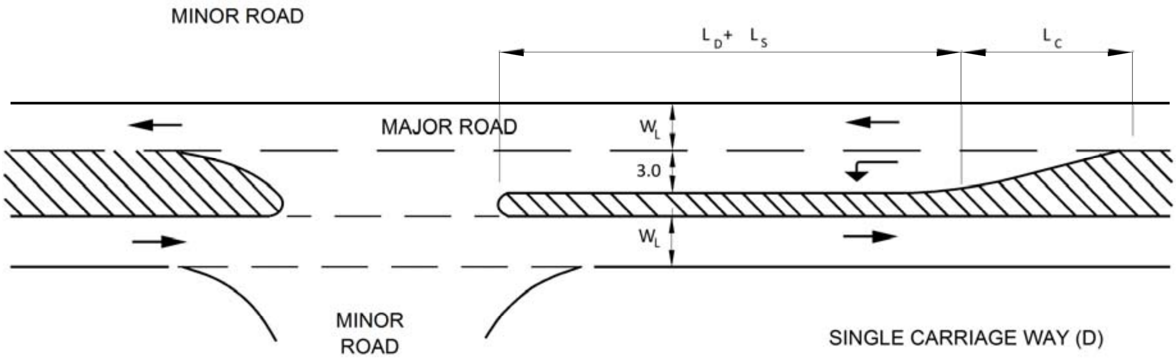
Intersection design speed(km/h)	120	100	80	70	60	50	40
Length of deceleration section (m)	160	120	100	80	60	40	30

The lengths of storage sections for left turning traffic are given in Table 3.4.3.

Table 3.4.3 Lengths of storage sections for left turning traffic

Traffic turning right (pcu/h)	Length of storage section (m)
0 - 150	20
151 - 300	40
over 300	N x 9.75 (where N is No. of pcu turning right per two minutes)

The width of the deceleration and storage sections shall be 3.0 meters.



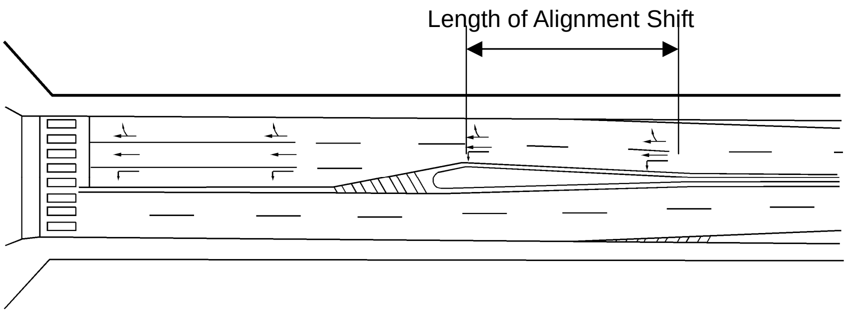
Where: L_C = Length of divering section
L_D = Length o deceleration sectio
L_S = Length of storage section
W_L = Width of through carriageway

Figure 3.4.9 Layout for Left Turn Lanes

In case of median width less than the width of left turn lane, alignment shift is required for providing left turn lane. The calculation of length of alignment shift section is shown in Table 3.4.4

Table 3.4.4 Length of Alignmnent Shift

Area / Design Speed (km/h)	Others		Urban Area	
	Calculation formula	Minimum Value	Calculation formula	Minimum Value
100		105	-	-
80	$(V \times \Delta W)/2$	85		55
60		60		40
40		35	$(V \times \Delta W)/3$	35
30	$(V \times \Delta W)/3$	30		30
20		25		25



(4) Turning Roadways

The widths of turning roadway for intersections are governed by the volumes of turning traffic and the types of vehicles to be accommodated. In almost all cases, turning roadways are designed for use by right-turning traffic. Minimum Edge-of-Traveled-Way Designs is one of typical types of right-turning roadways at intersections. Where it is appropriate to provide for turning vehicles within minimum space, as at channelized intersections, the corner radii should be based on minimum turning path of the selected design vehicles. Curve designs for edge of traveled way are shown in Table 3.4.5 and Table 3.4.6.

Table 3.4.5 Edge-of-Traveled-Way Designs for Turns at Intersections – Simple Curve Radius with Taper

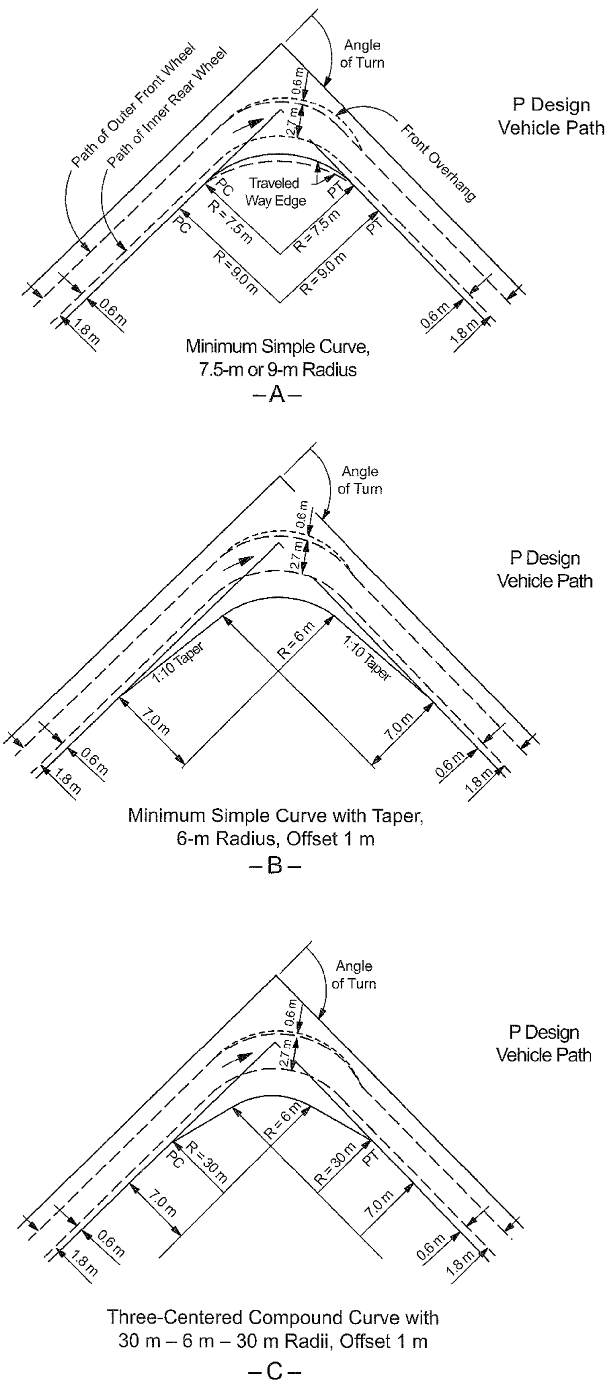
Angle of Turn (◦)	Design Vehicle	Simple Curve Radius (m)	Simple Curve Radius with Taper			Angle of Turn (◦)	Design Vehicle	Simple Curve Radius (m)	Simple Curve Radius with Taper		
			Radius (m)	offset (m)	Taper L:T				Radius (m)	offset (m)	Taper L:T
30	P	18	-	-		105	P	-	6	0.8	8:1
	SU-9	30	-	-			SU-9	-	11	1.0	10:1
	SU-12		-	-			SU-12	-	14	1.2	10:1
	WB-12	45	-	-			WB-12	-	12	1.2	10:1
	WB19	110	67	1	15:1		WB19	-	35	1.0	15:1
45	P	15	-	-		120	P	-	6	0.6	10:1
	SU-9	23	-	-			SU-9	-	9	1.0	10:1
	SU-12	35	-	-			SU-12	-	11	1.8	8:1
	WB-12	36	-	-			WB-12	-	11	1.5	8:1
	WB19	70	43	1.2	15:1		WB19	-	30	1.5	15:1
60	P	12	-	-		135	P	-	6	0.5	10:1
	SU-9	18	-	-			SU-9	-	9	1.2	10:1
	SU-12	30	-	-			SU-12	-	12	1.2	8:1
	WB-12	28	-	-			WB-12	-	9	2.5	15:1
	WB19	50	43	1.2	15:1		WB19	-	24	1.5	20:1
75	P	11	8	0.6	10:1	150	P	-	9	0.6	10:1
	SU-9	17	14	0.6	10:1		SU-9	-	9	1.2	8:1
	SU-12	27	18	0.6	10:1		SU-12	-	11	2.1	8:1
	WB-12	-	18	0.6	15:1		WB-12	-	9	2.0	8:1
	WB19	-	43	1.2	20:1		WB19	-	18	3.0	10:1
90	P	9	6	0.8	10:1	180	P	-	5	0.2	20:2
	SU-9	15	12	0.6	10:1		SU-9	-	9	0.5	10:1
	SU-12	24	14	1.2	10:1		SU-12	-	11	2.0	10:1
	WB-12	-	14	1.2	10:1		WB-12	-	6	3.0	5:1
	WB19	-	36	1.3	30:1		WB19	-	17	3.0	15:1

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Table 3.4.6 Edge-of-Traveled-Way Designs for Turns at Intersections – Tree-Centered Curves

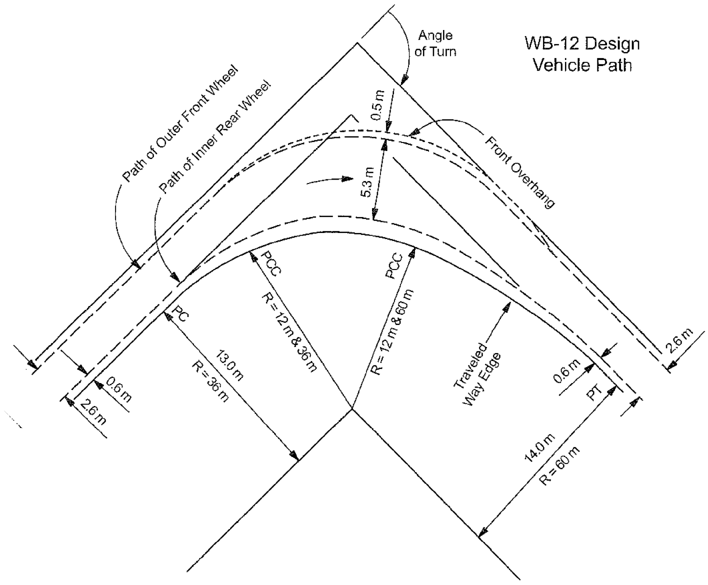
Angle of Turn (◦)	Design Vehicle	Three-Centered Compound		Three-Centered Compound		Angle of Turn (◦)	Design Vehicle	Three-Centered Compound		Three-Centered Compound	
		Curve Radii (m)	Symmetric offset (m)	Curve Radii (m)	Asymmetric Offset (m)			Curve Radii (m)	Symmetric offset (m)	Curve Radii (m)	Asymmetric Offset (m)
30	P	-	-	-	-	105	P	30-6-30	0.8	-	-
	SU-9	-	-	-	-		SU-9	30-11-30	1.0	-	-
	SU-12	-	-	-	-		SU-12	61-11-61	1.8	18-12-58	0.5-1.8
	WB-12	-	-	-	-		WB-12	30-11-30	4.5	110-23-180	1.2-3.2
	WB19	-	-	-	-		WB19	160-15-160	4.5	110-23-180	1.2-3.2
45	P	-	-	-	-	120	P	30-6-30	0.6	-	-
	SU-9	-	-	-	-		SU-9	30-9-30	1.0	-	-
	SU-12	-	-	-	-		SU-12	61-11-61	1.8	18-12-58	0.5-1.5
	WB-12	-	-	-	-		WB-12	36-9-36	2.0	30-9-55	0.6-2.7
	WB19	140-72-140	0.6	36-43-150	1.0-2.6		WB19	160-21-160	3.0	24-17-160	5.2-7.3
60	P	-	-	-	-	135	P	30-6-30	0.5	-	-
	SU-9	-	-	-	-		SU-9	30-9-30	1.2	-	-
	SU-12	-	-	-	-		SU-12	61-12-61	1.2	18-2-55	0.5-1.5
	WB-12	-	-	-	-		WB-12	36-9-36	2.0	30-8-55	1.0-4.0
	WB19	120-30-120	4.5	34-30-67	3.0-3.7		WB19	180-18-180	3.6	30-18-95	2.1-4.3
75	P	30-8-30	0.6	-	-	150	P	23-6-23	0.6	-	-
	SU-9	36-14-36	0.6	-	-		SU-9	30-9-30	1.2	-	-
	SU-12	61-11-61	1.5	18-14-61	0.3-1.4		SU-12	61-11-61	2.0	18-12-61	0.3-1.4
	WB-12	36-14-36	1.5	36-14-60	0.6-2.0		WB-12	30-9-30	2.0	28-8-48	0.3-305
	WB19	134-23-134	4.5	40-30-165	1.5-3.6		WB19	145-17-145	4.5	43-18-170	2.4-3.0
90	P	30-8-30	0.8	-	-	180	P	15-5-15	0.2		
	SU-9	36-12-36	0.6	-	-		SU-9	30-9-30	0.5		
	SU-12	61-9-61	2.1	18-14-61	0.3-1.4		SU-12	46-11-46	1.9	15-11-40	1.7-21
	WB-12	36-12-36	1.5	36-14-60	0.6-2.0		WB-12	30-6-30	3.0	26-6-45	2.0-4.0
	WB19	120-20-120	3.0	48-21-110	2.0-3.0		WB19	245-14-245	6.0	30-17-275	4.5-4.5

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)



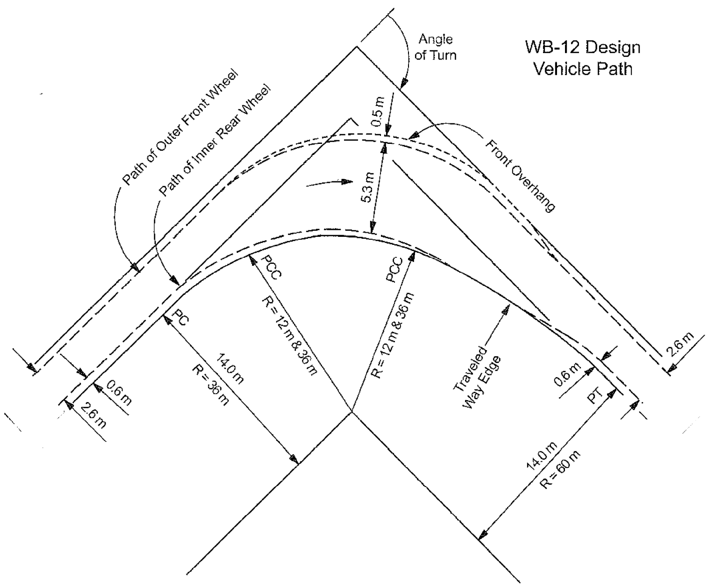
Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Figure 3.4.10 Minimum Edge-of-Traveled-Way Designs (Passenger Vehicles)



Three-Centered Compound Curve with
36 m – 12 m – 60 m Radii, Offset 1 m and 2 m

–A–

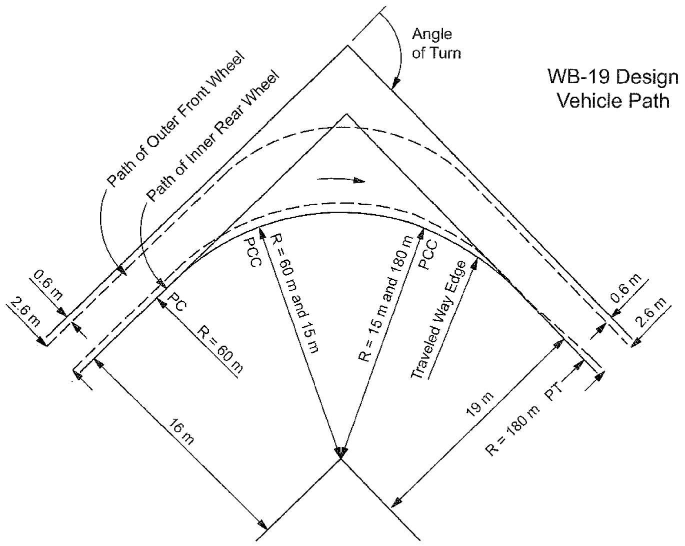


Three-Centered Compound Curve with
36 m – 12 m – 36 m Radii, Offset 2 m

– B –

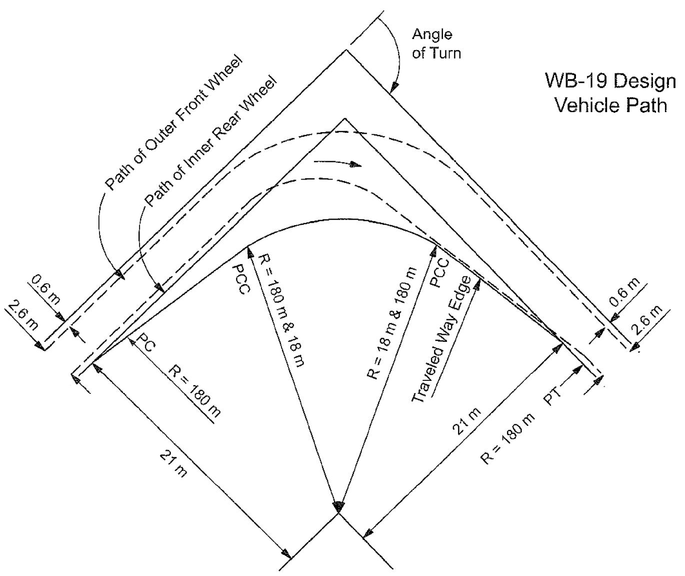
Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Figure 3.4.11 Minimum Edge-of-Traveled-Way Designs (WB-12)



WB-19 Interstate Semitrailer Combination,
Three-Centered Compound Curve,
60 m – 15 m – 180 m Radii, Offset 1 m and 4 m

– A –



WB-19 Interstate Semitrailer Combination,
Three-Centered Compound Curve,
180 m – 18 m – 180 m Radii, Offset 3 m

– B –

Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Figure 3.4.12 Minimum Edge-of-Traveled-Way Designs (WB-19)

(5) Traffic Islands and Minor Road Widening

Traffic islands should be provided where necessary, at major/minor priority junctions, for the following reasons:

(i) To assist traffic streams to intersect or merge at suitable angles.

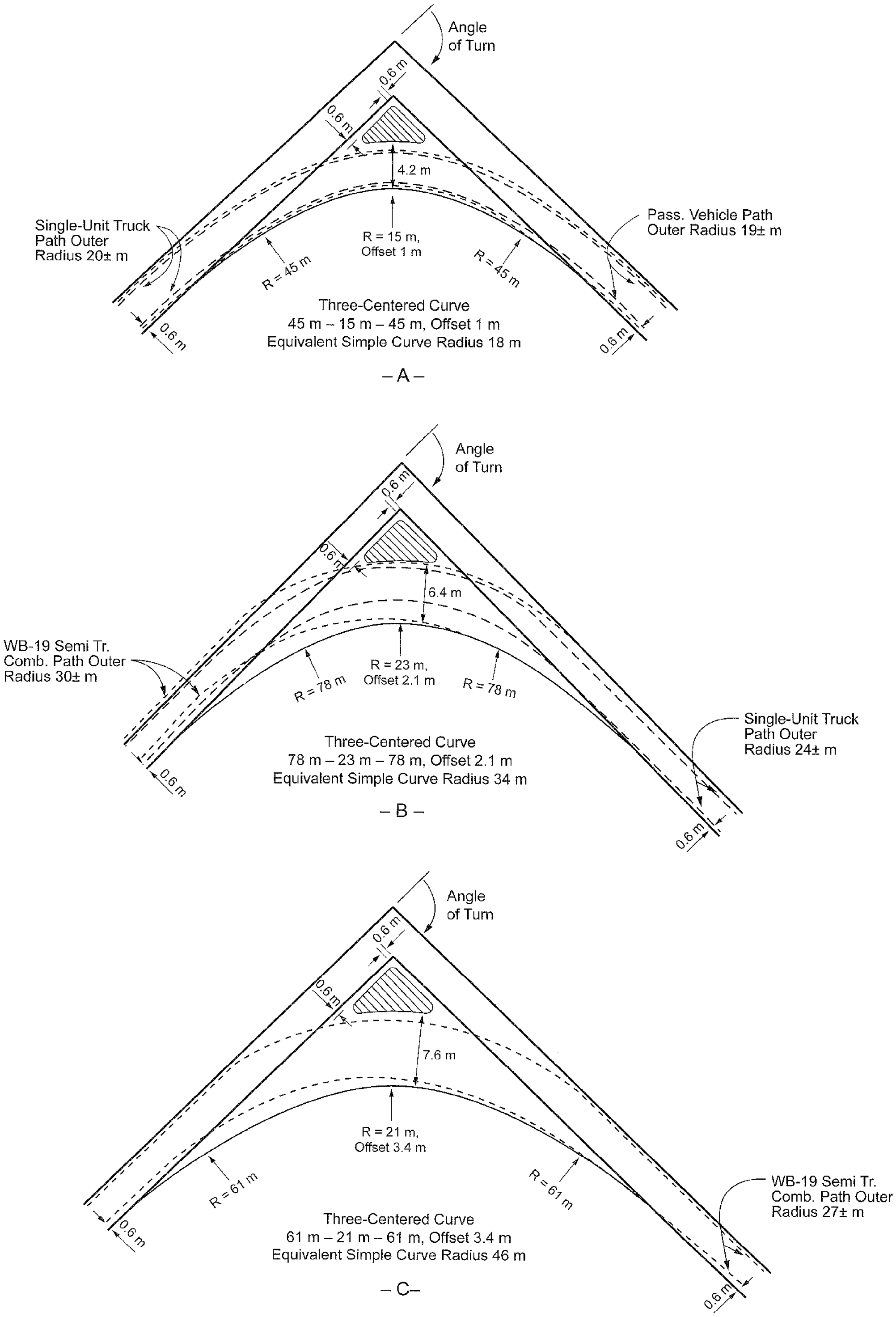
(ii) To control vehicle speeds.

(iii) To provide shelter for vehicles waiting to carry out certain manoeuvres such as turning left.

(iv) To assist pedestrians to cross.

Islands are either elongated or triangular in shape and are situated in areas not normally used as vehicle paths, the dimensions depending upon the particular junction or bus layout. Traffic islands bordered by raised kerbs should not be used in the major road unless lighting is provided but can be used without lighting in the minor road. To enable raised islands to be clearly seen they should have an area of at least 4.5 square meters and where necessary additional guidance should be given by carriageway markings in advance of the nose supplemented, if necessary, by speed humps.

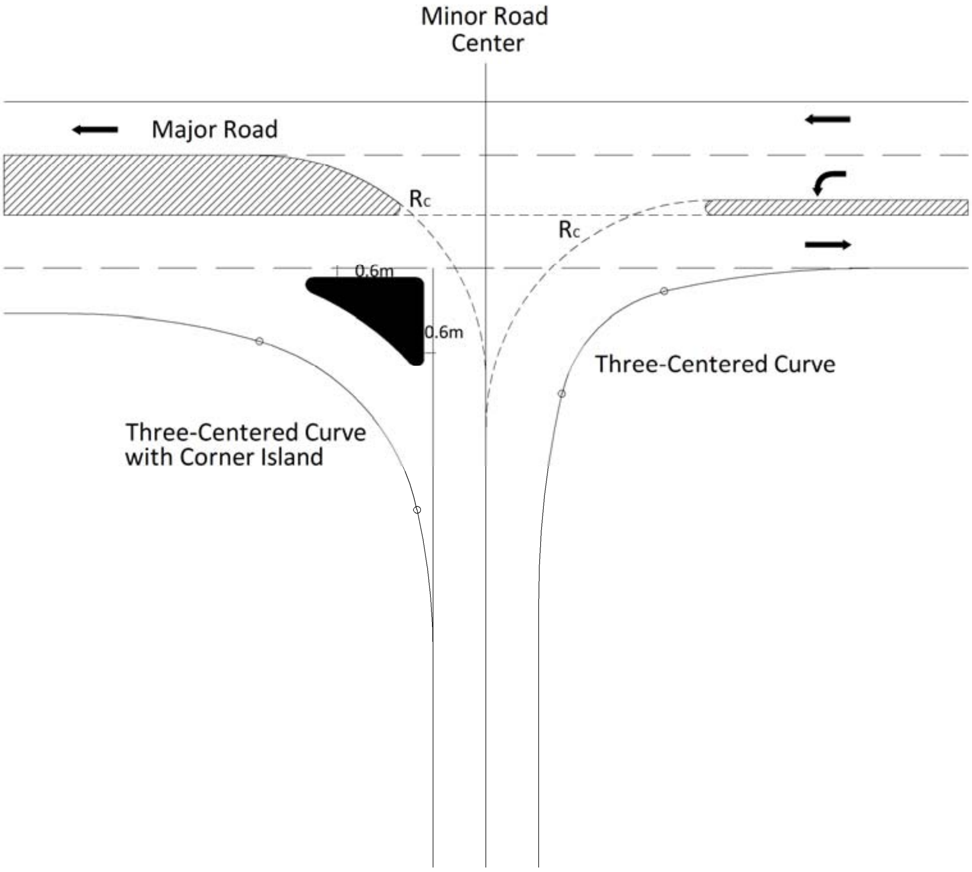
A turning roadway should be designed to provide at least the minimum side island and the minimum width of roadway. The turning roadway should be wide enough to permit the right and left wheel tracks of a selected vehicle to be within the edges of the traveled way by about 0.6m on each side. Generally, the turning roadway width should not be less than 4.2m. Figure 3.4.13 shows minimum turning roadway designs for 90-degree right turn.



Source: AASHTO, A Policy on Geometric Design of Highway and Streets (2011)

Figure 3.4.13 Minimum Turning Roadway Designs with Corner Islands at Urban Locations

Intersection Layout Type channelised intersection, as shown on Figure 3.4.14, is to be used whenever a separate left turn lane is required in accordance with the requirements of 3.4.5(3). It should be noted that this layout also makes provision for a right turn lane.



Notes:

1. Rc Control radius dependent up on vehicle turning characteristics.

   Recommended value = 15 m

2. Three-Centered Curve with corner island for minimum turning roadway design is shown in Figure 3.4.13.

3. Three-Centered Curve for minimum turning roadway design is shown in Table 3.4.6, Figure 3.4.10, Figure 3.4.11 and Figure 3.4.12.

Figure 3.4.14 Typical Intersection Layout

3.4.6 Roundabouts

(1) Typical type of roundabout

Roundabouts can be classified into three basic categories according to size and number of lanes to facilitate discussion of specific performance and design issues;

- Mini-roundabout
- Single-lane roundabout
- Multilane roundabout

Table summarizes and compares some fundamental design and operational elements for each of the three roundabout categories discussed herein.

Table 3.4.7 Comparison of Roundabout Types

Design Element	Mini-Roundabout	Single-Lane Roundabout	Multilane Roundabout
Recommended maximum entry design speed	25 to 30 km/h	30 to 40 km/h	40 to 50 km/h
Maximum number of entering lanes per approach	1	1	2+
Typical inscribed circle diameter	13 to 27m	27 to 46m	40 to 76m
Central island treatment	Mountable	Raised	Raised
Typical daily volume on 4-leg roundabout(vehicle/day)	0 to 15,000	0 to 20,000	20,000+

• Mini-roundabout

Mini-roundabouts are small roundabout used in low-speed urban environments, with average operating speed s of 50 km/h or less. Figure provides an example of a mini-roundabout. They can be useful in low-speed urban environmental in cases where conventional roundabout design is precluded by right-of-way constraints.

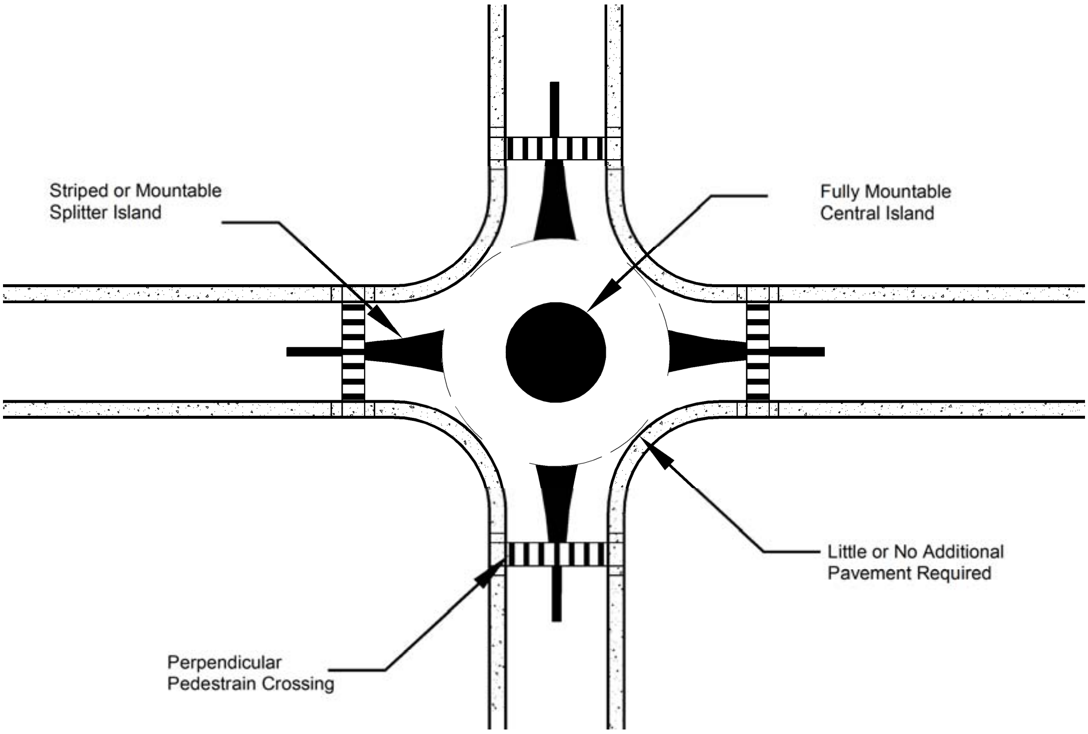


Figure 3.4.15 Typical Mini Roundabout

• Single-lane roundabout

This type of roundabouts characterized as having a single entry lane at all legs and one circulatory lane. Figure provides an example of a typical urban single-lane roundabout. They are distinguished from mini-roundabouts by their larger inscribed circle diameters and non-mountable central islands.

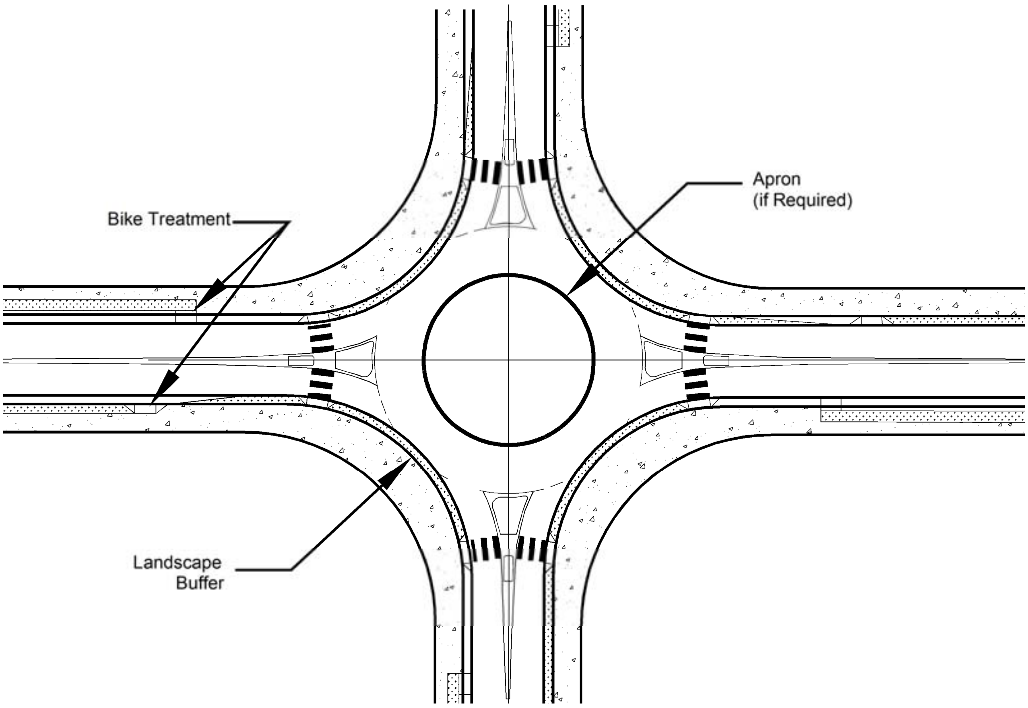


Figure 3.4.16 Typical Single-lane Roundabout

- Multilane roundabout

Multilane roundabouts include all roundabouts that have at least one entry with two or more lanes. In some cases, the roundabout may have a different number of lanes on one or more approaches. For example, a roundabout with both two-lane entries would still be considered a multilane roundabout. Figure provides an example of typical multilane roundabout.

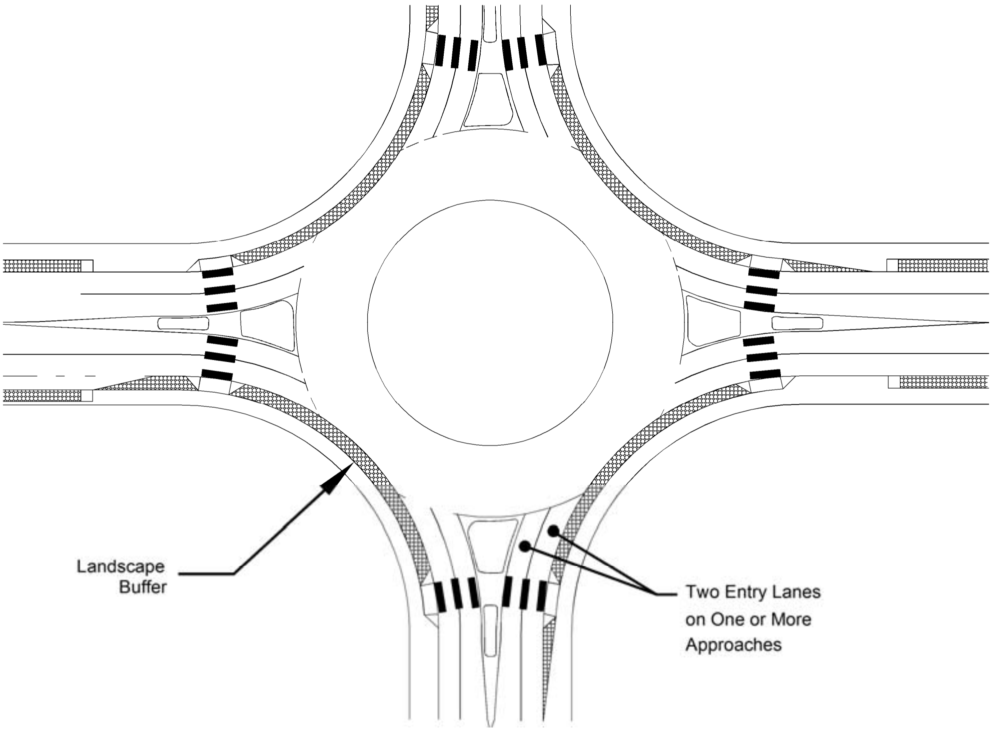


Figure 3.4.17 Typical Multilane Roundabout

(2) Design Speed

According to Roundabouts: an informational guide U.S. Department of Transportation, recommended maximum entry design speeds for roundabouts at various intersection site categories are provided in Table 3.4.8.

Table 3.4.8 Recommended Maximum Entry Design Speed

Site Category	Recommended Maximum Entry Design Speed
Mini-Roundabout	25 km/h
Urban Compact	25 km/h
Urban Single Lane	35 km/h
Urban Double Lane	40 km/h
Rural Single Lane	40 km/h
Rural Double Lane	50 km/h

(3) Geometric Elements

Representative elements are mentioned in this section. More detail elements are referred to Roundabouts: an informational guide U.S. Department of Transportation.

Figure 3.4.18 provides a review of the basic geometric features and dimensions of a roundabout.

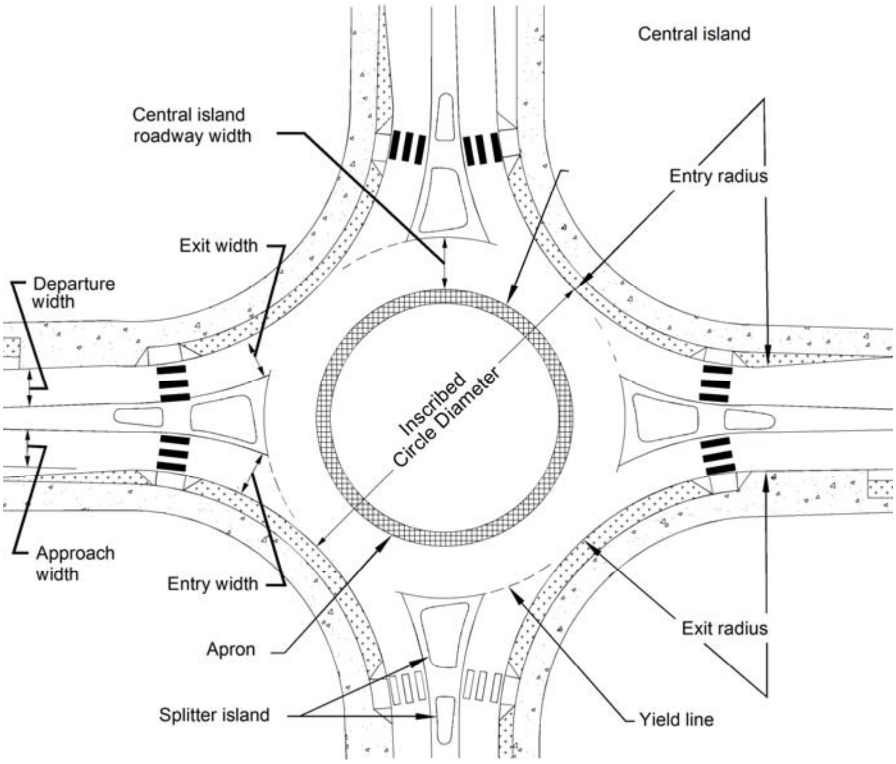


Figure 3.4.18 Basic Geometric elements fo a Rroundabout

1) **Inscribed Circle Diameter**

At single-lane roundabouts, the size of the inscribed circle is largely dependent upon the turning requirements of the design vehicle. In general, the inscribed circle diameter should be a minimum of 30 m to accommodate a WB-15 design vehicle. Smaller roundabouts can be used for some local street or collector street intersections, where the design vehicle may be a bus or single-unit truck. At double-lane roundabouts, accommodating the design vehicle is usually not a constraint. Generally, the inscribed circle diameter of a double-lane roundabout should be a minimum of 45 m.

Table 3.4.9 provides recommended ranges of inscribed circle diameters for various site locations.

Table 3.4.9 Recommended Inscribedcircle Diameter Ranges

Site Category	Typical Design Vehicle	Inscribed Circle Diameter Range*
Mini-Roundabout	Single-Unit Truck	13 - 25m
Urban Compact	Single-Unit Truck/Bus	25 - 30m
Urban Single Lane	WB-15	30 - 40m
Urban Double Lane	WB-15	45 - 55m
Rural Single Lane	WB-20	35 - 40m
Rural Double Lane	WB-20	55 - 60m

*Assumes 90-degree angles between entries and no more than four legs.

**It is for reference because maximum design vehicle is WB-19 in Lao.

2) **Entry Width**

Entry width is the largest determinant of a roundabout's capacity typical entry widths for single-lane entrances range from 4.3 to 4.9 m; however, values higher or lower than this range may be required for site-specific design vehicle and speed requirements for critical vehicle paths.

3) **Circulatory roadway width**

At single-lane roundabouts, the circulatory roadway should just accommodate the design vehicle. Appropriate vehicle-turning templates or a CAD-based computer program should be used to determine the swept path of the design vehicle through each of the turning movements. Table 3.4.10 provides minimum recommended circulatory roadway widths for two lane roundabouts where semi-trailer traffic is relatively infrequent.

Table 3.4.10 Minimum CirculatoryLane widths for Two-lane Roundabout

Inscribed Circle Diameter	Minimum Circulatory Lane Width*	Central Island Diameter
45m	9.8m	25.4m
50m	9.3m	31.4m
55m	9.1m	36.8m
60m	9.1m	41.8m
65m	8.7m	47.6m
70m	8.7m	52.6m

*Based on AASHTO Table III-20, Case III (A)(4). Assumes infrequent semi-trailer use (typically less than 5 percent of the total traffic). Refer to AASHTO for cases with higher truck percentages.

4) **Central Island**

The central island of a roundabout is the raised, nontraversable area encompassed by the circulatory roadway; this area may also include a traversable apron. Where aprons are used, they should be designed so that they are traversable by trucks, but discourage passenger vehicles from using them. They should generally be 1 to 4 m wide and have a cross slope of 3 to 4 percent away from the central island. To discourage use by passenger vehicles, the outer edge of the apron should be raised a minimum of 30 mm above the circulatory roadway surface.

3.4.7 Grade Separated Intersection

There is a wide variety of types of interchanges that can be employed under the various circumstances that warrant the application of interchanges. The major determinants of the type of interchange to be employed at any particular site are the traffic composition, access control, classification and characteristics of the intersecting road.

For detail design of grade separated intersection, a policy on Geometric Design of Highways and Streets (AASHTO 2011) is advisable.

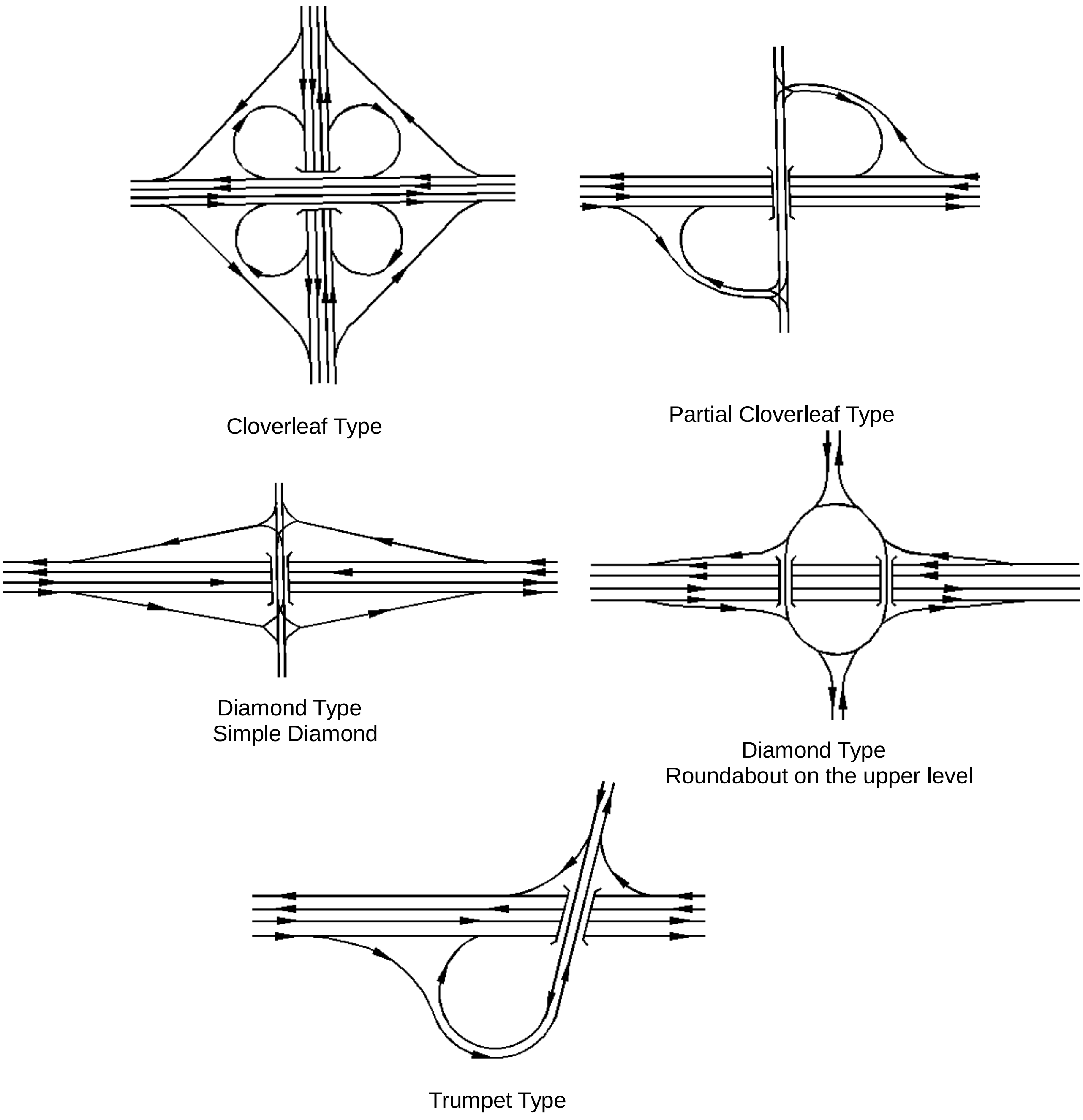


Figure 3.4.19 Typical Grade Separated Intersection

3.5 ROAD FURNITURE AND OTHER FACILITIES

3.5.1 General

Road furniture represents a collection of marginal elements intended to improve the driver's perception and comprehension of the continually changing appearance of the road.

- Traffic islands, kerbs and road markings delineate the pavement edges and thereby clarify the paths that vehicles are to follow. Safety fences prevent cars from leaving the road at locations where this would have the most severe consequences.
- Fences and gates along the road reserve are a means of controlling access.
- Marker posts assist in a timely perception of the alignment ahead and, when equipped with reflectors, provide good optical guidance at night.
- Traffic signs provide essential information to drivers for their safe and efficient maneuvering on the road.

3.5.2 Traffic Island

A traffic island is a defined area between traffic lanes for the control of vehicle movements and which may also be used as a pedestrian refuge.

Traffic island may take the form of an area delineated by barrier kerbs or a pavement area marked by paint or a combination of these.

Traffic islands are generally included in the design of intersections for one or more of the following purposes:

- Separation of conflicts
- Control of angle of conflict
- Reduction of excessive pavement areas
- Regulation of traffic and indication of proper use of intersection
- Arrangements to favor a predominant turning movement
- Protection of pedestrians
- Protection and storage of turning and crossing vehicles
- Location of traffic signs

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

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

3.5.3 Kerbs

The edging to vehicle paths at intersections, bus stops or parking bays can be clearly marked by the use of kerbs. Where there is a footpath or cycle track alongside the carriageway a continuous edge kerb combined with a reflectorized line is recommended. Special kerbs, e.g. those incorporating a drainage channel may be used with advantage in some circumstances.

For minimizing material and promoting material recycling, applying precast products should be considered.

3.5.4 Marker Posts

Marker posts are intended to make drivers aware of potential hazards such as abrupt changes in shoulder width, abrupt changes in the alignment, and approaches to structures etc. Generally, horizontal curves can be outlined sufficiently by marker posts positioned only on the outside of a curve. Reflectorized surfaces or buttons on marker posts greatly improve their visibility at night, when most needed. Marker posts are not intended to resist impact.

Marker posts should be constructed in the most economical way, in material which is not likely to be removed for alternative uses by the local population. Marker posts should be sited minimum 1.0m outside the edge of the shoulder.

3.5.5 Safety Fences

Considerations should be given to the provision of a safety fence at sections or points of traffic hazard, such as fixed objects along the edge of the shoulder, high fills, and steeps side slopes at escarpments or along water courses etc., where the hazard of hitting the safety fence is considered a desirable substitute for a more serious accident. Generally the safety fence shall be guard-rail of the Flex-beam type, but on low-cost, low-volume roads a specially designed masonry and/or concrete kerb (wall) may be a more-economical alternative.

It is generally accepted that side slopes of 1:4 are reasonably safe, whereas side slopes of 1:6 will provide insurance against overturning even under adverse conditions.

3.5.6 Other Fences and Gates

All-purpose rural roads have to provide access to land and property. Future development can probably be controlled but a major problem is that of providing safe access to existing development such as roadside shops, markets, farms and schools.

Often numerous unauthorized accesses tend to develop in undesirable locations. This unwanted development can be limited by proper fencing of the considered section at the edge of the road reserve.

At a gated access, recommended especially for schools and cattle farms, the gates should open away from the major road and sufficient space should be provided off the carriageway to accommodate a standing (long) vehicle; if necessary the gate should be set back to allow for this.

3.5.7 Traffic Signs and Road Markings

Traffic signs and road markings are important features in the design of a road which the designer must consider in the geometric layout of the road.

Although safety and efficiency of operation depend to a considerable degree upon the geometric design, the physical layout must also be supplemented by effective signing and making as a means of informing, warning and controlling traffic.

Traffic sign and road marking plans, coordinated with the horizontal and vertical alignments, intersections, sight distance obstructions, operating speeds and maneuvers, should be prepared as an integral part of the design process.

Road signs of all categories including road markings shall follow Road Signs and Marking manual 2017, which have also been adopted in Laos.

CHAPTER 4 ROAD PAVEMENT DESIGN

4.1 DEFINITIONS AND ABBREVIATIONS

4.1.1 Pavement

Figure 4.1.1 and Figure 4.1.2 the terms used in describing the principle pavement and cross-section component.

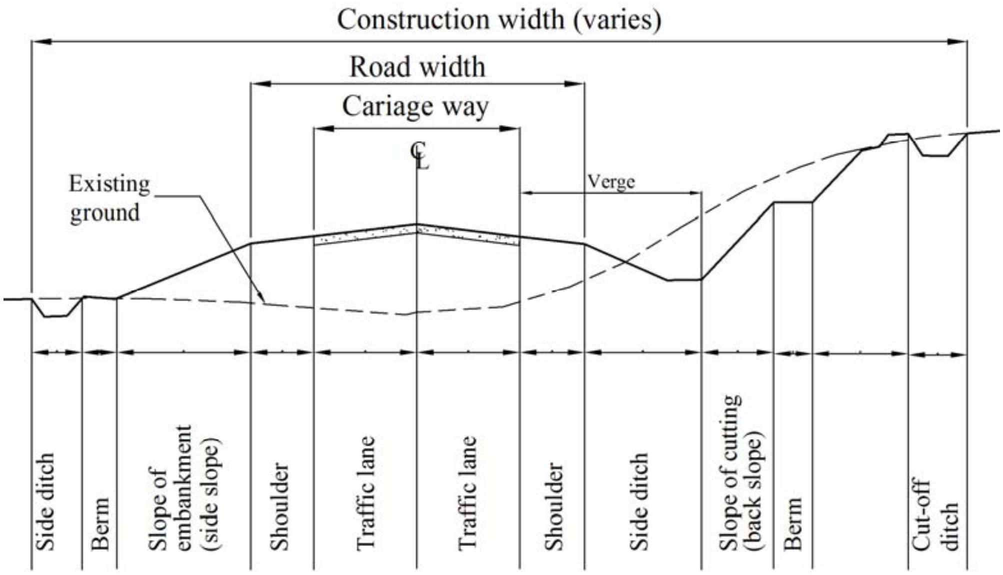


Figure 4.1.1 Cross-Section Terminology

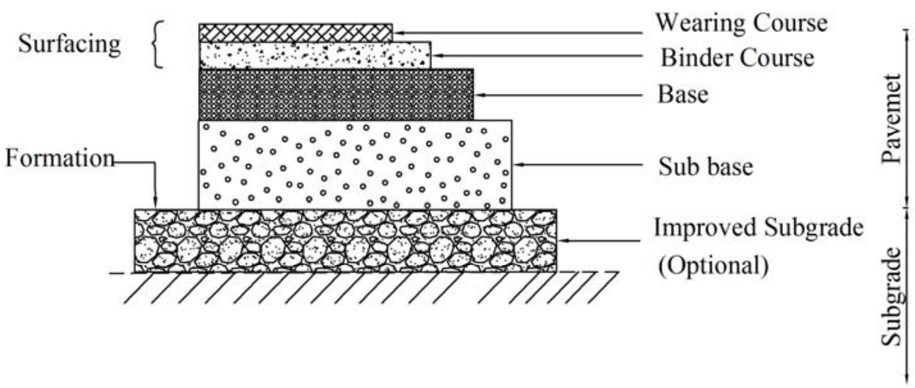


Figure 4.1.2 Pavement Terminology

Natural material all are old material below the subgrade or cut material level.

Subgrade all material below the pavement and may include in-situ material, fill and selected subgrade.

Selected subgrade a layer of selected fill material, the top of which is at formation level, placed where the natural in-situ or fill material is unsuitable for the direct support of the pavement.

Formation the surface of the ground in the final shape, upon which the pavement structure, consisting of sub base, base and surfacing is constructed.

Surfacing the uppermost pavement layer which provides the riding surface for vehicles. It will normally consist of one of the following: surface dressing, sand asphalt or asphalt concrete.

4.1.2 General Material

Borrow area - a site from which natural material, other than solid stone, is obtained for construction of the works. (The term borrow pit is also used).

Quarry - an open surface working from which stone is removed by drilling and blasting, for construction of the works.

Cement - stabilizes material suitable for base layer and consists of a mixture of natural gravels or coarse clayey sand with approximately 4-8 per cent of ordinary Portland cement such that a rigid material is produced. An acceptance criterion includes unconfined compressive strength.

Lean concrete - a high quality, well graded gravel and Portland cement mixture, mixed in a stationary plant and laid by a paves. It is used as a high quality base.

Rocks fill - rock material of such particle size that the material can only be placed in layers of compacted thickness exceeding 300 mm.

Graded crushed stone - a base or subbase material, conforming to a specified grading, strength, shape and soundness criteria.

Crushing ratio - a term applied to crushed stone which has been produced by crushing rounded alluvial material. Such a natural material must contain particles greater than the required product maximum. These oversize particles are normally removed and crushed. The crushing ratio is the weight of this oversized, crushed fraction divided by the total weight of alluvial material, expressed as a percentage. (If for any reason the oversized crushed fraction is not completely returned to the uncrushed material, the above percentage should be adjusted by multiplying by the percentage actually returned).

Gravel wearing course - top surfacing course made from gravel and applied to a road formation where no pavement or bituminous surfacing are to be placed. The term "gravel" includes one or a combination of the following materials: lateritic gravel, quartz tic gravel, calcareous gravel, some forms of partly decomposed rock, soft stone, clayey sands and crushed rock.

4.1.3 Bituminous Materials

Bituminous binders - petroleum derived adhesives used to stick stone chippings on to a road surface, in surface dressings or to bind together a layer of surfacing or base material. There are three principal types used in road work:

a) Straight-run bitumen - bitumen whose viscosity or composition has not been adjusted by blending with solvents or any other substance.

b) Cut-back bitumen - a bitumen whose viscosity has been reduced by the addition of a volatile diluent.

c) Bitumen emulsion a binder - in which petroleum bitumen, in finely-divided droplets, is dispersed in water by means of an emulsifying agent to form a stable mixture.

Surface dressing is a method of providing a running surface to a pavement and consists of applications of bituminous binder and single sized stone chippings. The usual form of this method on a new road is a double surface dressing with the triple second surface layer dressings of stone chips are also being used half the nominal sized of the first. Single and triple surface dressing are also used.

Emulsion slurry seal is a surfacing material, used by itself in one or two layers, or on top of a single surface dressing. It consists of fine aggregate, mineral filler and bitumen emulsion.

Fog spray is a light application of bitumen emulsion or cut-back, on top of a surface dressing. Its purpose is to improve the waterproofness of the surfacing and to assist in holding the chip pings.

Asphalt concrete is a group of bitumen - bound materials used as pavement surfacing. They normally consist of a mixture of coarse aggregate, fine aggregate and filler bound with straight-run bitumen. The proportions and grading of the coarse aggregate may be varied to produce different types of mix with differing properties.

Sand asphalt is a surfacing material consisting of a hot-mixed, hot-laid, plant mixture of natural sand and, in some cases, mineral filler and crushed fine aggregate, bound with straight-run bitumen. It is not suitable for heavily trafficked roads.

Gap-Graded asphalt is a hot laid, plant mixture of gap-graded filler and straight-run bitumen, used for pavement surfacing.

Binder course is the lower layer two-course asphalt concrete which is used as a surfacing. It usually differs from the upper, wearing course in having a slightly lower bitumen content, low stability and greater voids.

Dense bitumen macadam is a cold laid plant mixture of well graded aggregate, filler and straight-run bitumen which is used for base construction. The specifications are very similar to dense bitumen macadam.

Prime coat is an application of low viscosity bituminous binder to an absorbent surface, usually the top of the base. Its purpose is two-fold i.e. to waterproof the surface being sprayed and to help bind it to the overlaying bituminous course.

Tack cost is a light application of bituminous binder to bituminous or concrete surface to provide a bond between this surface and the overlying bituminous course.

4.1.4 Traffic

Private cars (cars) - all passenger motor vehicles seating not more than 9 persons, including the driver.

Light goods vehicles (LGV) - all goods vehicles of not more than 15 kN unladen weight.

Buses - all passenger motor vehicles seating more than 9 persons, including the driver.

Medium Goods Vehicles (MGV) - all two-axle goods vehicles of more than 15 kN unladen weight.

Heavy Goods Vehicles (HGV) - all goods vehicles having more than two axles.

Commercial vehicles includes buses and goods vehicle of more than 15 kN unladen weight.

Equivalent Standard Axle (E.S.A) is a design concept to enable the damaging effect of a range and number of different axle load, to be considered in the structural design of pavements. The equivalent standard axle imposes a load of 80 kN.

Equivalence factor of an axle (or vehicle) is the number of passages of an Equivalent standard Axle which would cause the same damage to a road as one passage of the axle (or vehicle) in question.

Design Period is the period during which the proposed pavement must carry the major reconstruction work, except for re-sealing. At the end of this period the pavement should still be in a sufficiently good condition that strengthening will result in further period of satisfactory traffic-carrying.

Traffic Classes - Classification of traffic group for the structural design of roads.

4.2 TRAFFIC

4.2.1 General

A major factor in road pavement design is the cumulative number of equivalent standard axles (ESA), defined as 8160 kg., over the design life period of the intervention. In order to determine this value a number of steps need to be taken:

- The axle load distribution amongst heavy vehicle traffic categories which will use the road should be assessed;
- These axle loads should be converted to an equivalent number of ESA;
- The initial number of ESA should be calculated;
- Where some future event is expected to alter the distribution of axle loads this should be taken into account;
- The annual traffic growth rate and design period should be estimated;
- The cumulative number of ESA can thus be estimated.

4.2.2 Axle load distribution

(1) Legislation

The legal axle load limits in force in Lao PDR are shown in follows:

<u>NR3, NR9, NR4 and ASEAN Highway Network roads when upgraded</u>:

Single axle, 4-wheel	11 tones
Tandem axle, 4-wheel (per axle)	10 tones
Triple axle, 4-wheel (per axle)	8.2 tones
Single axle, 2-wheel	7 tones

<u>Other National Roads:</u>

Single axle, 4-wheel	9.1 tones
Tandem axle, 4-wheel (per axle)	8.2 tones
Triple axle, 4-wheel (per axle)	6.8 tones
Single axle, 2-wheel	6.8 tones

NB. In the wet season (1 June to 30 November) the axle weight limit is reduced by 20% on unpaved roads.

4.2.3 Equivalent factors

(1) Axle load equivalent

The axle load Equivalence Factor (EF) relates a given single axle weight to its ESA equivalent. The following relationship may be used to convert a single axle load to its ESA:

$$EF = \left(\frac{Axle\ Load\,(kg)}{8160} \right)^{4.5}$$

Where : EF is the equivalence factor of the single axle considered.

All axles, including those forming part of tandem and tridem axles, should be weighed separately. The Vehicle Damage Factor (VDF) is then the ESA for each single axle load totaled over all axles of the vehicle category (axle configuration) concerned.

The equivalence factors depend to some extent on the strength of the pavement. It can nevertheless, be considered that the above equation satisfactorily accounts for the damaging power of traffic on the pavement.

Sometimes climatic weighting is applied to the equivalent factor. Wet regions would then have higher weighting factors (+30%) than dry regions (-30%). Until such time more experience has been gained in pavement behavior versus climatic regions, it is proposed to use a regional weighting factor of 1.0 for Laos roads, i.e. no further weighting applied.

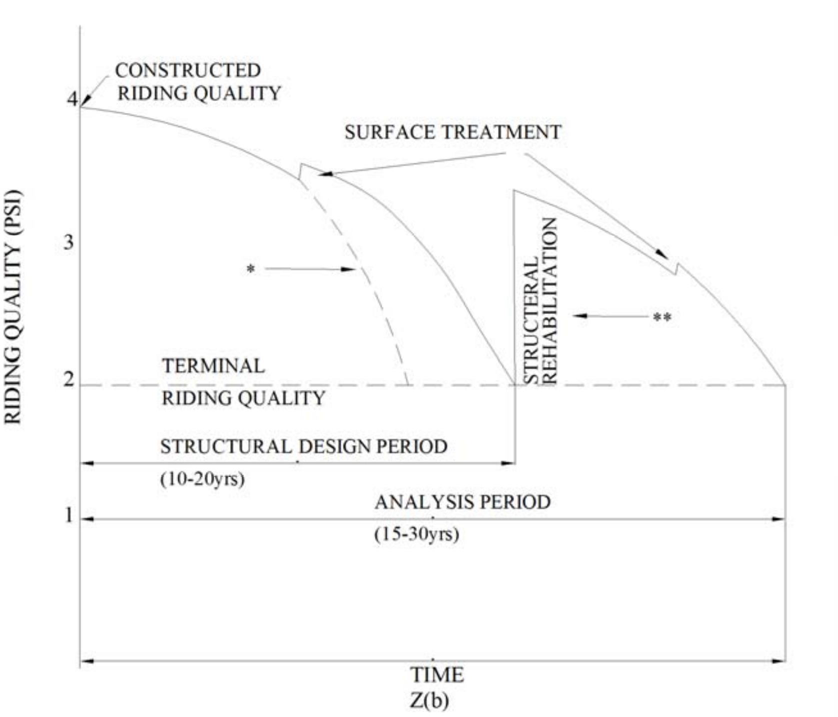
The load equivalence factor is furthermore taken when an existing pavement has reached its terminal riding quality.

The terminal quality, expressed as a degree of serviceability, is reached as the road deteriorates with time. This is due to combinations of deformation, settlements and eventually failures. The serviceability of a road any time, is expressed in terms of the Present Serviceability Index (PSI), usually ranging from 0-5, where the higher value represent the better riding quality and the lower values approaches the terminal riding quality stage. The minimum acceptable serviceability depends on the road class. Important rural and inter urban roads would have a minimum PSI of 2.0-2.2 and other less important rural roads could have as low as 1.5 PSI.

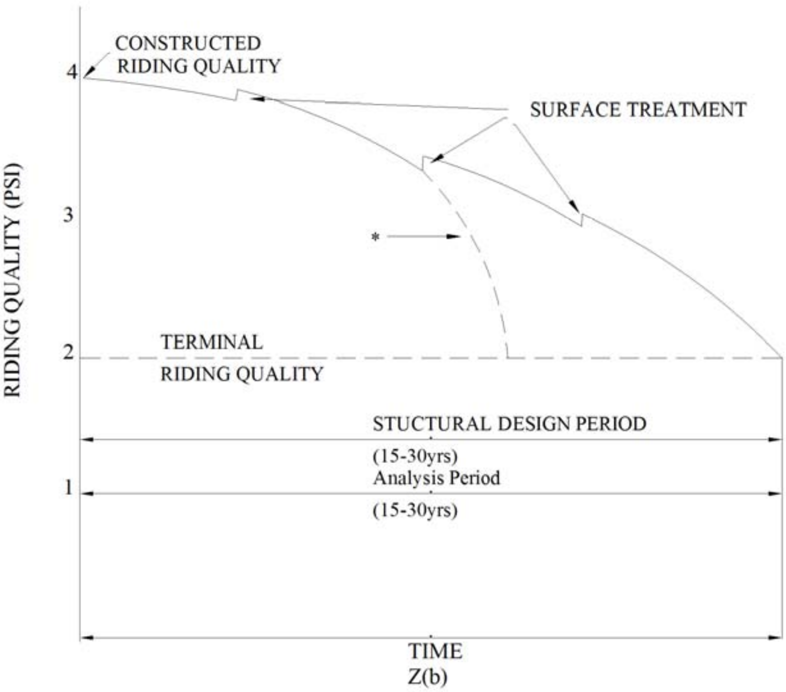
Figure 4.2.1 illustrates the design analysis period versus the Riding Quality expressed as Present Serviceability Index (PSI).

(2) Design Period

The analysis period is generally analysis period, normally a range of 15-30 years, includes design periods and structural rehabilitation periods. The design analysis period therefore, can consist of one or more structural design periods. Generally advisable design period of pavement is 10 to 20 years.



DESIGN I: Requires two resurfacings and one structural rehabilitation during the analysis period



DESIGN II: Requires three resurfacing and no strength ending during the analysis period

*If surfacing is not maintained and if water-susceptible material are used in the pavement

**Structural rehabilitation usually occurs at a later stage

Figure 4.2.1 Illustration of Design Periods and Alternative Design Strategies

(3) Vehicle equivalence factor

For each road project, the commercial vehicle equivalence factors should be evaluated through specific axle load surveys using portable weighbridges.

4.2.4 Evaluation of traffic for design purpose

(1) Estimating the initial daily number of commercial vehicles

It is necessary, as a first step to estimate the average daily number of each type of commercial vehicle that will use the road, in both directions, during the first year. The loads imposed by private cars and light goods vehicles do not contribute significantly to the structural damage caused to pavements by traffic. <u>Therefore, for structural design purposes, cars and light goods vehicles can be ignored.</u>

Routine traffic counts should be carried out annually. In addition to Classified Traffic Counts (CTS) and Axle Load surveys (ALS), at a number of census points

(2) Estimating the initial daily number of standard axles

This operation is concerned with finding the average daily traffic expressed in terms of equivalent standard axles, which will be using the road in the first year after opening. It will be obtained by multiplying the above average daily numbers of each type of commercial vehicle by the appropriate equivalence factor and then by summing up the numbers of standard axles of all the vehicle types. .

(3) Estimating the cumulative number of standard axles

To estimate the total number of standard axles to be catered for by the design, it is necessary to forecast the annual growth of the traffic and to decide what the design period should be described below:

1. <u>Forecasting the annual rate</u>

This is often a difficult and uncertain exercise. Some guidance can be obtained by studying the annual trends in traffic growth indicated by censuses regularly carried out in the region concerned. The study of national and regional development plans and other economic studies may also be necessary.

When more precise information is not available, an indication of likely traffic growth can be assessed from the national trends in the number of vehicles registered annually.

2. <u>Choosing a design period</u>

The concept of design period should not be confused with pavement life.

At the end of the "design period" the pavement will normally require to be strengthened in order to carry traffic for a further period. At the end of the "design period" the pavement will <u>not</u> be completely worn out or have deteriorated to the point that reconstruction is needed.

During the design period of the pavement, ordinary (routine) maintenance will be carried out, i.e. shoulders and drainage system maintenance, vegetation control, localized patching and periodic resealing.

The design aim is, therefore, to minimize the total expenditure on the pavement, including the initial construction costs and subsequent maintenance or strengthening costs discounted to present day value.

3. Calculating the cumulative of standard axles

The cumulative number of standard axles, T over the chosen design period N (in years) is obtained by:

$$T = 365 t_1 \frac{(1+i)^N - 1}{i}$$

Where:

t_1 : the average daily number of standard axles in the first year after opening:

i: the annual growth rate expressed as a decimal fraction.

4. Lane distribution

The design traffic loading shall be corrected for the distribution of vehicles between lanes in accordance with Table 4.2.1.

Table 4.2.1 Lane Distribution Factor

Number of Lanes in Each Direction	Percent of 18-kip ESAL in Design Lane
1	100
2	80 -100
3	60 – 80
4	50 - 75

4.2.5 Traffic classification

(1) Use of cumulative number of standard axles

A rational approach is to base the traffic classification on the accumulation equivalent standard axle values. However even this method has its drawbacks in that the initial axle load spectrum is assumed to remain fixed. Where it is fairly certain that some future event will alter the distribution of axle load, this should be taken into account.

This type of classification enables the effects of unexpected change in traffic volumes or axles-load distribution on the pavement life to be evaluated.

4.3 ENVIRONMENT

4.3.1 Climate

The climatic conditions (moisture and temperature) as well as the underlying subgrade conditions, has a considerable influence on road performance and should therefore be taken into account by the designer.

The climate will largely determine the weathering of natural rocks, the durability of weathered, natural road building materials and, depending on drainage conditions, also the stability of untreated materials in the pavement. The climate may also influence the equilibrium moisture content. The designer should always consider the climatic conditions and avoid using excessively water-susceptible or temperature-sensitive materials in adverse conditions.

4.3.2 Natural materials and soils

In order to minimize construction costs, natural materials should be used as much as possible. Every effort should be made to use available local materials before considering the importation of material from some distance. <u>It is therefore of prime importance to make a complete inventory of all available road making materials, at the pre-design investigation stage.</u>

4.4 EARTHWORKS

4.4.1 Cuttings

(1) General

Wherever a cutting is required, consideration should be given to the following factors that will affect its design and cost:

- Type of material to be excavated
- Volume and position of the different materials
- Level and flow of water table and springs
- Stability of slopes
- Drainage and protection against erosion

Any cutting to a depth of more than 5m requires a specific study, including boreholes or test-pits down to the formation level.

(2) Type of material to be excavated

The type of material to be excavated controls the construction methods, the use to which the material can be put, its suitability as subgrade material and the slopes that can be safely selected.

Tropical weathering generally results in the occurrence of two types of materials: residual soils and weathered rock. The depth and degree of weathering are often variable and the properties of the residual materials may be quite different even within the same horizon over a short distance. In addition, the pattern of the residual materials is often complicated by the processes of leaching of silica and accumulation of iron and aluminum sesquioxides, known as lateralization.

At the design stage, it is sometimes not easy to accurately differentiate rock from rippable material or rippable from normal material. In particular, the occurrence of residual boulders and especially their volumes are difficult to ascertain.

The depth of bedrock is clearly important not only because of its effect on the cost of the cuttings, but also because the presence of rock can provide a layer on which a perched water table can exist. Depending on the type of rock and its structure, springs may also be a problem.

(3) Water table and springs

A water table may be permanent, seasonal or perched. In any case, its presence and its characteristics (level, How of water and etc.) should be determined, as they may affect the method of excavation and the stability of the cut slopes and as they govern the drainage system required.

Similarly, the likelihood of springs occurring within the cut should be assessed.

(4) Determination of the angle of slope

The design of the slope angle of cut is a compromise between the following requirements:

(i) Stability

(ii) Erosion control

(iii) Appearance and visibility

(iv) Need of fill material

(v) Minimum cost

1) **Stability**

The analytical determination of the factor of safety against sliding or rotation of a proposed cut cross-section is a complex matter. It requires a knowledge of the material strength parameters, the slope geometry, the location of any water table and the presence of any external loading on the slope, In most cases slope angles are determined not analytically, but either by experience or "rule of thumb", The slopes angles given below are ones that have generally been found to be satisfactory, where there is no water seeping out of the face and no external loads, They are expressed as Vertical:Horizontal

Cohesionless sands	1:2
Residual soils (red friable clays)	1:0.7 if d < 4m
	1:1 if d > 4m
Weathered rock	1:0.5 to 1:0.25
Sound rock	1:0.2 to 1:0.1
	Where d is depth of cut

<u>Notes</u>

The structure of the parent rock may dominate the stability in residual soil.

Safe slopes in stratified rock chiefly depend on the orientation of the foliation or strata. Similarly in rocks with a strong pattern of jointing, the orientation and spacing of the discontinuities my well control the safe slope angle.

Where cutting are more than 8 m deep or the material and water-table situation are believed to be problematical, an analytical appraisal may be considered. This may well involve a detailed site investigation and associated laboratory testing.

2) **Erosion control**

This is sometimes a difficult requirement to reconcile with stability.

For instance, in cohesive materials, the erosion is less extensive for very steep slopes than for slopes at about 1:1 and it is therefore desirable to keep the slopes as steep as possible, unless the lopes is protected.

Generally, guidance on satisfactory slope angles from the point of view of both stability and erosion resistance can be obtained from survey of other cuttings and natural steep slopes in the vicinity.

3) **Other factors**

Partly for aesthetic and safety reasons a low angle slope is normally considered more desirable than a near vertical one, even if other factors will allow this latter course. The need for or the surplus of fill material will also have an influence on slope angles.

4.4.2 Embankments

(1) General

Wherever an embankment is required, consideration needs to be given to the following factors that will affect its design and cost:

- Foundation conditions
- Acceptable fill material
- Stability of slopes
- Settlement
- Method and rate of construction

(2) Foundation conditions and settlement

The foundation conditions beneath embankments require special attention to avoid shear failures and excessive settlements. Wherever an embankment is to be built on a wet, compressible soil such as mud, soft clay, etc., detailed investigations are necessary to determine the most suitable construction method, the rate of construction and any special precautions required.

It is essential to use methods of construction which ensure either the removal of the soft material or its substantial consolidation before the pavement is completed. The following methods can be used:

i) Excavation and displacement involving:

Total excavation of the compressible soil.

Displacements of very watery material by coarse rock fill.

ii) Consolidation involving:

Pre-loading with higher embankment

Installation of vertical sand drains.

Combination of pre-loading and sand drains.

Time lapse between placing fill and completing pavement.

(3) Fill material

Fill material will general be obtained from cuttings. If the material obtained this source is insufficient or unsuitable, extra material shall be obtained borrow areas.

The following materials are generally unsuitable for the construction of fills.

- All material containing more than 5 % by weight of organic matter (such as topsoil, material from swamps, mud, logs, stumps and perishable menials).
- All material with a swell of more than 3%.
- All clay of plasticity index exceeding 50. (However, some red friable cla having a plasticity index over 50 may successfully be used).
- All material having moisture content greater than 105 percent of the Optimum Moisture Content (Standard Compaction).

Rockfill as defined in section 4.1.2 can be use pervaded that boulder greater than 0.2 m3 in volume (600 mm size) are not used and this material is not pleased within 600 mm of formation level.

The best material either from cuttings or from borrow areas, should be reserved for the upper layer of fill.

(4) Stability of slopes

The vast majority of embankments up to 8m in height and resting on non- saturated soils have their slope angles determined by either experience or "rule of thumb". The following slopes, expressed as vertical: horizontal are recommended:

Cohesion less sand	1:3	if h< 1 m
	1:2	if h > 1 m
Other materials	1:3	if h < 1 m
	1:2	if 1< h < 3 m
	1:5.5	if 3 < h < 10 m

Where h is height of embankment

Embankments higher than 10 m or those of lesser height but founded on soft, wet materials, should be analyzed individually using appropriate geotechnical methods.

(5) Placing and compaction of fill

i) Construction on near level ground

Generally, all soft and organic material shall be removed and hollows shall be filled, to obtain a uniform surface to receive the fill.

Any backfilling required to obtain the uniform surface shall be compacted to a dry density of least 95% MDD (Standard compaction).

ii) Construction on slopes

Where the slope of existing ground is greater than 1 (vertical) to 3 (horizontal), horizontal benches in steps not less than 3 m wide should be cut into the existing ground. Immediately on completion of cutting the benches, the whole of the area to receive the fill shall be compacted to 95% MDD (Standard Compaction) down to a depth of 150 mm. The time between preparing the area and placing the fill must be kept to a minimum.

iii) Compaction of fill material

Materials other than rock fill shall normally be placed in layers of compacted thickness not exceeding 300 mm. Thicker layers may be permitted only where very heavy compaction equipment is available and after trial sections have proved that the required compaction may readily be obtained over their full depth.

The minimum layer thickness shall be twice the maximum particle size of the material.

Normally, the layers of fill material shall be compacted throughout to a dry density of at least 95% MDD (Standard Compaction), except for the upper 300 mm of the subgrade which shall be compacted to a dry density of at least 100% MDD (Standard Compaction).

For very high fills, higher compaction may be required to reduce settlement. The moisture content of the material shall be adjusted so that the above minimum compaction. Figures are obtained. Moisture contents well below the Optimum

Moisture Content (Standard Compaction) may be accepted, provided that the compaction equipment and method are such that the required compaction is achieved. It is strongly recommended that the moisture content at the time of compaction does not exceed 105%of the Optimum Moisture Content (Standard Compaction). This applies particularly to silty and clayey materials, which are prone to shrinkage and loss of strength, resulting from excessive moisture contents.

Normal laboratory compaction tests cannot be accurately carried out on materials containing a high proportion (say more than 25% of particles greater than 40mm size. On such coarse material, the minimum dry density required and the suitable moisture content shall determine from site compaction trials.

Where fairy homogeneous materials are used, the compaction requirements may consist of a method specification with the following parameter being fixed:

- The maximum thickness of compacted layer
- The characteristics of the compacting equipment
- The number of passes of each roller
- The permissible range of moisture content, all as determined from full scale compaction trails.

Where rockfill is used it should be placed in the bottom of embankment. The largest size of rock is be placed in layers of maximum compact thicknesses of 1 m. the interstices 11 then be filled with smaller rocks, spalls and approved finer material. The whole layer shall be compacted until the interstices are completely filled or until the required settlement is obtained. Heavy vibratory rollers are generally the most suitable machines for compacting rockfill.

It is most important that the specified compaction is achieved over the full width the embankment. Loose material left on the slopes may absorb water and may endanger the stability of the slopes.

4.4.3 Slope Protection

The guideline of maintenance of existing slope is reported in “Slope Maintenance Manual, MPWT, 2008” which is attached in APPENDIX-H. In this section, the main topic is summarized.

(1) Slope Instability Type

The most common occurrences of slope instability affecting the road network are shown in Figure 4.4.1.

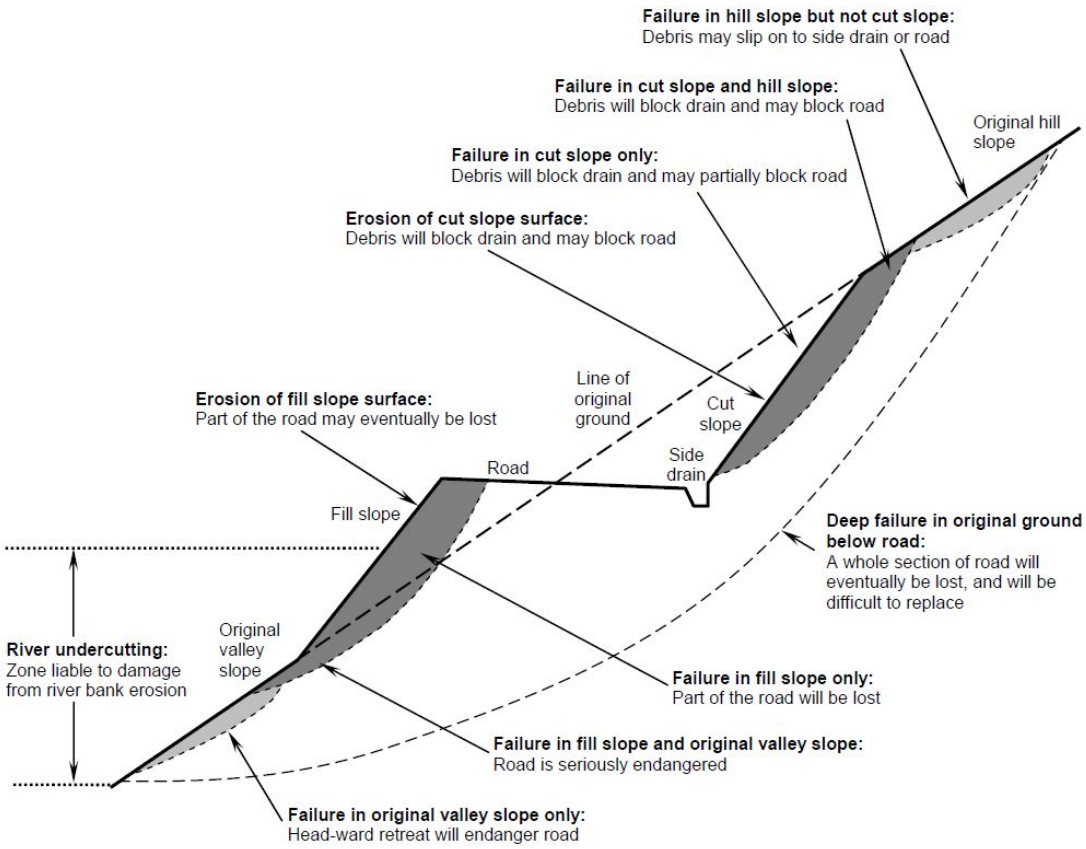


Figure 4.4.1 Typical Slope Instability

(2) Inspection and Investigation

Factors that need to be taken into account when undertaking road, slope and wall inspections and investigations are likely to include many of the following:

<u>Topographical</u>

- the steepness and shape of the slope
- the location of tension cracks and other signs of movement

<u>Hydrological</u>

- the presence of a river or stream at the base of the slope, particularly if this could cause toe erosion during periods of flood or high flow
- the presence of a drainage course at or above the crest of the slope
- any indications of a high or temporarily perched water table within the slope, e.g. seepages and springs.
- the effectiveness and condition of the existing drainage measures

<u>Rainfall</u>

- the pattern of rainfall in the immediate locality, particularly periods of prolonged and/or intense rainfall that could lead to saturation of the slope

<u>Geological (particularly for rock slopes)</u>

- rock type, weathering grade, jointing and fracture patterns
- presence of faults or shear zones
- the direction and angle of dip and joints in underlying bedrock compared to the angle and orientation to the slope, particularly if bedrock is exposed or is at a shallow depth beneath the surface, the persistence of the joints, the presence of clay filling
- the sequence of the underlying strata, particularly if this includes weak or impermeable layers

Geomorphological

- soil types and depths
- the presence of pre-existing landslides, the distribution of colluvial deposits and unstable/erodible soils

Land Use

- forest clearance and the extent and type of cultivation, particularly wet padi
- the presence of irrigation channels, ditches and water pipes
- excavations and fill slopes associated with commercial and residential developments adjacent to the road

If an instability problem is detected, then a detailed site inspection must be undertaken. The decision-making process is given in Figure 4.4.2.

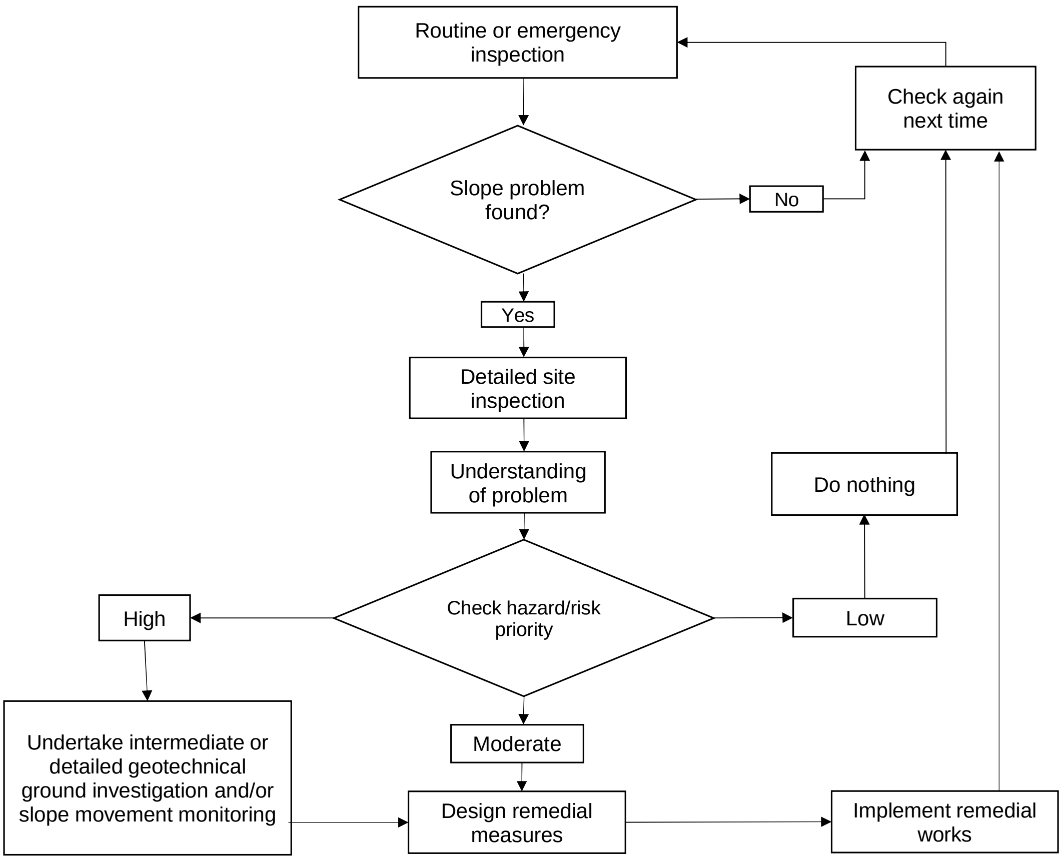


Figure 4.4.2 Decision-Making Process for site assessment and Problem Diagnosis

(3) Determination of Treatment Measures

This section focuses mainly on slopes composed of soil or highly weathered rock (having many of the behavioral characteristics of soils), and on associated retaining walls. The decision-making process in the determination of treatment measures is given in Figure 4.4.3.

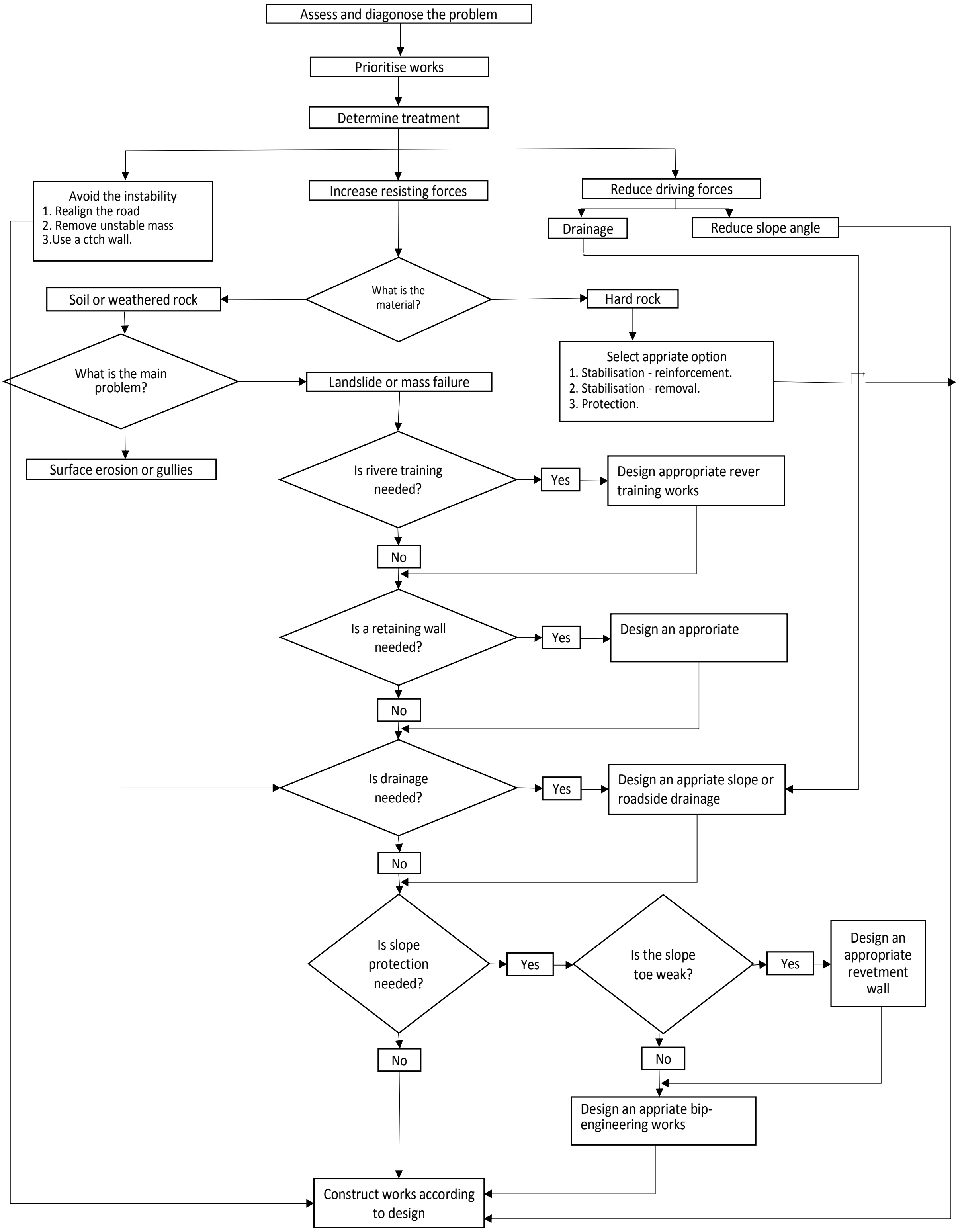


Figure 4.4.3 Decision-Making Process for Treatment Measures

The specific forms of instability shown in Figure 4.4.1 have certain treatments that can be used to remedy them (see Figure 4.4.4).

These take the following forms.

- Slope stabilization: The arresting of structural and mass movements within a slope. In engineering terms this means either the reduction of driving forces (e.g. excess weight at the top of a section of slope) or the increase of resistance through an external force (e.g. a retaining wall).
- Slope protection: The prevention of surface degradation on a slope. This means strengthening the surface (e.g. with a rip-rap stone covering) or reducing the energy of runoff water (e.g. by interrupting flow with a vegetation cover).
- Slope drainage: The provision of either shallow drainage to remove mainly surface water or deeper drainage to remove mainly groundwater. This strengthens the slope by increasing the internal resistance (i.e. by reducing pore water pressures).

Table 4.4.1 provides the main possibilities for technical engineering solutions to slope instability.

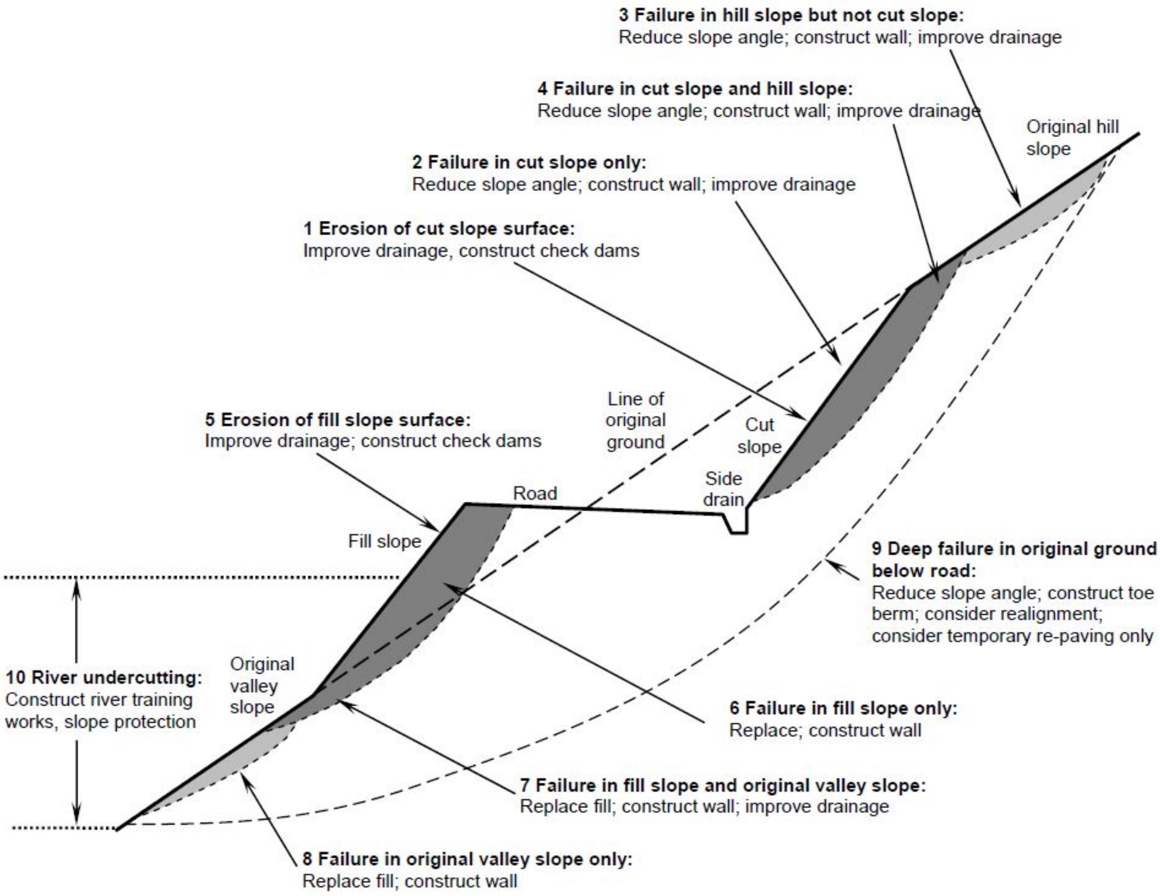


Figure 4.4.4 Engineering Solutions for Slope Stability

Table 4.4.1 Technical Treatment Requirements for Different Failure Types

Instability	Stabilization options	Drainage options	Protection options
Above the road			
1 Erosion of the cut slope surface	• None.	• Usually none. • Occasionally a cut-off drain above the cut slope can reduce water runoff; however, these are difficult to maintain and can contribute to instability if blocked or otherwise disturbed.	• In most cases, bioengineering is adequate, usually grass slip planting. • Where gullies are long or slopes are very steep, small check dams may be required. • Sometimes a revetment wall at the toe helps to protect the side drain.
2 Failure in cut slope only	• Reduce the slope grade if this is feasible, then add erosion protection. • A retaining wall to retain the sliding mass. • For small sites where the failure is not expected to continue, a revetment might be adequate.	• A subsoil drain may be required behind a wall if there is evidence of water seepage. • Herringbone surface drains may be required if the slope drainage is impeded.	• Bio-engineering is usually important to prevent surface erosion • and increase the resistance of the surface soil.
3 Failure in hill slope but not cut slope	• Reduce the slope grade if this is feasible, then add protection. • A retaining wall to support the sliding mass, as long as foundations can be found that do not surcharge or threaten the cut slope.	• A subsoil drain may be required behind a wall if there is evidence of water seepage. • Herringbone surface drains may be required if the slope drainage is impeded.	• Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
4 Failure in cut slope and hill slope	• Reduce the slope grade if this is feasible, then add protection. • A retaining wall to retain the sliding mass. This may need to be quite large, depending on the depth of the slip plane	• A subsoil drain may be required behind a wall if there is evidence of water seepage. • Herringbone surface drains may be required if the slope drainage is impeded..	• Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
Below the road			
5 Erosion of the fill slope surface	• None.	• Ensure roadside drainage is controlled.	• Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
6 Failure in fill slope only	• Re-grade or remove, replace and compact fill. • Before replacing fill, cut steps in original ground to act as key between fill and original ground. • A new road retaining wall may be the only option	• Ensure roadside drainage is controlled	• Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.

Instability	Stabilization options	Drainage options	Protection options
7 Failure in fill slope and original valley slope	• Re-grade or remove, replace and compact fill. • Before replacing fill, cut steps in original ground to act as key between fill and original ground. • A new road retaining wall may be the only option	• Ensure roadside drainage is controlled.	• Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
8 Failure in original valley slope	• Re-grade if sufficient space between road and valley side. • A new road retaining wall may be the only option	• Ensure roadside drainage is controlled	• Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
9 Deep failure in the original ground underneath the road	• Consider re-alignment of road away from instability • If slow moving, short-term option may be to re-pave or gravel the road.	• Ensure roadside drainage is controlled	• Bio-engineering is usually important to prevent surface erosion and increase the resistance of the surface soil.
10 Removal of support from below by river erosion	• May need extensive river training works to prevent further erosion.	• None.	• Slope protection (walls and rip-rap etc.) may be necessary.

In the majority of situations where slope failure has occurred, some form of structural retention will be required. Table 4.4.1 and Figure 4.4.5 give details of some retaining structures commonly used around the world.

Table 4.4.2 Features of Retaining Structures

System	Function	Type	Advantages	Limitations
Externally stabilized	Gravity walls	Masonry	Technique well known	Unable to accommodate movement without distress
		Mass Concrete	Simple to construct	Large quantities of concrete required
		Reinforced Concrete - Cantilever	Generally occupies less width	Requires reinforced concrete construction; good foundations; generally uneconomic above 8m height
		Reinforced Concrete - Counterfort	As above	As above, but can be constructed to greater heights
		Gabion	Technique well-known; can accommodate limited movement without distress; permeable	Moderate durability; not recommended as retaining walls below and immediately adjacent to paved road surface due to flexibility
		Crib	Attractive, environmentally friendly appearance	Possible problems of durability if timber cribs are used
	In-situ walls	Sheet pile Slurry walls	Occupies very limited space, no temporary excavation works required	High cost; requires specialist installation equipment; impermeability may create problems

System	Function	Type	Advantages	Limitations
		Bored-in-place piles		High cost; requires specialist installation equipment
Internally stabilized	Reinforced soil	Strips and grids	Can accommodate limited movement without distress; easy to construct	Occupies large space behind wall face
		Soil nailing	Used extensively when steepening existing cut slopes	Requires specialist installation equipment

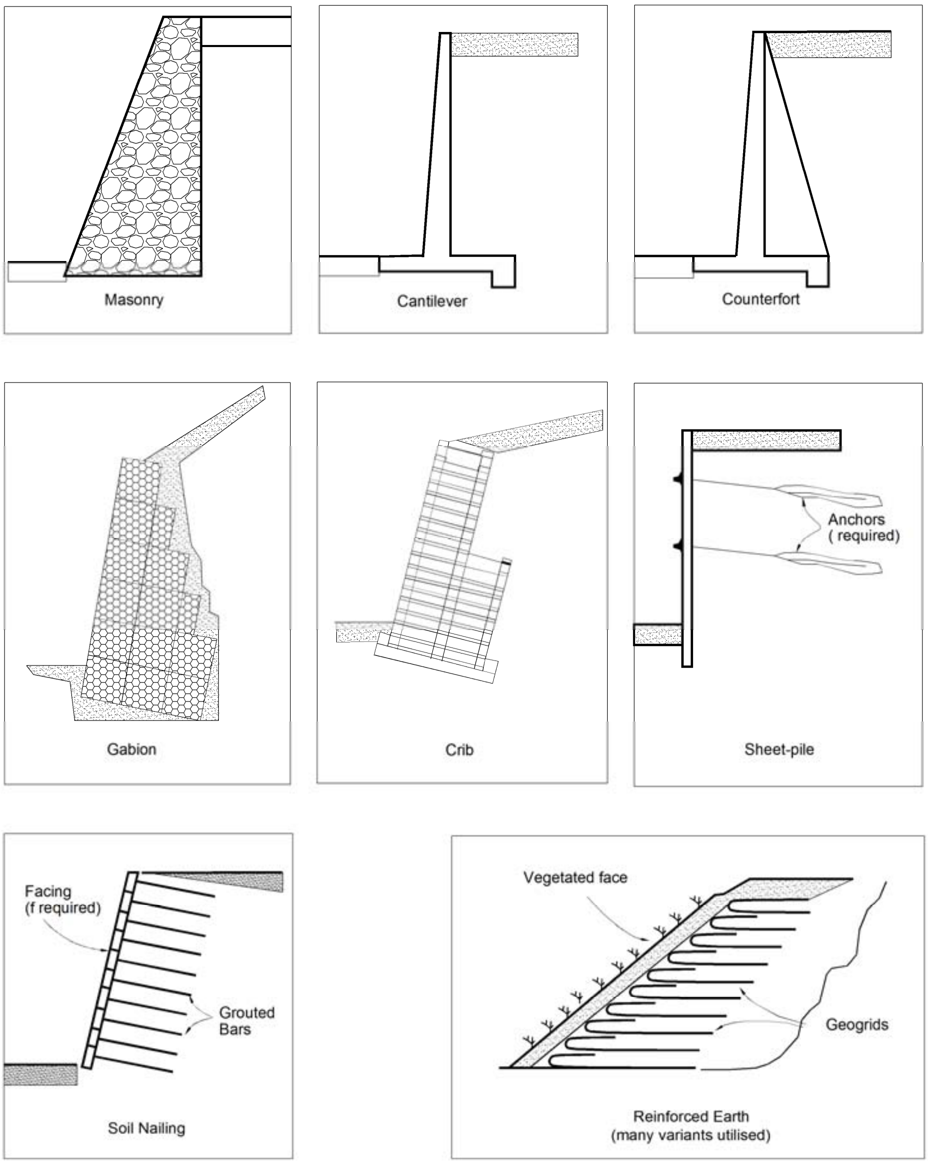
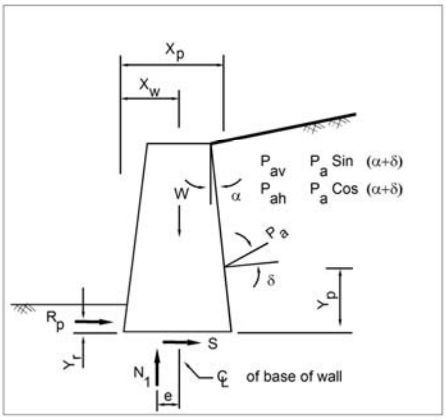
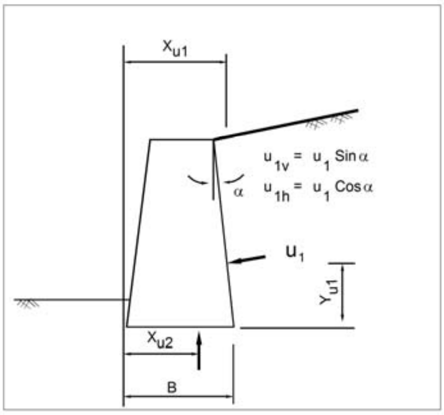


Figure 4.4.5 Common Forms fo Retainig Structure

As an example of counter measure, calculation of wall stability is shown in Figure 4.4.6.



Forces on retaining walls
(water forces not shown)



Water forces on retaining wall

< Sliding Failure >

Activating force : $F_a = P_{ah} + U_{1h}$

Resisting force : $Fr = N_1 \tan \delta_b + R_p$, where $N_1 = W + P_{av} + U_{1v} - U_2$

< Overturning Failure >

Overturning moment : $M_o = P_{ah}y_p - P_{av}x_p + U_{1h}y_{u1} - U_{1v}x_{u1} + U2x_{u2}$

Resisting moment : $M_r = W_{xw} + R_py_r$

Eccentricity : $e = B / 2 - (M_r - M_o) / N_1$

< Bearing Capacity Failure >

Effective normal load Q_n and shear load Q_s imposed on the foundation are given by $Q_n = N_1$ and $Q_s = Fa$ respectively.

Notes:

(1) The total weight W equals the weight of the wall plus the weight of the hatched portion of soil.

(2) The possibility of excavation in front of the wall should be considered in evaluating passive resistance. Where excavation is likely, a minimum trench depth of 0.5m should be allowed for in the calculation.

(3) Zero wall friction should be assumed for the vertical plane in soil on which the passive resistance acts.

Figure 4.4.6 Force Acting on a Gravity Wall

4.5 DRAINAGE

4.5.1 Drainage of surface water

(1) General

This chapter deals with the drainage of surface and groundwater, and design methods of protecting slopes and ditches from erosion. Cross drainage (culverts) is dealt with in Part V of the Road Design Manual.

(2) Side ditches

The design of these ditches is covered in Part V of the Road Design Manual where standard cross sections for different terrain and gradient/capacity curves are given.

(3) Cut-off ditches

It is usually desirable to construct a cut-off ditch at the top of cutting slopes to prevent water flowing down the face. The preferred type, consisting of a combined ditch and bank, is detailed in Part V of the Road Design Manual. The moderate slopes of 1 vertical: 2 horizontal used in this detail have been chosen to allow the inevitable movement of pedestrians and livestock, aiming at as little damage to the ditch as possible.

(4) Discharge channels

Depending on topographic conditions it is sometimes necessary to collect water at the top or either a cutting or an embankment and discharge it down the slope. For this purpose discharge channels shall be constructed and line with masonry, concrete or metal.

(5) Collection of water on embankments

On embankments, where water is to be discharged down the side slopes in discharge channels, it is necessary to lead all water to the tops of these channels. This can be achieved by some form of kerbing or a recessed channel.

The kerbing can be formed from masonry, precast concrete units or in-situ concrete.

(6) Embankment toe ditches

At the base of embankments, toe ditches may be necessary to remove water from the vicinity of the embankment or to prevent erosion of the fill. They should be designed on similar principles to side ditches as mentioned in above.

4.5.2 Drainage of ground water

(1) General

Ground water may be encountered in the following situations:

- in cuttings, a water table with a level above or near formation, or springs;
- in low-lying or poorly drained flat areas, a water table near formation, likely to affect the subgrade by capillary rise.

(2) Drainage remedies

i) choice of proper alignment

The best expedient for the prevention of drainage problems is carrying out proper survey of the areas concerned and choosing or modifying both vertical and horizontal alignments so that the formation is as far away as practicable from water tables and springs.

In particular, in low-lying or poorly drained areas, it is necessary that the road be raised by means of an embankment to avoid surface flooding.

ii) Subsoil drains

Longitudinal subsoil drains can be used to locally lower a water table. These will not normally consist of porous concrete, open jointed or perforated pipe laid in a trench with a surround and, backfill of free draining material, e.g. graded crushed stone (maximum size: 60mm), clean coarse gravel or sand. The pipe size will depend on the expected flow of water but will generally not be less than 100 mm internal diameter. The depth of the trench will depend on the level of the water table and the permeability of the soil but normally it should. be at least 1 metre deeper than the formation level and 500 mm wide.

In some cases where it is necessary to prevent surface water from entering subsoil drains, the upper 500 mm of the trench shall be backfilled with impermeable clayey material.

If the surrounding ground is likely to squeeze or wash into the free draining material, filter protection is required. This can be achieved by placing filter material as free-draining material in the trench.

Filter materials shall comply with the following requirements:

$5.S15 < F15 < 5.S85$

Where:

F15: the sieve size (in mm) through which 15% by weight of the filter material passes.

S15: the sieve size (in mm) through which 15% by weight of the natural soil passes.

S85: the size (in mm) through which 85% by weight of the natural soil passes.

It is important that the pipe be surrounded by appropriate filter material to prevent fines from clogging the openings.

A non-woven geo-fabric of an approved type may be placed around the draining material to prevent silt or fine particles from being washed into it.

It may also be useful to place non-woven geo-fabric around the pipe. The effective pore size of the fabric should comply with the above filter criteria.

Where pipe drains are used, inspection chambers with silt traps shall be constructed every 100 m along straight sections and at every change in direction. These will enable the pipe to be rodded or flushed out.

iii) Blanket drains

Blanket drains Blanket drains are used to remove seepage water appearing in the base of cuttings or in the subgrade. The blanket shall consist of a filter layer in contact with the soil, and a coarser collector layer. Nonwoven geo-fabric may also be used, to prevent fines from blocking the drainage layer. Protection by filter layers or non -woven geo-fabric may be required on both sides of the blanket drain.

iv) Seepage remedies

If during construction unanticipated local seepages or springs are encountered in cuttings they may be controlled by either a counterfort drain or sub-horizontal well. In its simplest form a counterfort drain consists of an excavated "slot" or deep trench running into the cut slope, which is then backfilled with free-draining material and in large cases a porous pipe.

The filter criteria already stated will apply and some arrangement must be made to lead away the intercepted water. Geo-fabrics can also be used as already described.

Sub-horizontal wells are formed by drilling into the cut slope at a slight upward angle to intercept water-bearing strata. The hole is then lined with a slotted or perforated pipe to keep it open and to carry the water out. Usual diameters range from 50 to 100 mm and lengths may reach 50m.

4.5.3 Erosion control

(1) General

Erosion problems may occur on the side slopes of embankments or cuttings, gravel shoulders or at any other point where surface runoff is concentrated or a spring occurs. The obvious cures are therefore well designed surface or sub- surface linage features and appropriate slope angles for the soils and rocks present.

(2) Protection and grassing

i) Top soiling and grassing

Sprigs of indigenous, "runner" type, grass can be planted on slopes by one or two methods:

- The slope shall be covered with a layer of tine topsoil free of stones greater than 50 mm. The minimum soil thickness should be 75 mm. The layer shall then be planted with grass.
- Sprigs and root "runners" of grass shall be planted at approximately 200 rom centers in pockets of topsoil, 75 mm deep. Planting should be carried out at the beginning of a rainy season.

ii) Surface treatments with seed and fertilizers

When difficulties are anticipated in establishing a healthy growth of grass on a sterile soil, a mixture of grass seeds and fertilizer should be applied. This can be done either as a wet or dry process. In the former process grass seed, fertilizer, mulch material water are mixed from a slurry which is then sprayed onto the ground. In the dry process grass seed and fertilizer are mixed and applied to the ground, followed by watering and possible application of mulching material.

iii)Gravel or stone blanketing

Erodible materials may be protected by placing coverings of gravel or stone blankets. The blanketing material should have a maximum size of 40 mm and be placed in (in even layer of at least 75. mm.

iv) Fascines (Boodles of sicks or twigs)

Placing fascines or branches over the most vulnerable areas, generally combined with some form of grass planning, will help stabilize the slope until it is covered by grass or other vegetation.

v) Serrated (grooved) slopes

Serrated slopes will aid the establishment of vegetation. Serrations may be constructed in any material that is rip able or that will hold a vertical or sub vertical face, until vegetation becomes established.

vi) Other protective works

More costly types of protection, such as stone pitching (possibly grouted), gabions, masonry or placing of concrete may also be used, but, in general, they are economically justified only where the overall slope stability has to be improved.

(3) Protection of ditches and channels

i) Critical length of unlined ditches

The critical length of unlined ditches shall be determined, with regard to erosion control.

The critical length is defined as the maximum length of unlined ditch, in which water velocities do not give rise to erosion.

The maximum velocity of water can be calculated from the slope, shape and dimensions of the ditch, volume of water and from the roughness coefficient of the material. Knowing the maximum permissible velocity for each type of material, the maximum length of ditch in this material can then be determined.

The recommended maximum permissible velocities for different type of materials are as follows:

Material	Max. permissible Velocity (m/s)
Fine sand	0.3*
Silt – Coarse sand	0.4 – 0.6*
Silty clay – Fine gravel	0.5 – 08*
Stiff clay	0.9 – 1.3*
Coarse gravel	1.2 -1.7*
Soft rock – Conglomerate	1.8 -2.5*
Hard rock – masonry – Concrete	3.0+

* Where the materials are grassed, the maximum permissible velocity is of the order of 1.5 m/s if a good cover is provided and 1.1 m/s if a sparse cover is provided.

ii) Methods of protection

Section of ditch beyond the critical must be protected from erosion by lining.

The following lining method can be used:

- Grassing
- Turfing
- Stone pitching (possibly grouted)
- Placing of masonry
- Concreting
- Reducing the gradient and constructing protected steps
- placing velocity breakers

iii) Sedimentation control

If water velocities are too low sedimentation may occur.

Ditches and drains should therefore be given sufficient gradient everywhere in so far as topography and erosion control will permit.

Sedimentation velocities for a few type of material are approximately the following:

- Silt	0.08 m/s
- Fine sand	0.15 m/s
- Coarse sand	0.20 m/s
- Fine gravel	0.30 m/s
- Gravel	0.65 m/s

4.6 SUBGRADE

4.6.1 General

For a rational approach to pavement design, the most important characteristic of the subgrade is its elastic modulus. However, the measurement of this modulus requires fairly complicated and time-consuming tests.

However, it has been proved that there is good correlation between the California Bearing Ratio (CBR) and the elastic modulus. Since the CBR test is fairly easy and widely used test, it has been adopted as the quantitative means of evaluating the subgrade bearing strength.

4.6.2 Determining Subgrade Strength

(1) Recommended subgrade CBR test procedure

The actual strength of the subgrade and, in particular, its actual CBR, depends on the type of material, its density and its moisture content.

For each type of material, it is therefore necessary to determine the relative compaction that should be obtained in-situ and the maximum moisture content likely to occur in the subgrade.

In order to obtain a complete knowledge of the relationship between density, moisture content and CBR, a “6 points” CBR test should be carried out on a representative sample of each type of subgrade material encountered. The tests are conducted in the following way:

The material shall be compacted at 3 different levels of compaction. The samples shall be molded at the moisture content which is expected at the time of in-situ compaction (in general, at the Optimum Moisture Content). At each level of compaction, one CBR shall be measured immediately on one unsoaked specimen and one CBR shall be measured on one soaked specimen. The time of soaking will depend on the anticipated subgrade conditions. The amount of water absorbed during soaking and the eventual swell shall also be measured.

The above method enables an estimate to be made of the subgrade CBR at different densities and thus helps in deciding the relative compaction required. It also indicates the loss of strength which soaking may cause.

(2) Subgrade compaction

The compaction requirements are generally as follows:

In some cases, it is advantageous to obtain relative compactions higher than the above figures, since compaction not only improves the subgrade bearing strength, but also reduces permeability. This applies, in particular, to clayey sands, silty sands and granular materials, the coarse particles of which are hard enough not to crumble under heavy compaction.

(3) Estimating the subgrade moisture content

The actual moisture content of the subgrade soil under the road pavement will depend on many factors, principally:

- local climate
- depth of the water table
- type of soil

- topography and the drainage
- permeability of the pavement materials
- permeability of the shoulders

(4) Determining the subgrade design strength

Unless a more accurate estimation of the ultimate subgrade moisture content can be made and backed by factual data, the subgrade strength shall be determined as follows:

(i) In areas where the mean annual rainfall exceeds 500 mm, the determination of the subgrade strength shall be based on CBR's measured after 4 days soak.

(ii) In dry areas, where the mean annual rainfall is less than 500 mm, the subgrade strength may be evaluated in terms of CBR's measured at Optimum Moisture Content (Standard Compaction). However, a design based on such unsoaked CBR's will be permitted only where it has been established that no prolonged soaking may occur. For this purpose, consideration shall be given to factors such as permeability of the natural ground and topography (in other words, to the ability of water to drain rapidly under all circumstance).

(iii) For calculattion of design CBR, outliers are removed and design CBR is calculated by folowing equation:

Design CBR = average of CBR's measured - standard deviation of CBR's measured

4.7 FLEXIBLE PAVEMENT DESIGN

4.7.1 Design Principles.

Pavement design is a complex subject. A pavement is built of materials with variable and diverse properties and the anticipated traffic loading are often difficult to predict

The pavement design and the selected design period should preferable enable the pavement to be rehabilitated, rather than be completely reconstructed, when terminal serviceability level has been reached (See Figure 4.7.1).

The design objective is to produce a structurally balanced pavement which, at minimum present worth of cost, will carry the traffic for the structural design period in the prevailing environment at an acceptable service level without major structural defects. If necessary, the pavement can be strengthened to carry the traffic over the full analysis period by means of various rehabilitation measures.

The overall aim should be to design a well balance pavement. Ideally each layer in a balanced pavement would be stressed to the same percentage of its capacity. This is however seldom achieved and the aim in practice is not to over stress any single layer. It is also important that the in situ materials be utilized as far as possible, particularly in the lighter classes of roads.

4.7.2 Design Consideration.

(1) Traditional Approach

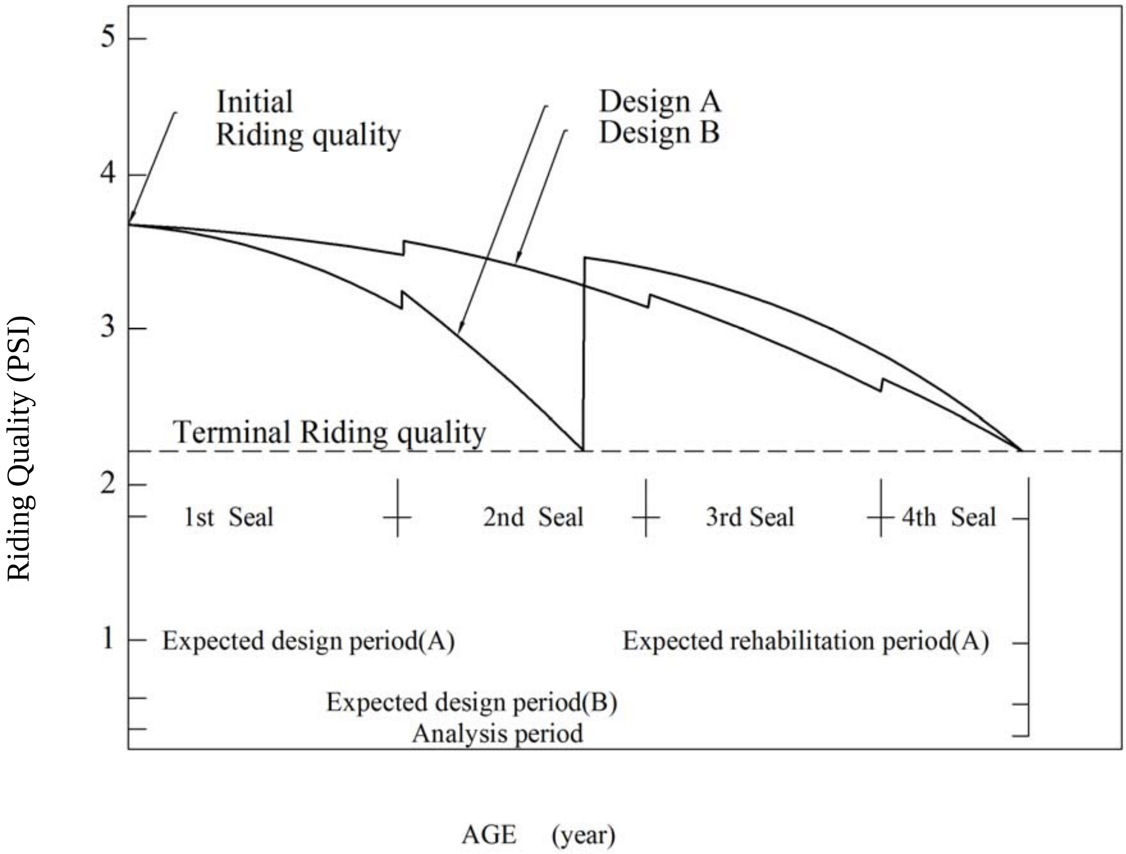
The traditional approach is that the material and strength requirements taper off fairly rapidly with depth. This rate of reduction is determined by the factors of traffic loading and pavement composition. The traditional pavement structure consisting of a relatively strong base with rapidly diminishing support is therefore not in all cases in agreement with the anticipated traffic loading or in situ conditions.

(2) Pavement Structure

The performance of pavement structure is dependent of the intervention and compatibility of the different layers. The pavement can be designed as a “shallow” or “deep” structure.

A shallow structure consists of one or two strong but rigid layer with relatively little support from the underlying layers. This design results in relatively high stresses and is fairly critical to overloading. A concrete pavement is example of a shallow structure.

A deep structure consists of a number of more flexible layers of lower strength. The underlying in situ material contributes more to the pavement strength. The stresses in the layers are lower and better distributed than in the case of a shallow structure. The deep structure is less sensitive to overloading and can deform without actually failing. This type of structure is usually more economical where good in situ subgrades are present.



Design A: Requires one resurfacing and one structural rehabilitation during the analysis period

Design B: Requires three resurfacing and no structural rehabilitation during the analysis period

Figure 4.7.1 Schematic Illlustration of Design life and Alternative Design and Construction Strategies

Neutral plane

The neutral plane divides the compressive and tensile zones in the pavement. The position of this plane is determined by the composition of the pavement layers.

By strengthening the lower layers (subbase zone) in the heavier pavement design, the neutral axis positioned lower. A larger portion of the upper part of the pavement is then placed in compression. The tensile stresses in the natural materials at the bottom of the structure are also reduced.

Material with a low tensile strength, such as crushed stone, should preferably be situated above the neutral plane where it will be mostly under compression.

Vertical stress

Vertical stress in the pavement decreases with depth. Careful attention must however be given to the strength, uniformity and density and density requirements of the lower lying layers for the heavier type of pavements (400 to 600 mm) in order to prevent deformations, settlements or failures.

Pavement deflection

Deflection of the pavement is reduced more economically by strengthening the lower (less costly) layers than by additionally strengthening the base or the upper subbase. This is particularly the case in using crushed stone or other processed upper layers.

(3) Design period

The pavement structures will be designed to carry certain cumulative traffic during a given period of time. When the pavement has carried the expected traffic, it will need to be strengthened so that it can carry traffic for a further period.

It is assumed that, during design period, only ordinary (routine) maintenance will be carried out. This will comprise maintenance of shoulders and drainage system, erosion and vegetation control, localized patching and periodic resealing of the wearing course. This maintenance is however highly essential and its neglect will seriously affect the pavement performance.

(4) Stage construction

A point of consideration is whether it is best to initially design a strong pavement, which will last throughout the design period without the need for strengthening, or to design a weaker, and therefore probably more economic pavement, with the aim of strengthening it at some intermediate or later stage to enable it to last the remainder of the design period.

Stage construction is however not recommended for heavy traffic classes as the risk of premature deterioration would be unacceptable for such important roads. Stage constructions should be considered only for medium to light traffic classes.

4.7.3 Pavement Design Method

There are several pavement design methods available to the designer. The following two design methods are proposed in this manual.

(1) Transport Research Laboratory (TRL), Overseas Road Note 31

The design method contains the pavement structure catalog which relates the CBR and the total number of standard axle load.

(2) AASHTO, Guide for Design of Pavement Structures 1993

The method refers to mechanistic design methods, the structural number method, the CBR various cover curves, stress strain calculations with E-modulus for various materials and layers.

Whatever method used, factors such as road category, design strategy, traffic, materials available, and the environment must be taken into account.

Some estimation of future maintenance measures is also necessary before a comparison of different structures can be made on basis of present worth of costs.

There are several methods of calculating the present worth of cost and the designer is recommended to use a method of his own choice in this regard.

4.7.4 Transport Research Laboratory (TRL), Overseas Road Note 31

(1) Subgrade Strength

For structural design purposes, when a material is classified according to the CBR, it is implied that no more than 10 percent of the measured value for such a material, will fall below the classification value.

Subgrade

The design subgrade CBR is normally limited to six design groups in the structural design method as Table 4.7.1.

Fill

When the road is designed as fill, the designer must use the best information available on the local materials that are likely to be used. Material with CBR values less than 3% should not be used inside structural road fills.

Cut

The design CBR of the subgrade in cut shall be the lowest CBR encountered within the controlled material depth.

(2) Traffic

The calculation procedure is mentioned in section 4.2. After calculation of ESAL, classification of traffic class is shown in Table 4.7.1.

Table 4.7.1 Traffic Class and Subgrade Strength Class

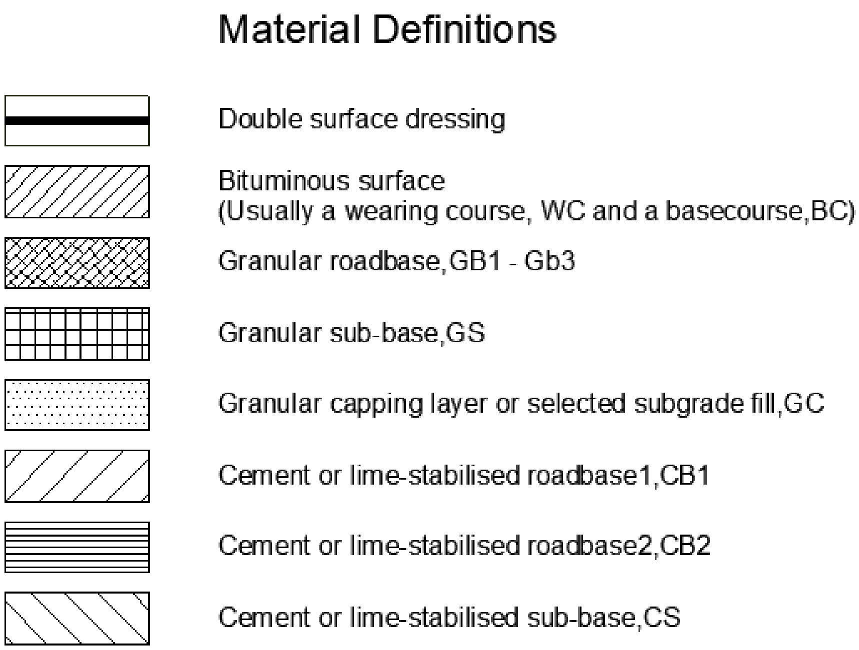
Traffic Classes		Subgrade strength classes	
Class	10^6 esa	Class	Subgrade (CBR %)
T1	< 0.3	S1	2
T2	0.3 -0.7	S2	3-4
T3	0.7 – 1.5	S3	5-7
T4	1.5 – 3.0	S4	8-14
T5	3.0 – 6.0	S5	15-29
T6	6.0 – 10	S6	30+
T7	10 – 17		
T8	17 - 30		

(3) The Pavement Material

The specification for the materials to be used in the various pavement layers is specified in the standard technical specifications.

(4) Pavement Structure Catalog

Pavement structure catalog is shown in Figure 4.7.2 to Figure 4.7.6.

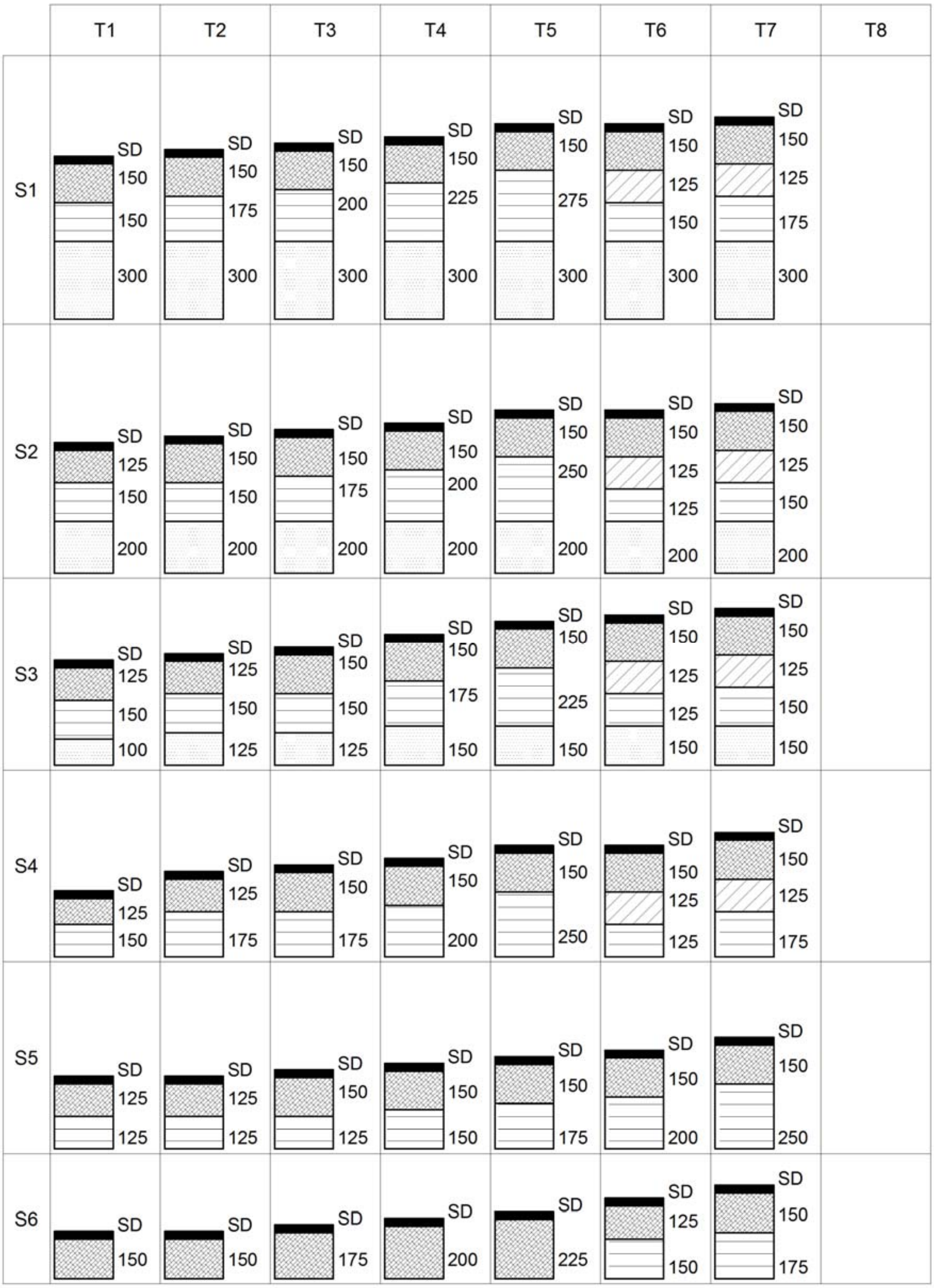


Note: 1 * Up to 100 mm of sub-base may be substituted with selected fill provided the sub-base is not reduced to less than the road-base thickness or 200mm whichever is the greeter.

The substitution ratio of sub-base to selected fill is 25mm : 32mm.

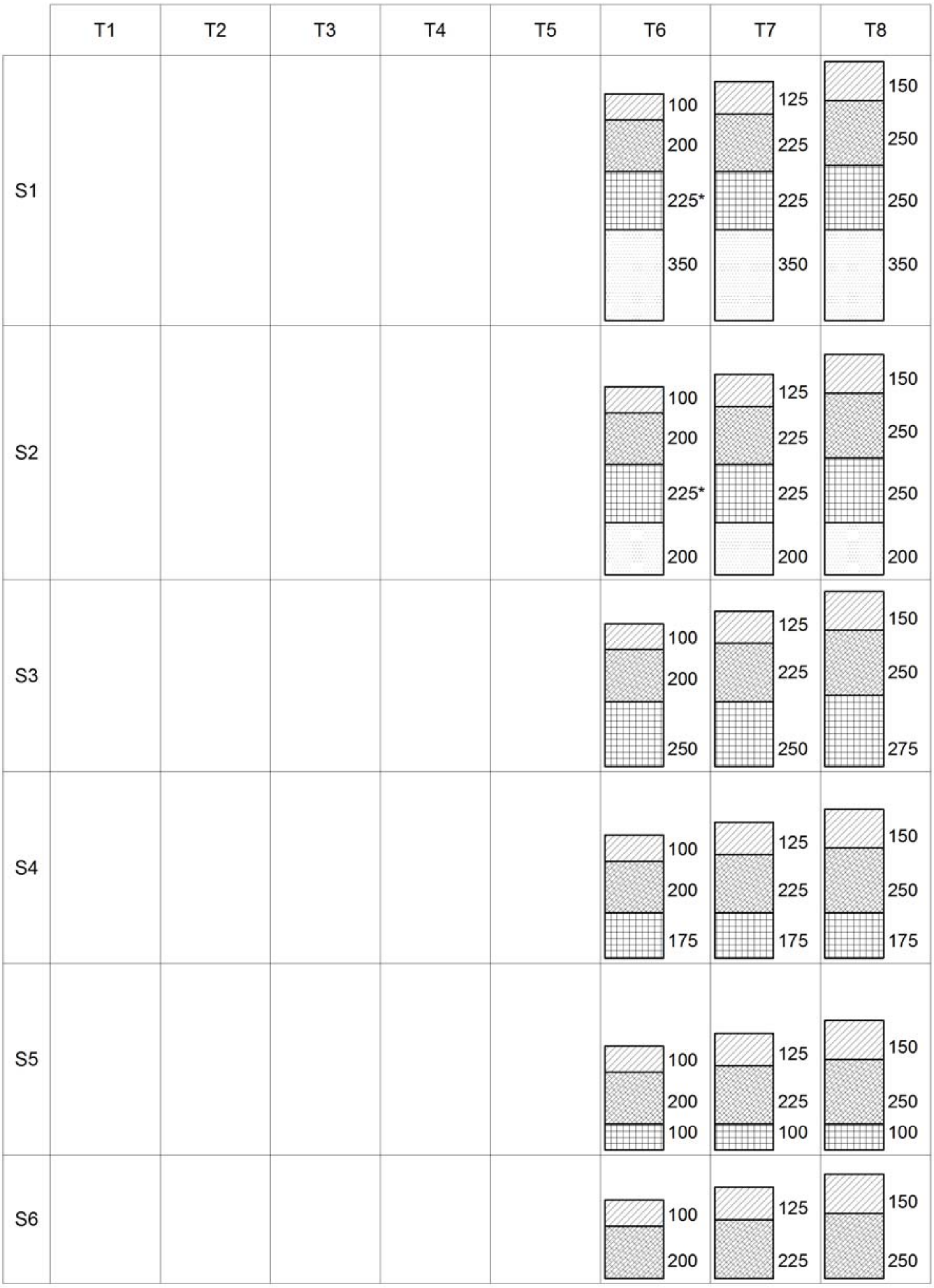
2 A cement or lime-stabilized sub-base may also be used.

Figure 4.7.2 Chart 1: Granular Roadbase / Surface Dressing



Note: Sub-base to fill substitution not permitted

Figure 4.7.3 Chart 2: Composite Road Base(Unbound & Cemented) / Surface Dressing

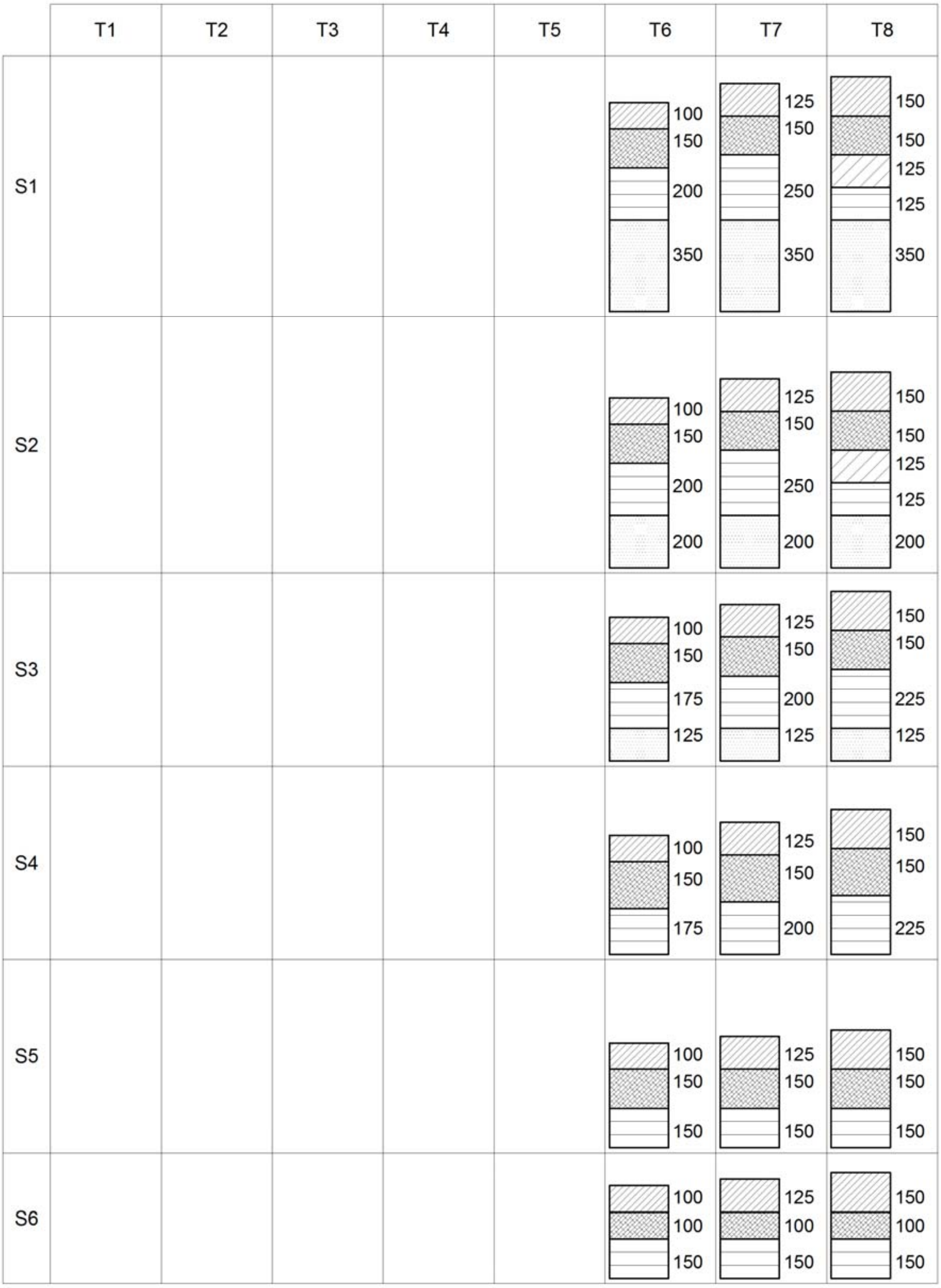


Note: 1 * Up to 100 mm of sub-base may be substituted with selected fill provided the sub-base is not reduced to less than the road-base thickness or 200mm whichever is the greeter.

The substitution ratio of sub-base to selected fill is 25mm : 32mm.

2 A cement or lime-stabilized sub-base may also be used.

Figure 4.7.4 Chart 3: Granular Roadbase / Structural Surface



Note: Sub-base to fill substitution not permitted

Figure 4.7.5 Chart 4: Composite Roadbase / Structural Surface

Note: A granular sub-base may be also be used.

Figure 4.7.6 Chart 5: Cemented Roadbase / Surface Dressing

4.7.5 AASHTO, Guide for Design of Pavement Structures 1993

(1) Structure Number

Required pavement strength, which is called the Structural Number (SN), will be calculated by the following formula in this method. Estimated accumulative axle loads of heavy vehicles (i.e. damage to pavement: W18) in the design/analysis period and bearing capacity of subgrade (MR) are principal factors to determine SN.

$$\log_{10}(W18) = Z_R \times S_0 + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

W18: Accumulative axle loads (Number of single axles passing with 18kip (= 8.16ton) weight in design/analysis period)

M_R: Resilient coefficient of subgrade

SN: Structure Number (Required strength of whole pavement structure)

Z_R: Reliability coefficient

S_0: Standard deviation

⊿PSI: Difference between initial serviceability index and terminal serviceability index of pavement

(2) Selection of Layer Thicknesses

Once the design structural number (SN) for an initial pavement structure is determined, it is necessary to identify as set of pavement layer thickness which, when combined, will provided the load-carrying capacity corresponding to the design SN. The following equation provides the basis for converting SN into actual thickness of surfacing, base and subbase:

$$SN_p = a_1 \times D_1 + a_2 \times D_2 \times m_2 + a_3 \times D_3 \times m_3$$

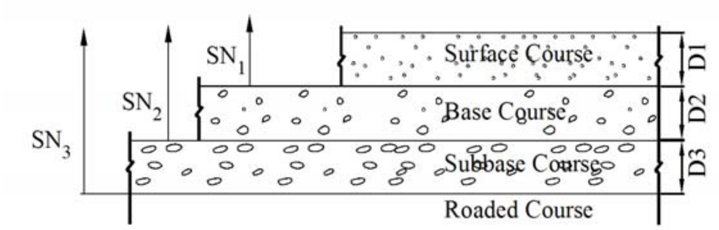
SN_p Structure Number of determined pavement structure

a_n Material coefficient of each layer (wearing course −sub base course)

D_n Thickness of each layer (inch)

m_n Drainage coefficient of each layer

For flexible pavements, the structure is a layered system and should be designed in accordingly. The structural should be designed in accordance with the principles shown in Figure 4.7.7.



$$D^*_1 \geq \frac{SN_1}{a_1}$$

$$SN^*_1 = a_1 D^*_1 \geq SN_1$$

$$D^*_2 \geq \frac{SN_2 - SN^*_1}{a_2 m_2}$$

$$SN^*_1 + SN^*_2 \geq SN_2$$

$$D^*_3 \geq \frac{SN_3 - (SN^*_1 + SN^*_2)}{a_3 m_3}$$

1) a, D, m and SN are as defined in the text and are minimum required values.
2) An asterisk with D or SN indicates that it represents the value actually used, which must be equal to or greater than the required value.

Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.7.7 Proceduere for Determining Thicknesses of Layers Using a Layered Analysis Approach

(3) Traffic

Traffic information required by the design equations used in this guide includes axle loads, axle configuration, and number of applications. The procedure used in this guide to convert a mixed traffic stream of different axle loads and axle configurations into a design traffic number is to convert each expected axle loads into an equivalent number of 18-kip single axle loads and to sum these over design period. General procedure of calculation of ESALs is mentioned in Section 4.2, but the calculation of equivalence factor is different from ORN31, the procedure for converting mixed traffic to ESALs is shown in APPENDIX-I.

(4) Serviceability

The primary measure of serviceability is the Present Serviceability Index (PSI), which is ranges from 0 (impossible road) to 5 (perfect road). Selection of the lowest allowable PSI or terminal serviceability index (p_t) is based on the lowest index that will be tolerated before rehabilitation, resurfacing, or reconstruction became necessary. An index of 2.5 or higher is suggested for design of major highway and 2.0 for highways with lesser traffic volumes. The original or initial serviceability (p_o) is recommended 4.2 for flexible pavements and 4.5 for rigid pavements in AASHTO.

(5) Reliability

Reliability is a means of incorporating some degree of certainty into the design process to ensure that the various design alternatives will last the analysis period. Table 4.7.2 presents recommended levels of reliability for various functional classifications. A standard normal deviate (Z_R) value corresponding to selected levels of reliability is shown in Table 4.7.2.

Table 4.7.2 Suggested Levels of Reliability for Various Function Classificaations

Functional Classification	Recommended Level of Reliability	
	Urban	Rural
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

Source: AASHTO, Guide for Design of Pavement Structures 1993

Table 4.7.3 Standard Normal Deviate (Z_R) Values Corresponding to Selected Levels of Reiability

Reliability R (%)	Standard Normal Deviate, Z_R
50	-0.000
60	-0.253
70	-0.524
75	-0.674
80	-0.841
85	-1.037
90	-1.282
91	-1.340
92	-1.405
93	-1.476
94	-1.555
95	-1.645
96	-1.751
97	-1.881
98	-2.054
99	-2.327
99.9	-3.090
99.99	-3.750

Source: AASHTO, Guide for Design of Pavement Structures 1993

(6) Standard Deviation (So)

A standard deviation (So) should be selected that is representative of local conditions. The general range of standard deviation (So) recommended in AASHTO is following:

Rigid Pavement: 0.30 – 0.40

Flexible Pavement: 0.40 – 0.50

(7) Subgrade

The definitive material property used to characterize roadbed soil for pavement design in the guide is the resilient modulus (M_R). It is recognized that many agencies do not have equipment for performing the resilient modulus test. The correlation is given by the following relationship. But, it is strongly recommended that user agencies acquire the necessary equipment to measure M_R. In any case, a well-planned experiment design is essential in order to obtain reliable correlations.

$$M_R(psi) = 1{,}500 \times CBR$$

(8) Material Properties for Structural Design

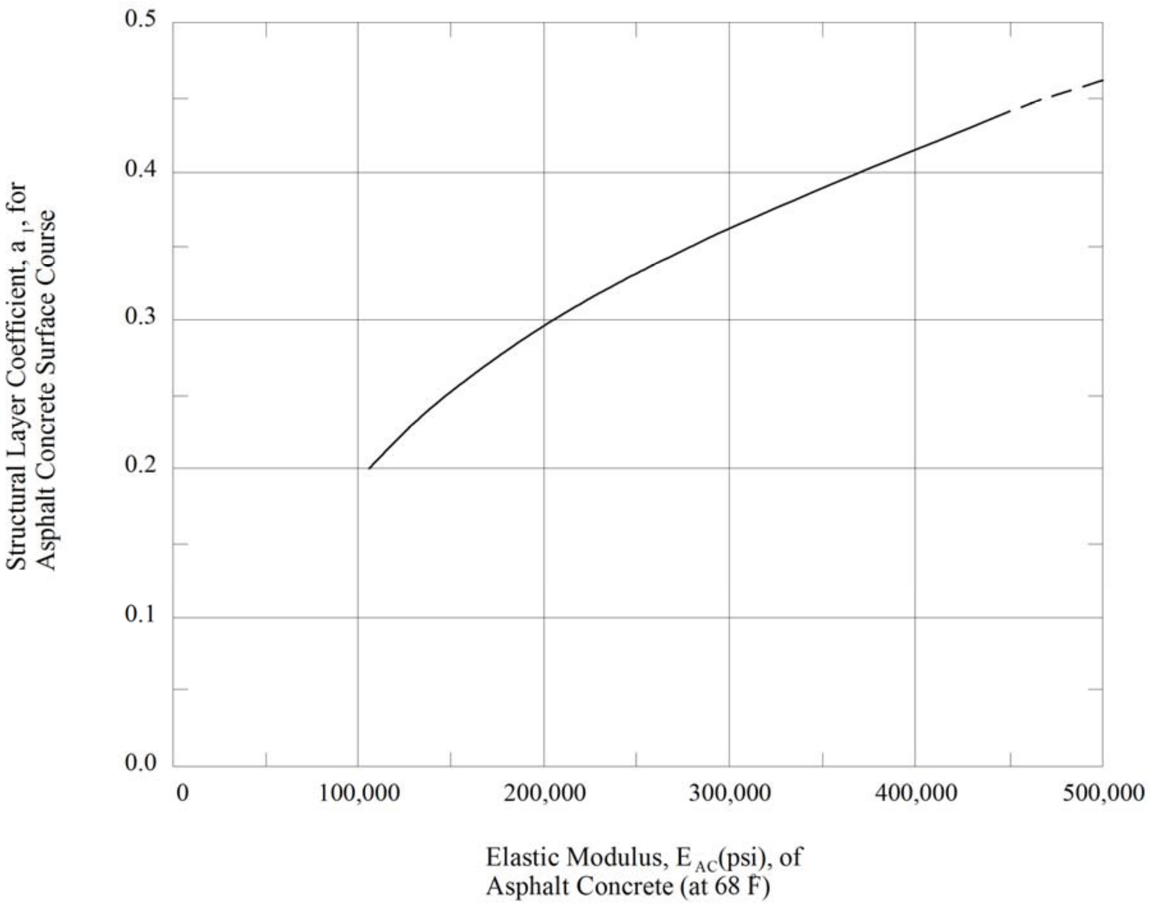
1) **Layer Coefficients**

A value for layer coefficient is assigned to each layer material in the pavement structure in order to convert actual layer thicknesses into structural number (SN). This following general equation for structural number reflects the relative impact of the layer coefficients (a_i) and thickness (D_i):

$$SN = \sum_{i=1} a_i D_i$$

2) **Asphalt Concrete Surface Course**

Figure 4.7.8 provides a chart that may be used to estimate the structural layer coefficient of a dense-graded asphalt concrete surface course based on its elastic (resilient) modulus (EAC) at 68 ° F. Caution is recommended for modulus values above 450,000 psi. Although higher modulus asphalt concrete is stiffer and more resistant to bending, it is also more susceptible to thermal and fatigue cracking.

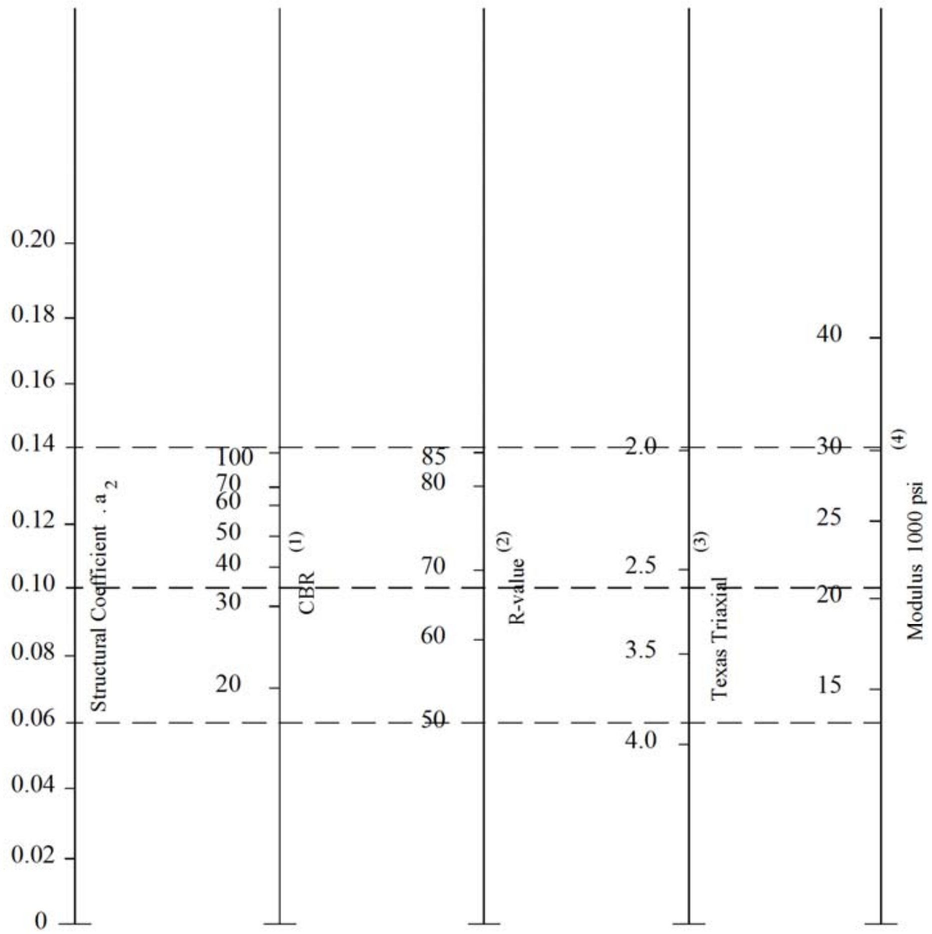


Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.7.8 Chart for Estimating Structural Layer Coefficient of Dense-Graded Asphalt Concrete Based on the Elastic (Resilient) Modulus (3)

3) **Granular Base Layers**

Figure 4.7.9 provides a chart that may be used to estimate a structural layer coefficient, a_2, from one of four different laboratory test result results on a granular base material, including base resilient modulus, E_{BS}. The AASHO Road Test basis for these correlations is:



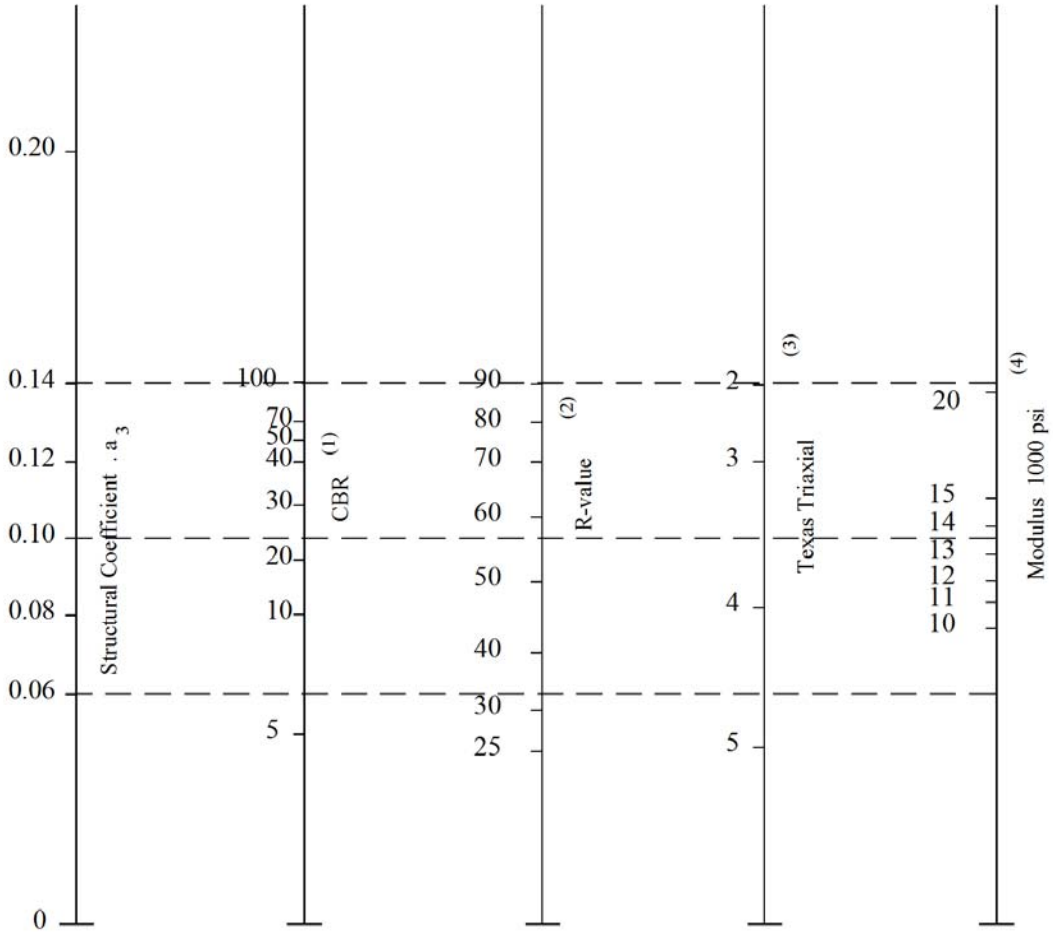
(1) Scale derived by averaging correlations obtained from Illinois
(2) Scale derived by averaging correlations obtained from California, New Mexico and Wyoming
(3) Scale derived by averaging correlations obtained from Texas
(4) Scale derived on NCHRP project (3).

Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.7.9 Variation in Granular Base Layer Coefficient (a_2) with Various Base Strength Parameters (3)

4) **Granular Subbase Layers**

Figure 4.7.10 provides a chart that may be used to estimate a structural layer coefficient, a_3, from one of four different laboratory test result results on a granular subbase material, including subbase resilient modulus, E_{BS}. The AASHO Road Test basis for these correlations is:



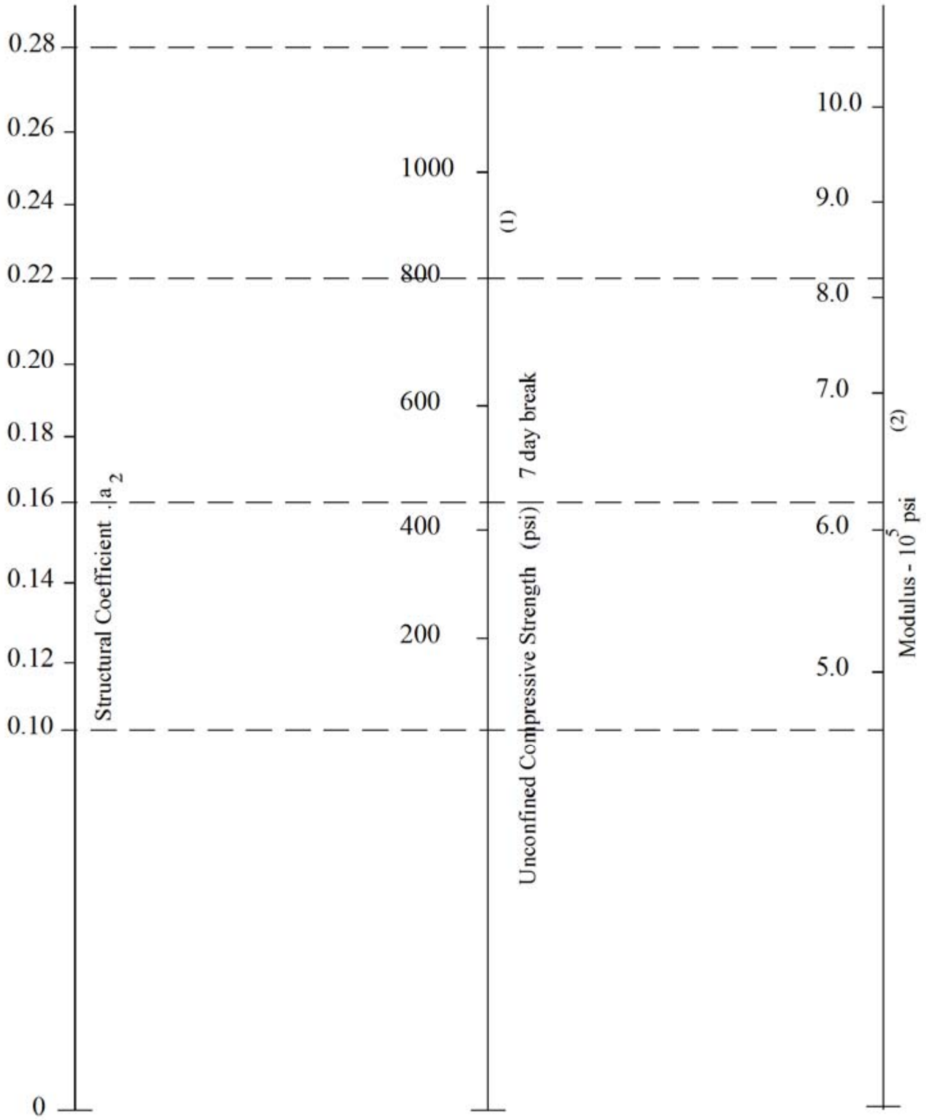
(1) Scale derived from correlations from Illinois.
(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming
(3) Scale derived from correlations obtained from Texas.
(4) Scale derived on NCHRP project (3).

Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.7.10 Variation in Granular Subbase Layer Coefficient (a_3) with Various Subbase Strength Parameters (3)

5) **Cement-Treated Bases**

Figure 4.7.11 provides a chart that may be used to estimate a structural layer coefficient, a2, for a cement-treated base material from either its elastic modulus, E_{BS}, or, alternatively, its 7-day unconfined compressive strength (ASTM D 1633).



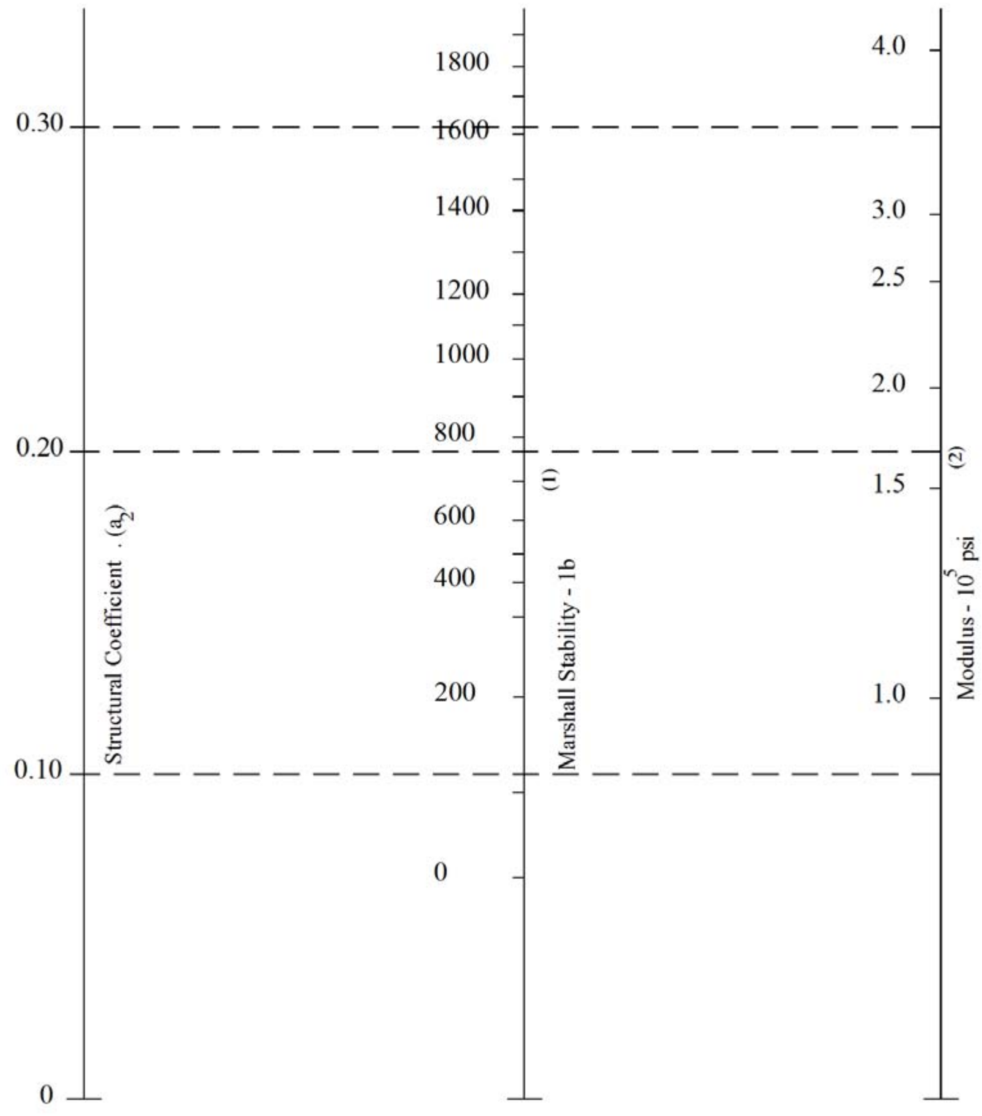
(1) Scale derived by averaging correlations from Illinois, Louisiana and Texas
(2) Scale derived on NCHRP project (3).

Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.7.11 Variation in Cement-Treated Bases with Base Strength Parameters (3)

6) **Bituminous-Treated Bases**

Figure 4.7.11 provides a chart that may be used to estimate a structural layer coefficient, a2, for a bituminous-treated base material from either its elastic modulus, E_{BS}, or, alternatively, its Marshall stability (AASHTO T 245, ASTM D 1559).



(1) Scale derived by correlations obtained from Illinois
(2) Scale derived on NCHRP project (3).

Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.7.12 Variation in Bituminous-Treated Bases with Base Strength Parameters (3)

(9) Drainage coefficient

Table 4.7.4 presents the recommended m_i values as a function of the quality of drainage and the percent of time during the year the pavement structure would normally be exposed to moisture levels approaching saturation.

Table 4.7.4 Recomended m_i Values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1-5%	5-25%	Greater Than 25%
Excellent	1.40-1.35	1.35-1.30	1.30-1.20	1.20
Good	1.35-1.25	1.25-1.15	1.15-1.00	1.00
Fair	1.25-1.15	1.15-1.05	1.00-0.80	0.80
Poor	1.15-1.05	1.05-0.80	0.80-0.60	0.60
Very poor	1.05-0.95	0.95-0.75	0.75-0.40	0.40

Source: AASHTO, Guide for Design of Pavement Structures 1993

4.8 RIGID PAVEMENT DESIGN

4.8.1 General

Rigid pavements are constructed of Portland cement concrete. Generally, concrete pavement can carry much more load repetitions than flexible pavement do and also have longer service period. However, the capital cost is higher and require maintainenance periodically.

4.8.2 Pavement Design Method

There are several pavement design methods available to the designer. The following two design methods are proposed in this manual.

(1) Transport Research Laboratory (TRL), Road Note 29

The design method contains the pavement structure catalog which relates the CBR and the total number of standard axle load.

(2) AASHTO, Guide for Design of Pavement Structures 1993

The method refers to mechanistic design methods, the structural number method, the CBR various cover curves, stress strain calculations with E-modulus for various materials and layers.

4.8.3 Transport Research Laboratory (TRL), Road Note 29

(1) Subgrade

In the design of concrete roads, three qualities of subgrade are considered as defined in Table 4.8.1.

Wherever practical the water-table should be prevented from rising to within 600mm of the formation level. This may be done by sub-soil drainage, or by raising the formation level by means of embankment.

It is important to provide efficient permanent drainage to remove water from the subgrade and sub-base, both during construction and during the life of the road.

Table 4.8.1 Classification of subgrades for concrete roads and minimum thickness of sub-base required

Type of subgrade	Definition	Minimum thickness of sub-base required
Weak	All subgrades of CBR value 2 per cent or less	150mm
Normal	Subgrades other than those defined by the other categories	80mm
Very stable	All subgrade of CBR value 15 per cent or more. This category includes undisturbed foundations of old roads	0mm

(2) Concrete Slab

The required thickness for reinforced and unreinforced concrete slab in terms of the cumulative number of standard axles to be carried for the three types of subgrades is determined by the chart shown in Figure 4.8.1. The designs given by this chart are based on a minimum compressive strength for concrete of 28 MPa at 28 days.

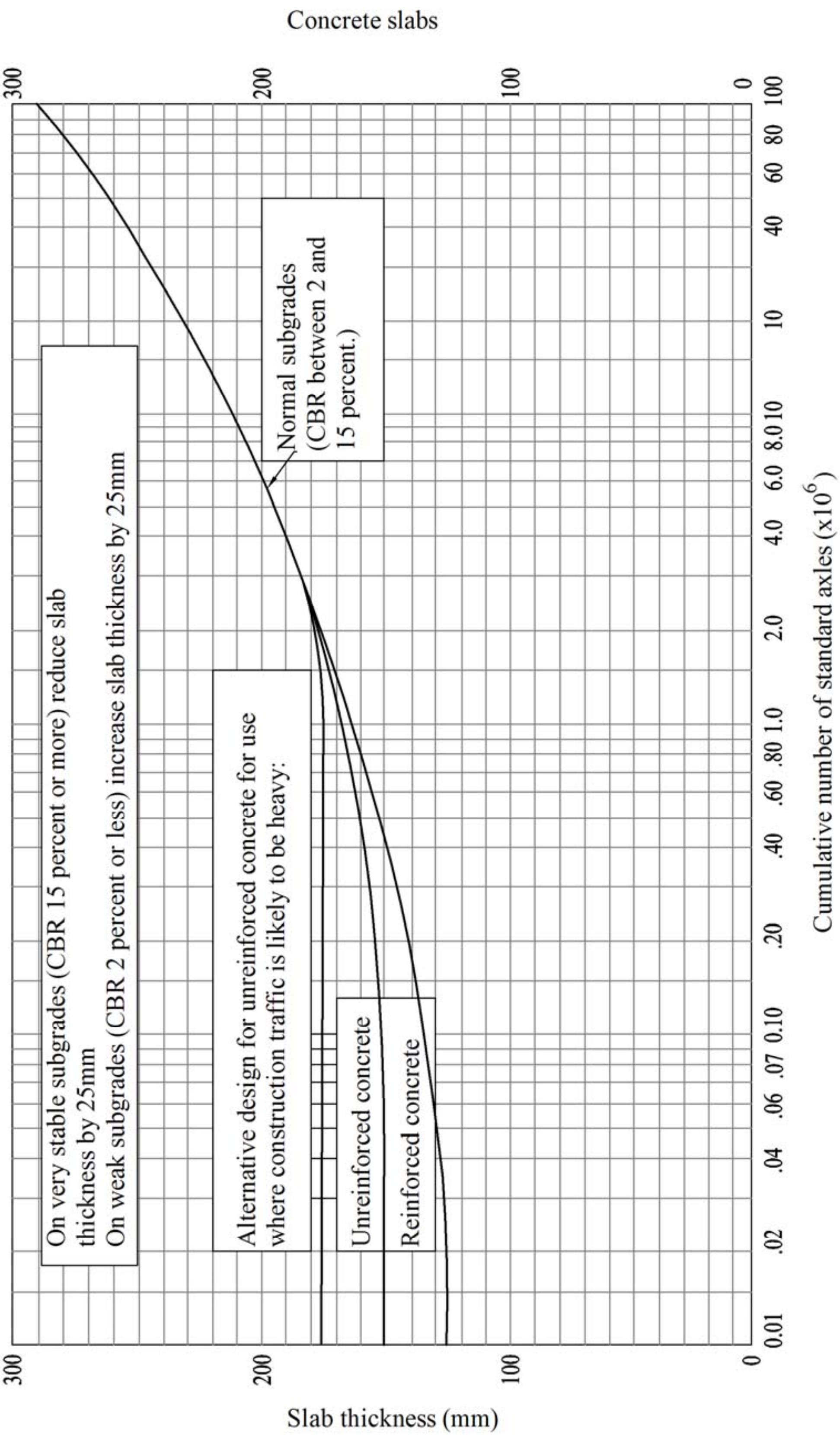


Figure 4.8.1 Thickness of Concrete Slab

(3) Reinforcement

For reinforced concrete the minimum weight of reinforcement required in relation to the cumulative number of standard axles to be carried is given in Figure 4.8.2 in terms of weight of long mesh reinforcement and area of steel bar unit width of pavement. The reinforcement should have 60mm cover from the surface except for slabs less than 150mm thick where 50mm cover should be provided. The reinforcement should terminate at least 40mm and not more than 80mm from the edge of the slab and from all joints except longitudinal joint.

At the transverse overlap of reinforcing mats the first transverse wire of one mat should lie within the last complete mesh of the previous mat and the overlap should be not less than 450mm. No overlap will be needed longitudinally between mats. When deformed bar reinforcement is used the overlap of the bars should not be less than 40 bar diameters.

Where a two or three lane carriageway width is constructed in one operation, reinforcing mat having transverse wires f 8mm diameter at 200mm centers may be used to span the longitudinal joints in place of tie bars. The 8 mm wires must be long enough to span at least 500 mm either side of the longitudinal joints.

Where a three lane carriageway is constructed in two widths, transverse reinforcement, consisting of 8 mm diameter wires at 200mm centers, which may be incorporated in special mats, should be used in slabs wider than 4.5m the length of this transverse reinforcement should be 600mm longer than a third of the slab width and should be placed centrally.

As a reference, relationship betweem traffic volume, thickness of concrete slab and reinforcement is mentioned in Pavement Design Handbook(2006), Japan. Noted point is that this design is summarsed based on traffic volume of heavy vhicle.

Table 4.8.2 Traffic Volume of Large Vehicle, Thickness of Concrete Slab and Long Mesh

Traffic Class	Traffic Volume of Heavy Vehicle	Concrete Slab			Spacing of Contraction Joint	Tie bar, Dowels
		Flexural strength	Thickness of Slab	Long Mesh Reinforcement		
$N_1 - N_3$	T < 100	4.4 MPa (3.9 Mpa)	15cm (20cm)	3 kg/m^2	8m, 5m in case of without mesh	Tie Bar and Dowel is used
N_4	100 <= T < 250	4.4 MPa (3.9 Mpa)	20cm (25cm)			
N_5	250 <= T <1,000	4.4 Mpa	25cm		10m	
N_6	1,000 <= T <3,000	4.4 Mpa	28cm			
N_7	3,000 <= T	4.4 MPa	30cm			

Reinforcement(Jointed Reinforced Concrete)

Souce: Pavement Design Handbook(2006), Japan

Table 4.8.3 Traffic Volume of Large Vehicle, Thickness of Concrete Slab and Reinforcement(Continuously Reinforced Concrete)

Traffic Class	Traffic Volume of Heavy Vehicle	Concrete Slab		Reinforcement			
		Flexural strength	Thickness of Slab	Longitudinal		Cross section	
				Diameter	Spacing (cm)	Diameter	Spacing (cm)
$N_1 - N_5$	T < 1,000	4.4 MPa	20cm	D16	15	D13	60
				D13	10	D10	30
N_6	1,000 <= T	4.4 MPa	25cm	D16	12.5	D13	60
				D13	8	D10	30

Souce: Pavement Design Handbook(2006), Japan

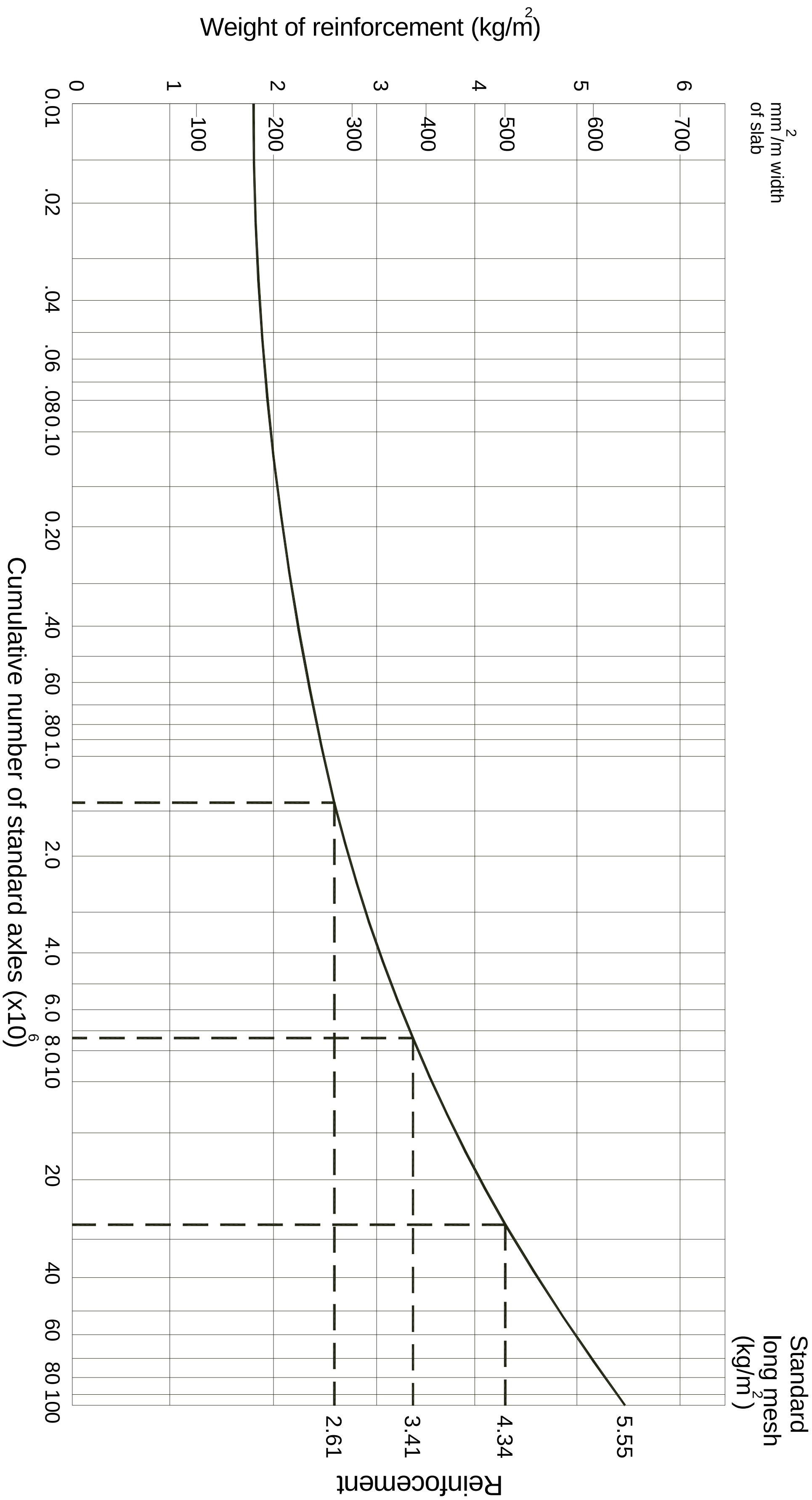


Figure 4.8.2 Reinforcement Minimum Weight for Concrete Slabs

(4) Spacing of Joints in Reinforced Concrete Slabs

The recommended maximum spacing of joints in relation to the weight of reinforcement is shown in Figure 4.8.3. Reinforcement must be discontinuous at both contraction and expansion joints. Longitudinal joints should be provided so that the slabs are not more than 4.5m wide, except where special reinforcement is used as given in 4.8.3(3).

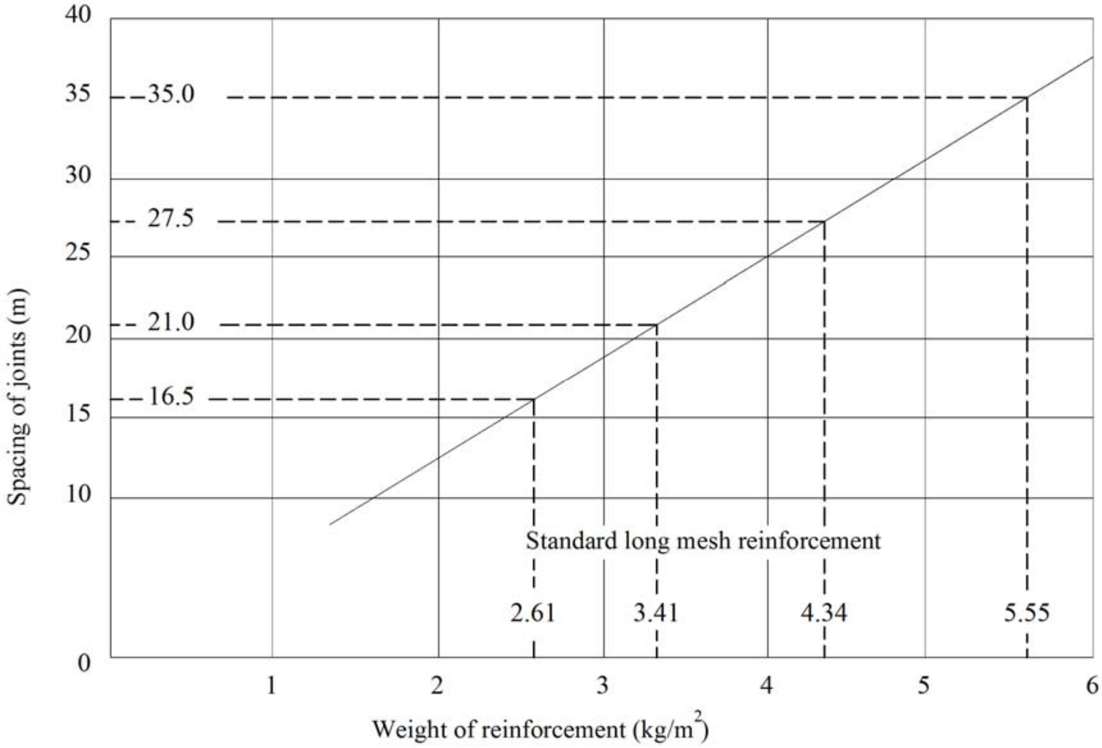


Figure 4.8.3 Maximum Spacing of Joints for Reinforced Concrete Slab

(5) Spacing of Joints in Unreinforced Concrete Slabs

The maximum spacing of expansion joints recommended is 60m for slabs of 200mm or greater thickness and 40m for slabs of lesser thickness, whit intermediate contraction joints at 5m intervals where aggregates other than limestone are used; where limestone is used throughout the depth of the slab, the maximum expansion joint spacing may be increased to 72m and 48m respectively with intermediate contraction joints at 6m intervals.

(6) Details of Joints in Concrete Slabs

All expansion and contraction joints should be have sliding dowel bars conforming to the requirements of Table 4.8.4. The dowels should be placed at 300mm center and half the length of the bars should be coated with a bond-breaking compound. The bars in expansion joints only should be provided with a cap at the debonded end, containing a thickness of 25mm of compressible material to allow the joint to open and close.

Longitudinal joint should have tie bars 12mm in diameter by 1m longer at 600mm center except in the case of roads designed to carry less than 0.15 million standard axles, when the spacing may be increased to 700mm.

Table 4.8.4 Dimention of Dowel Bars for Expansion and Contraction Joints

Slab thickness (mm)	Expansion joints		Contraction joints	
	Diameter (mm)	Length (mm)	Diameter (mm)	Length (mm)
150 – 180	20	550	12	400
190 – 230	25	650	20	500
240 and over	32	750	25	600

4.8.4 AASHTO, Guide for Design of Pavement Structures 1993

(1) General

The AASHTO design procedure is based on the AASHO Road Test pavement performance algorithm. Inherent in the use of the procedure is the use of dowels at transverse joints.

(2) Develop Effective Modulus of Subgrade Reaction

Before the design chart for determining design slab thickness can be applied, it is necessary to estimate the possible level of slab support that can be provided. This accomplished using Table 4.8.5 and Figure 4.8.4 to develop an effective modulus of subgrade reaction, k. The procedure is as following:

The first step is to identify the combinations (or levels) that are to be considered.

Subbase types

Different types of subbase have different strengths or modulus values. The consideration of a subbase type in estimating an effective k-value provides a basis for evaluating its cost-effectiveness as part of the design process.

Subbase thicknesses (inches)

Potential design thickness for each subbase type should also be identified, so that its cost-effectiveness may be considered.

Loss of support, LS

This factor is used to correct the effective k-value based on potential erosion of the subbase material. Standard range of loss of support for each material is shown in Table 4.8.5.

Table 4.8.5 Typical Ranges of Loss of Support (LS) Factors for Various Types of Material

Type of Material	Loss of Support (LS)
Cement Treated Granular Base (E= 1,000,000 to 2,000,000 psi)	0.0 to 1.0
Cement Aggregate Mixtures (E= 500,000 to 1,000,000 psi)	0.0 to 1.0
Asphalt Treated Base (E= 350,000 to 1,000,000 psi)	0.0 to 1.0
Bituminous Stabilized Mixtures (E= 40,000 to 300,000 psi)	0.0 to 1.0
Lime Stabilized (E= 20,000 to 70,000 psi)	1.0 to 3.0
Unbound Granular materials (E= 15,000 to 45,000 psi)	1.0 to 3.0
Fine Grained or Natural Subgrade Materials (E= 3,000 to 40,000 psi)	2.0 to 3.0

NOTE: E in this table refers to the general symbol for elastic or resilient modulus of the material

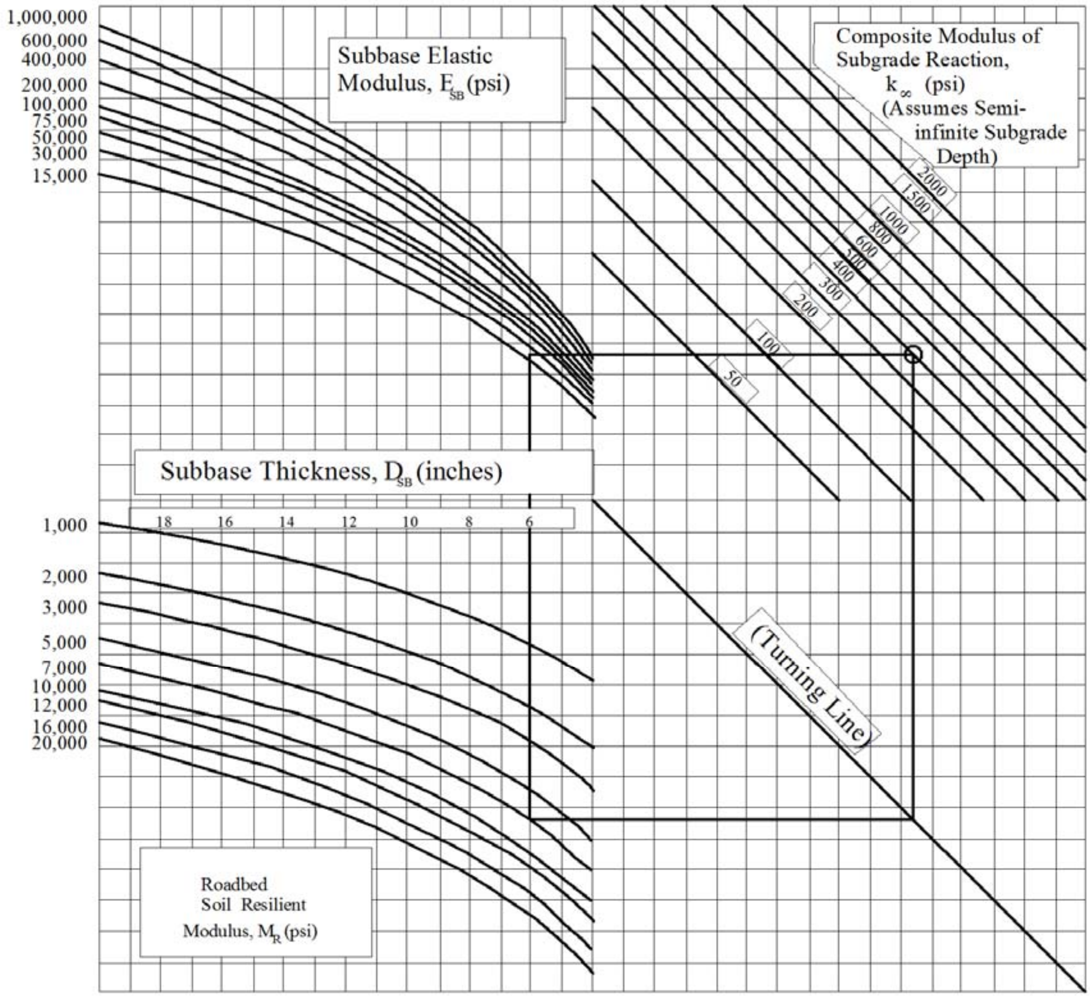
<u>Depth to rigid foundation (feet)</u>
If bedrock lies within 10 feet of the surface of the subgrade for any significant length along the project, its effect on the overall k-value and the design slab thickness for that segment should be considered.

The second step is to identify the subgrade soil resilient modulus values and subbase resilient modulus value as mentioned in section 4.7.5. To consider seasonal value reference is made to original AASHTO, Guide for Design of Pavement Structures 1993.

As a third step, assuming a semi-infinite subgrade depth, the composite modulus of subgrade reaction is estimated by using Figure 4.8.4 . Note that the starting point in this chart is subbase thickness, D_{SB}. If the slab is placed directly on the subgrade, the composite modulus of subgrade reaction is defined using following theoretical relationship between k-values from a plate bearing test and elastic modulus of the roadbed soil:

$$K = M_R / 19.4$$

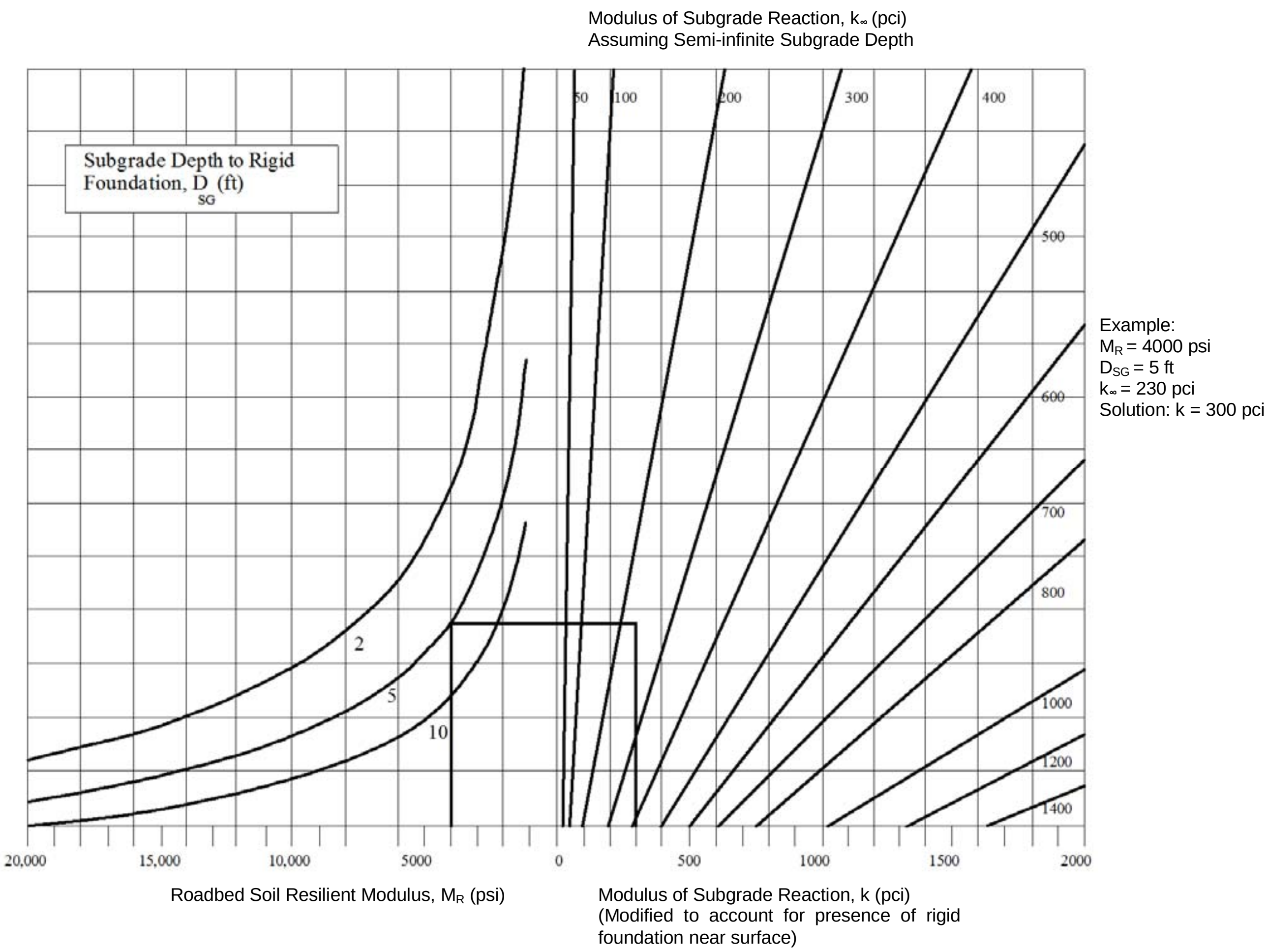
Example:
D_{SB} = 6 inches
E_{SB} = 20,000 psi
M_R = 7,000 psi
Solution: k_∞ = 400 pci



Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.4 Chart for Estimating Composite Modulu of Subgrade Reaction, K_∞, Asuuming a Semi-Infinite Subgrade Depth

The fourth step is to develop a k-value which includes the effect of a rigid foundation near the surface by using Figure 4.8.5. This step should be disregarded if the depth to rigid foundation is greater than 10 feet.

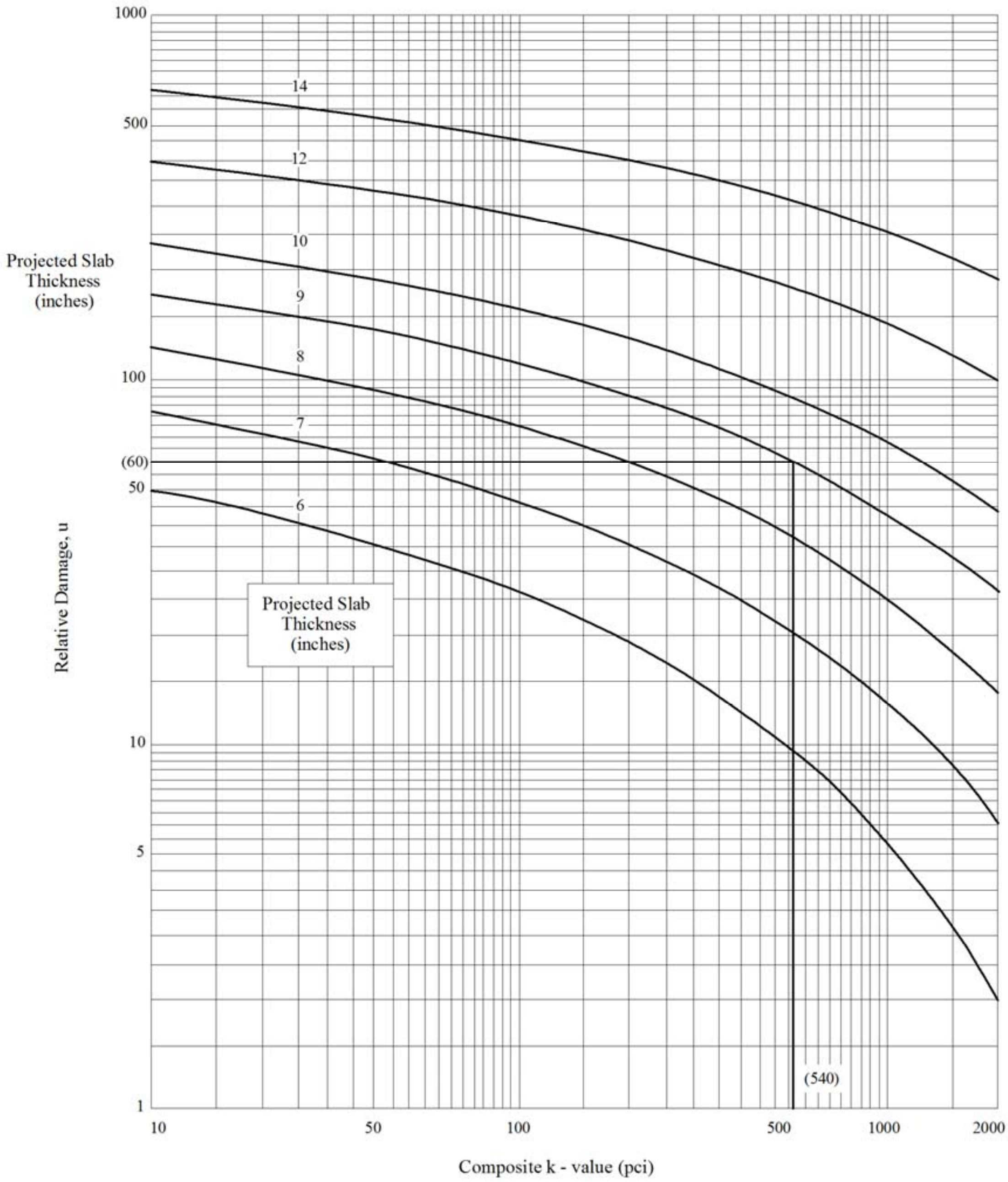


Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.5 Chart to Modify Modulus of Subgrade Reaction to Consider Effets of Rigid Foundation Near Surface(within 10 feet)

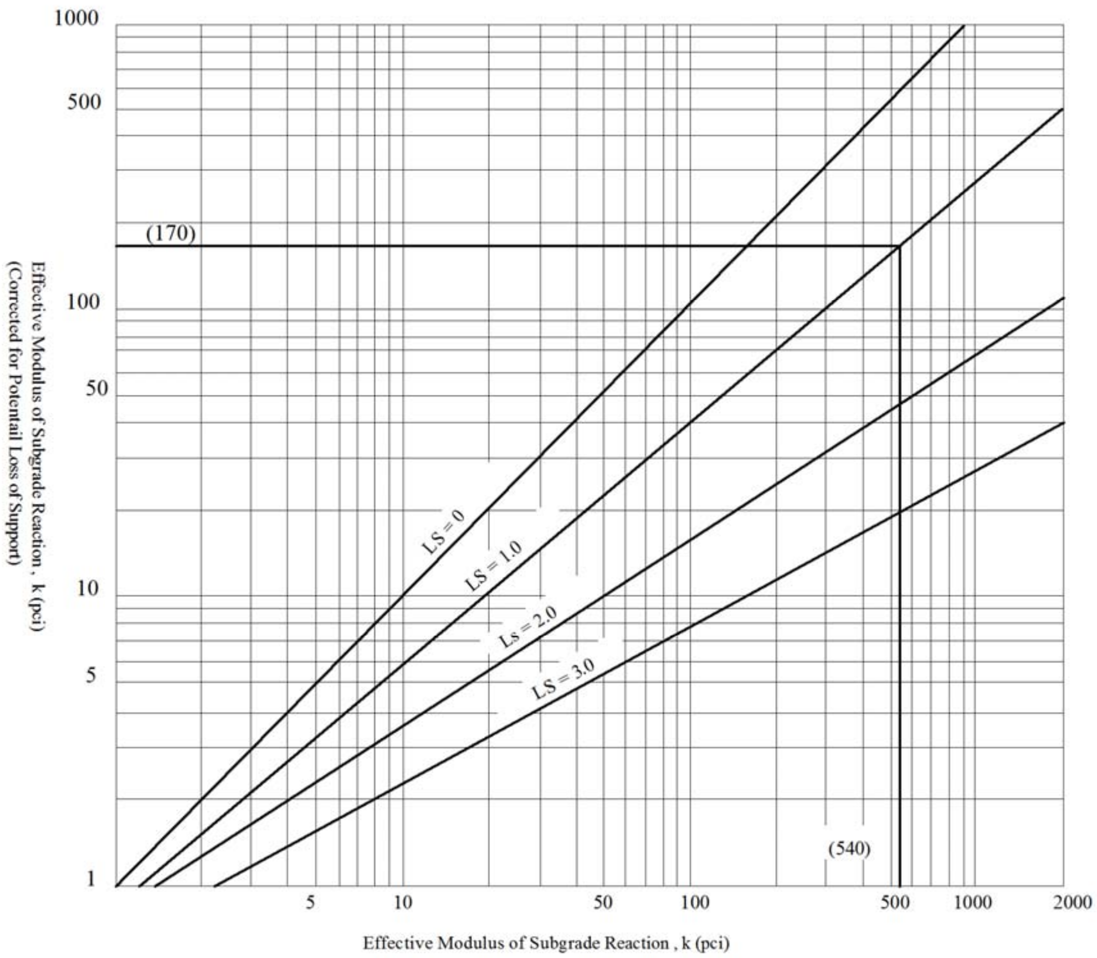
The fifth step is to estimate the thickness of the slab that will be required, and then use Figure 4.8.6 to determine the relative damage, u_r. The effective modulus of subgrade reaction, then, is the value corresponding to the relative damage (and projected slab thickness) in Figure 4.8.6.

The final step is to adjust the effective modulus of subgrade reaction to account for the potential loss of support arising from subbase erosion. Figure 4.8.7 provides the chart for correcting the effective modulus of subgrade reaction based on the loss of support factor, LS.



Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.6 Chart for Estimating Relative Damage to Rigid Pavement Based on Slab Thickness and Underlying Support



Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.7 Correction of Effective Modulus of Subgrade Reaction for Potential Loss of Subbase Support

(3) Determine Required Slab Thickness

The following equation is to determine the slab thickness of each effective k-value identified in the previous section. The designer may then select the optimum combination of slab and subbase thicknesses. In addition to the design k-value, other inputs required by this rigid pavement design include:

$$\log_{10}(W18) = Z_R \times S_0 + 7.35 \times \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.5-1.5}\right)}{1+\frac{1.624\times10^7}{(D+1)^{8.46}}} + (4.22 - 0.32 p_t) \times \log_{10}\left[\frac{S'_C \times C_d\left[D^{0.75} - 1.132\right]}{215.63 \times J\left[D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$

- The estimated future traffic, W18 (section 4.7.5(3)) for the design period
- The reliability, R (section 4.7.5(5))
- The overall standard deviation, S_o (section 4.7.5(6))
- Design serviceability loss, PSI= $p_i - p_t$ (section 4.7.5(4))
- Concrete clastic modulus, Ec

  The following equation is a correlation recommended by the American Concrete Institute for normal weight Portland cement concrete:

  $$E_c = 57{,}000(f_c')^{0.5}$$

where
E_c : PCC elastic modulus (in psi)
f'_c : PCC compressive strength in psi as determined by AASHTO Designation T-22 or T-140 or ASTM C 39.

- Concrete modulus of rupture, S'c
  The modulus of rupture required by the design procedure is the mean value determined after 28 days using third-point loading (AASHTO T97, ASTM C 78). Because of the treatment of reliability in this guide, it is strongly recommended that the normal construction specification for modulus of rupture (Flexural strength) not be used as input, since it represents a value below which only a small percent of the distribution may lie. If it is desirable to use the construction specification, then some adjustment should be applied, based on the standard deviation of modulus of rupture and the percent (PS) of the strength distribution that normally falls below the specification;

  $$S_c'(mean) = S_c + z(SD_s)$$

  Where

  S'_c : estimated mean value for PCC modulus of rapture (psi),
  S_c : construction specification on concrete modulus of rapture (psi)
  SD_s : estimated standard deviation of concrete modulus of rapture (psi)
  z : standard normal variate:
  0.841, for PS=20%
  1.037, for PS=15%
  1.282, for PS=10%
  1.645, for PS=5%
  2.327, for PS=1%

- Load transfer coefficient, J
  The land transfer coefficient, J, is a factor used in rigid pavement design to account for the ability of concrete pavement structure to transfer (distribute) load across discontinuities, such as joints or cracks. Table 4.8.6 establishes ranges of load transfer coefficients for different conditions developed from experience and mechanistic stress analysis. If dowels are used, as a general guideline, the dowel diameter should be equal to the slab thickness multiplied by 1/8 inch (e.g., for a 10-inch pavement, the diameter is 1 and 1/4 inch). The dowel spacing and length are normally 12 inches and 18 inches, respectively.

Table 4.8.6 Recomended Load Transfer Coefficient for Various Pavement Types and Design Condition

Shoulder	Asphalt		Tied P.C.C.	
Load Transfer Devices	Yes	No	Yes	No
Pavement Type				
1. Plain jointed and jointed reinforced	3.2	3.8-4.4	2.5-3.1	3.6-4.2
2. Continuously reinforced concrete pavement (CRCP)	2.9-3.2	N/A	2.3-2.9	N/A

Source: AASHTO, Guide for Design of Pavement Structures 1993

- Drainage coefficient, C_d

 The treatment for the expected level of drainage for a rigid pavement is through the use of a drainage coefficient, C_d, in the performance equation. As a basis for comparison, the value for C_d for conditions at the AASHO Road Test is 1.0. Table 4.8.7 provides the recommended C_d value.

Table 4.8.7 Recomended Values of Drainage Coefficient , Cd, for Rigid Pavment Design

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1-5%	5-25%	Greater Than 25%
Excellent	1.25-1.20	1.20-1.15	1.15-1.10	1.10
Good	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very poor	1.00-0.90	0.90-0.80	0.80-0.70	0.70

Source: AASHTO, Guide for Design of Pavement Structures 1993

(4) Rigid Pavement Reinforcement Design

Because the longitudinal steel reinforcement requirements between jointed reinforced (JRCP) and continuously reinforced concrete pavement (CRCP) are significantly different, the reinforcement designs are treated separately. It should be recognized, however, that the design for transverse steel in CRCP is exactly the same as the design for longitudinal and transverse steel reinforcement in JRCP. In all cases, the amount of reinforcement required is specified as a percentage of the concrete cross-sectional area.

1) **Jointed Reinforced Concrete Pavements**

The nomographs for estimating the percent of steel reinforcement required in a jointed reinforced concrete pavement is presented in Figure 4.8.8. The inputs required include:

- Slab length, L

This refers to the joint spacing or distance, L (feet), between free transverse joints. It is an important design consideration since it has a large impact on the maximum concrete tensile stresses and, consequently, the amount of steel reinforcement required. Because of this effect, slab length (joint spacing) is an important factor that must be considered in the design of any reinforced or un-reinforced jointed concrete pavement. For example, the maximum joint spacing for an 8-inch slab is 16 feet. Also, as a general guideline, the ratio of slab width to length should not exceed 1.25.

- Steel working stress, f_s

This refers to the allowable working stress, fs (psi), in the steel reinforcement. Typically, a value equivalent to 75 percent of the steel yield strength is used for working stress. For Grade 40 and Grade 60 steel, the allowable working stresses are 30,000 and 45,000 psi, respectively. For Welded Wire Fabric (WWF) and Deformed Wire Fabric (DWF), the steel yield strength is 65,000 psi and the allowable working stress is 48,750 psi. The minimum wire size should be adequate so that potential corrosion does not have a significant impact on the cross-sectional area.

- Friction factor, F

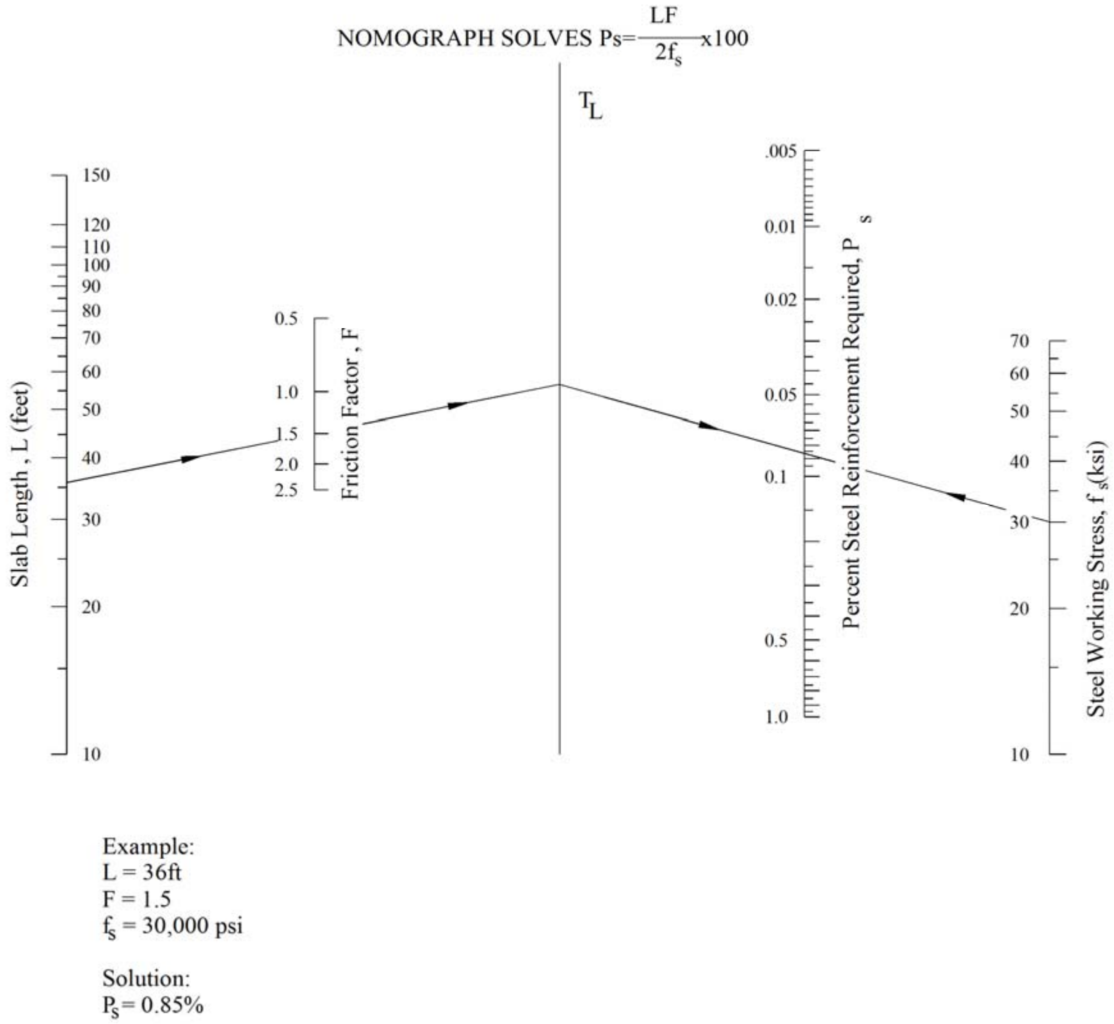
This factor, F, represents the frictional resistance between the bottom of the slab and the top of the underlying subbase or subgrade layer and is basically equivalent to a coefficient of friction. Recommended values for natural subgrade and a variety of subbase materials are presented in Table 4.8.8.

Table 4.8.8 Recomended Friction Factors

Type of Material Beneath Slab	Friction Factor (F)
Surface Treatment	2.2
Lime stabilization	1.8
Asphalt stabilization	1.8
Cement stabilization	1.8
River gravel	1.5
Crushed stone	1.5
Sandstone	1.2
Natural subgrade	0.9

Source: AASHTO, Guide for Design of Pavement Structures 1993

This chart applies to the design of transverse steel reinforcement in both jointed and continuously reinforced concrete pavements, as well as to the design of longitudinal steel reinforcement in JRCP. Normally for joint spacing, less than 15 feet transverse cracking is not anticipated, thus steel reinforcement would not be required.



Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.8 Reinforcement Design Chart for Jointed Reinforced Concrete Pavements

2) **Continuously Reinforced Concrete Pavement**

This section is for the design of longitudinal reinforced steel in continuously reinforced concrete pavements. The design procedure presented may be systematically performed using the worksheet in Table 4.8.11. In this table, space is provided for entering the appropriate design inputs, intermediate results and calculations for determining the required longitudinal steel percentage. The design inputs required by this procedure are as follows:

- Concrete indirect tensile strength, ft
  Steel reinforcement design is based on the tensile strength derived from the indirect tensile test which is covered under AASHTO T 198 and ASTM C 496 test specifications. The strength at 28 days should be used. For this design procedure, the indirect tensile strength will normally be about 86 percent of concrete modulus of rupture.

- Concrete shrinkage at 28 days, Z
  The value of shrinkage at 28 days is used for the design shrinkage value. As more water is added to a mix, the potential for shrinkage will increase and the strength will decrease. Since shrinkage can be considered inversely proportional to strength, Table 4.8.9 may be used as a guide in selecting a value corresponding to the indirect tensile strength determined.

Table 4.8.9 Approximate Relationship Between Shrinkage and Indirect Tensile Strength of Portland Cement Concrete

Indirect Tensile Strength (psi)	Shrinkage (in/in)
300 (or less)	0.0008
400	0.0006
500	0.00045
600	0.0003
700 (or greater)	0.0002

Source: AASHTO, Guide for Design of Pavement Structures 1993

- Concrete thermal coefficient, α_c

  Recommended values of PCC thermal coefficient (as a function of aggregate type) are presented in Table 4.8.10.

Table 4.8.10 Recommended Value of the Thermal Coefficient of PCC as a Function of Aggregate Types

Type of Coarse Aggregate	Concrete Thermal Coefficient ($10^{-6}/^{o}F$)
Quartz	6.6
Sandstone	6.5
Gravel	6.0
Granite	5.3
Basalt	4.8
Limestone	3.8

Source: AASHTO, Guide for Design of Pavement Structures 1993

- Reinforcing bar diameter, ϕ

  Typically, D 5/8in and D 6/8in deformed bar are used for longitudinal reinforcement.

- Steel thermal coefficient, α_s

  Unless specific knowledge of the thermal coefficient of the reinforcement steel is known, a value of 5.0×10^{-6} in./in./ $^{\circ}F$ may be assumed for design purposes.

- Design temperature drop, DT_D

  The temperature drop used in the reinforcement design is the difference between the average concrete curing temperature and a design minimum temperature. The design temperature drop which is entered in the longitudinal reinforcement design procedure is:

  $$DT_D = T_H - T_L$$

  Where

  DT_D: design temperature drop, $^{\circ}F$

  T_H: average daily high temperature during the month the pavement is constructed, $^{\circ}F$ and

  T_L: average daily low temperature during the coldest month of the year, $^{\circ}F$

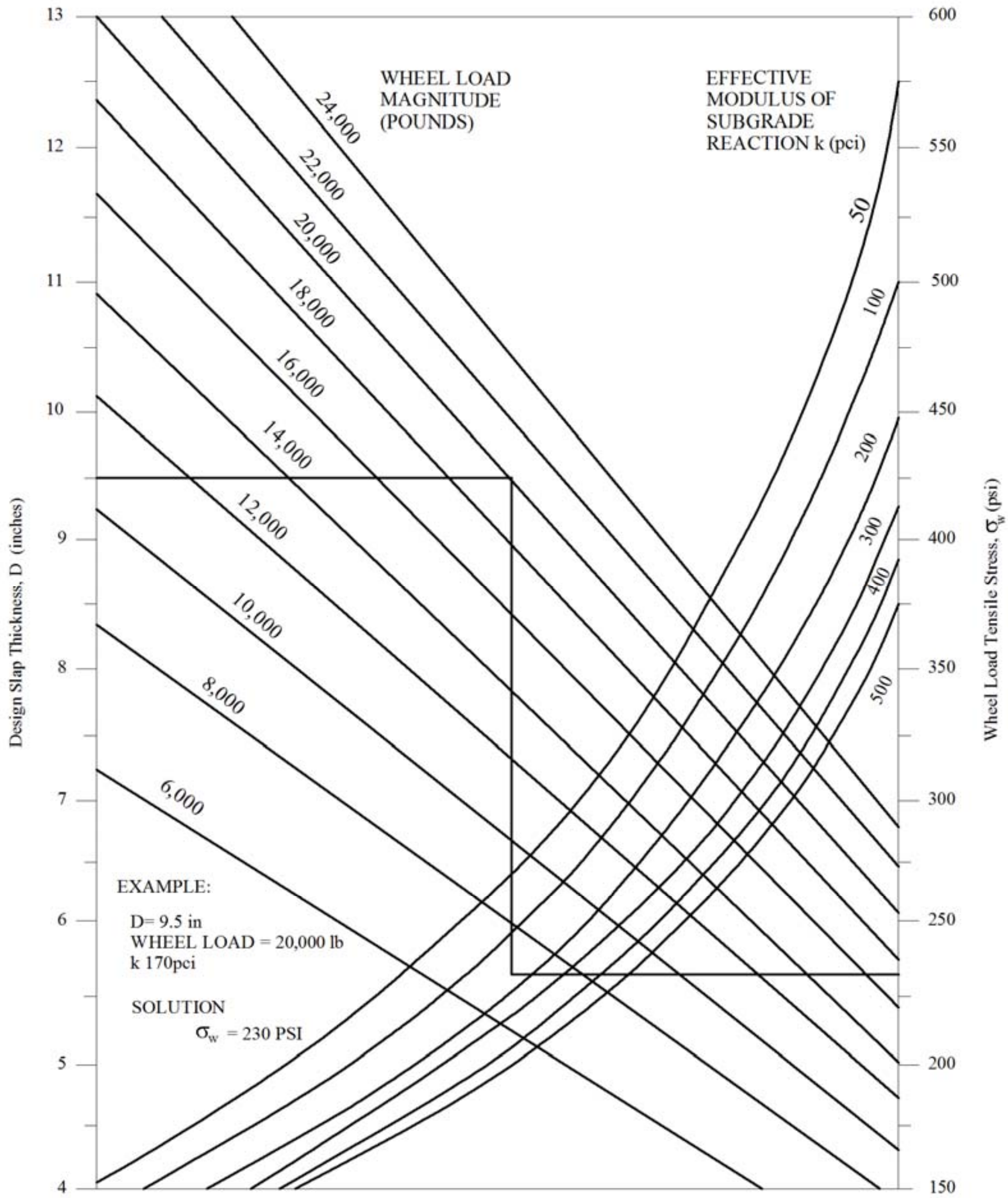
An additional input required by the procedure is the wheel load tensile stress developed during initial loading of the constructed pavement by either construction equipment or truck traffic. Figure 4.8.9 may be used estimate this wheel load stress based on the design slab thickness the magnitude of the wheel lead, and the effective modulus of subgrade reaction.

Table 4.8.11 Worksheet for Longitudinal Reinforcement Design

DESIGN INPUTS			
Input Variable	Value	Input Variable	Value
Reinforcement Bar/Wire Diameter, ϕ (in/in)		Thermal Coefficient Ratio, α_s/α_c (in/in)	
Concrete Shrinkage, Z (in/in)		Design Temperature Drop, DT_D (^oF)	
Concrete Tensile Strength, f_t (psi)		Wheel Load Stress, σ_w (psi)	

DESIGN CRITERIA AND REQUIRED STEEL PERCENTAGE				
Value of Limiting Criteria	Max. 8.0 Min. 3.5			
Minimum Required Steel Percentage				(P_{min})*
Maximum Allowable Steel Percentage				P_{max}

* Emter the largest percent across line.
** If Pmax, then reinforcement criteria are in conflict, design not feasible.
Source: AASHTO, Guide for Design of Pavement Structures 1993



Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.9 Chart for Estimating Wheel Load Tensile Stress

<u>Limiting Criteria</u>

In additional to the inputs required for the design of longitudinal reinforcing steel, there are three limiting criteria which must be considered: crack spacing, crack width, and steel stress.

The limits on crack spacing are derived from consideration of spalling and punch outs. To minimize the incidence of crack spalling, the maximum spacing between consecutive cracks should be no more than 8 feet. To minimize the potential for the development of punch outs, the minimum desirable crack spacing that should be used for design is 3.5 feet.

The limiting criterion on crack width is based on a consideration of spalling and water penetration. The allowable crack width should not exceed 0.04 inch.

Limiting criteria placed on steel stress are to guard against steel fracture and excessive permanent deformation. To guard against steel fracture, a limiting stress of 75 % of the ultimate tensile strength is set.

Value of allowable mean steel working stress for use in this design procedure are listed in Table 4.8.12 as a function of reinforcing bar size and concrete strength.

Table 4.8.12 Allowable Steel Working Stress, ksi

Indirect Tensile Strength of Concrete at 28 days, psi	Reinforcing Bar Size No.04 (1/2in.)	No.05 (5/8in.)	No.06 (3/4in.)
300 (or less)	65	57	54
400	67	60	55
500	67	61	56
600	67	63	58
700	67	65	59
800 (or greater)	67	67	60

*For DWF proportional adjustments may be made using the wire diameter to bar diameter.
Source: AASHTO, Guide for Design of Pavement Structures 1993

<u>Design Procedure</u>
The following procedure may be used to determine the amount of longitudinal reinforcement required:

Step1
Solve for the required amount of steel reinforcement to satisfy each limiting criterion using the design charts in Figure 4.8.10, Figure 4.8.11, and Figure 4.8.12.

Step 2
If P_{max} is greater than or equal to P_{min}, go to Step 3. If P_{max} is less than P_{min}, then

- Review the design inputs and decide which input to revise.
- Indicate the revised design inputs in the worksheet in Table 4.8.13. Make any corresponding change in the limiting criteria as influenced by the change in design parameter and record this in Table 4.8.13. Check to see if the revised inputs affect the subbase and slab thickness design. It may be necessary to reevaluate the subbase and slab thickness design.
- Rework the design nomographs and enter the resulting steel percentage in Table 4.8.13.
- If P_{max} is greater than or equal to P_{min}, go to Step 3. If P_{max} is less than P_{min}, repeat this step using the space provided in Table 4.8.13 for additional trials.

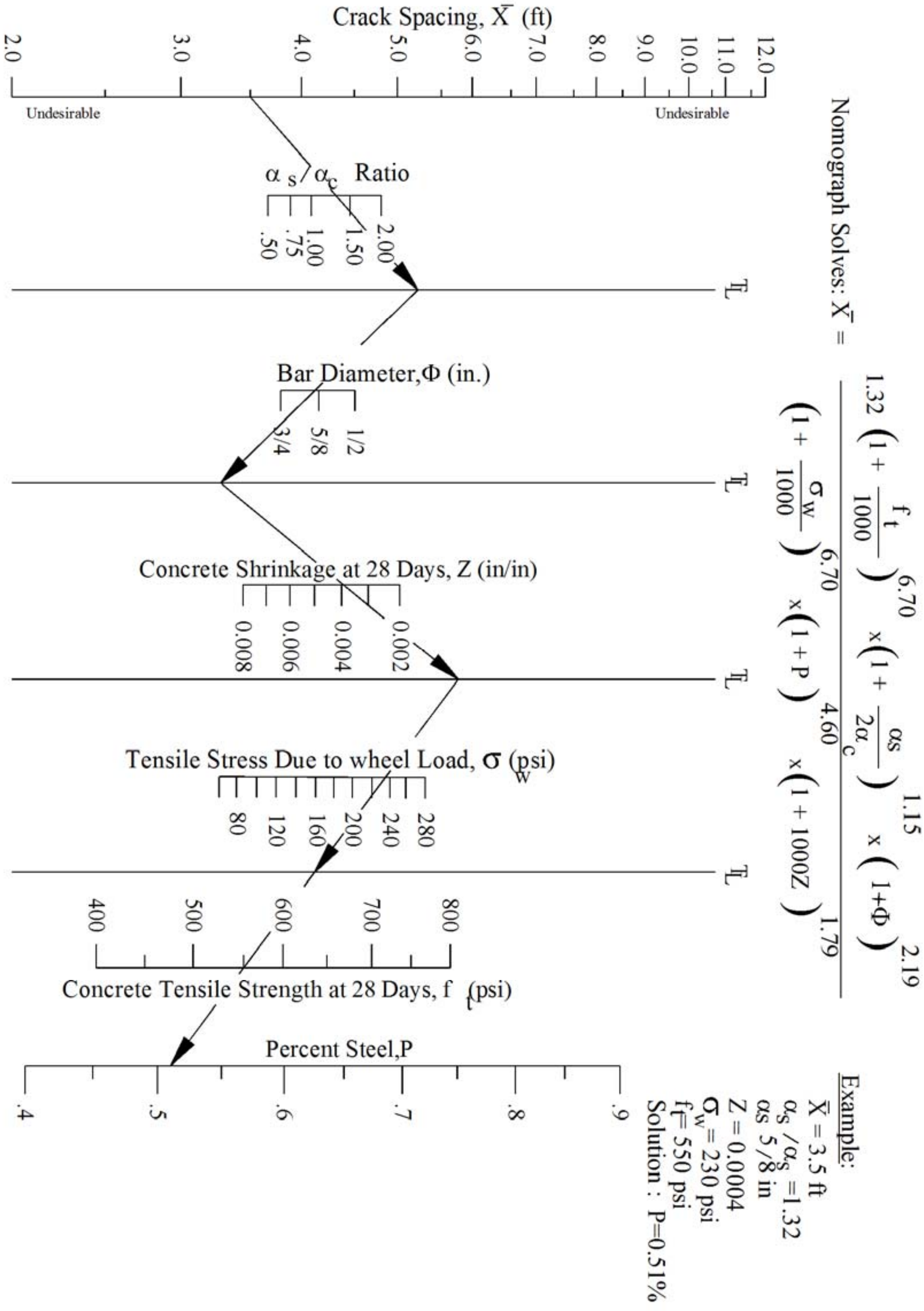
Step 3
Determine the range in the number of reinforcing bars:

$$N_{\min} = 0.01273 \times P_{\min} \times W_s \times D / \phi /^2, and$$
$$N_{\max} = 0.01273 \times P_{\max} \times W_s \times D / \phi /^2$$

where
N_{min} : minimum required number of reinforcing bar
N_{man} : maximum required number of reinforcing bar
P_{min} : minimum required percentage steel
P_{man} : maximum required percentage steel
W_s : total width of pavement section (inches)
D : thickness of concrete layer (inches)
ϕ : reinforcing bar (inches) which may be increased if loss of cross section is anticipated due to

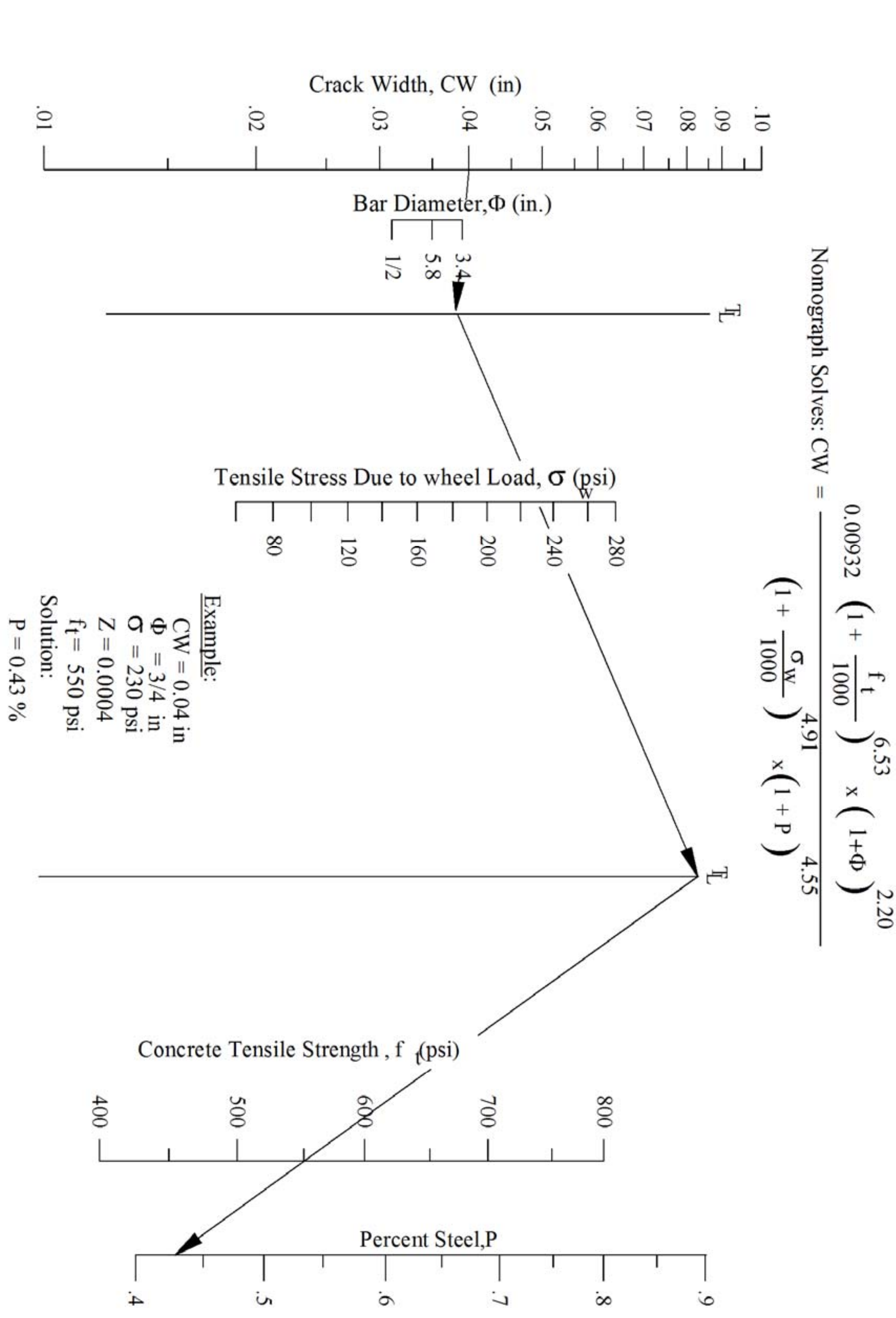
corrosion

Step 4
Determine the final steel design by selecting the total number of reinforcing bar in the final design section. N_{Design}, such that N_{Design} is a whole integer number between N_{min} and N_{max}. the appropriateness of these final design alternatives may be checked by converting the whole integer number of bar to percent steel and working backward through the design charts to estimate crack spacing, crack width, and steel stress.



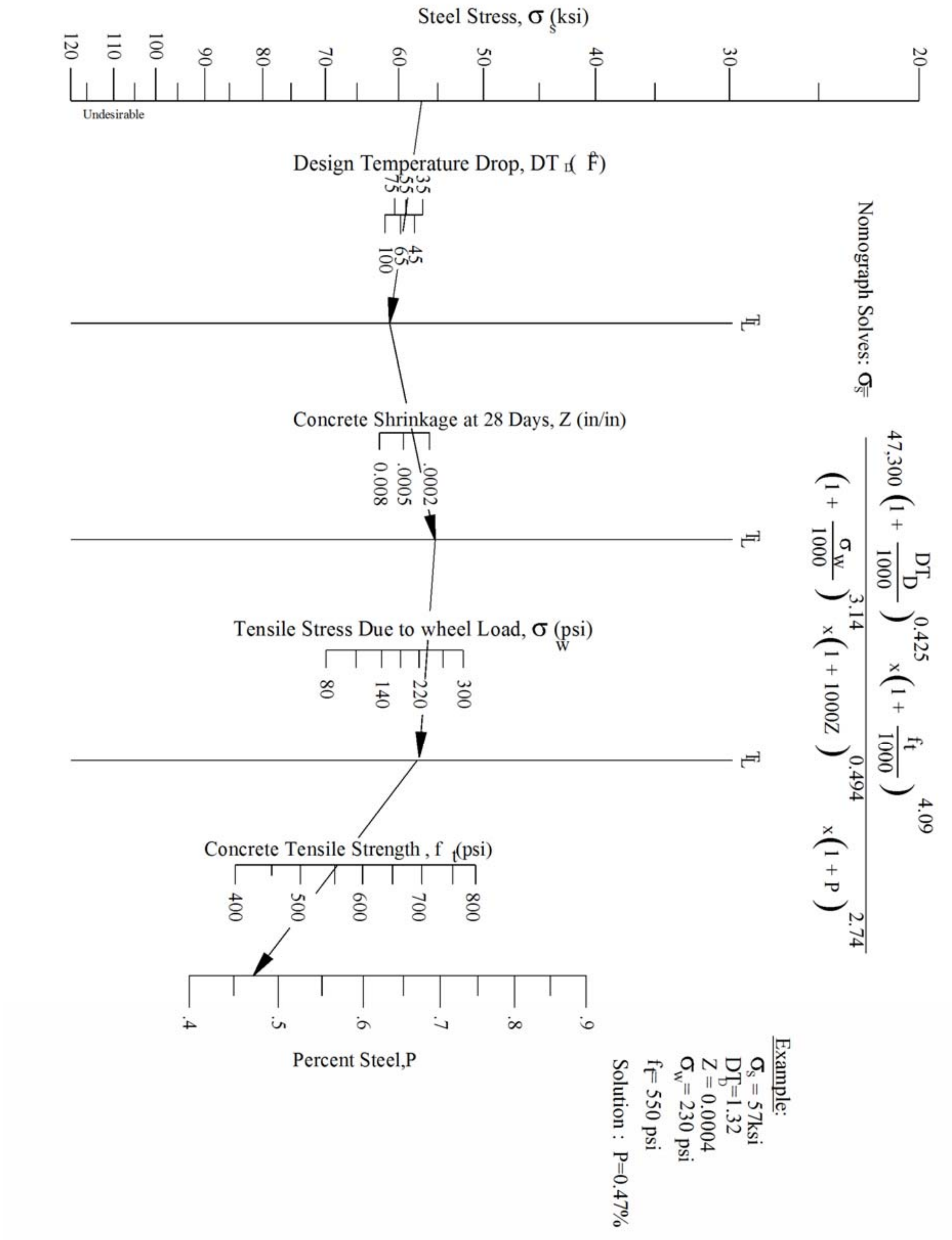
Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.10 Percent of Longitudinal Reinforcement to Satisfy Crack Spacing Criteria



Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.11 Minimum Percent of Longitudinal Reinforcement to Satisfy Crack Width Criteria



Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.12 Minimum Percent of Longitudinal Reinforcement to Satisfy Steel Criteria

Table 4.8.13 Example Application of Worksheet for Revised Longitudinal Reinforcement Design

Parameter		Change in Value from Previous Trial: Trial 2	Trial 3	Trial 4	Trial 5	Trial 6
[2]Reinforcing Bar/Wire Diameter, ϕ (inches)						
Concrete Shrinkage, Z (in./in.)						
[2]Concrete Tensile Strength, f_t (psi)						
Wheel Load Stress, σ_w (psi)						
[1]Design Temperature Drop, DT_D (^oF)						
Thermal Coefficient Ratio, α_s/α_c						
Allowable Crack Width Criterion, CW_{max} (inches)						
Allowable Steel Stress Criterion, $(\sigma s)_{max}$ (ksi)						
Required Steel % for Crack Spacing	Min.					
	Max.					
Minimum Required Steel % for Crack Width						
Minimum Required Steel % for Steel Stress						
Minimum % Reinforcement, P_{min}						
Minimum % Reinforcement, P_{max}						

[1] Change in this parameter will affect crack width criterion.
[2] Change in this parameter will affect steel stress criterion.
Source: AASHTO, Guide for Design of Pavement Structures 1993

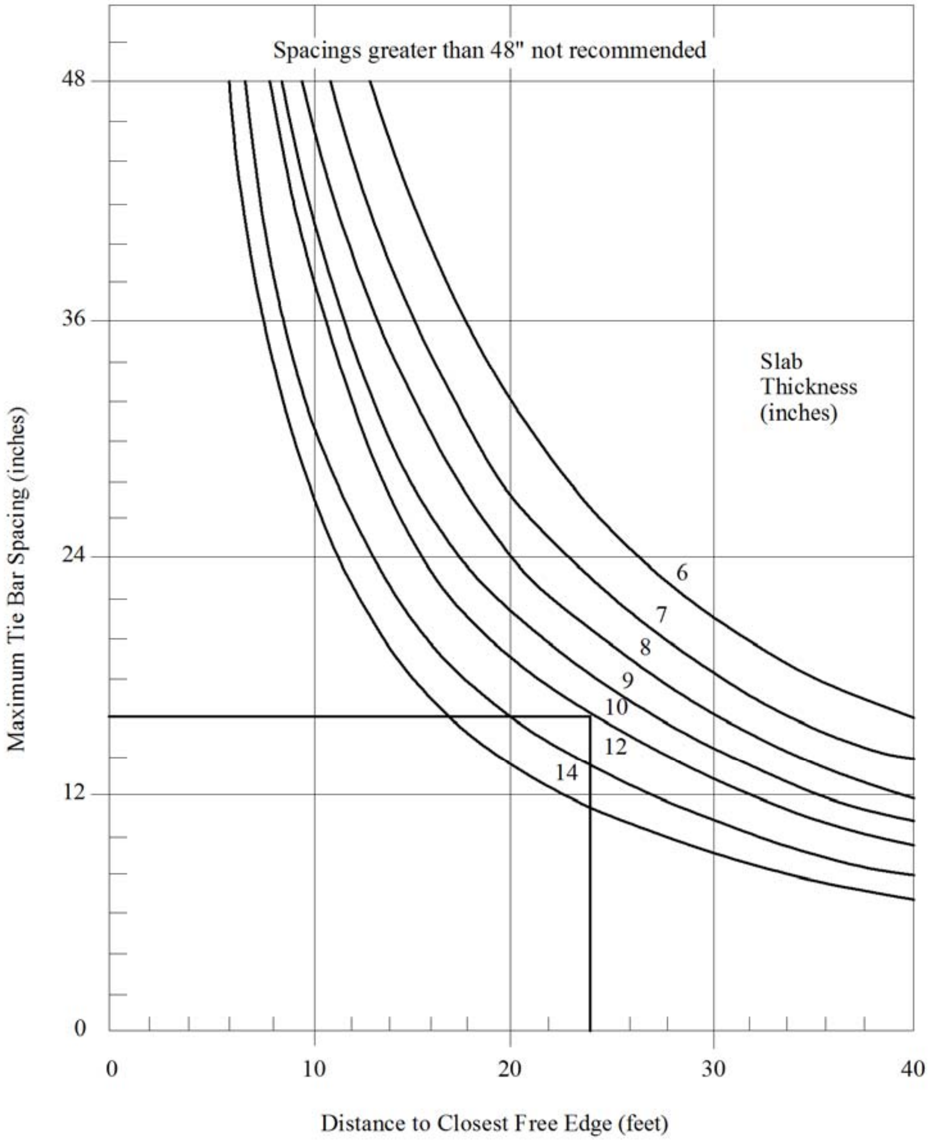
(5) Transverse Reinforcement

Transverse steel is included for conditions where soil volume change (due to changes in either temperature or moisture) can result in longitudinal cracking. For normal transverse reinforcement, figure may be used to determine the percent transverse steel. For normal transverse reinforcement, Figure 4.8.8 may be used to determine the percent transverse steel. The percent transverse steel may be converted to spacing between reinforcing bars as follows:

$$Y = \frac{A_s}{P_t S} \times 100$$

Where
Y : transverse steel spacing (inches)
A_s : cross sectional area of transverse reinforcing steel (in. ^{2})
P_t : percent transverse steel
D : slab thickness (inches)

Figure 4.8.13 and Figure 4.8.14 may be used to determine the tie bar spacing for 1/2 and 5/8 diameter deformed bars, respectively. These nomographs are based on Grade 40 steel and a subgrade friction factor of 1.5.

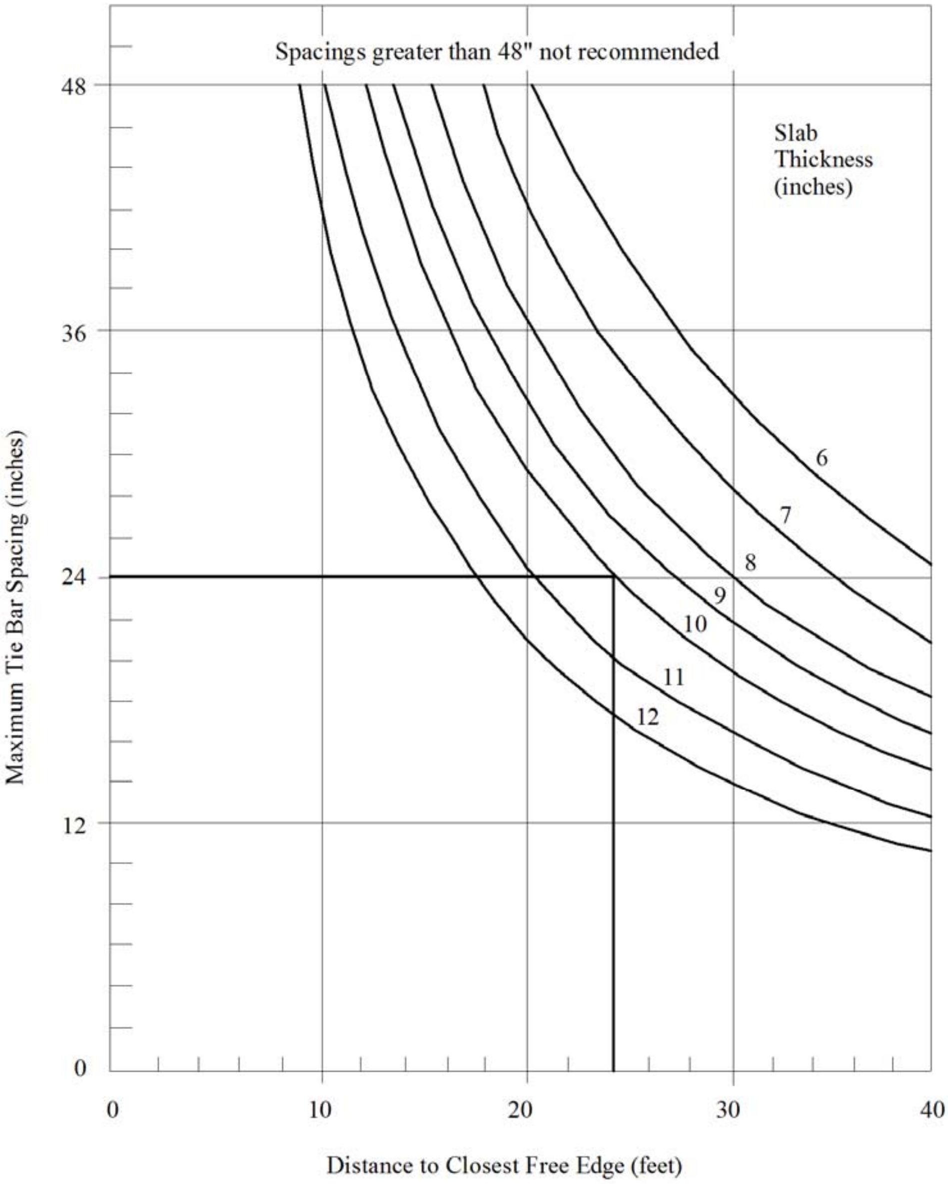


Example: Distance from free edge = 24 ft
D = 10 in.

Answer: Spacing = 16 in.

Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.13 Recommended Maximum Tie Bar Spacings for PCC Pavement Assuming 1/2-inch Diameter Tie Bars, Grade 40 Steel, and Subgrade Friction Factor of 1.5



Example: Distance from free edge = 24 ft
D = 10 in.

Answer: Spacing = 24 in.

Source: AASHTO, Guide for Design of Pavement Structures 1993

Figure 4.8.14 Recommended Maximum Tie Bar Spacings for PCC Pavement Assuming 5/8-inch Diameter Tie Bars, Grade 40 Steel, and Subgrade Friction Factor of 1.5

4.9 LOW STANDARD ROADS

4.9.1 General

Low standard roads will in certain cases be justified where traffic is very low.

This could for instance apply to roads in remote areas, probably not yet connected to other villages and to feeder roads in agricultural and forestry areas and other low trafficked special or strategic roads.

The designer shall justify economically the benefits of reducing the standards for such a road projects.

Recommendations are given in this chapter for the design and materials specification for such low standard for such low standard bitumen surfaced roads.

4.9.2 Low Standard Bases

(1) Thicknesses

It is normally neither practicable, nor economical, to reduce the base course thickness to and absolute minimum for low trafficked roads. Even for the lightest traffic, the minimum base should be 150 mm of natural gravel or 125 mm of treated material.

(2) Natural Gravels

(i) General

The normal bearing strength requirement is a CBR (after 4 days soak) at least equal to 70. For medium traffic Classes this requirement must be fulfilled.

However, for very light traffic, lower strength gravels may be used provided the pavement is well drained under all circumstances.

Two main reasons may govern the decision to use low standard gravel for the base construction of a bitumen road:

1) No gravel meeting the normal requirement of CBR at least equal to 70 is available.

2) In the region considered, gravels are so scarce that the periodic overlaying of gravel roads. Necessitated by the normal loss of material under traffic, will rapidly exhaust all gravel resources. Sealing gravel roads is then a means of preventing gravel loss. This type of construction is termed “sealed Gravel”.

In the first case, the decision to utilize low standard gravel is aimed at minimizing construction costs. In the second case, the economic purpose of sealing low standard gravel is mainly the reduction of maintenance costs.

In Laos, naturally gravels meeting the normal requirement of CBR over 70 are rare. It is therefore of prime importance to make the maximum use of all locally available gravels. This includes use of low standard lateritic gravel for base construction of very lightly trafficked roads.

(ii) Material requirements

The following relaxed specifications are recommended tor cumulative traffic of less than500, 000 standard axles during the selected design period, Provide that the pavement is well drained under all circumstances

- Maximum size : 10 to 40 mm
- Passing 0.075 : Maximum 35%
- Plastic Index : Maximum 20 in dry areas
  : Maximum 15 in wet areas

CBR (4 days soak) : Maximum 50

Such relaxed specifications should be applied only when it is uneconomic to use materials complying with the normal requirements. In particular, the possibility of improving low standard gravels with cement or lime should always be considered

Good payment drainage requires properly designed and maintained side ditches, properly shaped and maintained shoulders and impervious surfacing. The imperviousness of the surfacing is essential. Whereas low standard surfacing may be used in dry areas, double surface dressings or slurry seals are required in wet areas.

The passage of a significant number of overloaded axles is likely to cause rapid deformation of low standard bases. The decision to use low standard gravel should be taken only after a traffic survey has proved that the road will not carry significant numbers of heavy trucks.

*NOTE:

In some cases, the use of low standard gravel can be considered as the first step of a stage construction. When the pavement becomes inadequate, it may be overlaid with a base of normal standard material.

4.9.3 Low Standard Sub Bases

(1) Thicknesses

The absolute minimum safe thickness required for subbase is 100 mm

(2) Natural Materials

As for road base, substantial savings may be obtained by utilizing al locally available materials, including those of a low standard for very lightly trafficked roads.

The following relaxed specifications are suitable for cumulative traffic of less than about 500,000 standard axles during the selected design period.

- Maximum size : 60 mm
- passing 2 mm sieve : Maximum 95%
- Passing 0.075 mm sieve : Maximum 40%
- Plasticity : Maximum 25%
- CBR (4 days soak) : Minimum 15 (preferably 20)

Materials having a CBR of 7 – 15, should be considered as improved subgrade.

(3) Cement or Lime Treated Soils

Clayey and silty soils may satisfactorily be used after treatment research with lime to form the sub base of very lightly trafficked road, but further research is still required to demine the appropriate specifications.

4.9.4 Low Standard Surfacing

(1) Single Surface Dressing

Single surface dressing is suitable for very low traffic roads. The use of single surface dressing is not recommended for wet areas.

The material requirements and design are same as standard technical specifications.

(2) Sand Seal

A sand seal is an application of bituminous covered with sand or fine aggregate

(i) The sand or fine aggregate shall be free from organic matter, clay and other deleterious material and shall have a grading curve within the following envelope.

Sieve size (mm)	% by weight passing
6.3	100
5	95 – 100
4	90 – 100
2	50 – 100
1	20 – 80
0.6	10 – 25
0.425	3 – 25
0.3	0 -15
0.150	0 – 8
0.075	0 - 5

The fines (passing 0.425 mm) shall be non-plastic. The Sand Equivalent shall be greater than 40.

(ii) Suitable binders are cationic emulsion KI-60 and medium-curing cut-backs MC 800, MC 3000 or 800/1400. Natural sands are generally siliceous. It is then necessary to employ either cationic emulsion or an adhesion agent with cut-back bitumen.

(iii) The amount of residual bitumen required is normally between 0.9 and 1.2litre/m2. An excessive amount of sand is deliberately spread (6-7 liter/m^2). The sand particles penetrate into the binder, forming a sort of mix. It is a good practice to broom the whipped-off sand several times back onto the road, until it stays. Usually, 4 to 5 liter/m^2 of sand can be applied in a single layer.

(iv) Sand seals provide a cheap surfacing for low traffic roads. It should be especially considered in regions where stone suitable for surface dressing is not available and sand is abundant.

(v) To ensure imperviousness, it is recommended to apply double sand seal. The service life of a double sand seal should be at least 5 years, under traffic not exceeding about 200vehicles/day.

<u>* NOTE:</u>

A sand seal can be used in combination with surface dressing. In particular, an application of binder and sand over a chipping seal greatly contributes to holding the chippings and water proofing the surface.

(3) Gravel Seal

A gravel seal is an application of bituminous binder covered with graded granular material.

(i) The cover aggregate shall consist of natural gravel or graded crushed stone or a mixture of these (natural gravel being, by far, the cheapest).

It shall be free from organic matter and other deleterious material and the grading curve shall be within the following envelope:

Sieve size (mm)	% by weight passing
20	100
14	65 – 100
10	45 – 95
6.3	25 – 80
4	10 – 65
2	15 – 50
1	5 – 40
0.425	3 – 30
0.075	0 - 10

The Los Angeles Abrasion shall be less than 40% and the Aggregate Crushing Value less than 30%.

It is desirable that the gravel be non-plastic. Nevertheless, if it is impossible to economically obtain non-plastic gravel, plasticity indexes up to about 10 may be accepted.

The gravel is used at its natural moisture content, provided this does not exceed about 5%.

(ii) The most suitable binders are medium-curing cut-backs MC 800 or MC 3000.
An adhesion agent shall always be added to the bituminous binder

(iii) The Appropriate rate of spray of bituminous binder is usually between 1.5 and 2.0 litre/m^2 of residual bitumen.

(iv) Rolling is essential. The most suitable machines are pneumatic-tyred rollers, but vibratory rollers can also be employed provided carefully used Rolling and, subsequently, passing traffic cause the upward displacement of the binder, hence forming a bituminous mix, which is further compacted by the traffic.
Depending on the amount of traffic, it takes week or even months until the final state of a dence and stable mix is obtained.

An adhesion agent is necessary to allow the gravel to be used at its natural moisture content and to prevent the surfacing from being adversely affected by rain, especially during the initial “pumping” and compacting phase.

(v) Gravel seals provide a cheap alternative for surfacing low traffic roads, especially in the regions where no stone suitable for surface dressing can be found.

Depending on the traffic, single or double gravel seals will be required. The service life of a properly constructed double gravel seal should be at least 10 years.

(4) Emulsion Slurry Seal

The material requirements and construction procedures are summarized in follows.

(i) Type of emulsion
Anionic emulsion A4 (slow setting or rapid setting), or Cationic emulsion K3

(ii) Aggregate

- Aggregate shall be free of organic and deleterious matter
- Sand Equivalent: Min. 40
- Percentage of crushed dust:
  Slurry Class A Min. 50%
  Slurry Class B Min. 25%

Aggregate Grading			
Sieve other size (mm)	% by weight passing		
	Type I (Fine)	Typr II (Normal)	Type III (Coarse)
10	-	-	100
6.3	-	100	80-90
5	-	90-100	70-90
2	100	60-87	40-65
1	60-85	40-67	25-45
0.425	30-48	22-38	15-29
0.300	25-42	18-30	12-25
0.150	15-30	10-20	7-18
0.075	10-20	5-15	5-15

(iii) Emulsion slurry seal can be used as a surfacing for new roads, directly on a primed based. Type II slurry should normally be used. It is recommended that two layers of equal thickness be applied (Total: 125 to 180 m2/m3 or 6-8 litre/m2)

(iv) Emulsion slurry seal can also be used as a final seal on surface dressing. The slurry seal greatly contributes to holding the chippings and waterproofing the surface.

(v) The slurry coverage is a function of the chippings size and shape. It varies between 125 and 250 m2/m3.

(vi) If siliceous sand is to be used, it is necessary to employ either cationic emulsion or an adhesion agent with anionic emulsion.

(vii) The use of emulsion slurry seal may be envisaged wherever stone for chippings is scarce or chippings are defective.

(viii) Construction Procedures
MIXING: by concrete mixer or, preferably, slurry machine
LAYING: by slurry machine.
CURING: by traffic until cured to a firm condition (no pick-up by tyres)
ROLLING: if required, by pneumatic tyred roller.

(5) Mixed-in-Place Road Oiling Treatment

Road oiling is the application of slow curing cut-back SC 70 or SC250.The light oils penetrate into the base, ensures cohesion and prevents moisture absorption. Usual application rate range is from 3 to 4.5 l/m^2. The process is applicable to sufficiently porous materials.

The same effects can be obtained by mixing in place the upper part of the road base with a bituminous binder. The type of binder depends on the material to be treated: anionic emulsions A2 or A3 and medium-curing cut-backs MC 250 or MC 800 are the most common. In practice, materials with more than 30% fines (passing 0.075) mm) or a plasticity index exceeding 15 are not suitable to such treatment. A maximum thickness of about 50 mm compacted materials is recommended.

4.10 GRAVEL ROADS

4.10.1 Design of Gravel Roads

(1) Design Method

Gravel roads cannot be designed in the same way as paved roads, since there is no proper failure criterion for a gravel wearing course and the damaging effects of different axle loads on a gravel road are not well known.

The required gravel thickness may as a guide, be determined as follows:

1) Determine of the minimum initial thickness necessary to avoid excessive compressive strain in the subgrade (D_1).

2) Determine the extra wearing course thickness needed to compensate for the gravel loss under traffic during the period between re-gravelling operations (D_2)

3) Determine the total gravel thickness required by adding the above two thicknesses ($D_1 + D_2 = D$).

(2) Minimum Thickness Required

It is necessary to limit the compressive strain in the subgrade to prevent excessive permanent deformation at the surface of the road. Table 4.10.1 indicates the minimum gravel thickness required for each class of subgrade bearing strength and for each range of initial commercial traffic.

Table 4.10.1 Minimum Initial Gravel TD_1 (mm)

Subgrade Strength CBR	Initial daily number of commercial vehicles (both directions)			
	<15	15 - 50	50 – 150	150 - 500
3 – 5	350	425	500	575
5 – 10	225	275	3250	375
7 – 13	175	225	250	175
10 - 18	150	175	200	225
15 – 30	125	150	175	200

The following points should be noted:

(i) The subgrade bearing strength shall be determined as indicated in section 4.6.2. This includes the use of improved subgrade which is likely to prove advantageous on poor quality soils and heavy traffic.

(ii) No provision has been made for traffic over 500 commercial vehicles/day, because it is thought that such levels of traffic justify the construction of paved roads.

(3) Gravel Loss

According to TRRL Laboratory Report 1111, an estimate of the annual gravel loss is given by the following equation:

$$GL_A = f\left(\frac{T_A{}^2}{T_A{}^2 + 50}\right) \times (4.2 + 0.092T_A + 3.5R_L{}^2 + 1.88VC)$$

Where: GL_A is the annual gravel loss measured in mm

T_A is the total traffic volume in the first year in both directions, measured in thousands of vehicles

R_L is the average annual rainfall measured in m

VC is the total (rise + fall) as a percentage of the length of the road

F = 0.9 for lateritic gravels

= 1.1 for quartzitic gravels

= 0.7 for volcanic gravels (weathered lava of tuff)

= 1.5 for coral gravels

(4) Total Thickness Required

The wearing course of a new gravel road shall thickness D calculated from:

$D = D_1 + D_2\ (=N*GL_A)$

Where: D_1 is the minimum thickness given in Table 4.10.1

N is the period between re-gravelling operations (in Years)

GL_A is the annual gravel loss.

The minimum Gravel Wearing Course thickness D_2, shall in all cases not be less than 125 mm, preferably 150 mm.

It should be noted that the D_1 layer thickness, is a more a pavement structural layer and that layer D_2 is the Gravel Wearing Course complying with Chart GWC.

The D_1 material can be made up of subgrade and sub base materials appropriate for the traffic loading and the design period selected.

The re-gravelling operations should be programmed to ensure that the actual gravel thick for the Gravel Wearing Course (D_2), is maintained and that the total thickness CD), never falls below the minimum thickness D_1.

(5) Shoulders and Cross-section

It is recommended that the shoulders be made from the same material as the gravel wearing course.

As cross fall of 4%-6% shall normally be given to gravel wearing courses and shoulders.

4.11 MAINTENANCE AND REHABILITATION

4.11.1 General

The selection of an appropriate maintenance treatment or rehabilitation strategy is based on a number of considerations. Firstly, the cause of deterioration in the existing pavement must be correctly identified and its importance assessed. For example, the deterioration may result from some deep seated structural insufficiency or construction defect. In such cases consideration must be given to full or partial reconstruction of the pavement to correct the situation. Secondly, attention should be given to the nature, extent and severity of the deterioration to check what effect it will have on the treatments that are being considered. For example, thin asphalt surfacing on their own will not provide a satisfactory repair where reflection cracking is likely, nor will any form of thin surfacing provide a significant improvement to riding quality where this is poor. Finally, the strategy must be economically viable taking into consideration both the costs of maintenance and the vehicle operating costs over a number of years.

It should not be assumed that when a road is in poor condition it inevitably needs strengthening. When traffic is low, for instance, the existing road structure is often thick enough to prevent long term rutting. In this case the maintenance treatment selected should address the cause, or causes, of the deterioration without necessarily adding strength to the pavement. It is important, therefore, to check the ability of the existing road pavement to carry the predicted traffic loading using at least two of the methods described below. Where either of the methods are shown to accurately predict the present performance of the road under study then the method is equally applicable for the design of strengthening works in the event that the road is shown to be too weak to carry the future traffic.

4.11.2 Analytical approach

The traffic carrying capacity of an asphalt pavement is governed by how effective the pavement layers are in preventing;

- fatigue cracking of the asphalt surfacing;
- shear failure of the granular materials;
- fatigue cracking or crushing of lightly cemented materials; and
- wheel path rutting resulting from subgrade failure.

Theoretical models to predict the behavior of granular and lightly cemented materials under the action of traffic are not well defined and therefore specifications for such layers have always been set in such a way that failures are unlikely. This has mitigated against the use of lower quality materials and has theoretically restricted the range of likely failure modes. The performance of road pavements has traditionally been dependent on the stress/strain values at two locations in the structure. The horizontal tensile strain at the bottom of the asphalt layer controls one type of fatigue cracking and the vertical compressive strain at the top of the subgrade controls rutting.

For roads having a thin bituminous seal the traffic carrying capacity is determined only by resistance to rutting. The performance of the surface seal will generally depend on environmental effects rather than traffic loads. The traffic carrying capacity of an asphalt surfaced road will be determined by both its resistance to fatigue cracking and wheel path rutting. However, research has shown that the predominant form of surface distress of asphalt surfacing in tropical climates is not fatigue cracking starting at the bottom of the asphalt layer but `top-down' cracking which is initiated at the surface of the layer (Rolt et al, 1986) (Smith et al, 1990). The type and severity of this form of cracking is a complex function of material properties and both environmental and traffic stresses and its development has yet to be successfully described by means of a practical analytical model. `Top-down' cracks often develop long before other types of cracks and thus the performance of asphalt surfaced roads rarely agrees with the analytical models. Nevertheless rehabilitation design should take account of all possible modes of future failure and therefore it is important to ensure that traditional fatigue failure of the surfacing and failure through inadequate protection of the subgrade do not occur within the design life required. In order to do this, analytical procedures properly calibrated to local conditions provide a suitable method. The analytical approach uses a mathematical model to describe the pavement. Almost all methods make use of the multilayer linear elastic model, although more sophisticated models can also be used. This model requires, as input, the thickness, elastic modulus and Poisson's ratio of each layer of the pavement. Very thin layers such as an existing seal are normally incorporated with the underlying road-base or ignored. Asphalt surfacings are usually assigned moduli based on mix constituents and binder properties at the design temperature although direct laboratory measurements of modulus can also be made on samples of material extracted from the road. Other moduli values can be either calculated from the back-analysis of Falling Weight Deflectometer (FWD) deflection bowls or assigned values following DCP testing and/or the laboratory testing of samples taken from trial pits. Stresses or strains at the critical points in the pavement are then calculated under the application of a standard load designed to replicate a 40kN wheel load (80kN axle load). These strains are then used to calculate the `life' of the structure using relationships (Powell et al, 1984)(Shell, 1985) between stress/strain and pavement life of the form:

Asphalt fatigue criteria

$$LogN_f = a + bLog\varepsilon_r$$

Where,

N_f = Fatigue life in esa

ε_r = Horizontal tensile strain at the bottom of the asphalt layer

a and b = constants associated with material properties

Subgrade deformation criteria

$$LogN_d = a + bLog\varepsilon_z$$

Where

Nd = Deformation life in esa

ε_z = Vertical compressive strain at the top of the subgrade

a and b = constants associated with material properties

Where the forms of these relationships are shown to predict the present performance of the road pavement, they can be used with more confidence to estimate the future traffic carrying capacity. After adjustment of the pavement model they can then also be used to determine overlay thickness, where necessary.

4.11.3 Structural approach

In this method the traffic carrying capacity of the road is estimated by comparing the existing pavement structure and its condition with established design charts. The thickness of the various pavement layers should first be established using the DCP and trial pits, and the in situ strength of the pavement layers and the subgrade determined by a combination of deflection and DCP data. These tests should be carried out shortly after the wettest period of the year, when the pavement can be expected to be in its weakest condition. If this is not possible, adjustments will need to be made to the deflection data and material properties to reflect the season during which the data were collected. The in situ strengths of the pavement layers obtained in this way, in particular the upper granular layers, should always be verified by laboratory tests to ensure they conform to normally accepted specifications. The effective structural number of the pavement can then be obtained by using techniques described in the AASHTO Guide for Design of Pavement Structures (AASHTO, 1993).

The required strengthening measures are then established by comparing the effective structural number of the pavement with the required structural number of a pavement for the future traffic, obtained from an appropriate design method, at a representative value of in situ subgrade strength. If the AASHTO guide is used then a mean value of the resilient modulus of the subgrade, suitably corrected (AASHTO, 1993), is used. If Road Note 31 is preferred, then the lower 10 percentile of the in situ subgrade CBR should be used, measured when the pavement is in its weakest condition. Where the comparison of the effective structural number, past traffic and design recommendations is shown to be consistent with the present condition of the road pavement, then the engineer can be more confident in designing the thickness of any necessary strengthening overlay by this method.

There are presently a number of methods of determining the structural number of a road pavement directly from FWD deflection bowl characteristics (AASHTO, 1993) (Jameson, 1992) (Rohde, 1994) (Roberts and Martin, 1996). With the development and refinement of these procedures it is likely that the rehabilitation of road pavements using the structural number approach will become increasingly popular.

4.11.4 Deflection approach

This method is to calculate overlay thickness based on result of deflection survey of pavement surface by Benkelman Beam. Measurement interval of deflection survey point is recommended 10m.

$$D = (d+2\sqrt{V}) \times f$$

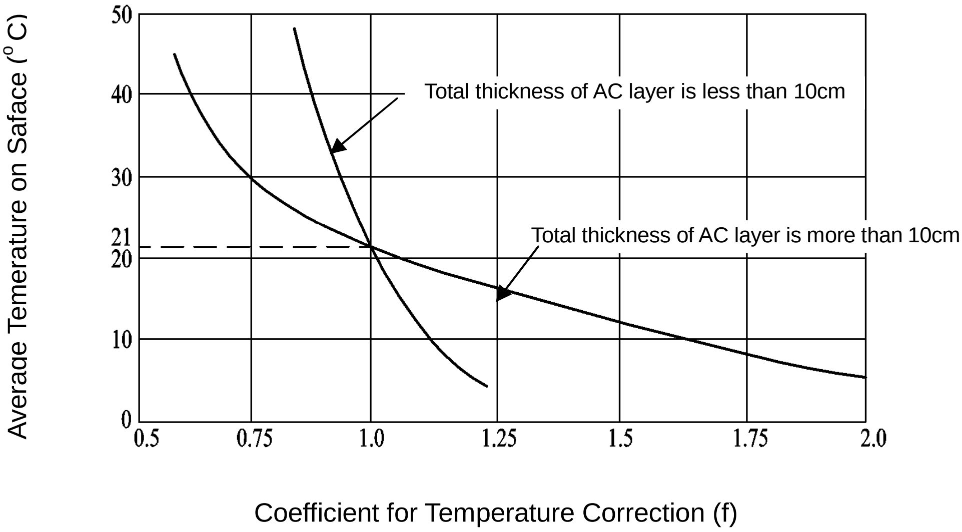
Where,

D: deflection on existing surface (mm)

d: average mesurement value (Wheel load 5 t)

$\sqrt{V}$: square root of unbiased variance of measurement value

f: coefficient for temperature on surface



Source: Japan Road Association, Road Maintenance Manual

Figure 4.11.1 Coefficient for Temperature Correction

Deflection on Existing Surface (mm)	Category of Large Vehicle Traffic Volume (vehicle / day-direction)				
	Class L < 100	Class A 100≤ <250	Class B 250≤ <1,000	Class C 1,000≤ <3,000	Class D 3,000 ≤
< 0.6	-	-	-	4	4
≤ 0.6	-	-	4	6	8
≤ 1.0	-	4	6	10	12
≤ 1.5	4	6	10	12	15
≤ 2.0	6	10	12	15	-

Source: Japan Road Association, Road Maintenance Manual

Figure 4.11.2 Thickness of Overlay based on Deflection (Unit: cm)

4.12 MATERIALS SAMPLING AND TESTING PROGRAMMES

4.12.1 General

There are usually three stages involved before a road is ready for construction. The three stages are normally as follows;

1) Pre-Feasibility Study
2) Feasibility Study
3) Detailed Design

This chapter describes the proposed materials sampling and testing programmers applicable to the Feasibility study stage and the detailed design stage. The pre-feasibility stage is a "go-between" stage often used by the clients' in-house capacity as a planning work waiting implementation funding.

Essentially, the only difference from the feasibility study stage is the amount of work being done, that is, at the feasibility study stage, approximately half the frequency of sampling and testing of the detailed design stage, is normally done.

4.12.2 Feasibility Study Stage

The purpose of feasibility study is to assess the effects of a particular project, to evaluate its economic return and to examine the conditions for financing it.

Material and pavement design for the feasibility study basically consist of:

(i) Defining the nature and the main characteristics of the anticipated alignment (soils & materials).

(ii) Assessing and describing the general nature of the materials to be cut and filled and describing problems likely to arise during the earthworks (e.g. rock cuts, moisture content, drainage, erosion; stability of slopes, etc.)

(iii) Preparing and inventory based on mapping and visual inspection of road-making material available in the vicinity of the project and determining their general engineering characteristics.

(iv) Describe the foundation conditions for the main structures, and

(v) Propose an approximate road design.

The amount and type of investigations and tests required will depend largely on the peculiarities of each project and on the quantity of data already available (such as geological maps, soils maps, reports on road projects in the area considered. etc.).

4.12.3 Detailed Design Stage

(1) Earthworks and Subgrade

(i) <u>Sampling</u>

At least one sample shall be taken per 500 meters along the length of the proposed alignment with more frequent samples where necessary to record changes of soil type or to provide an adequate assessment of the sub grade strength. A good knowledge of the materials to be cut is also essential.

For these purposes, holes shall be excavated mostly in proposed cut areas, down to at least 0.5 m below the anticipated formation level, unless lock is encountered, the position of each test hole shall be accurately determined and reported. In hilly or mountainous terrain, deep holes will be required to accurately determine the mater to be cut. It is sometimes impossible to dig trail pits to the depth of the anticipated formation level. It is then recommended to use a hand or pow auger to drill holes to the depth required.

In every hole, all layers, including top soil, shall be accurately described and their thicknesses measured and recorded. All layers of more than 300 mm (except top soil) shall be sampled. In every hole in cuts, one sample shall be taken at the approximate level of the formation. The other samples shall be representative either of the anticipated fill materials or of the anticipated subgrade in fills.

The sample shall be taken over the full depth of the layer by taking a vertical slice of material. The log of each test hole shall e accurately drawn and included in the Material Report.

To assess the quantities of the various earthwork categories (i.e. rock, rip rap or normal material), it will in some cases be necessary to drill boreholes. This type of investigation may advantageously be supplemented by a seismic survey.

(ii) <u>Testing of soils on new alignments</u>

<u>Basic testing</u>

Sufficient material shall be obtained of each sample to carry out the following tests:

a) Grading to 0.075 mm sieve

b) Atterberg Limits

c) Compaction test (Modified Proctor (AASHTO T 180)

d) CBR and swell on samples molded at 100% M.D.D (Standard compaction) and O.M.C (Standard compaction).

Note: CBR's shall normally be measured after 4 days soak, except in arid areas(annual rainfall less than 500 mm) where they may be measure at OMC or after a reduced soaking period, depending on the equilibrium moisture content predicted under the pavement in the area. The moisture contents after soaking shall be measured.

<u>Classification of the subgrade soils and testing of samples representative of each soil category</u>

The results from the above basic testing, combined with the relevant field observations, will enable a classification of the subgrade soils to be made. A category of soil should include the soils of the same type having fairly consistent geotechnical characteristics (Grading, Atterberg Limits, Compaction and CBR).

Usually, the number of soil categories will not exceed 4 or 5 for a given road project.

For each soil category, one representative large sample shall then be taken

Such large specimens shall be obtained by re-sampling from appropriate test pits.

Each large sample shall be submitted to the following tests:

a) Grading to 0.075 mm sieve.

b) Atterberg Limits

c) Compaction test (Heavy Compaction: 4.5 kg rammer)

d) " 6 points "CBR test as summarized below:

For a “6 points” CBR test the material shall be compacted using 3 compactive efforts for example 10 blow, 30 blows and 65 blows per layer and 4 days soaked CBR be measured as per specified percentage of compaction (example: CBR at 90%, 95% or 100% of MDD) from the relation of 4 days soaked CBR (AASHTO T 193) versus Dry Density (AASHTO T 180) Graph.

This method enables an estimate to be of the subgrade CBR at different densities and thus assists in determining the relative compaction to be specified. It also indicates the loss of strength which soaking may cause.

Treatment tests (when appropriate)

If treatment of some of the alignment materials is contemplated, for use either as improved subgrade or as subbase, the treatment test shall be carried out, as indicated in section 4.12.3(3) on the above large samples typical of each relevant soil category.

(iii) Testing of subgrade on existing gravel road alignments

Basic testing

See Section 4.12.3(1)

Classification of subgrade soils and testing of samples representative of each soil category

See Section 4.12.3(1)

(2) Existing Gravel Wearing (where appropriate)

No further sampling or testing is required at this stage. Indeed, existing gravel wearing course are subject to changes both in quantity and quality, under the action of traffic and weather. They should be considered as possible extra sources of material, to be re-evaluated at the construction stage.

(3) Soil and Gravel Borrow Pits

(i) General

Information obtained at the preliminary design stage will enable a selection of the most suitable borrow areas to.be made. Consideration shall be given to the following factors:

- quality of the materials
- location of the propose borrow areas, so as to minimize haul and obtain the most economic use of materials
- ease of working (land acquisition, clearance of the site, access, overburden, thickness of exploitable horizon, etc.)

(ii) Field investigations and sampling procedures

Pits shall be dug at every point on a 30 m grid, through the full depth of the layer(s) proposed for use.

The position of each proposed borrow pit shall be indicated on a key plan. A site plan of each propose borrow pit shall be prepared, showing the position of each trial pit the characteristic features of the site and the mean of access and location.

In every trial pit, all, layers, including top soil and overburden, shall be accurately described and their thicknesses measured and recorded. All layers proposed for use shall be sampled.

The sample shall be taken over the full depth of the layer proposed for use by taking a vertical slice of material.

The log of each trial pit hall be accurately drawn and included in the "Materials Report”.

(iii) Frequency of sampling and testing

Samples for identification tests

Sampling shall be carried out so as to obtain at least one sample per 1,000 m3 of material proposed for use. At least one sample shall be taken from each positive trial pit, even if the volume represented is small.

Each sample shall be submitted to the following tests:

a) Grading to 0.075 mm sieve
b) Atterberg Limits

Large samples for Compaction and CBR tests:

Large samples for compaction and CBR tests shall be obtained either by the Mix-method or by the Re-sampling method. As described in Section 4.12.3(3).

At least one large sample, whether mixed or re-sampled, is required per 5,000 m3 of material proposed for use.

Each large sample shall be submitted to the following tests:

a) Grading to 0.075 mm sieve
b) Atterberg Limits
c) Compaction test(Heavy Compaction: 4.5 kg rammer)
d) CBR and swell at 4 days soak, on specimens molded at OMC (Heavy Compaction) at 3 levels of compaction, normally around 90, 95 and 100% MDD (Heavy Compaction). The moisture content after soaking shall be measured.

Note: For the types of gravel susceptible to crushing during compaction, the grading of the specimen compacted closest to 95 % MDD (Heavy Compaction) shall be determined after compaction and CBR testing and compared with the grading before compaction of the specimen prepared for CBR.

In addition to the foregoing tests, the Los Angeles Abrasion and the Aggregate Crushing

Value of the coarse particles shall be determined for at least one typical sample from each site of gravelly material.

Treatment tests (when appropriate)

Information obtained at the Preliminary Design stage combined with the results of the above tests, will enable the design Engineer to decide which borrow pit materials require treatment and the nature of that treatment (i.e. type of additive and approximate percentage needed, method of mixing, etc.)

Treatment tests shall then be carried out on the relevant large samples (as defined above)

First, if it is suspected that the chemical composition of the material may give carried rise to detrimental reactions, the following chemical tests shall be carried out:

e) Organic matter content

f) pH value

g) Sulphate content

Then, if the material appears to lend itself to treatment, the representative large sample shall be mixed with the additive chosen. 3 amounts of additive shall be selected so as to give a representative picture of the treated material's characteristics. The following tests shall be carried out:

h) Compaction test (Heavy Compaction) on the large sample mixed with the amount of additive expected to be appropriate (in general, the intermediate value of the three).

i) CBR and/or UCS at 7 days cure plus 7 days soak on specimens molded at OMC and 95 % MDD (as determined by test (h) above) with 3 amounts of additive. CBR tests apply to improved materials, whereas UCS apply to stabilized ones (The results of the between" Improvement" and" stage Stabilization" to be made)

j) Atterberg Limits on one set of 3 specimens (3 amounts of additive) from (i) above.

At least one large sample per 5,000 m3 of material proposed for treatment shall be submitted to tests (h), (i) and (j) above.

(4) Stone Quarries

(i) general

Information obtained will enable a selection of the most suitable quarry sites to be made on the basis of stone quality and quantity, location, access and ease of working.

(ii) Investigations, drilling and sampling

Each selected potential quarry site shall be investigated as follows:

- Trial holes shall be dug or drilled on a 30 m grid to prove overburden.
- Boreholes shall be drilled to prove quantity and quality of stone. It is recommended that, normally, the cores diameter be 76 mm (Coring bits required: HWG, formerly HX). In any case, the minimum coring diameter shall be 55 mm (NWG, formerly NX), so as to recover stone in sufficient quantity for testing. The log of each borehole shall be accurately recorded, drawn and included in the Materials Report.

- Consideration should be given to the use of a bulldozer or other mechanical excavator to prove the availability of solid rock. Such an excavation may also be shown to tenderers during a conducted site visit.
- Samples of fresh rock shall be obtained by hand, or pneumatic drilling from existing faces and outcrops. Great care shall be taken to avoid sampling from a superficial horizon of weathered rock and. to ensure the samples are representative of the stone to be used.
- In addition, whenever possible, deeper samples shall be obtained by blasting.

Depending on the consistency of the stone and whether it is an existing or a new quarry, 5 to 10 samples are required per quarry

A site plan of each potential quarry shall be prepared, showing the characteristic features of the site (outcrops, existing faces etc.) and the means of access and location.

The position and level of each borehole and each sampling, point shall be accurately determined and recorded on the site plan.

(iii) Testing

Each sample shall contain sufficient material to carry out the following tests:

a) Los Angeles Abrasion

b) Aggregate Crushing Value

c) Sodium Sulphate Soundness

d) Plasticity Index on L A A fines

e) Specific Gravity (over-dry method)

f) Bitumen Affinity (for stone proposed for use with bitumen)

Moreover, one large sample shall be obtained from each quarry, so as to be representative of the stone to be used.

This large sample shall be crushed with a small crusher (and not broken by hand), to a maximum size depending on the proposed use of the stone (usually ranging from 20 to 40 mm). The crushed stone shall be submitted to tests (a), (b), (c), (d), (e), (f) above and, in addition, to the following tests:

a) Grading to 0.075 mm sieve

b) Flakiness Index

c) Sand Equivalent

d) Compaction test (Vibrating Hammer method), when appropriate.

(5) Mass of Samples Required

The total mass of sample required on the test to be carried out the grading of the material (its maximum particle size, in particular) and its susceptibility to crushing during compaction.

For general guidance, Table 4.12.1 below shows the minimum mass of sample required for various sequences of tests and typical materials, namely:

- Fine grained soil (Maximum size : 2 mm)
- Coarse grained gravel (Maximum size: 40 mm), not susceptible to crushing during compaction.
- Coarse grained gravel (Maximum size: 40 mm), susceptible to crushing during compaction.
- Solid stone.

The masses indicated in Table 4.12.1 include some allowance for drying, wastage and rejection of coarse fragments where necessary.

Table 4.12.1 Minimum Mass of Sample Required (Soil and Gravel)

Test Required	Fine grained soil (max. size 2 mm)				Coarse grained gravel (max. size 40 mm) not susceptible to crushing				Coarse grained gravel (max. size 40 mm) susceptible to crushing			
Grading	-0-	-0-	-0-	-0-	-0-	-0-	-0-	-0-	-0-	-0-	-0-	-0-
Atterberg limits	-0-	-0-	-0-	-0-	-0-	-0-	-0-	-0-	-0-	-0-	-0-	-0-
Compaction		-0-	-0-	-0-		-0-	-0-	-0-		-0-	-0-	-0-
CBR (1 point)		-0-				-0-				-0-		
CBR (3 point)			-0-	-0-			-0-	-0-			-0-	-0-
Treatment Tests				-0-	-0-			-0-				-0-
Minimum Sample Mass	5	20	35	80	20	40	60	150	20	60	80	180

Table 4.12.2 Minimum Mass of Sample Required (Solid Stone)

Tests Required	Solid Stone	
L.A.A	-0-	-0-
A.C.V	-0-	-0-
S.S.S	-0-	-0-
S.G	-0-	-0-
Bitumen Affinity	-0-	-0-
Crushing	-0-	-0-
F.I		-0-
S.E		-0-
Compaction		-0-
Minimum Sample Mass (kg)	50	200

4.12.4 Standard Method of Testing

(1) Soils and Gravels

1) **Preparation of disturbed samples for testing**

Samples shall be prepared for testing as indicated in relevant AASHTO test method or clause 1.5 of BS 1337, except that:

a) A 40 mm sieve shall be used instead of a 37.5 mm sieve,

b) The mass (in g) of sample required for sieve analysis is around 400 D

c) Samples containing particles than 20 mm shall be prepared for compaction and CBR tests as follows:

Sieve an adequate quantity of the representative material over the 50 mm and 20 mm sieve. Weigh the material passing the 50 mm sieve and retained on the 20 mm sieve and replace it with an equal mass of material passing the 20 mm sieve and retained on the 5 mm sieve. Take the material for replacement from the remaining portion of the main sample.

Note: Preparation of gravel samples

The aggregation of particles shall be broken with a wooden hammer or rubber pestle. Care shall be taken that no discrete particles are crushing in this operation.

2) **Determination of the moisture content**

This test shall be carried out in accordance with the relevant AASHTO test method or as 1377, Test 1(A) – Standard method (oven drying method).

3) **Determination of the Liquid Limit**

This test shall be carried out in accordance with the relevant AASHTO test method or BS 1377, Test 2 (A) – Method using the con penetrometer or test 2(B) Method using the Casagrande apparatus.

4) **Determination of the Plastic Limit and the Plasticity Index**

The plastic limit shall be determined as indicated in the relevant AASHTO test method or BS 1377 Test 3

The plasticity index shall be determined as indicated in the relevant AASHTO test method or BS 1377. Test4.

5) **Determination of the liner shrinkage**

This test shall be carried out in accordance with the relevant AASHTO test method or as 1377, Test 5.

6) **Determination of the particle size distribution**

This test shall be carried out in accordance with the relevant AASHTO test method or as 1377, Test 6.

7) **Determination of the particle size distribution**

The quantitative determination of the particle size distribution in a material down to the fine sand size shall be carried out in accordance with the relevant AASHTO test method or BS 1377, Test 7 out (A) in - Wet sieving method, except that the following series of sieves, based on ISO 565 – 1972 (E), shall be used:

80 – 63 – 50 – 40 – 28 – 14 – 10 – 6.3 – 5 – 4 – 2 – 1 -0.6 – 0.5 – 0.425 – 0.300 – 0.150 – 0.075 mm

The quantitative determination of the particle size distribution in a soil from the coarse and size down shall be carried out in accordance with BS 1377, Test 7 (D) – hydrometer method.

8) **Determination of the organic matter content**

This test shall be carried out in accordance with the relevant AASHTO test method or as 1377, Test 8.

9) **Determination of the total sulphate content**

This test shall be carried out in accordance with the relevant AASHTO test method or as 1377, Test 9.

10) **Determination of the pH value**

This test shall be carried out in accordance with the relevant AASHTO test method or as 1377, Test 11. (A) – Electrometric method.

11) **Determination of the density - moisture content relationship using a 2.5 kg rammer (Standard Compaction Test)**

This test shall be carried out in accordance with AASHTO T99 except in place sieves shall be used of the 4.75 mm and 19.0 mm test sieve the 5.00 mm and 20.0 mm test sieve shall be used:

- Method A (102 mm mold) for materials passing a 5 mm sieve
- Method D (152 mm mold) for materials passing a 20 mm sieve

Samples containing larger particles than 20 mm shall be prepared as indicated in Section 4.12.4(1)1).

When the material is susceptible to crushing during compaction, a separate and new sample shall be used for each compaction test.

Note: In Method A, molds and rammers to the relevant AASHTO test method or BS 1377 (diameter: 107 mm, volume: 1 liter) may also be used. The number of blows shall then be increased from 25 to 27, so as to obtain the same compacting energy per unit volume.

12) **Determination of the density-moisture content relationship using a 4.5 kg rammer (Heavy Compaction Test. also called "Modified AASHTO")**

This test shall be carried out in accordance with AASHTO T180 except in place of the 4.75 mm and 19.0 mm test sieves the 5.00 mm and 20.0 mm test sieves shall be used:

- Method A (102 mm mold) for materials passing a 5 mm sieve
- Method D (152 mm mold) for materials passing a 20 mm sieve

Samples containing larger particles than 20 mm shall be prepared as indicated in Section 4.12.4(1)1).

When the material is susceptible to crushing during compaction, a separate and new sample shall be used for each compaction test.

Note: In Method A, molds and rammers to the relevant AASHTO test method or BS 1377 (diameter: 107 mm, volume: 1 litre) may also be used. The number of blows shall then be increased from 25 to 27, so as to obtain the same compacting energy per unit volume.

13) **Determination of the density-moisture content relationship using**

This test shall be carried out in accordance with the relevant AASHTO test method or BS 1377, Test 14 – Vibrating Hammer method, except that a 40 mm sieve shall be used instead of a 37.5 mm sieve.

14) **Determination of the California Bearing Ratio**

This test shall be carried out in accordance with the relevant AASHTO T99 except that in place of the 4.75 mm and 19.0 mm test sieve the 5.00 mm and 20.0 mm test sieve shall be used.

Samples containing larger particles than 20 mm shall be prepared as indicated in Section 4.12.4(1)1).

15) **Determination of the Sand Equivalent**

This test shall be carried out in accordance with AASHTO T176 except that in place of the 4.75 mm test sieve the 5.00 mm and 20.0 mm test shall be used

- Mechanical shaker method or Manual shaker method.

16) **Determination of the dry density on the site**

This test shall be carried out in accordance with the relevant AASHTO test method or BS 1377. Test15.

The Sand Replacement Method, i.e. test procedure 15 (a) or (B) should be used wherever possible except the 115 mm diameter sand pouring cylinder with a core dimensions to AASHTO Tl91, However, there might be cases in which other procedures, i.e. 15(D) core Cutter Method, 15(E) Immersion in water Method or 15(F) Water Displacement Method, have to be employed (for example, in auger drilled holes).

(2) Stone, Aggregate, Sand and Fillers

Sampling shall be carried out, the samples prepared and the following tests performed in accordance with the relevant AASHTO test method or BS 812:

(1) Determination of the particle size distribution, except that the ISO series of sieves shall be used.

(2) Determination of clay, silt and gust in fine or coarse aggregate.

(3) Determination of the Flakiness Index.

(4) Determination of the relative density and water absorption.

(5) Determination of the bulk density, voids and bulking.

(6) Determination of the moisture content. Standard Method (Oven drying method)

(7) Determination of the Aggregate Crushing Value.

(8) Determination of the soluble chloride content.

(9) The following tests shall be carried out in accordance with the following methods:

(10) Organic impurities in sands for concrete: AASHTO T21.

(11) Resistance to abrasion of coarse aggregate by use of the Los Angles machine. AASHTO T96 (ASTM C 131) and for large size coarse aggregate ASTM C 535.

(12) Soundness of aggregates by use of sodium sulphate AASHTO T 104 (ASTM C 88)

(13) Sand – Equivalent of graded aggregates AASHTO T 176 (Mechanical Shaker or manual Shaker Method) except the 4.75 mm test sieve shall be replaced by a 5.00 mm test sieve.

(14) Average Least Dimension of an aggregate.

The Average Least Dimension of a sample of aggregate shall be determined as follows:

(a) Method

By means of a riffler, divide out a representative sample of such a size as to give at least 200 aggregate particles of each of the fractions to be tested.

Sieve the sample through a sieve with an aperture size half the nominal size on the aggregate to be tested and discard the particles passing the sieve (see (c)).

By means of callipers with platens of at least 5 mm in diameter (or square) measure to smallest dimension of each particle retained on that sieve, accurate to. 0.1 mm and record the measurements and the number of particles tested.

(b) Calculations

Calculate the average least dimension to the nearest 0.01 mm as follow:

Average least dimension (mm) = A/B

Where

A = sum of the smallest dimension of all the particles in mm

B = number of particles

Report the average least dimension to the first decimal place.

(c) Note:

Nominal size can be considered as the smallest sieve through which at least 85 per cent of the aggregate will pass.

(3) Cement and lime

Ordinary and Rapid Hardening Portland cement shall be samples and tested in accordance with the relevant AASHTO test method.

Lime shall be sampled and tested in according with the relevant AASHTO test method or BS 890.

(4) Cement or Lime Treated Materials

1) **Preparation of samples for testing**

Samples shall be prepared for testing as indicated in the relevant AASHTO test method or Clause 1.5.2 of BS 1924 expects that:

(i) A 40 mm sieve shall be used instead of a 37.5 mm sieve.

(ii) Samples containing large particles than 20 mm shall be prepared for compaction and CBR tests. (The fraction coarser than 20 mm shall be replaced by an equal weight of 5/20mm material).

2) **Determination of the moisture content**

This test shall be carried out in accordance with the relevant AASHTO test method BS 1924. Test1 (A) Standard Method (over drying method).

3) **Determination of the density – moisture content relationship using a 25 kg rammer (Standard Compaction Test)**

Five or more samples of treated material shall be prepared. The samples shall each be mixed thoroughly with different amounts of water to give a suitable range of moisture contents, i. e. such that the O.M.C is within the range.

Each sample shall then be tested in accordance with AASHTO T99 except that in place of the 4.75 mm and 19.0 mm test sieves the 5.00 mm and 20.0 mm test sieves shall be used:

- Method A (102 mm mold) for materials passing a 5 mm sieve
- Method D (152 mm mold) for materials passing a 20 mm sieve

When cement is used, compaction must start within one hour and be complete within 2 hours after the start of mixing operation.

4) **Determination of the density – moisture content relationship using a 4.5 kg rammer (Heavy Compaction Test)**

Five or more samples of treated material shall be prepared. The samples shall ach be mixed thoroughly with different amounts of water to give a suitable range of moisture contents, i.e. such that the O.M.C is within the range.

Each sample shall then be tested in accordance with AASHTO T180 except that in place of the 4.75 mm and 19.0 mm test sieves the 5.00 mm and 20.0 mm test sieve shall be used:

- Method A (102 mm mold) for materials passing a 5 mm sieve
- Method D (152 mm mold) for materials passing a 20 mm sieve

When cement is used, compaction must start within one hour and be complete within 2 hours after the start of mixing operation.

5) **Determination of the density – moisture content relationship using a vibrating hammer**

This test shall be carried out in accordance with BS 1924 – Test 5 – Vibrating Hammer Method, except that a 40 mm sieve shall be used instead of a 37.5 mm sieve.

6) **Determination of the Unconfined Compressive Strength (UCS) of fine and medium grained treated materials**

This test shall be carried out in accordance with the relevant AASHTO test method or BS 1924 - Test 10. It is further specified that the specimens shall be compacted to a predetermined density, as indicated in the relevant AASHTO test method or Clause 5.1.4.1 of BS 1924 and during the curing period, the specimens shall be kept at a temperature of 27 ± 2°C.

7) **Determination of the Unconfined Compressive Strength (UCS) of medium and coarse grained treated materials**

This test shall be carried out in accordance with the relevant AASHTO test method or BS 1924 - Test 11, except that a 40 mm sieve shall be used instead of a 37.5 mm sieve. It is further specified that the specimens shall be compacted to a predetermined density, as indicated in Clause 5.2.3, 1 of BS 1924 and during the curing period, the specimens shall be kept at a temperature of 273±2°C.

8) **Determination of the effect of immersion in water on the U.C.S of treated material**

This test shall be carried out in accordance with the relevant AASHTO test method or BS 1924 - Test 12. It is further specified that, during the curing and soaking periods, the specimens shall be kept at a temperature of 27±2°C.

9) **Determination of the California Bearing Ratio of treated material**

Samples containing larger particles than 20 mm shall be prepared.

The CBR test shall then be carried out in accordance with the relevant AASHTO test method or BS 1924 - Test 13. CBR molds complying with AASHTO M193 may be used instead of the BS molds if the equivalent AASHTO M193 compaction procedures are used:

It is further specified that:

a) The specimens shall be dynamically compacted with a 4.5 kg rammer, the number of blows being governed by the relative compaction chosen (Dynamic Compaction, Method 1).

b) If it is required to soak the specimen, the mold shall be immersed in water to allow free access of water to top and bottom of the specimen. During soaking, the water level in the mold and the soaking tank shall be maintained approximately 25 mm above the top of the specimen.

c) During the curing and soaking periods, the specimens shall be kept at a temperature of 27±2°C.

10) **Determination of the cement content treated materials**

This test shall be carried out in accordance with the relevant AASHTO test method or BS 1924, Test 14.

11) **Determination of the time content of lime treated materials**

This test shall be carried out in accordance with the relevant AASHTO test method or BS 1924, Test 15.

(5) Bituminous Binders

1) **Sampling procedures**

Sampling of straight - run bitumen and cut - backs shall be carried out in accordance with AASHTO method T40 (ASTM D140).

Sampling of bitumen emulsion shall be carried out in accordance with BS 434, except that where a delivery is made in drums or barrels, the number of samples shall be as indicated in AASHTO Sampling method T40, Paragraph 1 l. l.

2) **Testing Procedures**

(a) Tests on straight-run bitumen shall be carried out in accordance with the following test procedures:

Penetration	AASHTO T49 (ASTM D5)
Softening point (Ring and Ball)	AASHTO T53 (ASTM D2398)
Flash and fire points	
(Cleveland open Cup)	AASHTO T48 (ASTM D92
- Loss on heating	AASHTO T47 (ASTMD6)
Ductility	AASHTO T51 (ASTM D113)
Water	AASHTO T55 (ASTM D95)
Thin film oven test	AASHTO T 179 (ASTM D1754)
- Solubility in organic	
Solvents	AASHTO T44 (ASTM D2042)
Specified gravity	AASHTO T228 (ASTM D70)

(b) Tests on cut-back bitumen shall be carried out in accordance with the following test procedures:

Kinematic viscosity	AASHTO T20l (ASTM D270)
Flash point (Tag open cup) (RC - MC)	AASHTO T79 (ASTM D310)
Flash point (Cleveland open cup) (SC)	AASHTO T48 (ASTMD92)

Distillation	AASHTO T78 (ASTMD402)
Water	AASHTO T55 (ASTMD95)
Specific gravity	AASHTO T228 (ASTMD3142)
Asphalt residue of 100 pen (SC)	ASM D243
Tests on residue from distillation	
- Penetration	AASHTO T49 (ASTM D5)
- Ductility	AASHTO T51 (ASTM Dl 13)
- Solubility	AASHTO T44 (ASTM D2042)
- STV viscosity	the relevant AASHTO test method or BS 3235

(c) Test on bitumen emulsion shall be carried out in accordance with the relevant AASHTO test method or BS 434 test procedures:

Residue on 0.710 mm sieve

Residue on 0.50 mm sieve

Stability to mixing with coarse aggregate

- Stability to mixing with cement

Binder content

- Engler viscosity
- Redwood II viscosity

Storage stability (short period)

- Storage stability (long period)
- Particle charge

(6) Bituminous Mixes

1) **Sampling procedures**

Sampling of bituminous mixtures shall be carried out in accordance with AASHTO method T 168 (ASTM D 979).

2) **Testing procedures**

Test on bituminous mixtures shall be carries out in accordance with the following test procedures:

- Moisture and volatile distillates	AASHTO T110 (ASTMD1461)
- Quantitative extraction of bitumen	AASHTO T146 (ASTMD2172) or BS 598
- Specific gravity of compacted mixture	AASHTO T166 (ASTMD1188)

	And D2726
- Recovery of bitumen from solution	AASHTO T170 (ASTMD1856)
	Or BS 598
- Coating and stripping	AASHTO T182 (ASTMD1664)
(with adhesion agent)	ASTMD2727
- Maximum specific gravity	AASHTO T209 (ASTMD2041)
- Degree of pavement compaction	AASHTO T230
- Marshall stability	AASHTO T245 (ASTMD1559)
- Hubbard – Field Stability	ASTMD1138
- Brazilian test	see below

The Brazilian Test

The Brazilian test comprises the measurement of the force required to crush a cylindrical sample along one of its diameters between two parallel flat plates. The sample are to be prepared in accordance with and to the size required by the Marshall test (ASTM D 1559) compacted using 50 blows of the rammer on each face of the sample. The temperature of the sample at the time of test is to be 25°C ±2OC and the plattens during the test are to be moved together at a content rate of 0.86 mm/sec. The indirect tensile stress is calculated as:

$$\frac{2 \times Load\ at\ failure}{perimeter \text{ of sample} \times \text{thickness of sample}}$$

CHAPTER 5 HYDRAULIC DESIGN

5.1 GENERAL

There are two main steps in the design of cross drainage structures for roads. The first step is to perform a hydrologic analysis of the particular drainage area to determine the quantity of water to discharge.

The second step is to design the drainage structure by performing hydraulic design calculations using the discharge values obtained from the hydrologic analysis in the first step.

The determination of discharge flow rates is a complex issue that often leads to series of estimations.

This manual, which does not intend to cover all aspects of drainage is limited to basic design for drainage of roads.

5.2 HYDROLOGY

5.2.1 Hydrologic Data

i) topographic maps in suitable scales,(1:50,000 or larger scale).

ii) hydrological base data such as flow stage and discharge from the water course crossed by or located close to the road alignment.

iii) additional information on the water course e.g. downstream or upstream crossings over the same river.

ii) satellite imagery, digital elevation models.

iii) photographs up- and downstream of the proposed crossing point.

iv) water use information of the area concerned, e.g. dams, irrigation etc.

v) rainfall data, including Intensity-Duration-Frequency curve (IDF curve).

vi) high-water marks of streams or rivers-etc. in the study area.

vii) further information on past flood events and past flood levels from local people.

5.2.2 Hydrologic Analysis

The collected data is used to determine following characteristics for a particular catchment area (drainage basin):

- Drainage area details such as obstructions, vegetation, water storage etc.
- Determination of catchment boundaries and run-off coefficients "C" for each segment of the area.
- Determination of the drainage area "A".
- Determination of the catchment or basin length that is the length from the furthest point in the basin to the point of discharge.
- Determination of catchment topography, highpoints, low points and slopes.
- Determination of the degree of rainwater infiltration, percentage imperviousness e.g. for water travelling over exposed rock areas versus dense vegetation areas.

Note:

The importance of the basin length is that a shorter travel time for storm water run-off usually has a higher discharge impact at the design point, than a longer rain travel time. The shape of the catchment also has influence on the discharge at the design point as shown in Figure 5.2.1 below.

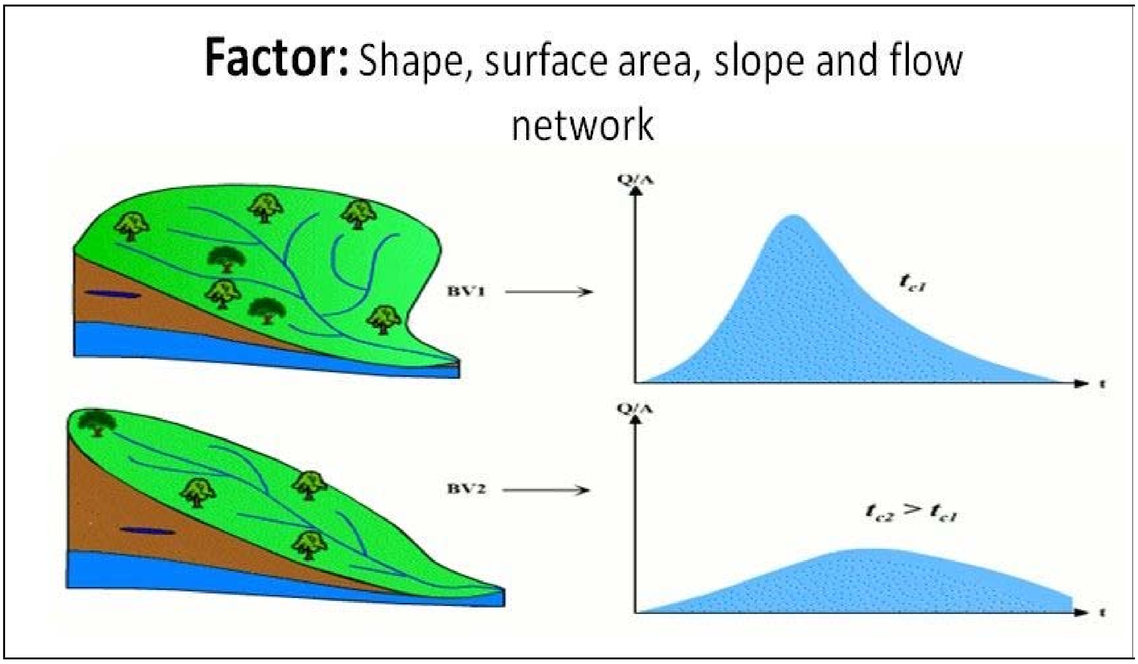


Figure 5.2.1 Influence of catchment shape on run-off hydrograph

5.2.3 Flood Discharge Estimates

The "Rational Method" is used for calculation of the maximum water discharge for a specific run-off area, formula as follows:

(1) $$Q_p = 0.278 \times C \times I \times A \quad \text{in m3 / sec.}$$

Where: Q = flow in cubic meters per second.

C = run-off coefficient, expressing the fraction of the rainfall that is assumed to become direct run-off.

I = intensity of rainfall in mm/hour, for the duration corresponding to the time of concentration (Tc) of each catchment area. See equation (2) below.

A = the drainage catchment area in km2.

Tc, the time of concentration is the time period (duration) required for the rain water to reach the outlet from the most remote point of the area.

The formula used is:

(2) $$T_c = \left(\frac{1}{H} \times 0.87 \times L^3 \right)^{0.385} \quad \text{in hours.}$$

Where: L = the stream flow length of the catchment area in km.

H = the corresponding level difference in meter (m).

Alternatively the Time of Concentration can be determined graphically by using Figure 4.3.

Notes:

The Rational formula is based on the theory that the run-off rate is linearly related to rainfall. This means that the run-off rate would become constant if a uniform rain of a constant

intensity falls on an impervious specific area.

Since the error of run-off estimate increases with increasing size of the drainage area, the

Rational Method is normally limited to an area size of smaller than 25 km2 (2500 Ha).

For larger areas the formula should be used with strict care, splitting up factors and areas and other empirical, graphical or statistical formulas should be considered.

Typical values of run-off coefficients for use in either urban or rural areas respectively are shown in Table 5.2.1, Table 5.2.2 below

Table 5.2.1 Typical C Coefficients Urban Areas

Description of Area	Run-off Coefficients
Business Downtown Areas Neighborhood Areas	 0.70-0.95 0.50-0.70
Residential Single-Family Areas Multiunits, Detached Multiunits, Attached	 0.30-0.50 0.40-0.60 0.60-0.75
Residential (Suburban)	0.25-0.40
Apartment Dwelling Areas	0.50-0.70
Industrial Light Areas Heavy Areas	 0.50-0.80 0.60-0.90
Parks, Cemeteries	0.10-0.25
Playgrounds	0.20-0.35
Railroad Yard Areas	0.20-0.40
Unimproved Areas	0.10-0.30
Streets Asphaltic Concrete Brick	 0.70-0.95 0.80-0.95 0.70-0.95
Drives and walkways	0.75-0.85
Roofs	0-75-0.95
Lawns; Sandy Soil Flat, < 2% Average, 2-7% Steep, > 7%	 0.05-0.10 0.10-0.15 0.15-0.20
Lawns; Heavy Soil Flat, < 2% Average, 2-7% Steep, > 7%	 0.13-0.17 0.18-0.22 0.25-0.35

Table 5.2.2 Typical C Coefficient Rural Areas

Watershed Characteristics			
A Relief	B Soil infiltration	C Vegetal Cover	D Surface Storage
0.40 Steep rugged terrain. Average slopes greater than 30%	0.20 No effective soil cover, either rock or thin soil mantle, negligible infiltration capacity	0.20 No effective plant cover. Bare or very sparse soil cover	0.20 Negligible, surface depression few and shallow, drainage ways steep and small. No ponds or marshes
0.30 Hilly with Average slopes of 10 to 30%	0.15 Slow to take up Water, clay or other soil of low infiltration capacity such as heavy gumbo	0.15 Poor to fair, clean cultivated crops or poor natural cover. Less than 10% of area under good cover	0.15 Low, well defined system of small drainage ways. No ponds or marshes
0.20 Rolling with average slopes of 5 to 10%	0.10 Normal deep loam	0.10 Fair to good, About 50% of area in good grass land woodland or equivalents cover	0.10 Normal, considerable surface depression storage, typical of prairie lands, lakes, ponds. Marshes less than 20% of area
0.10 Relatively flat land, average slopes 0 to 5%	0.05 High, deep sand or other soil that takes up water readily and rapidly	0.05 Good to excellent. About 50% of area in good grass land, woodland or equivalent cover	0.05 High, surface depression storage high, drainage system not sharply defined, large flood plain storage, large number of ponds and marshes

Note: Run-off coefficient is equal to sum of coefficients from the appropriate block in rows A, B, C and D. These run-off coefficients shall be proportioned to the percentage of area covered

Example: A watershed consists of 20 ha of light industrial areas,30 ha of apartment dwelling areas and 15 ha of residential single family areas.
Total Area = 65 ha

Area	Type	Percent Of Total	Coeff.
20 ha	Light Industrial	20/65 = 0.31	0.65
30 ha	Apartment Dwelling	30/65 = 0.46	0.60
15 ha	Residential Single family	15/65 = 0.23	0.40

5.2.4 Calculation using the Rational Method

1. From suitably scaled maps, measure the area “A” using a planimeter or other means such as GIS or mapping software. For many area segments along a specific route, it is advisable to split the area segments along the water dividers selected from contoured maps. Determine the area "A" for each discharge point on the road.

2. Select the run-off coefficients "C" for each area segment and calculate the average coefficient "C" for areas leading to the discharge point. Run-off coefficients should be selected on the basis of satellite imagery which shows recent developments in land use. Maps might be too old.

3. Determine the rainfall intensity factor "I" for the storm (rainfall) duration Tc for the specified rainfall return frequency.

Example 4.1

A catchment area (drainage basin) consists of a 3.1 km2 (3100 Ha) gently sloping farmland near the town of Attapue. The overall average slope has been estimated to 0.3%. The length of catchment or the length from the most remote point to the drainage position is 2.5 km. A 25 year rainfall return frequency is selected for the design.

Figure 5.2.2 below illustrates the relationship between catchment geometry, shape of flood hydrograph and the data requirement for the Rational Method.

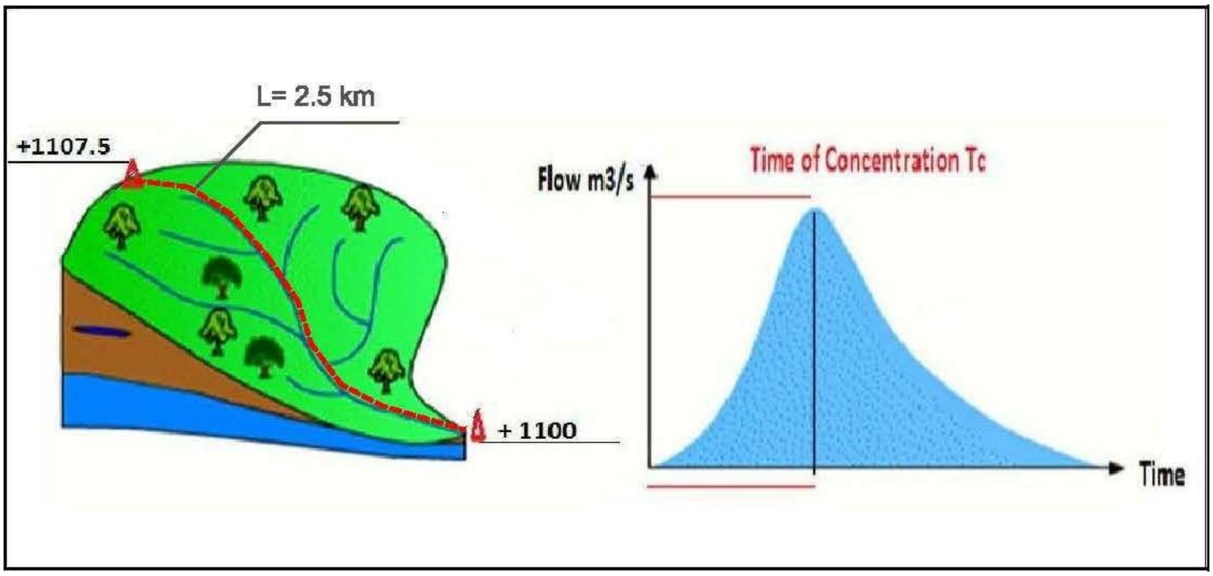


Figure 5.2.2 Relationship between catchment geometry and shape of run-off hydrograph

Area	3.1 km2
H	7.5 m
L	2.5 km

Solution for Example 4.1

i) UsingFigure 5.2.3 Tc is estimated for L = 2.5 km and H = 7.5 meter.

Tc = 1.25 Hours or 75 Minutes:

ii) Using the Intensity-Frequency curve for Attapue (Figure 5.2.5), enter the Tc value of 75 minutes against the 25-years frequency curve to obtain rainfall Intensity “I”.

I = 67 mm per hour.

iii) Using Figure 5.2.4 for the estimated run-off coefficient "C", or the calculated (more accurate) average coefficient, determines the "C" factor.

C is selected as 0.23in Figure 5.2.4, for a 67 mm intensity rain against Band 3 timber lands of moderate to steep slopes.

iv) Enter the determined values of C, I, and A, into the Rational Formula to obtain the design discharge value.

$$Q = 0.278 \times 0.23 \times 67 \times 3.1 = 13.28 \text{ m3/sec.}$$

Note:

Figure 5.2.4 should only be used for overall estimation purposes. For actual design values it is recommended to calculate the "C" values. For larger drainage structures it is advisable to perform hydrological subdivisions of the catchment area.

A change in the run-off coefficient, say from 0.23 in above example to 0.30, will increase the discharge value by +30%! Great care should therefore be taken in the selection of run-off coefficients.

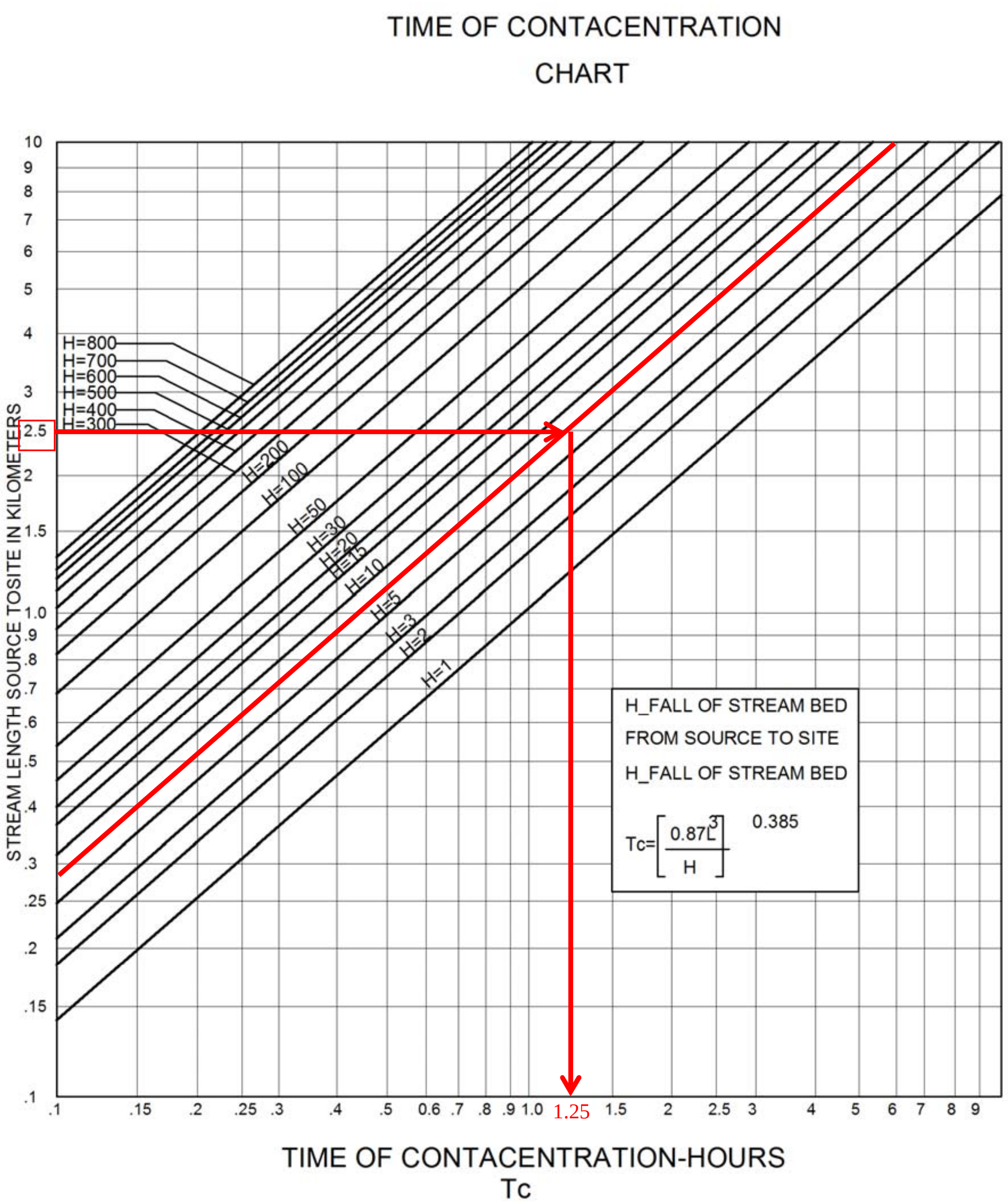


Figure 5.2.3 Diagram for manual determination of Time of Concentration

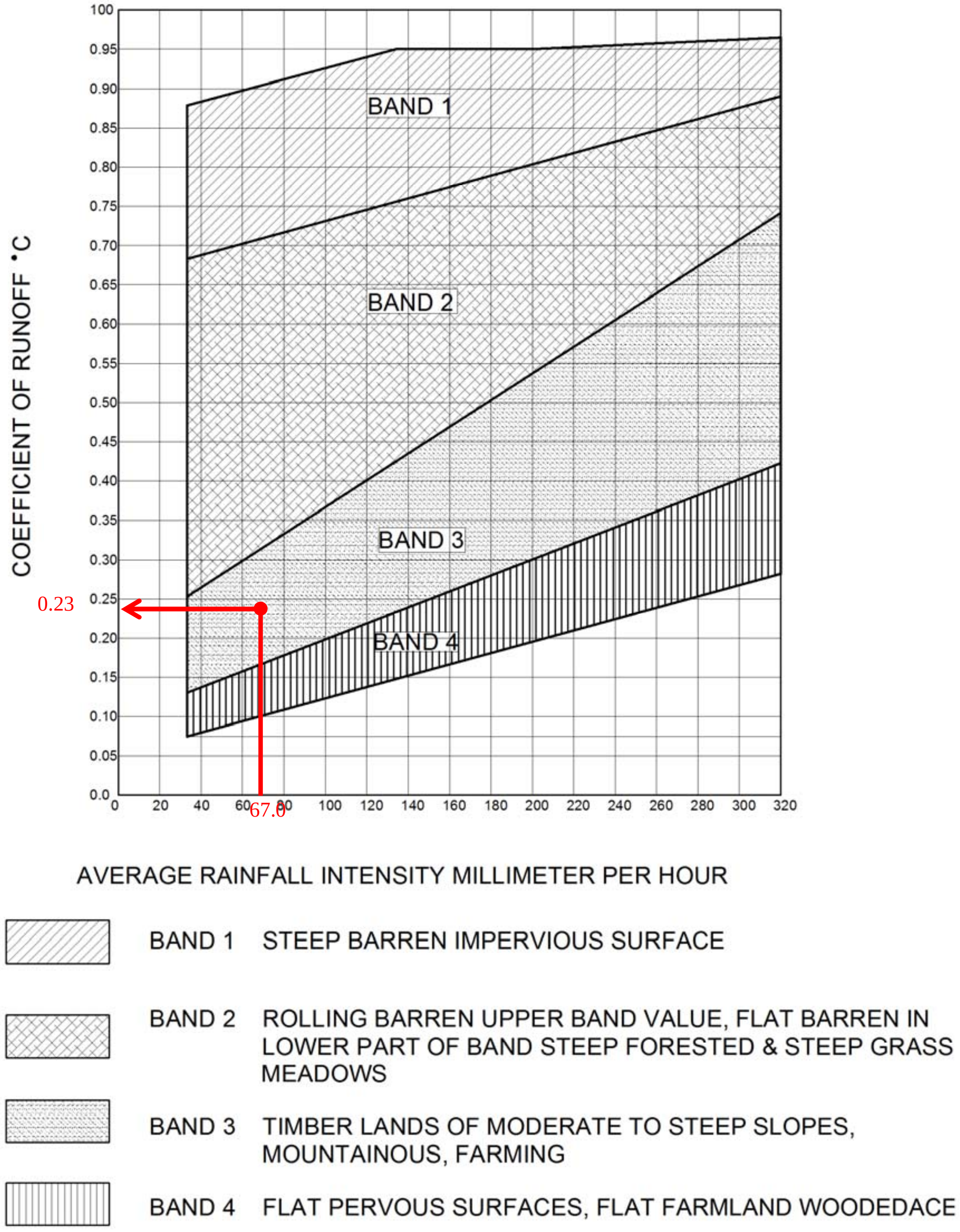


Figure 5.2.4 Diagram of estimation run-off Coefficient C in relation to time of concentration

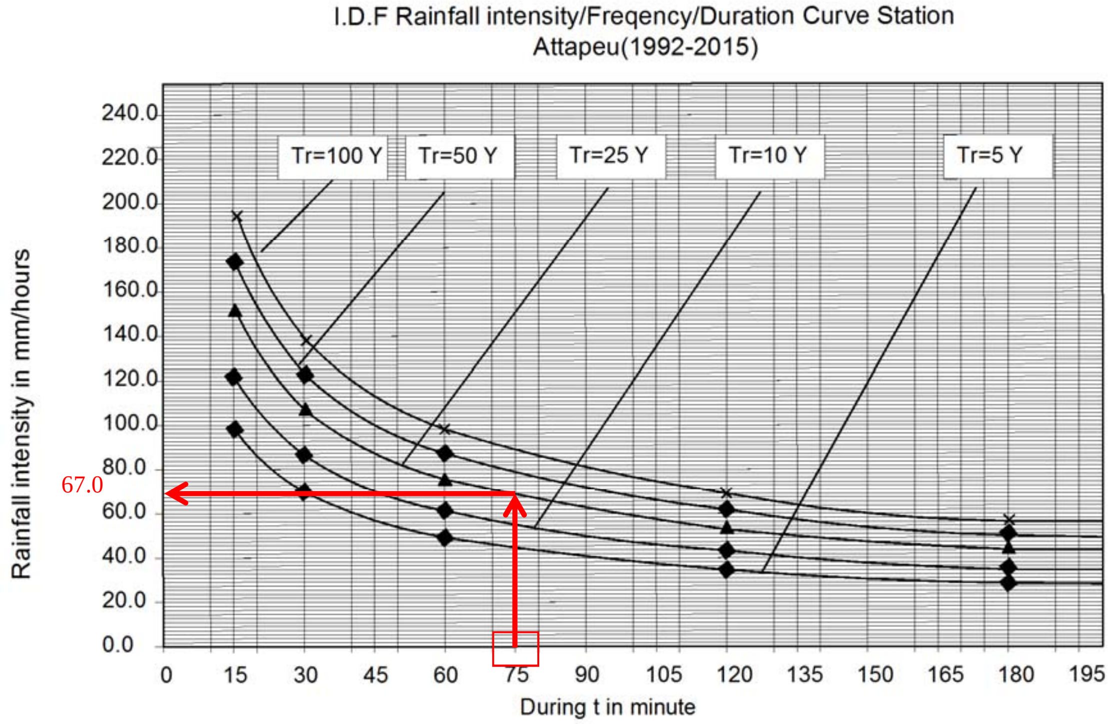


Figure 5.2.5 Intensity Duration Frequency Curve Attapue

Example 4.2

Another catchment area located in the eastern part of Laos has terrain which combines mountains, rolling and gentle sloping run-off terrain segments, giving variable slope and runoff values. Calculate the discharge, but this time use Table 5.2.2. and combine run-off coefficients proportionately to the percentage of areas covered.

Area No.	A in km 2	A %	C	Sum of C	Average C
			A+B+C+D From Table 4.2		A%/sum of "C"
A1	0.3	0.08	(0.3+0+0.2+0)	0.5	0.04
A2	1.2	0.32	(0.1+0.1+0.15+0.15)	0.45	0.14
A3	1.5	0.41	(0.1+0.05+0.05+0.05)	0.25	0.1
A4	0.7	0.19	(0.3+0.15+0.15+0.05)	0.65	0.12
Total	3.7	1.0			Average "C" = 0.4

Drainage area length L= 4.0 km

Elevation difference H = 3 m

Using Figure 5.2.3or Formula F2, Tc is estimated for L = 4.0 km and H = 3.0 m, giving

Tc = 3.0 hours, or 180 Min.

Using the 25 years IDF - curve a rainfall intensity of 42 mm/h is obtained. Entering the values into the Rational Formula a design flow of

0.278 x 0.4 x 42 x 3.7 = 17.28 m3/ sec. is calculated.

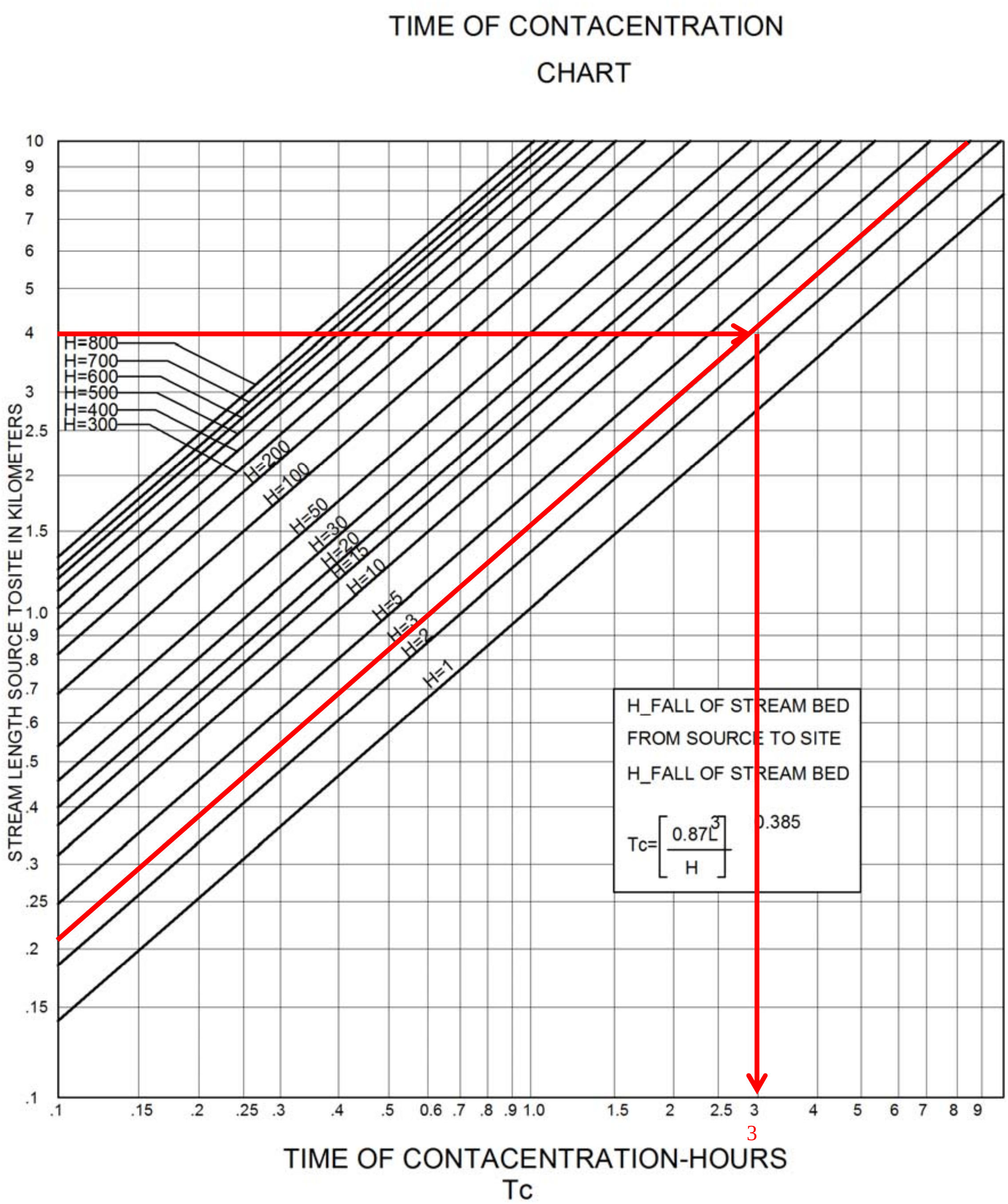


Figure 5.2.6 Diagram for manual determination of Time of Concentration

5.2.5 Opening Area Calculation

The discharge values obtained from the Rational Equation can also be used to check existing culvert structures and small bridges by performing “Opening Area” calculations.

The following example illustrates this.

Example 4.3

A design check is required for drainage opening and drainage structures. A provincial road crosses streams with following data:

i) At point km 75+200, known stream velocity = 1.7 m/sec.

ii) At point km 78+400, known stream velocity = 2.0 m/sec.

iii) At point km 80+100, known stream velocity = 2.0 m/sec.

The drainage basin consists of timber land with moderate to steep slope, situated in gently rolling terrain.

Calculation steps:

1. Determine catchment area “A” from maps or GIS. Data found was:

i) km 75+200, A = 18.0 km2, L = 4.5 km, H = 75 meter

ii) km 78+400, A = 5.4 km2, L = 2.5 km, H = 50 meter

iii) km 80+100, A = 2.6 km2, L = 2.0 km, H = 5.0 meter

2. .Estimate “Tc” time of concentration using Figure 5.2.3, or use the formula (F2): The data obtained was:

i) km 75+200, Tc = 1.02 Hrs, = 61 minutes

ii) km 78+400, Tc = 0.60 Hrs, = 36 minutes

iii) km 80+100, Tc = 1.13 Hrs, = 68 minutes

3. Determine the “I” Intensity of rainfall from a suitable IDF curve.

For this example the following results are used:

i) km 75+200, I1 = 75 mm/Hr

ii) km 78+400, I2 = 97 mm/Hr

iii) km 80+100, I3 = 70 mm/Hr

4. Select the appropriate "C" Run-off Coefficient (in this example using Figure 5.2.4) as follows:

C1 =0.25, C2 = 0.28, C3 = 0.24.

5. Inserting above values into the Rational Equation Q = 0.278 x C x I x A, gives following 25 years frequency discharge values:

   i) Q25 = 93.8 m3/sec.
   ii) Q25 = 40.7 m3/sec.
   iii) Q25 = 12.1 m3/sec.

6. The required drainage opening area (A) is obtained by Q/A, (discharge/stream velocity) as follows:

   i) km 75+200, 93.8 m3/sec./1.7m/sec. = 55.2 m2
   ii) km 8+400, 40.7 m3/sec./2.0m/sec. = 20.4 m2
   iii) km 80+100, 12.1 m3/sec./2.0 m/sec. = 6.0 m2

7. Structures adopted to match above discharge openings are therefore;

   i) km 75+200, = 55.2 m2

A slab type of bridge of 3 spans x 10 m with clearance to suit channel

and flow area above HWL (High Water Level), which gives a safety factor

FS = 72 m2/ 55.2 m2 = 1.30

i) km 78+400 = 20.4 m2

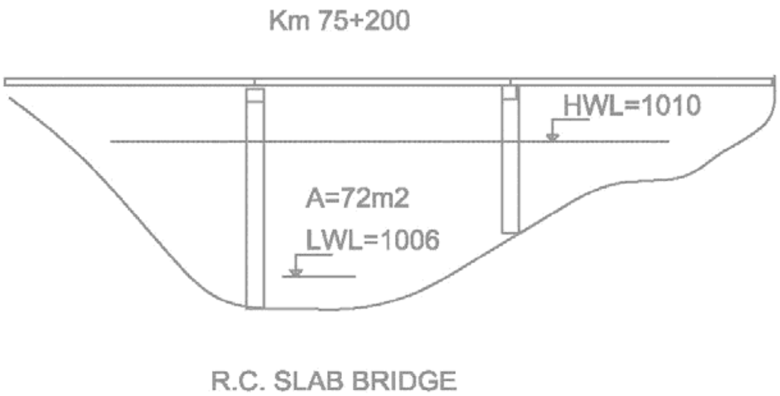
Reinforced Concrete (R.C.) box culverts, 2 Nos. size 3.6 x 3.6 m, which gives a safety factor FS = 25.9 m 2/20.4 m2 = 1.26.

ii) km 80 +100, = 6.0 m2

R.C. circular culverts, 2 Nos. diameter 2250 mm for 3.98 m2 each and a

FS = 7.96/6.0= 1.32 or space permitting, 4 Nos. diameter 1500 mm

for 7.07 m2/6.0 = FS = 1.17.



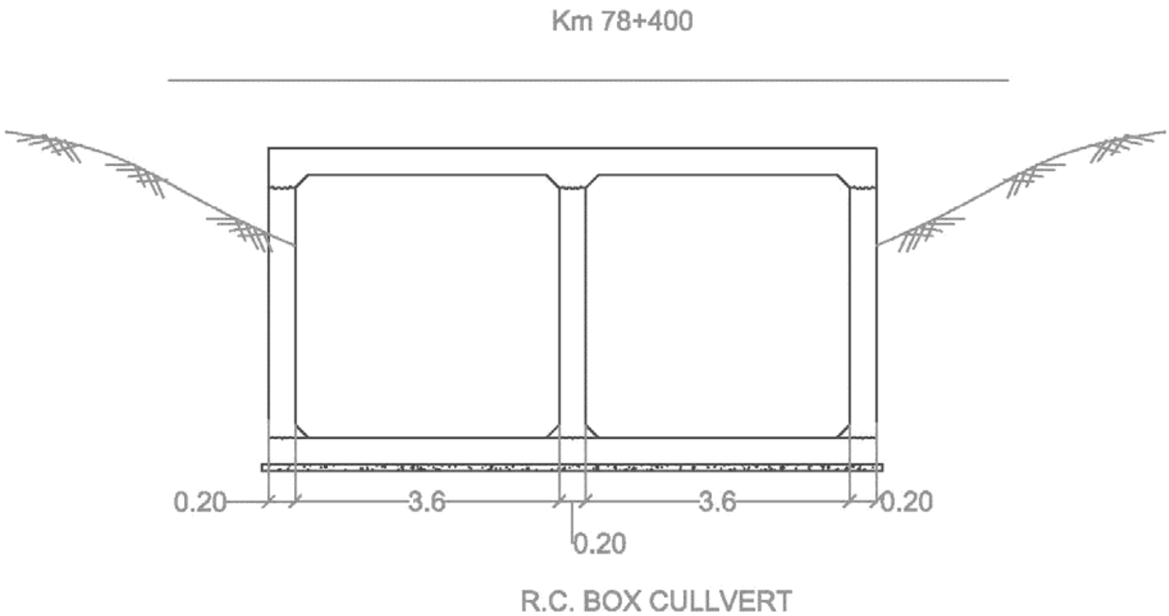


Figure 5.2.7 Culverts checked in Example 4.3

5.2.6 Modified Rational Method

The modified Rational Method is based on the same principle as the original formula. The difference is the introduction of an Area Reduction Factor, which is equal to the value of 1 for small catchments and is reduced as a function of catchment size. The aim of the factor is to counter the overestimation of catchment run-off produced by the Rational Method when applied to large catchments.

The method is easy and straight forward. There are a number of equations available in the literature. Normally the factor is linked to the storm duration and the size of the catchment area. For the application in connection with the Rational Method the formula developed by Kuichling (1889) is commonly used.

The run-off calculation for the Rational Method thus becomes:

$$Q_p = 0.278 \times C \times I \times A \times ARF$$

Where Q_p: is peak discharge in m3/s

0.278: is a constant conversion to express discharge in the required units.

C: is the run-off coefficient.

I: is the rainfall intensity in mm/h during the time of concentration (Tc).

A: is catchment area in km2 .

ARF: is Area Reduction Factor from (Table 5.2.3).

Table 5.2.3 Area Reduction Factor

Area (km2)	Area Reduction Factor
0-25	1
>25-50	0.95
>50-100	0.9
>100-150	0.85
>150-200	0.8

5.2.7 General Tropical Flood Model (GTFM)

This section describes the theory and method of application of the Generalised Tropical Flood Model. This method applies for the mid-size catchment and large catchment with a total catchment area over 25 km^2. The upper limit of catchment area for the application of this model is in the region of 200 km^2. The values of the parameters required for the Generalised Tropical Flood Model have been taken from the recommendations contained in Watkins L H and Fiddes D, Highway and Urban Hydrology in the Tropics, Pentech, Press, London, 92-100 (1984).

The Generalised Tropical Flood Model is expressed by the formula:

$$Q_p = \frac{CA \times P \times A \times F}{360 \times TB}$$

Where Q_p: is peak discharge in m3/s.

CA: is the percentage run-off coefficient.

P: is the design storm rainfall (i.e. total rainfall in mm not intensity in mm/h) of hydrograph base time (TB hours).

A: is the catchment area in km2.

F : is the peak flow factor to convert the average flow

generated by the model to peak flow Table 5.2.11

ARF : is the area reduction factor (as defined in Table 5.2.4).

TB : is the hydrograph base time in hours.

The values of the parameters required for the Generalised Tropical Flood Model have been published by Fiddes (1984) These are reproduced in Table 5.2.4 toTable 5.2.11 and the application is described below.

Percentage run-off coefficient 'CA'

The percentage run-off coefficient 'CA' is express by the formula:

$$CA = CS \times CW \times CL$$

Where CS: Is the standard value of contributing run-off coefficient, from

Table 5.2.7, and is dependent on Soil Class I, from Table 5.2.5and Slope Class S, from Table 5.2.6

CW: is the catchment wetness factor which is dependent on soil moisture recharge (SMR) and which is tabulated in Table

4.10. Because Laos is within a wet zone (SMR > 75 mm) the value adopted is 1.00.

CL: is the land use factor from Table 5.2.9. The value 1 can be taken for most catchments.

Base Time'TB'

The hydrograph base time can be thought of as being made up of three components: the storm duration, the time taken for the surface run-off to drain into the stream system; and the flow time down to the culvert or bridge site. Base time 'T_B' is expressed by the formula:

$$T_B = \frac{C \times A^{0.5}}{S^2} + T_S$$

Where C: is a constant, which is 30 for humid zone catchments.
A: is the catchment area in km^2 .
P: is the Slope Class 'S '.
T_S: is the surface cover flow time from Table 4.8.

Area reduction factor 'ARF'

The area reduction factor (ARF) is introduced to account for the spatial variability of point rainfall over the catchment. In simple terms the average rainfall intensity at any instant for a catchment will be less than the rainfall measured at a single point (rain gauge) in the catchment, and the difference increases with increasing size of catchment. Therefore this is not significant for small catchments but becomes so as catchment size increases. The relationship adopted for 'ARF' is:

$$ARF = 1 - 0.04T^{-0.33}A^{0.50}$$

Where T: is duration in hours

A: area in km2

This equation applies for storms of up to 8 hours duration. For longer durations on large catchments the T = 8 value is adopted. The values adopted for design are presented in Table 5.2.4~~Table 5.2.4Table 5.2.4Table 5.2.4~~.

Table 5.2.4 Areal Reduction Factor, General Tropical Flood Model

Storm	**Catchment Area '*A*' (km^2)**										
Duration '*T*'	**1**	**2**	**3**	**5**	**10**	**25**	**50**	**100**	**250**	**500**	**1000**
8 h	0.98	0.97	0.97	0.96	0.94	0.90	0.86	0.80	0.68	0.55	0.37
4 h	0.97	0.96	0.96	0.94	0.92	0.87	0.82	0.75	0.60	0.44	0.20
2 h	0.97	0.96	0.95	0.93	0.90	0.84	0.78	0.68	0.50	0.29	
1 h	0.96	0.94	0.93	0.91	0.87	0.80	0.72	0.60	0.37	0.11	
30 min	0.95	0.93	0.91	0.89	0.84	0.75	0.64	0.50	0.20		
15 min	0.94	0.91	0.89	0.86	0.80	0.68	0.55	0.37			
10 min	0.93	0.90	0.87	0.84	0.77	0.64	0.49	0.27			
5 min	0.91	0.87	0.84	0.80	0.71	0.54	0.35	0.08			

GTFM Parameters

Table 5.2.5 Soil permeability classification

Soil class (*I*)	Description
1	Impermeable - rock surface.
2	Very low permeability. Clay soils with high swelling potential; shallow soils over largely impermeable layer, very high water table.
3	Low permeability. Drainage slightly impeded when soil fully wetted.
4	Fairly permeability. Deep soils of relatively high infiltration rate when wetted.
5	Very permeable. Soils with very high infiltration rates such as sands, gravels and aggregated clays.

Table 5.2.6 Catchment slope classification

Slope Class (S)	5.2.8 Average catchment slope (%)
1	0 - 0.2
2	0.2 - 1.0
3	1.0 - 4.0
4	4.0 - 10.0
5	10.0 - 20.0
6	> 20.0

Table 5.2.7 Basic run-off coefficient for humid catchments (C_S) (%)

Soil class (I)	1	2	3	4	5
Slope class (S)					
1	49	37	25	13	1
2	57	45	33	21	9
3	65	53	41	29	17
4	73	61	49	37	25
5	81	69	57	45	33
6	89	77	65	53	41
Note:	Values in the above table are based on the relationship: PRO = 53 - 12 I + 8 S, which applies to humid catchments				

Table 5.2.8 Surface cover flow time (TS)

Catchment type	**TS (h)**
Arid zone	0.0
Poor pasture / scrub (large bare soil patches)	0.0
Good pasture	1.0
Cultivated land (down to river bank)	2.0
Forest (a) shallow impermeable soils	2.0
(b) very steep (S5, S6) permeable soils	2.0
(c) other	12.0
Swamp filled valleys	20.0

Table 5.2.9 Land use factor (CL)

Catchment type	**CL (h)**
Semi arid zone	1.00
Largely bare soil (humid zone)	1.50
Intensive cultivation	1.50
Grass cover	1.00
Dense vegetation (particularly in valleys)	0.50
Forest (a) shallow impermeable soils	1.00
(b) very steep (S5, S6) permeable soils	0.67
(c) other	0.33

Table 5.2.10 Catchment wetness factor (CW)

	Catchment wetness factor (CW)	
Rainfall zone	Perennial streams	Ephemeral streams
Semi-arid zone	1.00	1.00
Wet zone (SMR $\geq$ 75 mm)	1.00	1.00
Dry zone (SMR < 75 mm)	0.75	0.50

Table 5.2.11 Peak Flow Factors (F)

Peak Flow Factors (F)	Peak flow factor (F)
Arid zone	3.0
Humid zone	2.5
Forest	1.7

Example Calculation

1	Description	Unit	Calculation	Comments, explanations and data source
2	Road No.		18B	Project Road
3	Catchment No.		18B_CA103	Catchment identification
4	Surface	km2	122	From map, GIS, Internet
5	High Point	m	1046	From map, GIS, Internet
6	Low Point	m	220	From map, GIS, Internet or construction survey point.
7	H_difference	m	826	Calculated from row 5 - raw 6.
8	Channel Length (CL)	m	15000	Measured from map, GIS, Internet
9	TS - Value	h	2	Cultivated land (down to river bank), value: 2, Table 5.2.8
10	Soil (I)	Class	4	From Table 5.2.5
11	Slope (S)	Class	4	From Table 5.2.6
12	C	%	37	From Table 5.2.7
13	Landuse CL		0.5	Dense vegetation, FromTable 5.2.9
14	Catchment Wetness (CW)		1	From Table 5.2.10
15	Slope	m/m	0.055	From Calculation (row 5-row 6)/row 8
16	TRRL Factor		30	Constant factor for humid catchments
17	Base Time	h	27.5	From formula: $T_B = \frac{C \times A^{0.5}}{S^2} + T_S$
18	Design Period	Years	25	From ToR or Design Manual
19	24 h Design Rainfall	mm	364.8	Total 24 hour rainfall for design period (24 hour total), 25 years + 15% safety margin. Attapeu Station
20	Rainfall Ratio at Base time		1.06	Total rainfall at base time = 14.12 *27.5 = 388.5. 388.35 / 364.8 = 1.06..Intensity at base time has to be taken from the IDF curve graph, or from a fitted equation. Equation Attap. 25 year curve : I = 573.64*(Tc*60)^-0.5
21	Precipitation Base Time	mm	387	From Calculation row 19*row 20
22	Area Reduction Factor		0.77	Area Reduction factor from Formula: (Eqn4.4) 1 - 0.04 T^-0.33A^0.50
23	Q avg.	m3/s	135.8	From Formula:= (row22*row12*row21*row4)/(360*row17) = (0.77*37*387*122)/360*27.5 =
24	Peak		1.7	From Table 5.2.11
25	Storm water Run-off	m3/s	231	Final 25 year flow

Evaluation of Example Calculation

	Rational Method	**Modified Rational Method**	**General Tropical Flood Model**
Parameter	Run-off coefficient:0.15	Run-off coefficient:0.15, Area reduction coefficient : 0.85	Parameter as in design example shown above.
25 year design run-off	295 m3/s	250m3/s	231 m3/s

The comparison of the calculation result gained with the three methods shows that the Rational Method arrives at very high discharges, even when the run-off coefficient is rather low at 0.15.

For an optimized design a check run with all the methods is the best practice for design tasks.

5.3 HYDRAULIC DESIGN OF CULVERTS

5.3.1 Definitions and Symbols

This chapter provides 2 procedures for the hydraulic design of highway culverts:

• A quick design based on culvert capacity charts.

• The use of culvert design software such as HY8 Culvert Hydraulic Analysis Program.

A culvert is a structure that is designed hydraulically to take advantage of submergence to increase hydraulic capacity. It is also a structure used to convey surface run-off through embankments. A culvert can be a structure, as distinguished from bridges, that is usually covered with an embankment and is composed of structural material around the entire perimeter. These include steel and concrete pipe culverts and concrete box culverts. However, a culvert can also be a structure supported on spread footings with the streambed serving as the bottom of the culvert. These include some multi-plate steel structures and concrete slab culverts.

5.3.2 Principles of Design

The following principles are specific to culverts:

• All culverts shall be hydraulically designed.

• Overtopping flood selected is gene rally consistent with the class of road and the risk at the site.

• Survey information shall include topographic features, channel characteristics, high- water information, existing structures, and other related site-specific information.

• Culvert location in both plan and profile shall be investigated to avoid sediment build- up in culvert barrels.

• Culverts shall be designed to accommodate debris or proper provisions shall be made for debris maintenance.

• Material selection shall include consideration of materials availability, and the service life including abrasion and corrosion potentials.

• Culverts shall be located and designed to present a minimum hazard to traffic and people.

• The detail of documentation for each culvert site shall be commensurate with the risk and importance of the structure. Design data and calculations shall be assembled and retained for future reference.

5.3.3 Design Criteria

The following design Criteria should be observed for new culverts on important roads in Laos.

Drainage Structure Typ	**Design Storm / Flood Frequency**
Pipe culvert < 2m Diameter	25 Years
Box Culverts and short bridges ≥ 10m span	50 years
Bridges	100 years
Side drains and ditches	10 year design

As far as **Local Roads** are concerned the following standards are recommended.

Drainage Structure Type	**Design Storm / Flood Frequency**
Pipe culvert < 2m Diameter	25 Years
Box Culverts < 2m span	25 Years
Box Culverts and short bridges> 10m span	50 years
Bridges	100 years
Side drains and ditches	10 year design

- Outlet velocity should be limited to 2.5 m/s.
- Maximum water level of 0.5 m below the road shoulder to protect the pavement formation.

5.3.4 Site Criteria

The type of drainage structure specified for a particular location is often determined based on economic considerations. The following can serve as a guide in the selection of the type of structure, proceeding from the most expensive to the least expensive. Note that bridges are included in the text of this section to allow for a more complete progression in the treatment of this topic.

Bridges are used where they are more economical than a culvert, perhaps due to the need to bury a culvert under a high level of fill. They are also employed to satisfy land use requirements, to mitigate environmental harm possible with a culvert, to avoid floodway or irrigation canal encroachments, and to accommodate large debris.

Culverts are used where bridges are not hydraulically required, where debris is tolerable, and where they are more economical than a bridge. Culverts can be concrete box culverts, reinforced concrete pipe culverts, or corrugated metal culverts.

Concrete box culverts are constructed with a square or rectangular opening, and with wingwalls at both ends. They are usually specified for larger flows, where the area of the opening is larger than that available for manufactured concrete or metal pipe culverts. They may also be used where the cost estimate indicates that concrete box culverts constructed on site are less expensive than manufactured and/or imported pipe culverts. An alternative sometimes employed is to use metal arch pipe, and for larger openings this can be more economic than concrete.

Although metal pipe culverts are usually less expensive than concrete pipe culverts, a cost estimate may indicate that this is not the case.

Certain corrosive soils can create problems with metal pipes, and this would have a tendency to create a shift in favor of concrete pipes. However, the corrosive effects are mitigated through the application of bitumen coating to the metal pipes. This adds slightly to the cost of the metal pipe.

The use of headwalls and/or wingwalls with pipe culverts is generally dependent on factors such as the slope and stability of the channel. Pipe culverts can often be placed particularly on lower volume roads without headwalls or wingwalls.

5.3.5 Design Limitations

Allowable Headwater

Allowable Headwater is the depth of water that can be ponded at the upstream end of the culvert that will be limited by one or more of the following:

- will not damage up stream property,
- not higher than 500 mm below the edge of the shoulder,
- equal to an Headwater/Culvert Depth not greater than 1.5,
- no higher than the low point in the road grade.

Tailwater Relationship of Channel

- The hydraulic conditions downstream of the channel determine the tailwater depth relationship for different discharges.
- Backwater curves at sensitive locations or single cross sections should be used. For important structures several downstream cross sections are required.
- Critical depth and equivalent hydraulic grade line can be used if the culvert outlet is operating with a free outfall.
- The high water elevation that has the same frequency as the design flood if events are known to occur concurrently (statistically dependent) should be used to evaluate the influence of confluences.

Maximum Velocity and Minimum Velocity

The maximum velocity at the culvert exit shall be consistent with the velocity in the natural channel or shall be mitigated with channel stabilization and energy dissipation. It is generally recommended to limit the exit velocity to 2.5m/s.

The minimum velocity in the culvert barrel should result in a tractive force greater than critical mass of the transported streambed material at low flow rates. When streambed material size is not known 0.8 meters per second should be used as approximation.

If clogging is probable, consider installation of a sediment trap.

5.3.6 Design Features

Culvert Sizes and Shape

The culvert size and shape selected is to be based on engineering and economic criteria related to site conditions. The following absolute minimum sizes shall be used to avoid maintenance problems and clogging:

- Urban areas: Cross drainage, min.dia. 450 mm.
- Rural areas: Cross drainage min.dia 800 mm, 600 mm for access culverts.

Land use requirements can dictate a larger or different barrel geometry than required for hydraulic considerations.

Multiple Barrels

Multiple barrel culverts should fit within the natural dominant channel with only minor widening of the channel to avoid conveyance loss through sediment deposition in some of the barrels. When the approach flow is supercritical, either a single barrel or special inlet treatment is required to avoid adverse hydraulic jump effects. It is good practice to install one barrel at the flow line of the stream while other barrels are set slightly higher to reduce sedimentation.

Where ever possible double cell pipe culverts should be replaced with single cell box culverts in order to avoid the problem of piling up of debris against the ineffective middle section.

Material Selection

Concrete is the preferred material for construction of culverts, however, other materials may be more suitable for a particular location, hydraulic roughness, bedding condition, or project. In evaluating the suitability of alternate materials, the selection process shall be based on a comparison of the total cost of alternate materials over the design life of the structure that is dependent upon the following:

- Durability (service life)
- Cost
- Availability
- Construction and maintenance ease
- Structural strength
- Traffic delays
- Abrasion and corrosion resistance, and
- Water tightness requirements

A pipe material other than concrete may be accepted as an alternate if the substitution is supported by evidence that the hydraulic capacity, strength, durability, abrasion, and corrosion resistance of the concrete pipe specified is equaled or exceeded. In addition, any substitution must be analyzed in terms of cost and availability. Corrugated metal pipe, if permitted, shall be protected at the ends by headwalls. Use of corrugated metal pipes with projecting ends is not recommended.

5.3.7 Quick Culvert Design

For the design of minor drainage structures, especially in rural areas where increased culvert headwaters cannot cause great damage and in preliminary design it is usually sufficient to use culvert capacity tables for a first quick assessment of opening area requirements.

A simplistic method for the design of concrete culverts is to use capacity tables shown in APPENDIX–L, which are calculated from following Manning's formula.

$$Q = A \times V = A \times K \times R^{\frac{2}{3}} \times S^{\frac{1}{2}}$$

$$K = \frac{1}{n}$$

Where

Q= Discharge (m^3/sec)

A= Cross section of flow area (m^2)

V= Water velocity in meter per seconds (m/s)

R= Hydraulic radius = A/WP where WP is the wetted perimeter of flow area (m)

S= Longitudinal slope of flow in meter per meter (H/L)

n = Channel roughness coefficient

Roughness coefficient for Manning used is 0.015 for reinforced pipe and 0.017 for box culverts and 0.019 for slab culvert (with paved waterway at inlet, bed and outside the slab culvert). A minimum efficiency factor of 200% in accommodating the available discharge is used for both new pipe and new slab culverts.

Example 1 Pipe Culverts

Say we have a discharge of 3.38 m^3/s, and a slope of 2.5%. Using a 200% efficiency factor the discharge used for design is 7.76 m^3/s. The table has limits of slope of up to 2% since this slope is critical and slopes beyond this level are supercritical and do not pose a problem of increased headwater depth. From the table 3x120 cm pipes which could accommodate a discharge of 8.10 m^3/s are chosen.

Example 2 Box Culverts

Given discharge = 13.2 m^3/s, Slope =0.7%

The discharge is doubled for 200% efficiency, discharge = 26.4 m^3/s

From the table a box culvert of 3.0 m span and 2.0 m height is chosen which has a capacity of 26.65 m^3/s. Depending on actual geometry of crossing at site, the geometry of opening can be altered.

Typical drawings of pipe/box cuverts are shown in APPENDIX-M.

Another simplistic method for the design of concrete culverts using a diagram is available in Figure 5.3.1. the diagram for circular pipes has been drawn from Manning's formula.

Flow Chart Guidelines

The main chart (the large chart) reads discharge in liters/ second against the slope S which is variable in meter per meter against the roughness coefficient (n) of 0.013, 0.012 and 0.011. The average value of 0.012 is recommended.

The inside chat (the smaller chart) reads flow velocity in meter per second (m/s) against the inside pipe diameter in millimeter (mm) and against the main chart as noted above. As stated on the chart, by knowing any of two factors, the other relevant factors can be determined.

The chart is recommended for use in determining less critical culverts where Inlet and Outlet Control is not warranted.

Using the values from Example 4.3, Road Station km 80 + 100 values of Q= 12.1 m3/sec. and flow velocity of 2.0 m/s. Reading horizontally from 12 100 liters/sec. until flow velocity of 2.0 meters/sec. is intercepted, gives 1 No diameter 2700 mm circular culvert.

This corresponds to 5.73 m2 culvert area and which allows such alternatives as 2 Nos. 1950 mm (5.98m2), 3 Nos. 1650 mm (6.41m2).

It has to be pointed out that this method of culvert design assumes full flow in the pipe, which is not the standard condition.

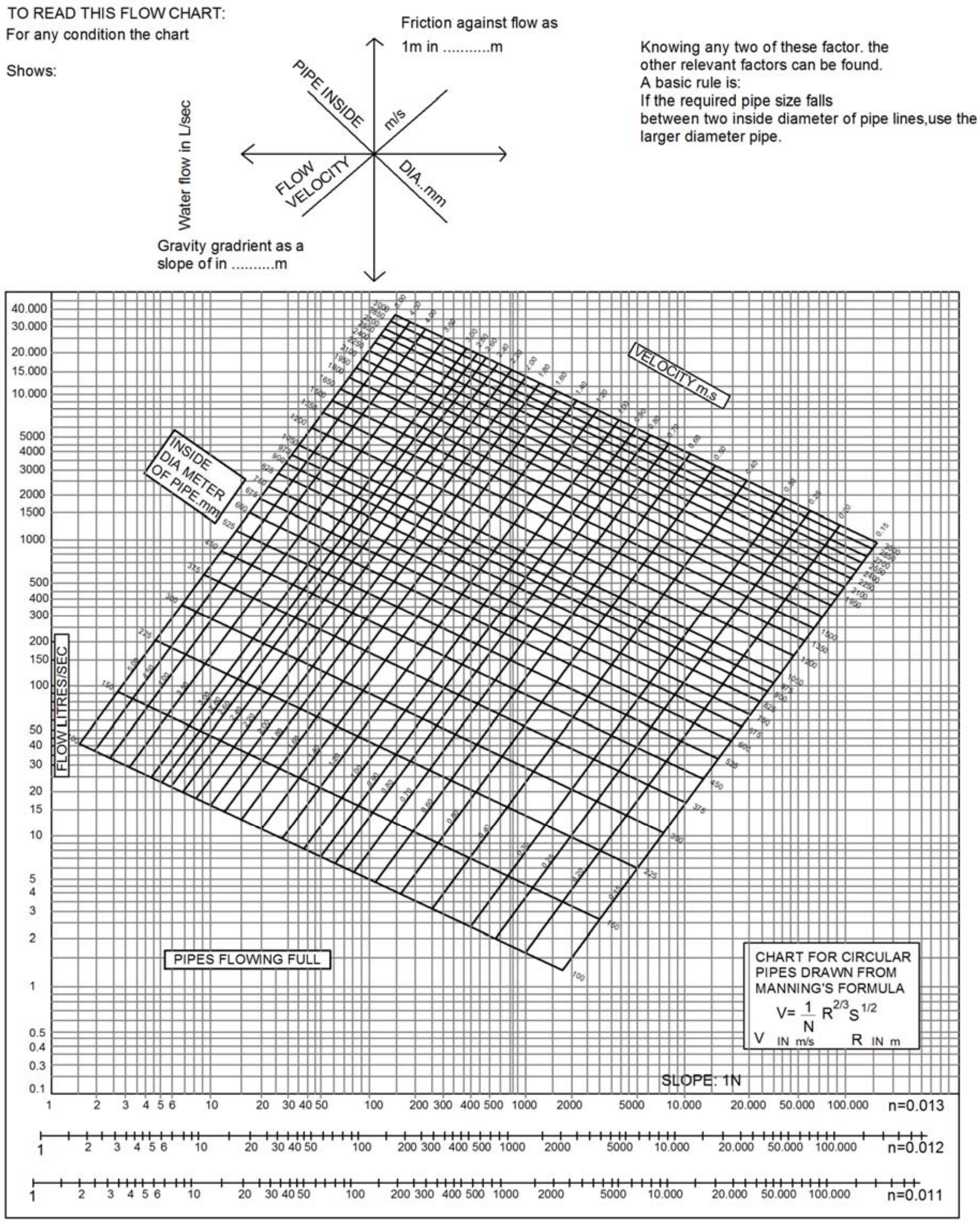


Figure 5.3.1 Culvert – Capacity Diagram

5.3.8 Detailed Culvert Design

An exact theoretical analysis of culvert flow is extremely complex because the following is required:

- analyzing non-uniform flow with regions of both gradually varying and rapidly varying flow,
- determining how the flow type changes as the flow rate and tailwater elevations change,
- applying backwater and drawdown calculations, energy, and momentum balance,
- applying the results of hydraulic model studies, and
- determining if hydraulic jumps occur and if they are inside or downstream of the culvert barrel

As culvert hydraulics are extremely complicated various software products are freely available to assist the lengthy process of culvert design as described above. The most widely used applications have been developed by the American Federal Highway Authority[1].

These computer programs are freely available on the internet and allow for easy and quick confirmation of manual calculations. For important designs the use of culvert design software is highly recommended.

Widely used applications are HY8[2] for culvert design and HEC RAS 4.1[3] for all kinds of water level calculations concerning floodplain crossings, multiple culvert calculations and bridge hydraulics.

For urban drainage network calculation the free software program EPA Storm Water Management Model (SWMM)[4] is recommended.

[1] FHWA, https://www.fhwa.dot.gov/
[2] HY8, ttps://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/
[3] HEC RAS 4.1, http://www.hec.usace.army.mil/software/hec-ras/
[4] EPA Storm Water Management Model (SWMM), https://www.epa.gov/water-research/storm-water-management-model-swmm

5.4 SIDE DITCHES AND CUT-OFF DRAINS

5.4.1 General

The types of side ditches and cut-off ditches which are recommended to be used are shown on Figure 5.4.1 while Table 5.4.1 gives guidelines regarding the choice of each particular type.

The guidelines given in Table 5.4.1 are based upon general economic and aesthetic considerations. However, the type of side ditch selected must be checked to ensure that it will carry the expected flow without running so deep as to wet the road pavement nor so fast as to cause scour.

Due to their location, cut-off ditches are usually difficult to maintain and should therefore, whenever possible, be constructed as "natural permanent depressions" with as gentle side slopes as possible.

5.4.2 Expected flow in side ditches

The side ditches must be designed to carry the stormwater run-off originating from the carriageway, shoulder, drain and cut slope. Where cut-off ditches are not provided, any run-off from beyond the cut slope must also be included.

The expected flow, or run-off, should be estimated using the formula

$$Q = 0.278 \times C \times I \times A \quad \text{m}^3\text{/sec.}$$

Where

Q= flow in cubic meters per second (m3/sec.)

C= run-off coefficient, expressing the fraction of the rainfall that is assumed to become direct run-off (see Table 5.2.1Table 5.2.2)

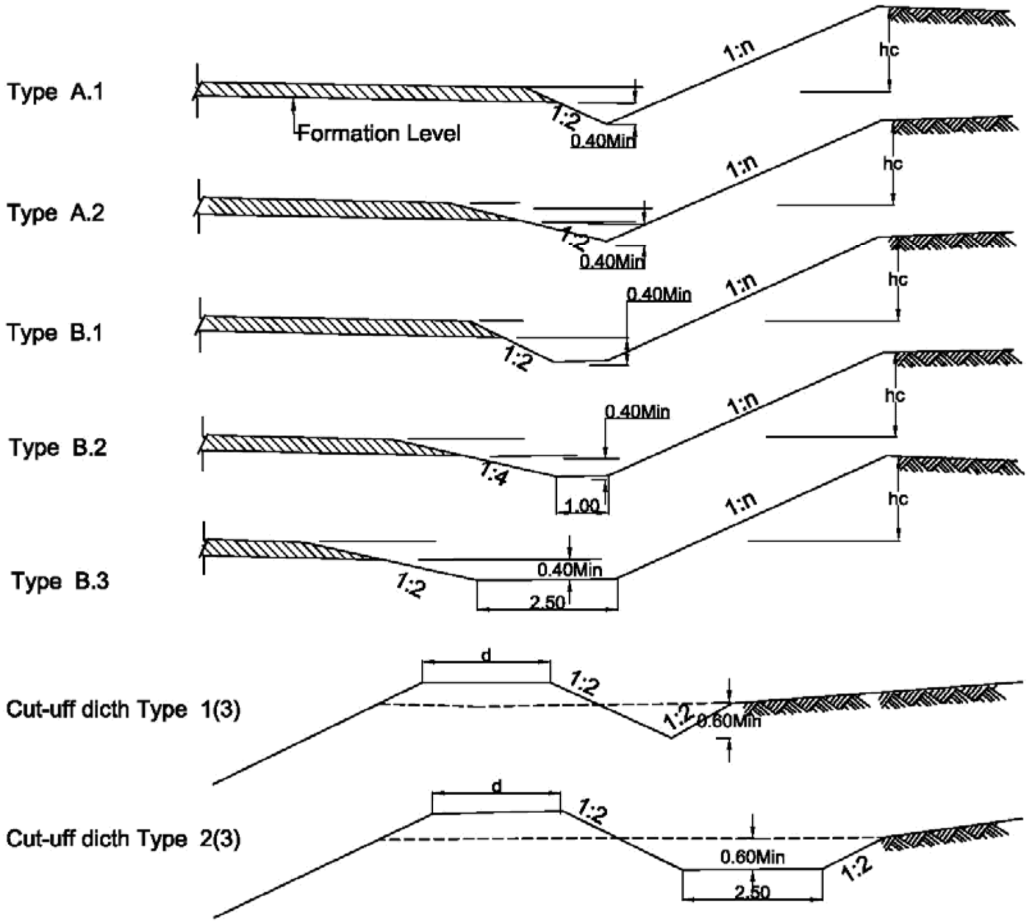
I= Intensity of rainfall in mm/hour, for a 5 or 10 minute storm with a return period of 2 years.

I= 150 mm/h is proposed.

A= the drainage catchment area in km^2.

5.4.3 Capacity of Side Ditches

The capacity of a side ditch should be estimated using the Manning Strikler formula which is shown in section 5.3.7.



Height of cut hc	Slope (1:n) of cutting in earth[1]	Distance, d, between edge of cutting and cut-off ditch
hc< 1.00	1 : 3	d = 3.0
100 <hc≤ 3.00	1 : 2	d = 4.0
hc> 3.00	1 : 1.5[2]	d ≥ 5.0

(1) In rock the slope 1 : n will vary from 1: 0.33 to 1 : 0.10
(2) If earth not stable at 1 : 1.5, the slope may be decreased to 1 : 2
If natural slope steeper than 1 : 1.5, then the natural slope may be used as the cut slope.
(3) The slopes into the ditch should be as indicated or flatter.

Figure 5.4.1 Side Ditches and Cut-off Ditches

Table 5.4.1 Guidelines for the selection of side ditch and cut-off ditch types

Side ditch type	To be used under the following condition	Remark
A1	Hilly to mountainous terrain with heavy Earthwork.	- Back slope to be varied, according to stability of cut material. Slope should be stable and enable vegetation to establish.
A2	Rolling terrain with moderate earth work. Hilly to mountainous terrain where flatter ditch than A1 is required due to capacity and/or velocity limitations.	- As for A1
B1	Hilly to mountainous terrain where a flatter ditch than A1 is required due to capacity and/or velocity limitations.	- As for A1 - Width may be increased if fill material is needed
B2	Rolling terrain with moderate earthwork where a flatter ditch than A2 is required due to capacity and /or velocity limitation.	- As for B1
B3	Flat terrain with little earthwork. Rolling terrain with moderate earthwork where a flatter ditch than B2 is required to capacity and /or velocity limitation.	- As for B2
Cut-off ditch Type	To be used under the following conditions.	Remarks
1 2	Moderate catchment area and little chance of siltation Large catchment area and in areas liable to the silting and/or damage to the ditch profile by pedestrians, cattle etc.	

Limiting values for the velocity of flow (v) to prevent scour, together with the corresponding Roughness Coefficients, are given inTable 5.4.2 for the different types of ditch material which will normally be encountered.

Table 5.4.2 Typical maximum permissible velocity in erodible ditches and corresponding roughness coefficients

Material in the ditch	Max permissible velocity (m/s)	Roughness Coefficient K
Sand, Loam, Fine gravel, volcanic ash	0.6*	45
Stiff clay	1.1*	50
Course gravel	1.5	40
Conglomerate, hard shale, soft rock,	2.0	25
Hard rock	3.0	25
Masonry	3.0	40
Concrete	3.0	60

* As a general rule, it is recommended that a Roughness factor of 30 is used, unless extensive pre-design investigations have been carried out to prove other factors. To facilitate the rapid calculation of side ditch capacities, Figure 5.4.2 and Figure 5.4.5 are given. These figures show, for each side ditch type, how the capacity varies for different ditch gradients, roughness, coefficients, velocities and depths of flow.

5.4.4 Scour Protection

It is important to note that a side ditch will only perform as designed if the design cross-section is maintained, i.e. excessive scour must be prevented. In practice, due to local inconsistencies in roughness and surface level, no side drain in any but the hardest of materials will be immune from scour. Thus, for long lengths of side ditch at gradients in excess of 4-5%, scour checks should be considered.

5.4.5 Precast Product

For minimizing material and promoting material recycling, applying precast products should be considered in the design.

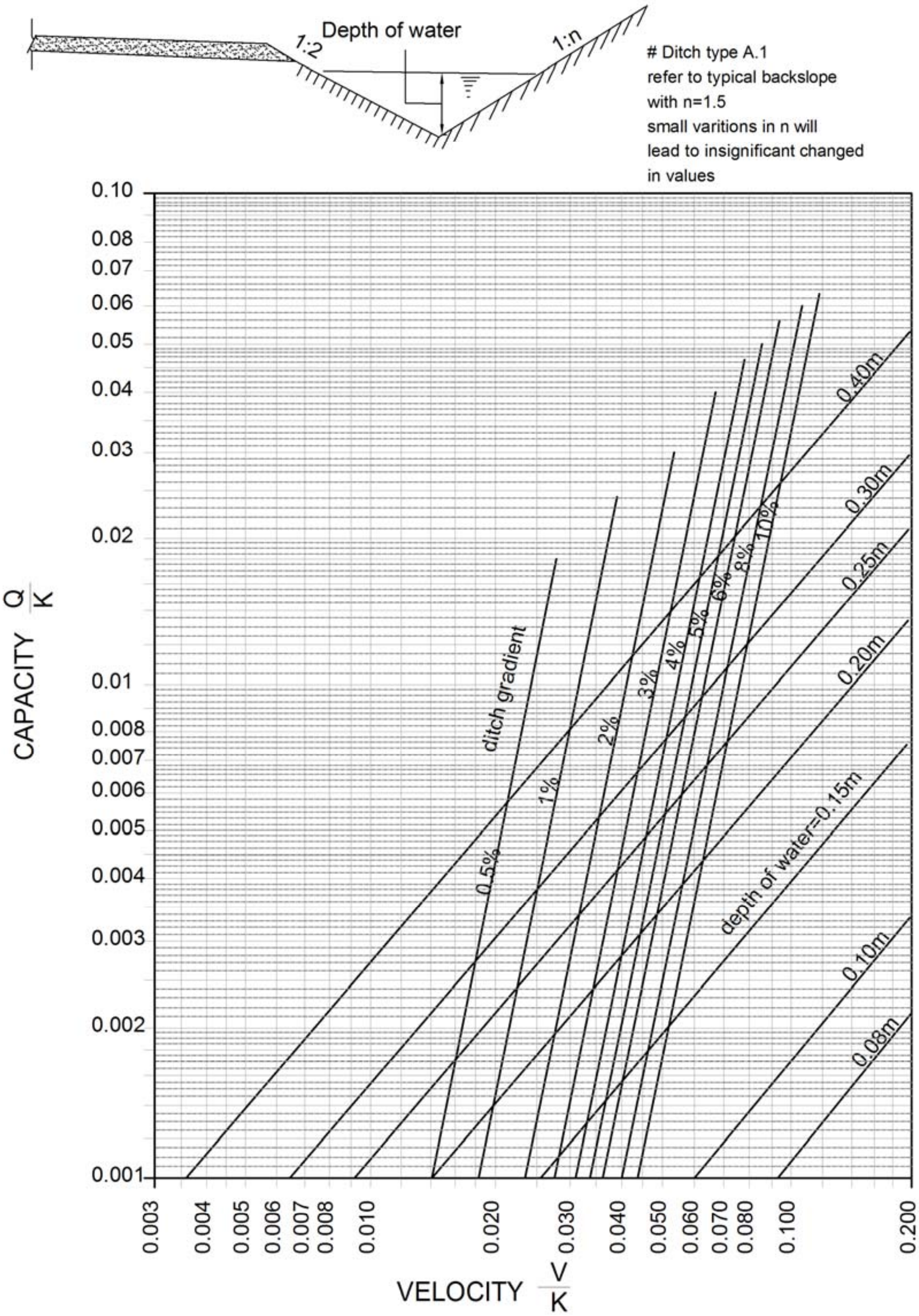


Figure 5.4.2 Capacity of Side Drain Type A.1

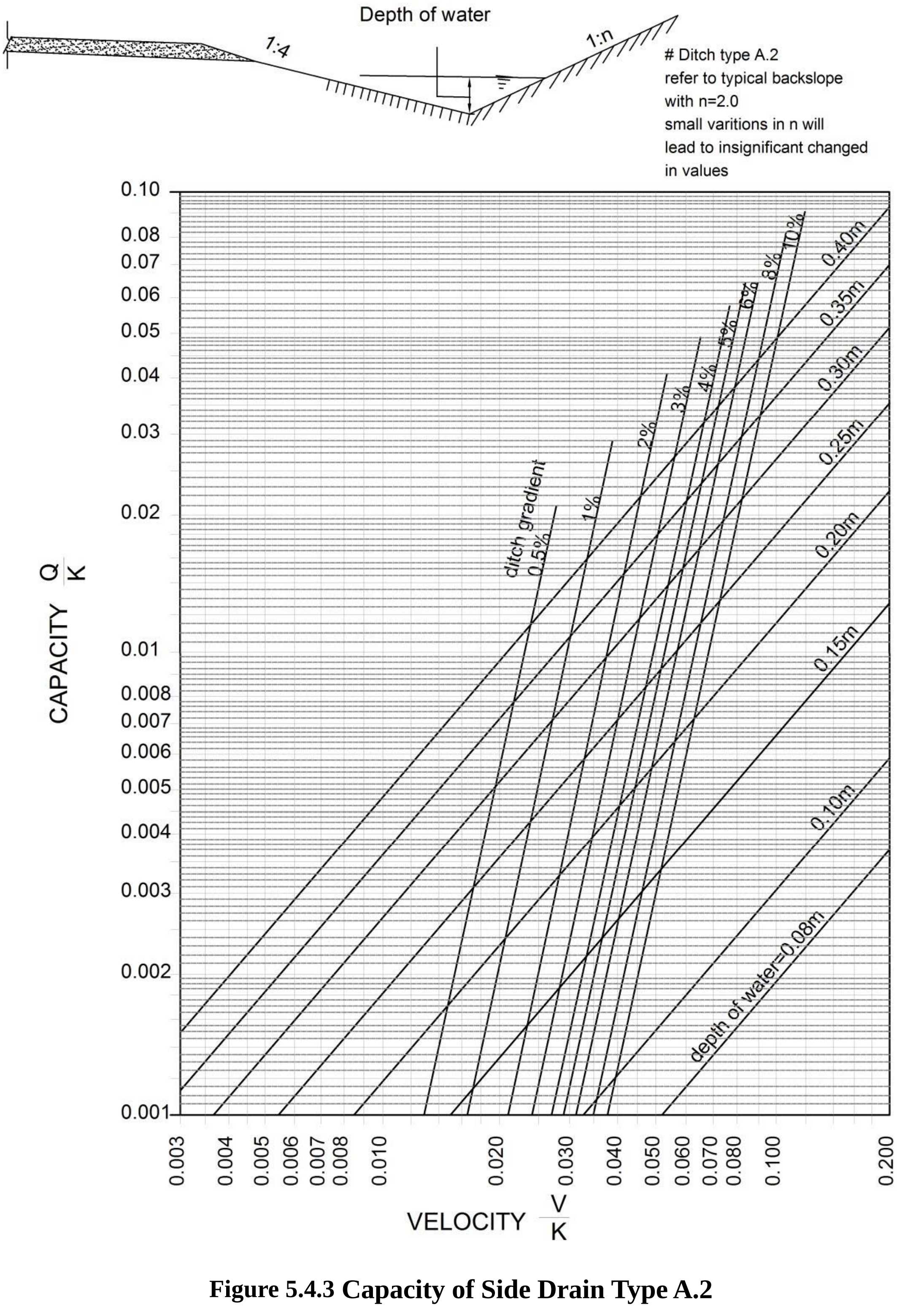


Figure 5.4.3 Capacity of Side Drain Type A.2

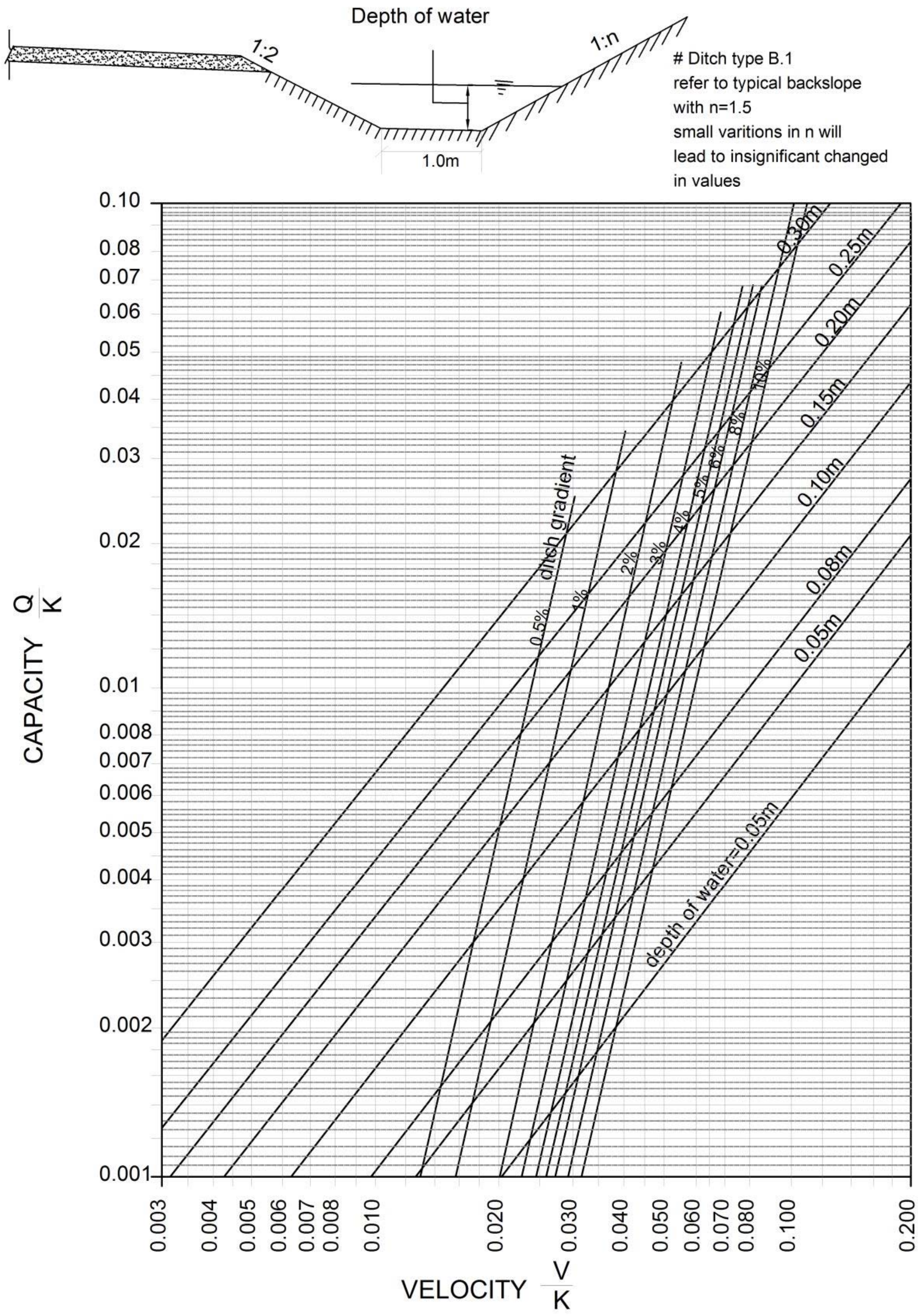


Figure 5.4.4 Capacity of Side Drain Type B.1

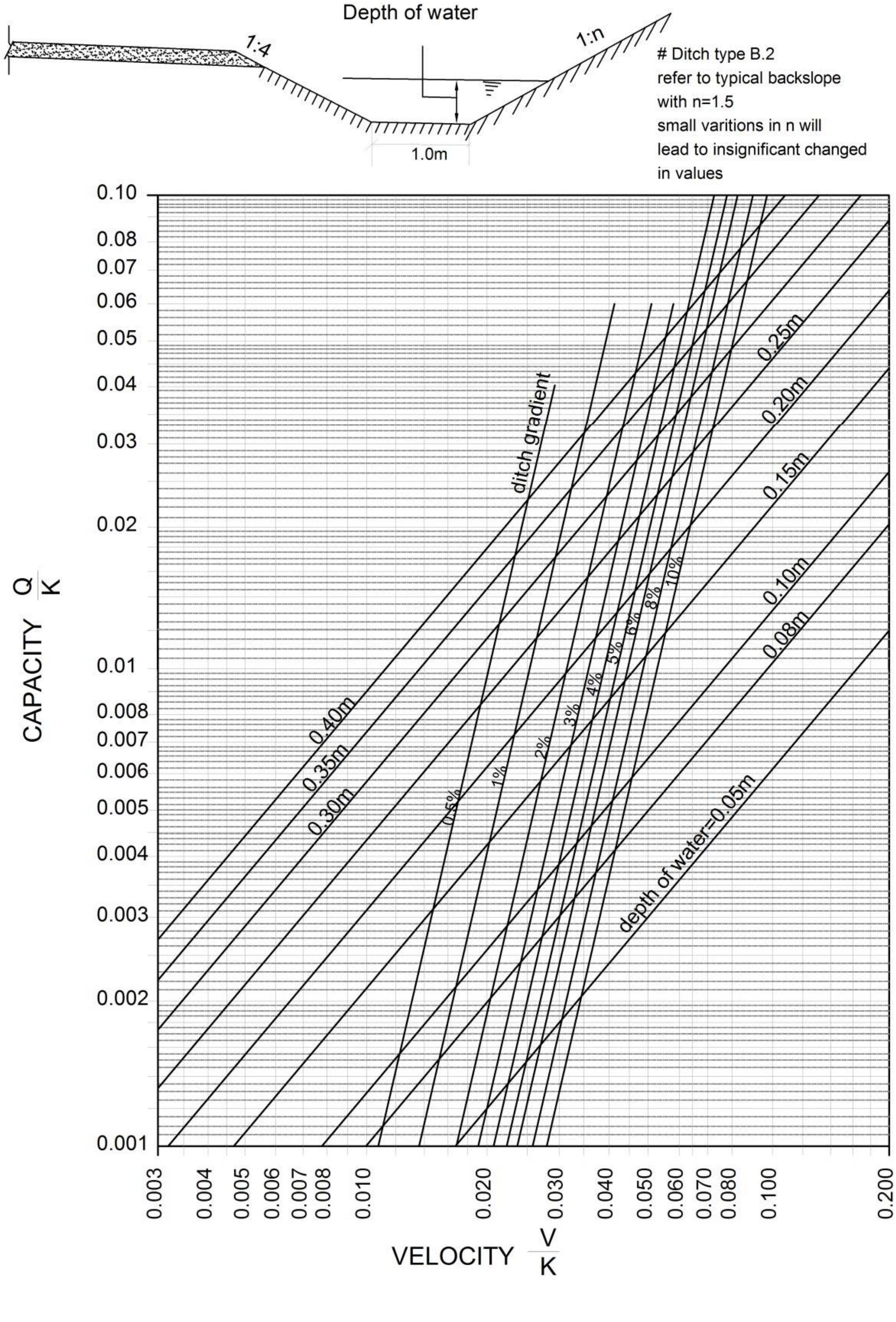
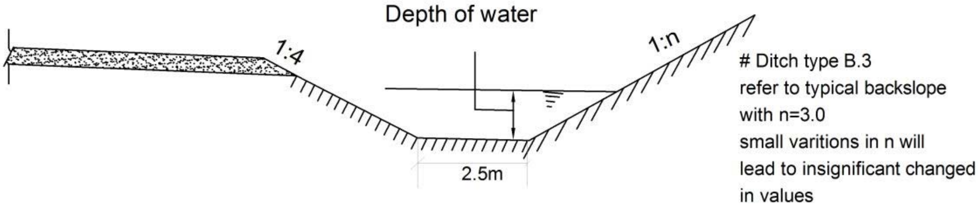


Figure 5.4.5 Capacity of Side Drain Type B.2



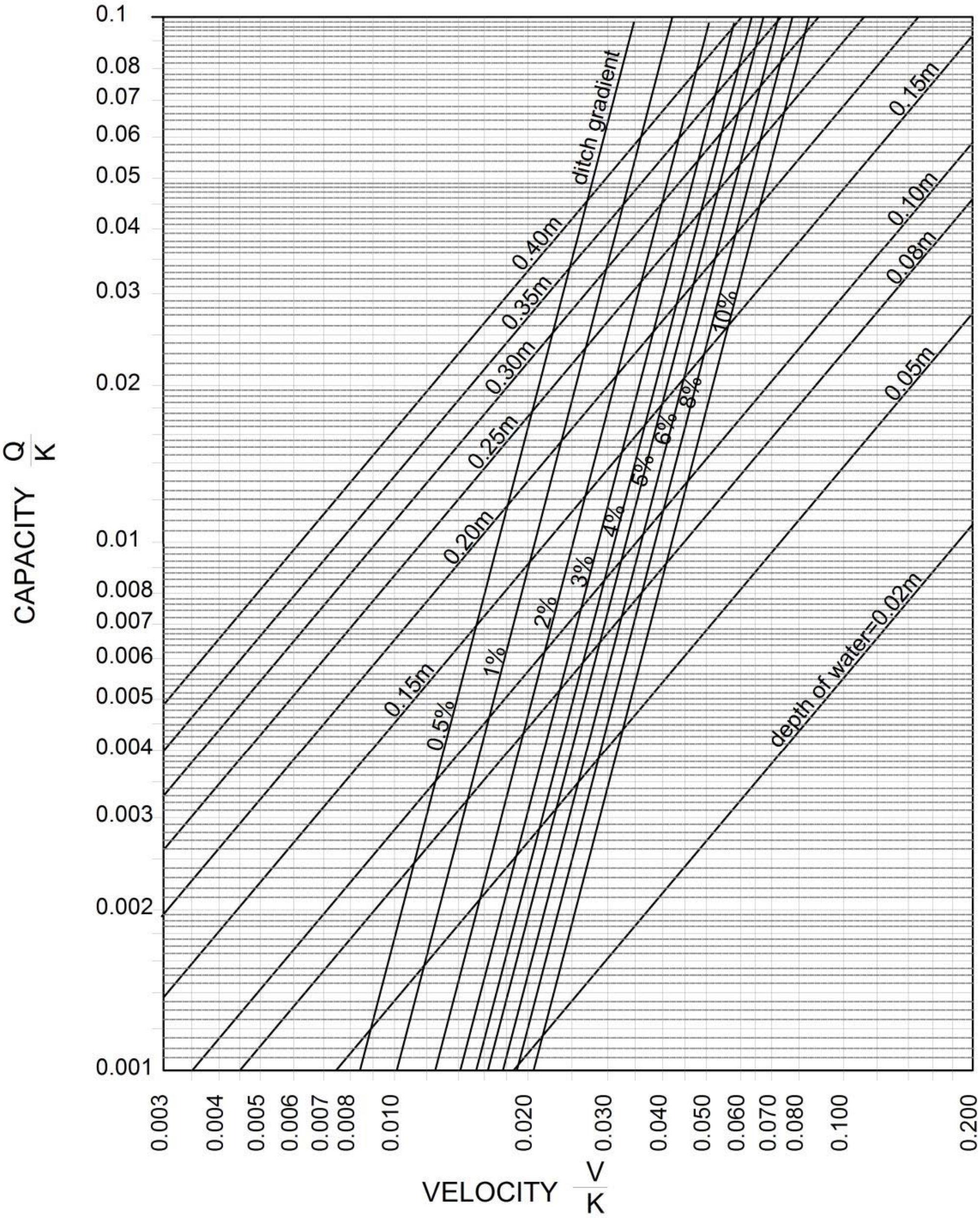


Table 5.4.3 Capacity of Side Drain Type B.3

5.5 CLIMATE CHANGE ISSUES CONCERNING ROAD DRAINAGE DESIGN

The hydrological techniques discussed in the hydrological section of this design manual are based on Extreme Rainfall Return Periods and Intensity – Duration – Frequency – (IDF) Curves. One basic assumption of the statistical methods applied for calculating extreme events is that the data used for these calculations is random and trend free. Only by accepting these assumptions is it possible to predict future climatic conditions on the basis of observed past experience and historic data records.

However, at meteorological stations all over Asia certain trends and increased variation of maximum temperatures, rainfall intensities and the spatial distribution of rainfall events have been observed in recent years. It seems that there is an observed increase in the number of extreme events, such as cyclones.

Although these developments cannot yet be analysed in detail due to insufficient data there is a consensus amongst experts that climate change is happening in Laos and that considerable changes in the rainfall regime are to be expected. Overall predictability of climatic events has decreased and the element of insecurity has increased, as reported by various international publications.

The 2011 World Bank Climate Change Country Profile for Laos (WB-GFDRR- 2011) reports of possible increase of annual rainfall up to 30% for southern and eastern areas of Laos, based on regional climate change models.

A publication by the USAID funded Mekong - ARCC (Mekong Adaptation and Resilience to Climate Change) indicates an increase of annual precipitation of 8 to 18% in the region of Khammouan. The publication also stresses the increasing occurrence of extreme intensity rainfall events.

The hypothesis of Climate Change throws doubt on the validity of the existing and established techniques. Basically, the statistical population may be changing, and therefore it becomes unreliable to predict future probability of storm events on historical records from an earlier climatic period. The risk to roads in Laos is that drainage provision that was satisfactory in the past may not be adequate in future.

As a consequence the following measures are recommended for dealing with the issue of Climate Change for road drainage design:

- Adding a 15 % increase to calculated 10, 25, 50 or 100 year return storm events in order to react to increases in short duration rainfall intensities.
- Careful review of IDF curves produced prior to 1990, which most probably used data from the 1960s. In case of doubt newer IDF curves are to be used, even is records are of fewer years.
- Integration of recent flooding events in project areas, even if specific data of these events has not been integrated into the official climatic database.
- Prioritizing flood safety over short term economic savings, if alternative road alignments are available for the same connection. This might include abandoning established flood vulnerable road alignments in favour of high investment flood secure options.

CHAPTER 6 RIVER BRIDGES DESIGN

6.1 GENERAL

The objective of this manual is to provide a practical help for the engineers engaged in bridge planning in Laos by explaining the requirements to be met by the bridge. This manual provides explication on the requirements for planning of the bridges while classifying them into requirements whose compliance is mandatory and those whose compliance is advisable, and arranging them in line with the review procedure.

6.2 CONCEPT OF ABUTMENT AND PIER LOCATIONS

6.2.1 Basic Concept

(1) Prerequisite

The prerequisite for reviewing the abutment and pier locations is that the following points are already made clear through the review of Section 1:

- Bridge overhead clearance higher than the height of H.W.L + freeboard
- Necessity of river crossing
- Longitudinal and horizontal alignment of road and railway
- Method to secure the administration road
- Recommendations that could not be met and the validity
- Necessity of effects analysis through hydraulic model experiment, numericalanalysis, etc.

This section describes the concept of decision of abutment and pier locations, in which the substructure of bridge constructed on the dike will be an abutment while the substructure provided within the flow section will be the piers.

(2) Review procedure

In this review on the location of abutments and piers, four stages shown below will be taken:

- Necessity of abutments and piers (see 6.2.2)
- Determination of the abutment location (see 6.2.4)
- Determination of the pier location (see 6.2.5)
- Determination of the necessity of hydraulic review

In the review on abutment and pier locations, two ambivalent items are often compared, such as minimizing the number of piers to minimize the effects of pier construction on the river channel, resulting in adverse effect on the bridge economic efficiency due to increase in the span length.

The standard review flow is shown Figure 6.2.1.

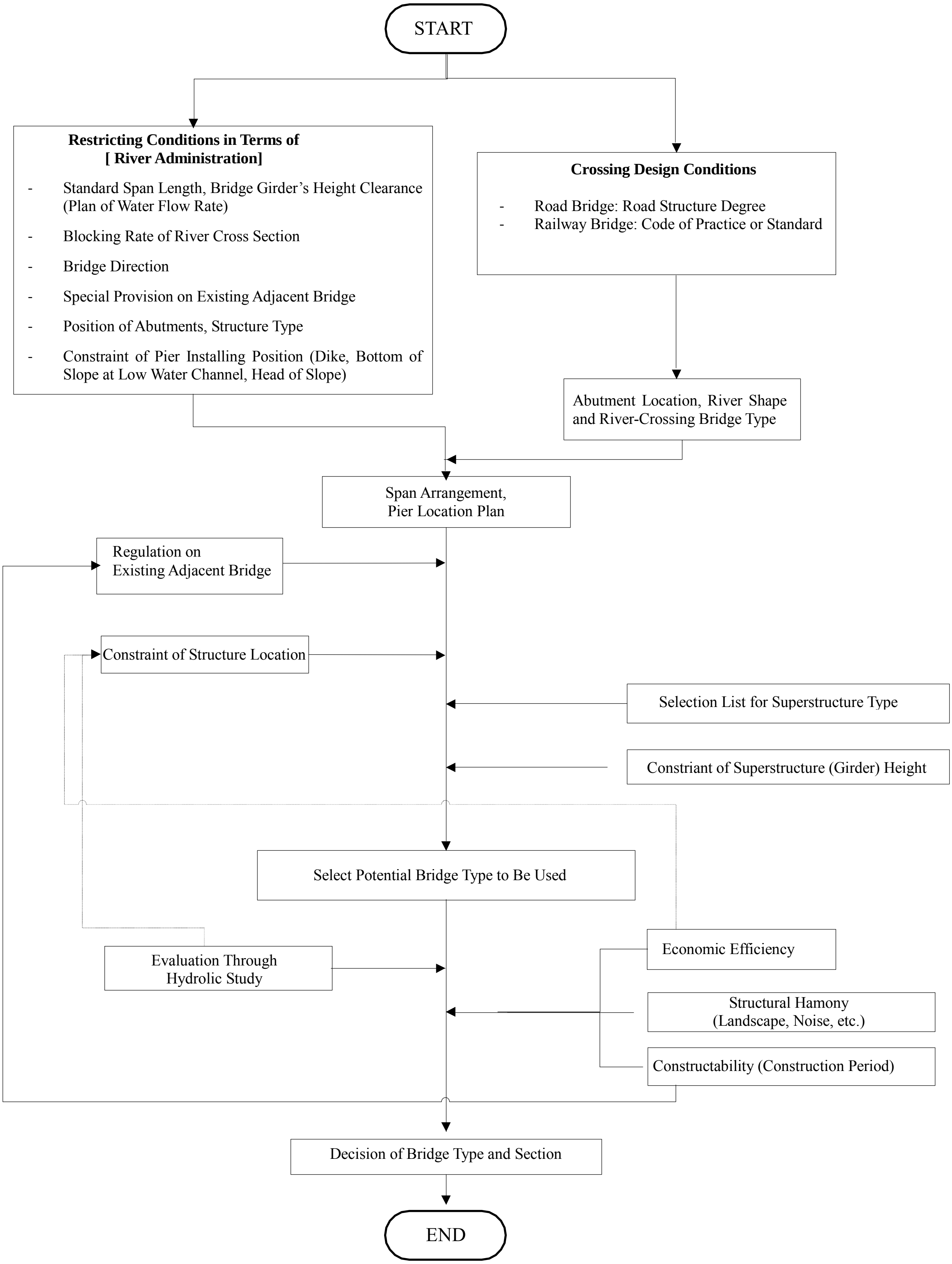


Figure 6.2.1 Determination of Pier & Abutment Location Flow

6.2.2 Necessity of Abutments and Piers

[Recommendation]

・Abutments and piers should not be provided within the river area as much as possible.

Generally, provision of abutment means insertion of foreign materials (concrete, etc.) into the bank area that is made up mostly from soils, which is considered possibly to be a weak point during floods. In addition, piers may reduce the flow section (the river cross-section) during floods, causing increase in the flood stage. Driftwood may also pile up to cause heading-up, which, together with deep scour of river bed, may inflict adverse effects on the neighboring dike. Bridging without constructing the abutments or piers, if possible, would be the more desirable plan. It is advisable therefore to review, on the basis of previous alignment plan, if bridging is possible without constructing the abutments and piers.

As an example, Table 6.2.1 shows the case without need of abutments and piers and the case requiring construction of abutments and piers. It is advisable to review the necessity of abutments and piers while referring to the example. Particularly important for the necessity of piers is how the span arrangement should be considered. As shown in References 1 and 2, the cases with the span length exceeding 100 to 150 m are limited in number and such span length is employed only under special conditions (like a bridge crossing a strait) and is difficult to employ in terms of cost.

Table 6.2.1 Cases with/without abutments and piers

	[Cases without abutments and piers]	[Cases with abutments and piers]
Abutments	Longitudinal alignment allows provision of abutment outside the river area and provision of girders at a location higher than the design dike height. Abutment in river is not necessary in terms of pier arrangement.	Intersection with the crown road is necessary, which makes it necessary to provide abutment on the design dike.
Piers	The river width is so narrow that piers are not to be provided from standpoint of bridge technology.	The river width is so wide as to make span arrangement indispensable from standpoint of bridge technology.

On the basis of review on the necessity of abutments and piers, the combination of the necessity of abutments and piers may be classified into four cases as shown in Table 6.2.2. Among them, the case in which neither abutments nor piers are necessary does not require hydraulic review and also the review on the location of abutments and piers as described below.

Table 6.2.2 Combination of cases with and without abutments and piers

Abutments	Piers	Hydraulic review	Remarks
Without	Without	Not necessary	
	With	Necessary	
With	Without	Necessary	Hydraulic review not necessary when water flow does not act directly on the abutment.
	With	Necessary	

Reference-1

Table 6.2.3 Railway Bridge: Span length and girder height by bridge type (reference)

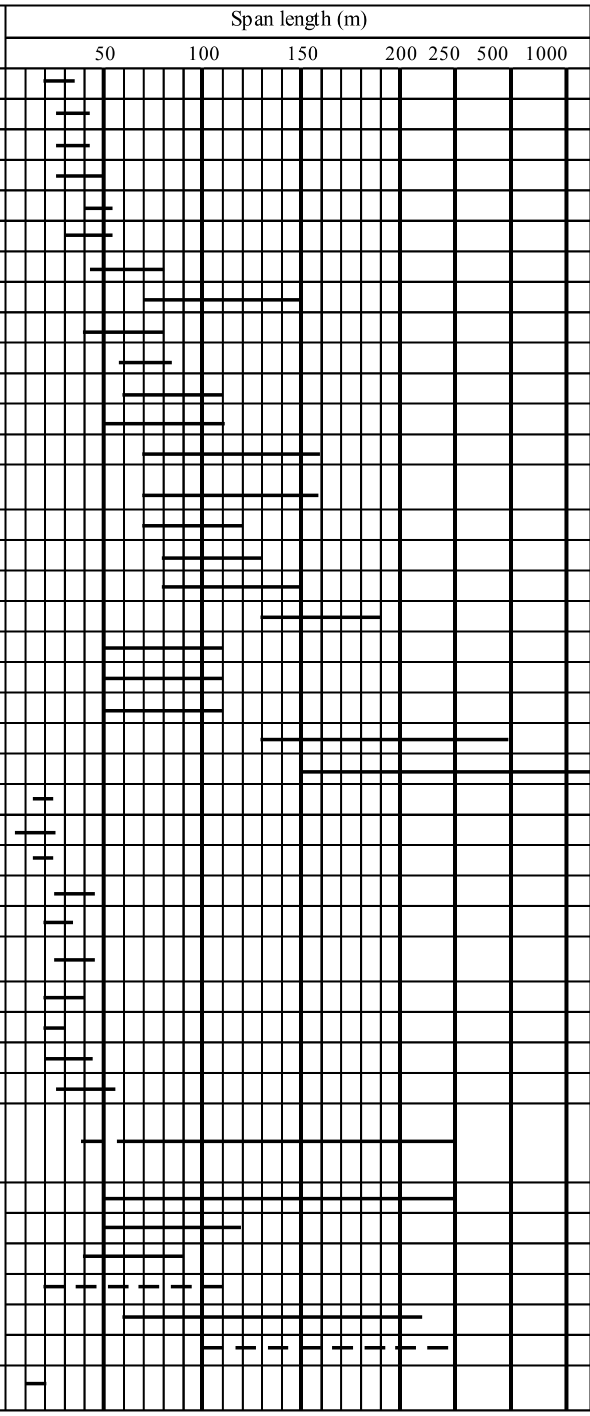
				Applicable Span Length（Proposal） 20m 40m 60m 80m 100m 120m	Top of Slab ～ Superstructure Bottom (Proposal)
Concrete Girder	RC Girder			1)	L/9～L/15 1)
	PC Box Girder			1)	L/12～L/18 1)
	PC T Girder			1)	L/10～L/18 1)
	PC Girder (Slab at Bottom of Girder)			1)	0.6～0.7m 1)
	PC Cable Stayed Bridge (Slab at Bottom of Girder)			2)	1.5 m 2)
Steel Girder	Steel Plate Girder	i Section	Slab at Bottom of Girder	3)	0.8m 5)
			Slab at Top of Girder	3)	L/10～L/19 7)
		Box Section	Slab at Bottom of Girder	3)	0.8m 2)
			Slab at Top of Girder	3)	L/10～L/19 4)
	Concrete Slab	Non-Composite Girder		3)	(L/20～L/25)+ Slab Thickness 8)
		Composite Girder	I Section Girder	3)	(L/20～L/25)+Slab Thickness 3)
			Box Section Girder	3)	
	Truss		Slab at Bottom of Girder	3)	(1.2～1.5m)+(0.25～0.3m) 3) Secondary Girder Height Concrete Slab
			Slab at Top of Girder	3)	L/6～L/8（Main Girder Height/Span) 2)
	Arch		Slab at Bottom of Girder	3)	(1.2～1.5m)+(0.25～0.3m) 6) Secondary Girder Height Concrete Slab
			Slab at Top of Girder	3)	L/6（Arch Rise/Span) 2)
Steel-Concrete Composite Structure (SRC)			Slab at Bottom of Girder	3)	0.6～0.8m（Double Track) 3) Secondary Girder Height Concrete Slab Cover
			Slab at Top of Girder	3)	(L/25 ～ L/30)+(0.25 3)～ 0.3m) Secondary Girder Height Concrete Slab Cover

1) Refer to Drawings and Structural Planning Guidance of Railway Technical Research Institute
2) Refers to Case Study from Structural Planning Guidance of Japan Railway Construction Public Corporation
3) Refers to Structural Planing from Japan Railway Construction Public Corporation
4) Refers to Existing Design Drawing List of Steel Bridge Design Document
5) Considering similary to Box Section- Slab at Bottom of Girder (Steel Plate Girder)
6) Considering similary to Composite Box Section- Slab at Top of Girder (Steel Plate Girder)

Reference-2

Table 6.2.4 Road bridge : Span length and girder height by bridge type (reference)

Span length (m): 50, 100, 150, 200, 250, 500, 1000



Type					Girder height
Steel bridge	Plate girder system	Simple type	H-beam bridge		1/25
Steel bridge	Plate girder system	Simple type	Non-composite plate girder bridge		1/18
Steel bridge	Plate girder system	Simple type	Composite plate girder bridge		1/18
Steel bridge	Plate girder system	Simple type	Non-composite box girder bridge		1/20
Steel bridge	Plate girder system	Simple type	Composite box girder bridge		1/20
Steel bridge	Plate girder system	Continuous type	Non-composite plate girder bridge		1/18
Steel bridge	Plate girder system	Continuous type	Non-composite box girder bridge		1/23
Steel bridge	Plate girder system	Steel slab girder bridge			Plate Garder 1/25 Box girder 1/27
Steel bridge	Rigid frame bridge				-
Steel bridge	Truss system	Simple truss bridge			1/9
Steel bridge	Truss system	Continuous (cantilever truss) bridge			1/10
Steel bridge	Stiffened arch system	Deck bridge	Langer girder bridge		-
Steel bridge	Stiffened arch system	Deck bridge	Lohse girder bridge		-
Steel bridge	Stiffened arch system	Half-through bridge	Lohse girder bridge		-
Steel bridge	Stiffened arch system	Through bridge	Langer girder bridge		-
Steel bridge	Stiffened arch system	Through bridge	Trussed Langer girder brodge		-
Steel bridge	Stiffened arch system	Through bridge	Lohse girder bridge		-
Steel bridge	Stiffened arch system	Through bridge	Nielsen Lohse girder bridge		-
Steel bridge	Arch system	Deck Half-through Through bridge	Solid sub-arch bridge		-
Steel bridge	Arch system	Deck Half-through Through bridge	Braced rib arch bridge		-
Steel bridge	Arch system	Deck Half-through Through bridge	Tied arch bridge		-
Steel bridge	Cable-stayed bridge				-
Steel bridge	Suspension bridge				-
Prestressed concrete bridge	Precast girder erection	Pre-tension	Simple bridge	T girder	1/18
Prestressed concrete bridge	Precast girder erection	Pre-tension	Simple bridge	Slab girder	1/24
Prestressed concrete bridge	Precast girder erection	Pre-tension	Continuous girder bridge	I (slab) girder	1/18
Prestressed concrete bridge	Precast girder erection	Post-tension	Simple bridge	T girder	1/18
Prestressed concrete bridge	Precast girder erection	Post-tension	Simple bridge	Composote I garder	1/15
Prestressed concrete bridge	Precast girder erection	Post-tension	Composite bridge	Composote I garder (composite slab)	1/13-1/25
Prestressed concrete bridge	Precast girder erection	Post-tension	Continuous girder bridge	T girder	1/18
Prestressed concrete bridge	Erection with staging		Simple bridge Continuous gader bridge	Hollow slab	1/20-1/25
Prestressed concrete bridge	Erection with staging		Simple bridge Continuous gader bridge	2-main girder	1/16-1/22
Prestressed concrete bridge	Erection with staging		Simple bridge Continuous gader bridge	Box girder	1/16-1/18
Prestressed concrete bridge	Cantilever erection		Continuous (With hinge) Rigid frame bridge	Box girder	Support 1/16-1/20 Center 1/30-1/40
Prestressed concrete bridge	Others		Arch bridge	Hollow slab T (slab) girder Box girder	-
Prestressed concrete bridge	Others		Truss bridge	Hollow slab T (slab) girder Box girder	-
Prestressed concrete bridge	Others		Rigid frame bridge	Hollow slab T (slab) girder Box girder	-
Prestressed concrete bridge	Others		Stressed ribbon bridge	Hollow slab T (slab) girder Box girder	-
Prestressed concrete bridge	Others		Cable-stayed bridge	Hollow slab T (slab) girder Box girder	-
Prestressed concrete bridge	Others		Suspension bridge	Hollow slab T (slab) girder Box girder	-
PC bridge	Hollow slab bridge				1/20

(Note) ▬▬ : Generally applied range

** Standard applicable span table (From the Road design procedure, Design of Chubu Regional Development Bureau; Partially modified*

6.2.3 Vertical Clearance Under Bridge Girder

Vertical clearance under bridge girder must be higher than the height of H.W.L + freeboard to secure safe alighment of road. The characteristic H.W.L and freeboard of the targeted river must be used in bridge plan as shown in Table 6.2.5. Since girder height varies from bridge type as shown in Reference-1 and Reference-2, the larger value of 2 m or 1/10 of the bridge length can be assumed as the girder height if the bridge type is undecided. However, it must be checked whether if the clearance is higher than the required height after the bridge type is fixed.

Table 6.2.5 Required Freeboard Under Bridge Girder

Design peak flow rate (m^3/s)	Freeboard (m)
Less than 200	0.6
200 to 500	0.8
500 to 2,000	1.0
2,000 to 5,000	1.2
5,000 to 10,000	1.5
More than 10,000	2.0

Navigation clearance is another criterion apart from the freeboard which indicates necessary space required for such as drifted logs or debris flown during the event of flooded river. It is required to discuss with Ministry of Public Works and Transport, Department of Water Way.

6.2.4 Determination of the Abutment Location

[Prerequisite]

The abutment must not be provided in the flow section (not higher than the design high-water level) for the river of 50 m or more in width, backwater section, or high-water section.

The abutment must not be provided on the surface side of river from the top of the dike front slope in sections other than above.

The surface of upper side of abutment must be parallel to the dike normal line.

The bottom of abutment must be anchored to the dike ground.

[Recommendation]

Piers must not be provided in the dike.

(1) Location of the front side of abutment

The location of abutment provided on the dike is specified as follows for the river width of 50 m or more and the river case of less than 50 m as follows:

River width of 50 m or more, backwater section, high-water section

: The structure must not be ahead of the intersection point (high-water normal line) between the slope and HWL.

River width of less than 50 m

: The structure must not be ahead of the dike normal line.

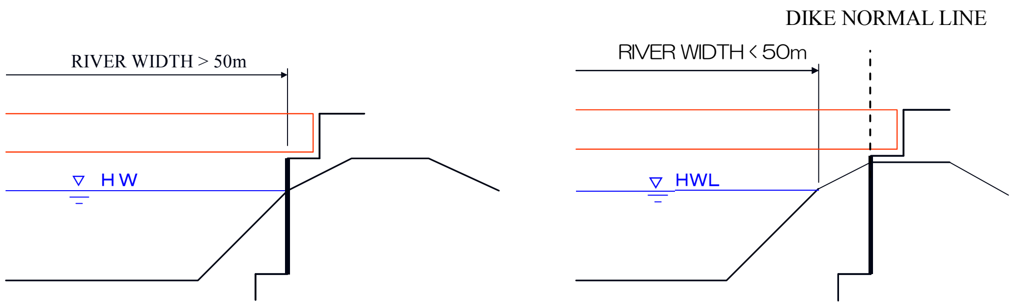


Figure 6.2.2 Image of Abutment Installing Position

(Left Figure: River Width > 50m, Right Figure: River Width < 50m)

(2) Direction to provide the abutment

In principle, the river-side surface of abutment provided on the dike is specified to be parallel to the dike normal line.

However, the application of the above stipulation may be precluded by taking such measures as dike reinforcement through dike back-slope widening, etc. shown in Figure 6.2.3 as “a measure necessary to prevent excessive interference to the dike structure,” in the following cases; for a skew bridge with small angle of skew and the dike normal line not parallel with the center line of stream at high-water, or when the abutment not parallel with the dike normal line has to be provided by necessity.

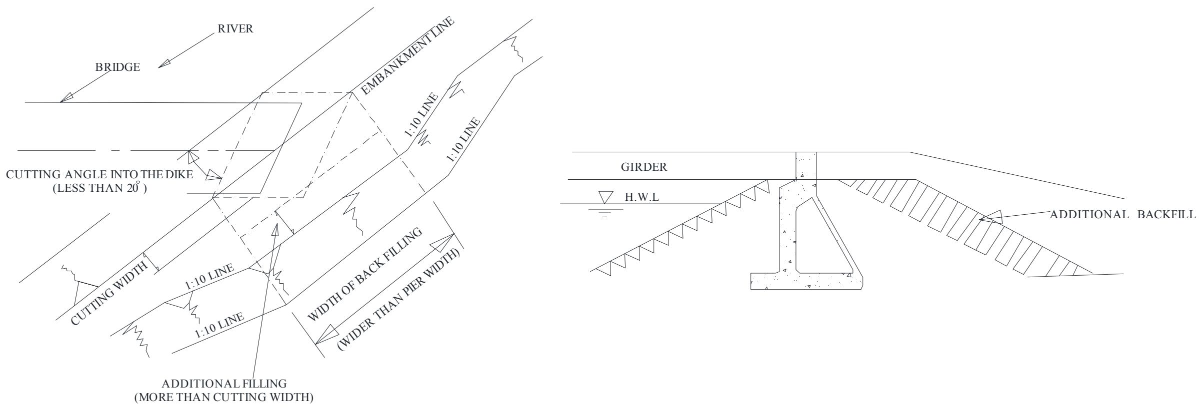


Figure 6.2.3 Reinforment For Cutting into Dike Location

(3) Bottom of abutment

The bottom of abutment provided on the dike must be anchored onto the dike ground. Apart from prohibition on the certain bottom installation height, the abutment with pile bent foundation must not be provided in the dike (see Figure 6.2.4)

Specific stipulations are shown below:

The abutment bottom to be provided in the embankment of the river must not be higher than the dike ground height.

The “dike ground height” is assumed as the height connecting the foot of riverside slope and that of back-slope as shown in Figure 6.2.4. For the excavated channel, this is assumed to be a line connecting a location having the width equivalent to the dike crown with the foot of riverside.

The abutment bottom may be below the ground (rock mass) when the ground is rock mass and can be clearly segregated from the dike ground.

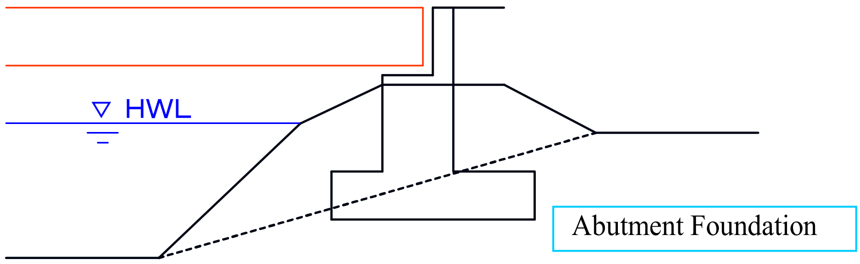


Figure 6.2.4 Image of Abutment Foundation Position

Provision of pile bent foundation in the dike is prohibited as the abutment structure because of the two reasons below. Among them, a. is considered the most important reason. Insertion of foreign materials into the earth dike that is uniform by nature as a whole may cause substantial defects because they do not behave in a unified manner respectively.

a. During earthquake, the massive objects behave in a unified manner with the dike body, but the flexible objects develop substantial displacement.

b. Piling is considered to develop crack in the tension area produced near the top of slope, but their detection and perfect treatment are extremely difficult.

(4) Piers not to be provided in the dike

In principle, the piers are not to be provided in the dike because of following reasons;

In the case of an elevated bridge, there is no need of providing piers in the dike, which is different from the abutment.

The dike and piers differ in the traffic-induced vibration in the normal state and in earthquake-induced vibration, causing the formation of a gap in the contact surface between the dike and piers, resulting in water leakage.

Note that, if unavoidable, provision of the piers abut not exerting adverse effects on the dike such as casing pipe structure is allowed only if dike reinforcement is done on the back-slope side.

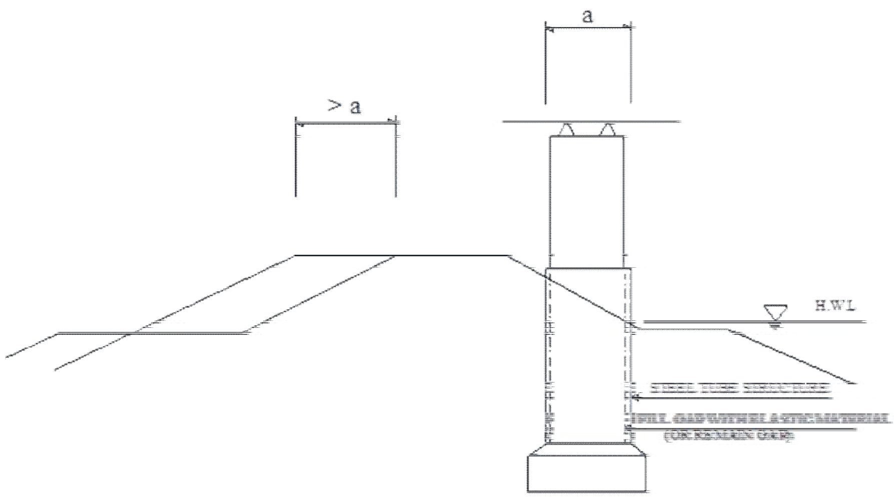


Figure 6.2.5 Example of Piers Abut (Casing Pipe Structure)

6.2.5 Determination of the Pier Location

[Prerequisite]

The piers must be provided in locations not causing adverse effects on the dike or the toe of slope of low-water bank slope foot.

The span length of bridge must be no less than the standard span length.

[Recommendations]

For the bridge to be provided in the vicinity of an existing bridge, the new piers should be provided on the line of sight of piers of the existing bridge.(Special rule for vicinity bridge)

The pier's location should be distanced by 10 m or more from the low-water bank or dike slope toe and the top of bank slope shoulder of low-water channel.

The piers located in the low-water channel or within 20 m from the top of bank slope of high-water channel should have a foundation crown height lowered by 2 m or more than the design river-bed height (or the deepest river-bed height).The crown height of the foundation for piers located in high-water channel other than above should be lowered by 1 m or more than the design high-water channel height.

The direction of piers to be provided should be parallel to the flood flow direction during flood of a design scale.

The horizontal section of piers should be of a slender ellipse or other similar shapes as much as possible.

As listed above, the pier location must secure the span length exceeding the standard span length and meet restrictive conditions concerning the location. On the other hand, the span length must fall into a range possible with the bridge type selected in terms of the economic efficiency, workability, etc. For the standard span length by bridge type, refer to References 1 and 2.

Generally, multiple plans appropriate to various restrictive conditions are chosen, which are evaluated comprehensively to determine the arrangement of piers.

In such an event, the aerial photos of existing floods at the existing bridge, if available, will enable understanding of the flow direction during floods by means of vector analysis. This in turn enables selection of the pier location with minimum adverse effects on the river while taking into account the area (other than the main channel) not so easily influencing the flow during floods and the direction of piers to be provided that matches the flow direction.

(1) Conditions concerning the span length between piers

For piers to be provided, the span length must as a rule, be determined so that it becomes longer than the "standard span length."

1) Definition of the span length

The span length of bridge between piers and between the piers and abutment is defined as follows.

- Center distance between piers in neighboring channels (distance up to the parapet wall surface in the case of abutment)
- Center distance between piers (for the skew bridge) in neighboring channels projected on a vertical plane crossing the river at a right angle relative to the flow direction during flood.

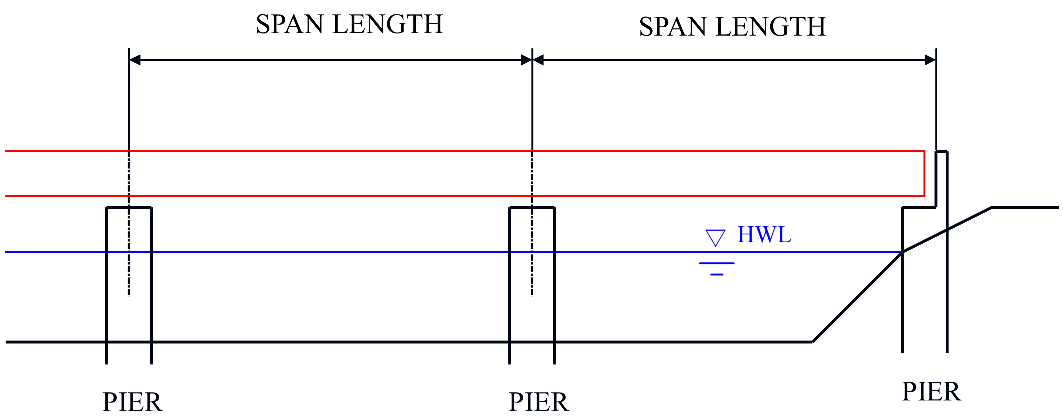


Figure 6.2.6 Definition of Span Length

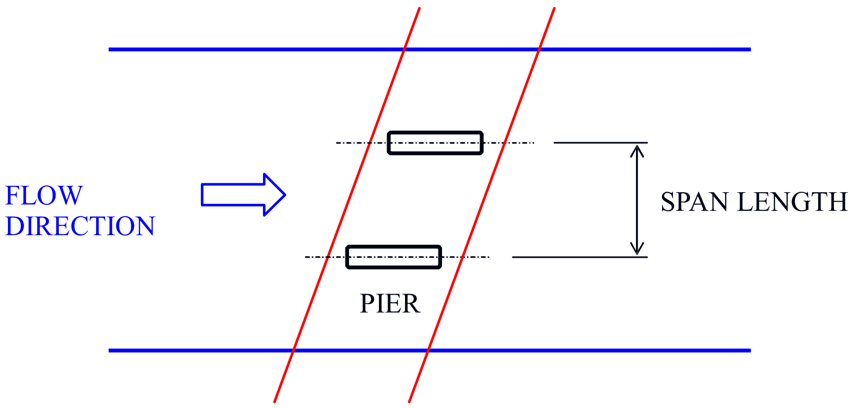


Figure 6.2.7 Definition of Span Length for Skew Bridge

(2) Standard span length

The standard span length is the value determined according to the following definition.

> The center distance between piers (for the skew bridge) in neighboring channels projected on a vertical plane crossing the river at a right angle relative to the flow direction during flood must as a rule be no less than the value determined as follows:
>
> $$L= 20 + 0.005 \times Q$$
>
> where, L: Span length (m), Q: Design: Design flood discharge (m^3/s)

The span length of river bridges must as a rule be the distance no less than the value obtained above. The span length may not be subject to the above standard value if the length selected is free from any adverse effects in terms of flood control because the bridge is planned for mountainous narrow passes and due to other river and topographical conditions.

Meanwhile, in case that the actual span length becomes 5m longer than the standard span length calculated by the above formula, the number of span can be increased but not more than one span (Mitigational option to decrease the span length up to max 5m)

Reference-3

Stipulation for deregulation for medium and small rivers

For the medium to small rivers with the design flood discharge of less than 2,000m^3/s, the value obtained from the standard span-length calculation equation is too strict. Therefore, it was determined to allow reduction of the span length to the value shown in Table 6.2.6 according to the conventional concept, provided that there is no concern of excessive obstructions in terms of river administration. For a case with the flow of less than 500m^3/s and the river width of 25 m or more, less than 30 m, the bridge may be planned with two spans with the maximum length of 12.5 m.

Table 6.2.6 Road bridge : Deregulation Stipulation for Medium and Small Rivers

Design flood discharge (m^3/s)	Less than 500		500 or more, less than 2000
River width (m)	Less than 30	30 or more	-
Required interval (m)	12.5	15	20

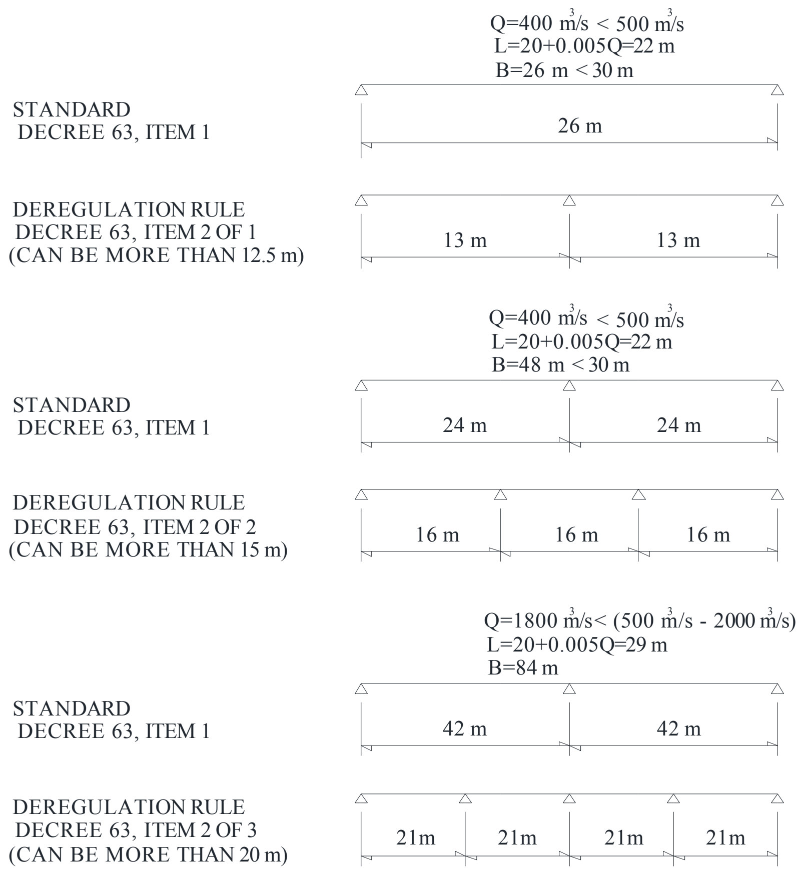


Figure 6.2.8 Example of Deregulation stipulation for medium and small rivers

(3) Special provisions for neighboring bridges

For new construction of a bridge in the neighborhood of crossing structures, such as the existing bridge, weir, etc., due care must be taken to minimize turbulence of flood flow line occurring depending on the positional relationship of these facilities and to prevent compounding of eddy currents occurring on upstream and downstream sides. Accordingly, the following stipulations are provided concerning the span arrangement of a neighboring bridge:

When the distance between the existing bridge and the neighboring bridge is less than the standard span length, the piers of the neighboring bridge must be provided on the line of eyesight of piers of existing bridge.

When the distance between the existing bridge and the neighboring bridge is no less than the standard span length and less than the river width (maximum 200 m), the piers of neighboring bridge must be provided on the line of eyesight of existing bridge or the line of eyesight running through the center of existing bridge span.

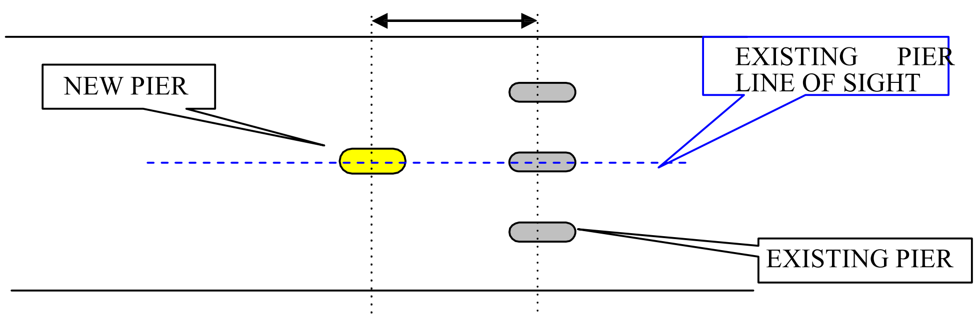


Figure 6.2.9 Special Provision for Neighboring Bridge (For the case when distance from new construction bridge is less than standard span length)

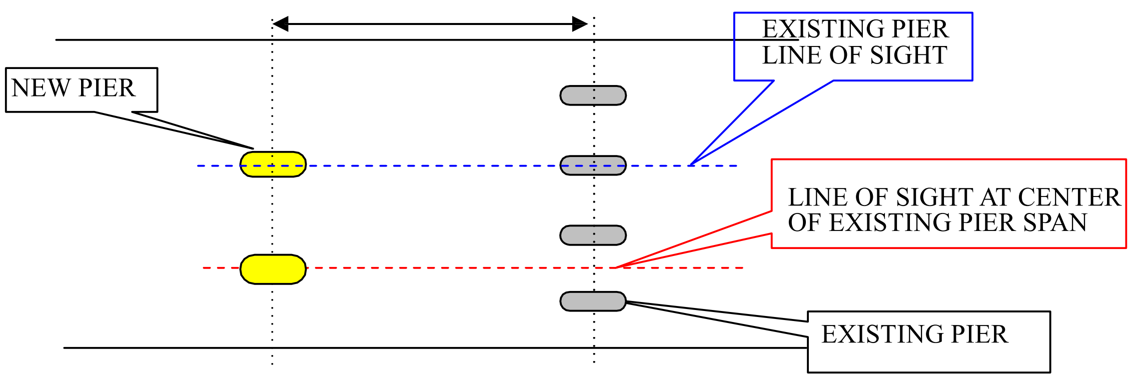


Figure 6.2.10 Special Provision for Neighboring Bridge

(For the case when distance from new construction bridge is more than standard span length, but less than River Width)

The special provisions for neighboring bridges include the "deregulation stipulations for special provisions for neighboring bridges" for the case when distance from new construction bridge is less than Standard Span length and the case when distance from new construction bridge is more than Standard Span length, but less than River Width.

(4) Conditions concerning the pier location and the foundation crown height

1) **Pier location**

The pier location is roughly determined by the span length. However, when the pier location is close to the bank or the foot of dike, the bank and the foot of dike tend to be scoured readily. It is essential to pay due attention on the following points in addition to the span length when the pier location is to be determined.

- 10 m (5 m for the river with the design high-water flow of less than $500m^3/s$) or more distance to be taken from the bank or the foot of dike respectively.

- Strengthening of the embankment and provision of the bed protection work or high-water protection work, as required, if piers are to be provided in above locations by necessity.

2) **Pier foundation crown height**

The pier foundation crown height is to be determined according to the stipulations below because such foundation is to be provided in the location deeper than the depth equivalent to the scour depth during floods.

a. Piers located in the low-flow channel: ”Design river bed height (or the deepest floor bed height) – 2 m or deeper”

b. Piers located in the high-water channel

  - Piers located within 20 m from the bank slope top: As per the conditions for the low-flow channel
  - Piers distanced more than 20 m from the bank slope top: ”Design high-water channel - 1 m or deeper”
  - The above conditions are illustrated in Figure 6.2.11.

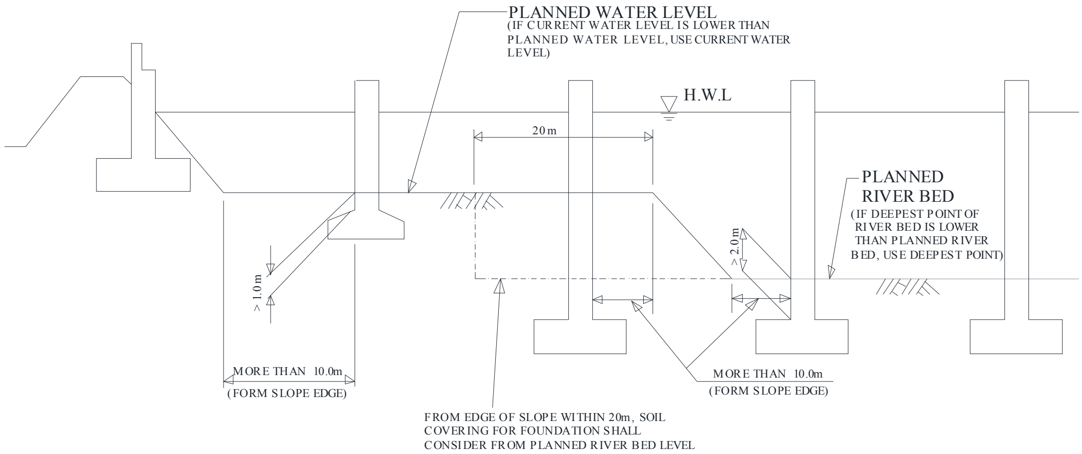


Figure 6.2.11 Constraints of Pier Position・Pier Foundation Depth

Reference-4

Impediment ratio of river flow

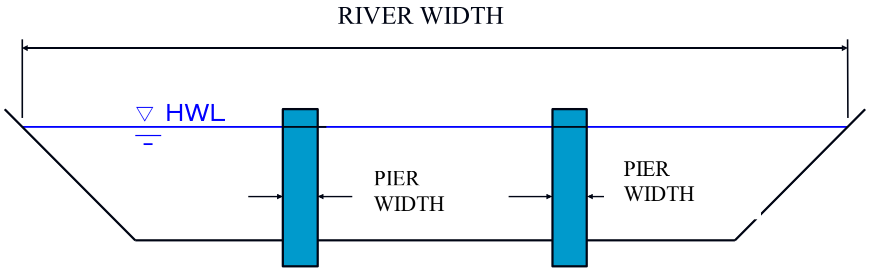
The impediment ratio of river flow is defined as a ratio of total piers width relative to the river width. The total width of river widths and piers is as follows:

(1) River width: Distance between the design high-water level and the intersections with the dike slope, as measured in the direction normal to the flow direction

(2) Total width of piers: Total of pier widths at the design high-water level, as measured in a direction normal to the flow direction

(3) The impediment ratio of river flow of bridge is targeted at the value less than the value shown below.

  – Ordinary bridges:5% or less as a rule
  – Railway bridges and expressway bridges: Less than 7% (as a special value)



Figue 6.2.12 River Width・Pier Width

When the piers are to be provided in a direction parallel to the dike normal line in the section where the dike horizontal alignment does not agree with the flow direction during flood, such as the curved section, the flood-flow impediment ratio of piers relative to the actual pier width is larger as shown in Figue 6.2.12. Therefore, for new construction of a bridge in the curved section, it is recommended to correct beforehand the bridging direction on the basis of flood flow understood from the hydraulic model experiment and the plain flood flow analysis.

Particularly when the bridge has many lanes and wide pier width, the impediment ratio of river flow increases suddenly because of several times of biasing of flow direction. It is particularly necessary to pay attention to the wider piers.

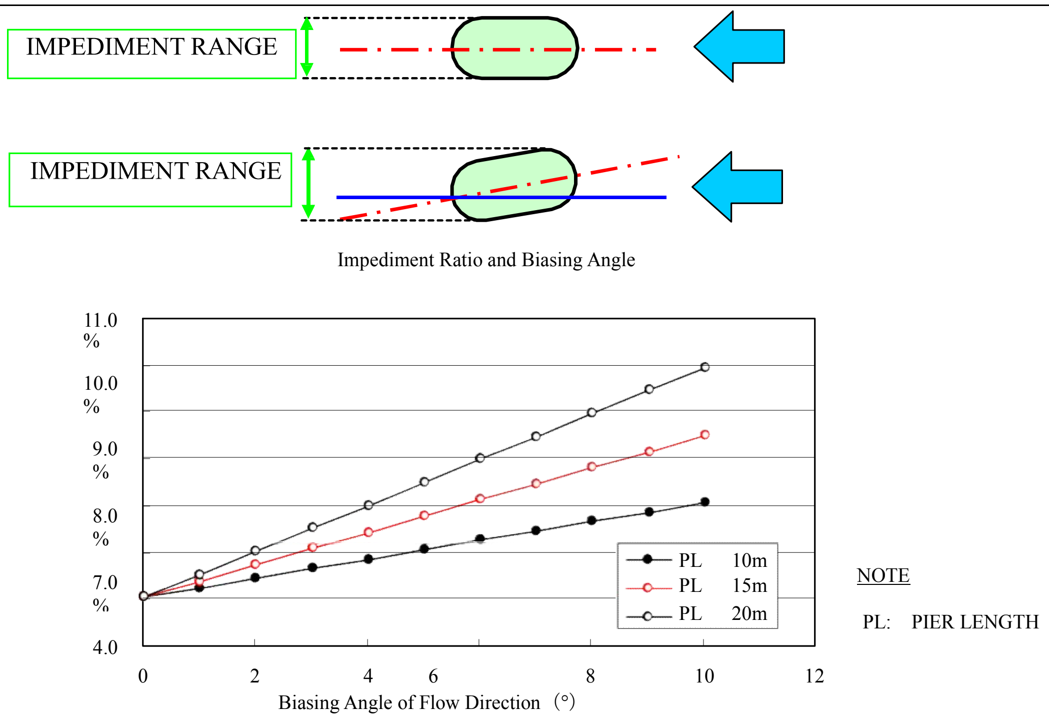


Figure 6.2.13 Impediment Width When Pier Installing Direction Does not Pararel to Flood Flow Direction

6.2.6 Summary

As is evident from description up to now, the basic bridge plan is established in terms of the bridge location, abutment and pier arrangement, etc. On the basis of the plan thus established, the preliminary design and schedule plan are to be drafted.

6.3 CONCEPT OF THE PROTECTION WORK, SUCH AS BANK PROTECTION, BED PROTECTION

6.3.1 Basic Concept

(1) Prerequisite

The prerequisite for review of the protection work, such as bank protection, bed protection, etc., is that the location of abutments and piers has been established already after completion of the review in section 6.2. This section deals with the concept of bank protection, bed protection, and protection work against scouring around piers.

(2) Contents of the review

The contents of review on the protection work vary depending on whether or not the hydraulic review has been considered necessary up to now. The standard stipulations that the protection work must meet are explained in 6.3.2.

6.3.2 Stipulations Concerning the Protection Work, such as Bank Protection and Bed Protection

[Prerequisite]

- Adequate bed protection work or high-water channel protection work must be provided to prevent scouring of river bed or high-water channel due to turbulence of flow water caused by installation of piers.
- Bank protection must be provided on both upstream and downstream sides of the bridge as the protection of bank against turbulent flow or driftwoods under effects of piers. Also taken into account are the measures to reinforce the weakened dike due to installation of abutment and the slope protective work as an alternative to lawn whose growth has become impossible as the bridge obstructs sunshine.

Provision of abutments and piers not only causes turbulent or unbalanced flood flow, but also forms weak points against seepage failure inside the dike. Therefore, it is essential to protect adequately the dike, high-water channel, and river bed.

The standard concept is explained below concerning the revetment to protect dike, protection of high-water channel, and bed protection to prevent souring of river bed.

Note that this does not apply when the bank or dike is free from the possibility of scouring due to geological situations, etc. and when there is no other problem in terms of flood control.

(1) Revetment

The dike for the area around piers and revetment for low-water bank are provided for the purposes listed below:

a. Protection of dike from turbulent flow or driftwoods under effects of piers

b. Reinforcing measures against weakening of dike due to provision of abutment

c. Slope sunshine.protection as an alternative to lawn whose growth has become impossible as the bridge obstructs

The area in which revetment is to be provided is basically the area shown in Figure 6.3.1 to Figure 6.3.3. Note that the revetment range is the reference of the minimum standard and must be enlarged as required for the river channel bend and rapid rivers.

The range of provision should be set while taking into account the range in which the hydraulic model experiment or plane two-dimensional river flow analysis shows increase in the velocity when compared with the case before provision of piers and the range of effects of the wavy water front caused by piers on the bank.

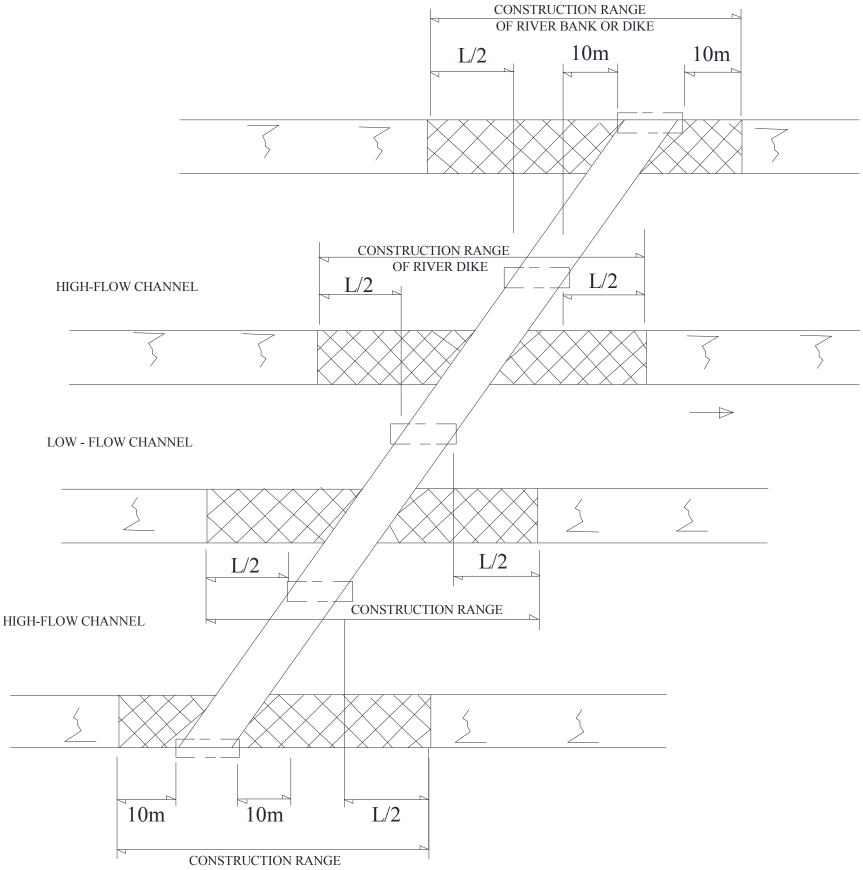


Figure 6.3.1 Necessary Length for Dike Reinforcement Due to Installation of Bridge

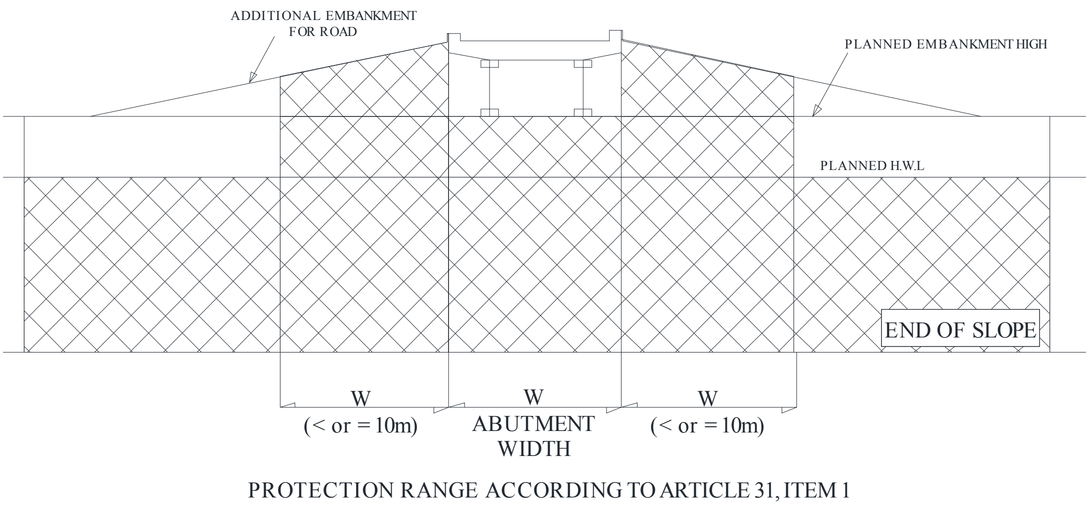


Figure 6.3.2 Necessary Height of Dike Due To Bridge Installation

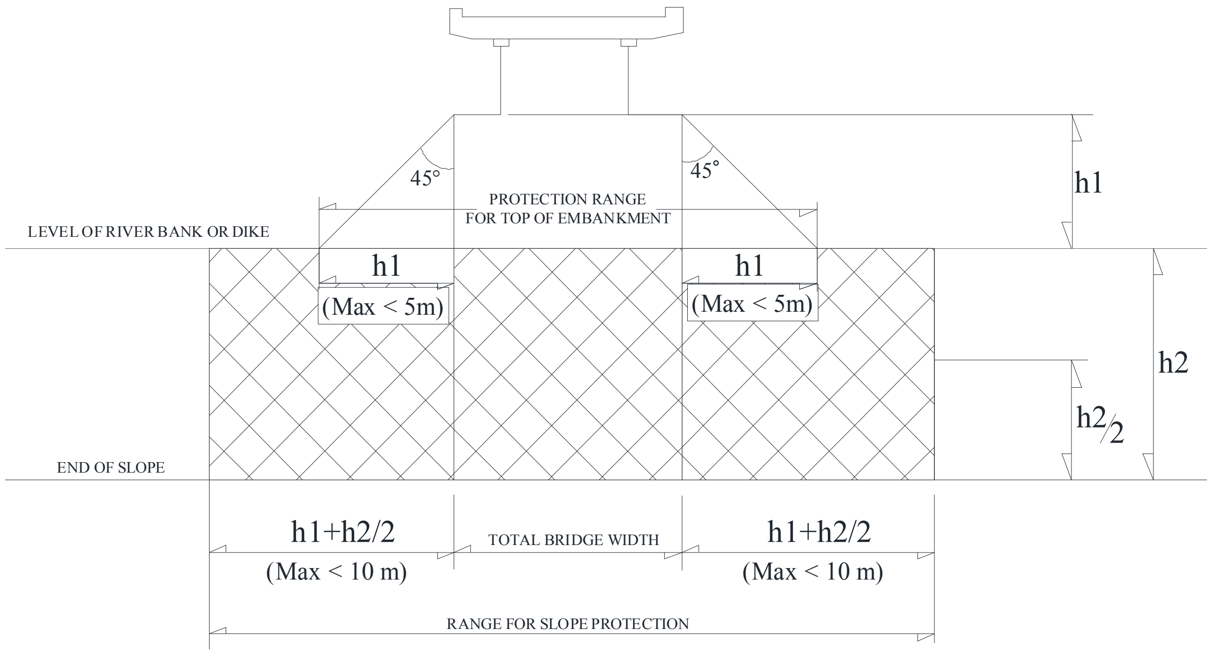


Figure 6.3.3 Range of Protection for Dike or River Bank Under Bridge

(2) Bed protection and high-water channel protection works

Bed protection and high-water channel protection works are provided, as required, around the piers to prevent scouring of river bed or high-water channel by turbulent flow after provision of piers.

The standard range of provision is as follows. If scouring of river bed is expected, the range of provision of revetment should be set on the basis of prediction of the scouring range according to the technique shown in Figure 6.3.3.

a. The range of revetment and high-water channel protection work is roughly the area about 5 m or more from the piers.

b. The high-water channel protection work is to be provided when scouring is considered excessive.

The high-water channel protection work is constructed to prevent scouring by flow while generally using cylinder mattress, articulated concrete mattress, etc. Basically, soil-covering is performed while taking into account harmonization with the surrounding landscape, protection of river ecosystem and environment.

6.3.3 Pier Scouring Protection Work

The depth of footing of pier foundation at the bridging location is specified as shown in Figure 6.3.4. The value shown in the figure is the minimum value of pier design. Considering the effects of pier scouring on the river, the foundation height of piers should be as deeper as possible.

The scouring phenomenon occurs around the piers as the flow is to be stabilized relative to the obstructing piers, and the configuration causing natural scouring has a function of stabilizing flow. Accordingly, water-level increase due to piers or effects of the flow on the bank can be suppressed if it is possible to provide the pier foundation height deep enough to reach the depth of maximum scouring caused by river flow.

Since the pier foundation height governs the pier construction cost, there is not much examples of providing the pier foundation while assuming the maximum scouring depth. If the maximum scouring depth assumed for the flow is deeper than the envelope of the deepest riverbed -2 m, the foot protection block and riprap work are often provided as the pier scouring protection work.

When scouring reaches the foundation, the pier diameter exerting influence on the scouring depth determines the scale of the foundation. Since the diameter of foundation structure is generally larger than the pier diameter, scouring proceeds rapidly, damaging the stability of piers. Accordingly, it is often the case that the scouring protection work is provided above the foundation so as to prevent scouring from reaching the foundation.

The pier scouring protection work, such as the foot protection work, etc. must be provided while taking into account a fact that such protection work forces the flow around the piers to deflect. For example, relative to the longitudinal flow causing scouring around the piers, the foot protection work causes lateral deflection in the flow direction. This must be taken into account duly because it may expand the scouring range or may cause raising of water level toward the upstream pass of the bridge by eddy generated in line with deflection.

The depth at which the foot protection is provided is above the foundation in consideration of scouring caused by piers. If the foot protection is provided at shallow levels, the pier causes great deflection, resulting in enlargement of scouring holes. It is recommended therefore to provide the foot protection work to the depth, which can reduce the scouring range through moderation of the flow by piers while allowing scouring of piers to a certain extent, as shown in Figure 6.3.5.

Depending on the shape of piers, river channel state, river bed shape, etc., the depth of foot protection work may exert influence on the river flow regime. For the bridge for which the scouring depth of piers is difficult to estimate, the hydraulic model experiment is often made for confirmation.

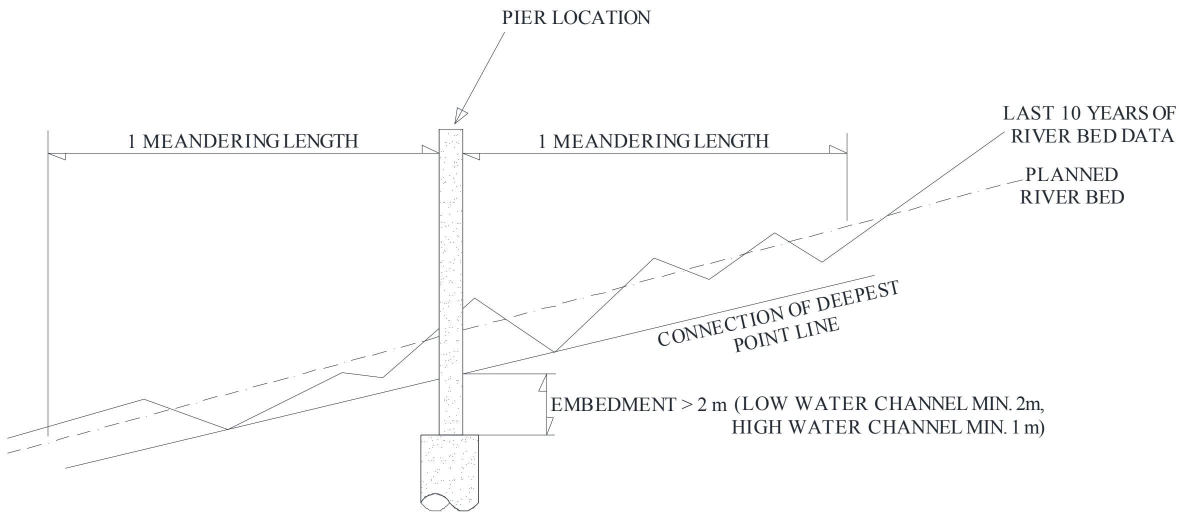


Figure 6.3.4 Embedment Depth of Bridge Piers

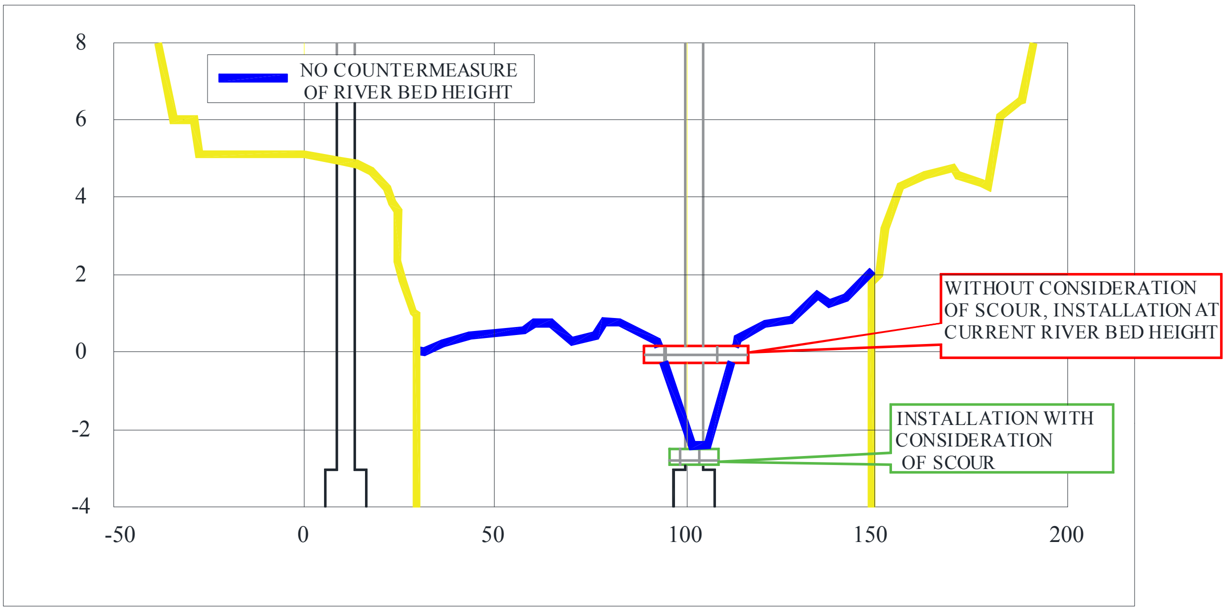


Figure 6.3.5 Example of Installation Depth for Scour Protection Works

6.4 CONCEPT OF THE COFFERDAM CONSTRUCTION

6.4.1 Basic Concept

(1) Prerequisite

In the course of schedule planning after completion of the review up to Section 6.3, cofferdam construction is pointed out as an issue specific to the work within the river area. Particularly when the piers are to be provided in the low-water channel, the plan of cofferdam construction must be reviewed carefully because it exerts substantial effect on the construction costs and period. This section describes the concept of this cofferdam.

(2) Contents of review

Review of the cofferdam construction must ensure selection of an adequate cofferdam while taking into account various types of cofferdams (see Figure 6.4.1). When the adverse effects of providing the cofferdam in the river channel because of its large scale, etc., cannot be ignored, the work to mitigate such effect must be considered.

(3) Considerations for review of the cofferdam

For the cofferdam, the work method is mostly selected according to the piers size and type. The work method ranges from a simple case of using sandbags, cases of using steels, such as steel sheet piles, steel pipe sheet piles, to cases of large cofferdams using cells, caissons, etc.

Generally, this is used during work and removed at completion of the work, similarly to the case of earth-retaining walls. In certain cases, this is used as a part of main body structure or the main body structure itself, such as the steel pipe sheet pile foundation, cells, caissons. It is essential that the cofferdam is constructed for earth retaining and as the wall having water tightness.

When the cofferdam is compared with the earth-retaining work, the greatest difference is that the external force to be dealt with is different. In the case of earth-retaining work, major external forces to be dealt with are soil pressure, water pressure, and surface charge, none of which fluctuates substantially by nature. On the other hand, for the cofferdam constructed in the river work, fluctuation of external forces must be taken into account because the water level rises suddenly due to rainfall, snow melting, or discharge from the upstream dams. In certain cases, effects of scouring with flow water or waves, wave pressure, etc. must be reviewed.

The substantial difference from the case of earth-retaining wall is that the cofferdam is subject to restrictions in terms of the construction time and period. In the case of river work, it is not allowed to leave any structure obstructing the flow within the river during a period from June to October (so-called flood season). Accordingly, the work of cofferdam construction is not allowed generally except for a case when the construction work in winter is not possible, for example, in the snow cold zones. Namely, the construction work is frequently conducted in the no-flood (drought) season from November to May. It is necessary to select the type and construction method of cofferdam under the conditions restricted in terms of schedule.

Essential points to be taken into account for planning and construction of the cofferdam is that the water pressure plays the leading role for the cofferdam more than the case of earth-retaining work, so that the time span from detection of abnormality to breakage is short. Namely, the temporary work must be planned and executed with due consideration of the fact that accidents occurring in the cofferdam construction tends to be large-scale ones.

6.4.2 Water Level for Cofferdam

The water level at which the cofferdam is to be performed is set while referring to the time maximum water level during non-flood season for the past five years because the pier work within the river is generally planned for the non-flood season. Note that the second highest water level in the past decade may be employed when the water level concerned showed abnormal flood in the past five years.

Setting the height of cofferdam to the high level will ensure the improved safety for the pier construction, but will prove dangerous for the surrounding river channel or raising of the water level. It is essential to take into account this fact when setting the height of cofferdam.

6.4.3 Types of Cofferdam

The types of cofferdam are shown in the figure below.

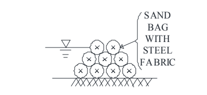
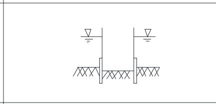
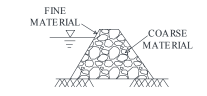
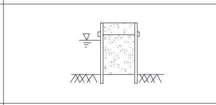
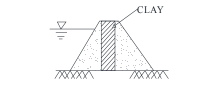
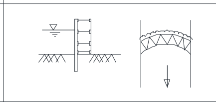
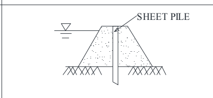
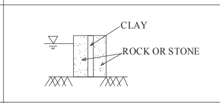
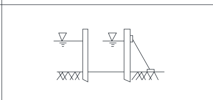
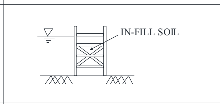
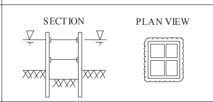
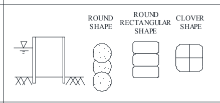
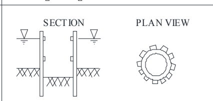
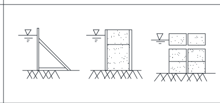
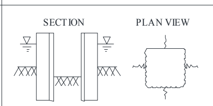
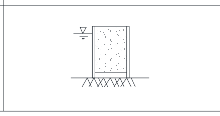
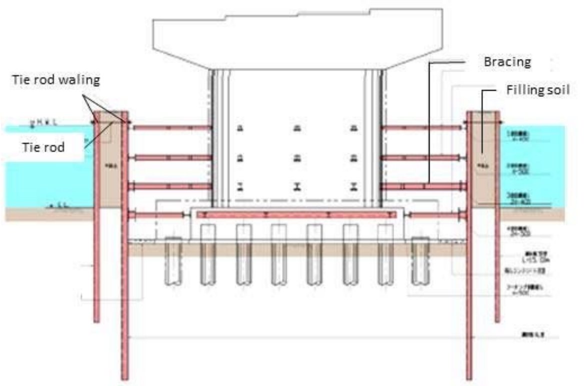
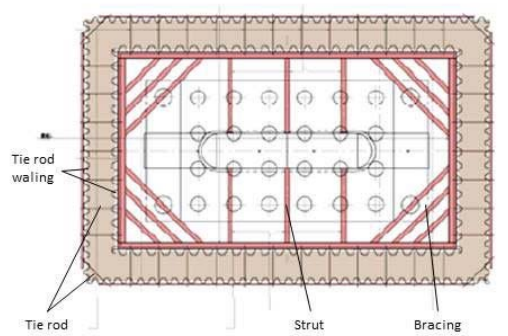
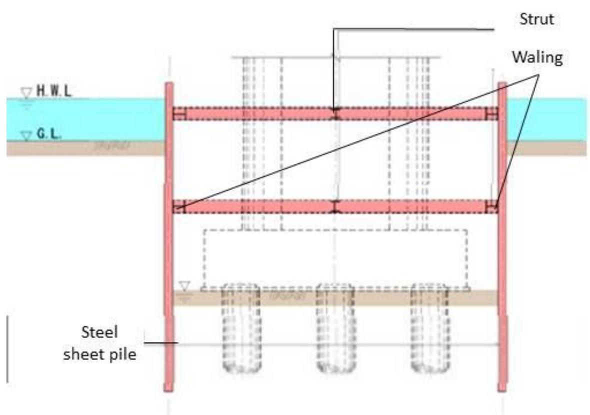
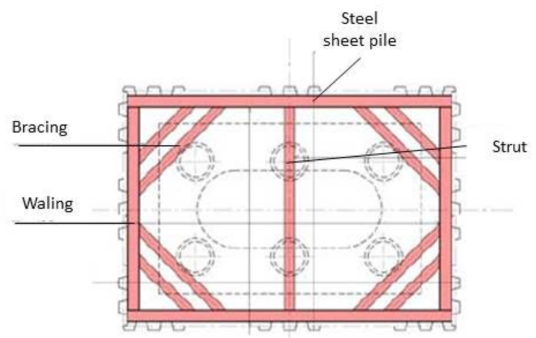
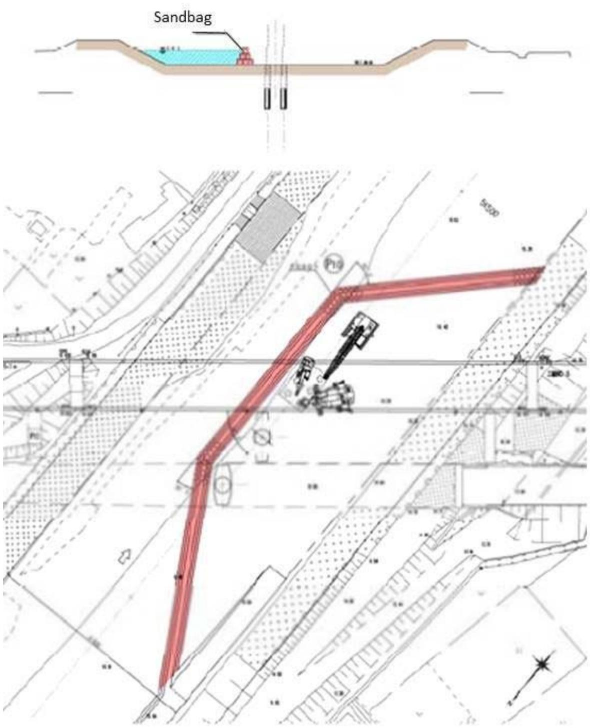
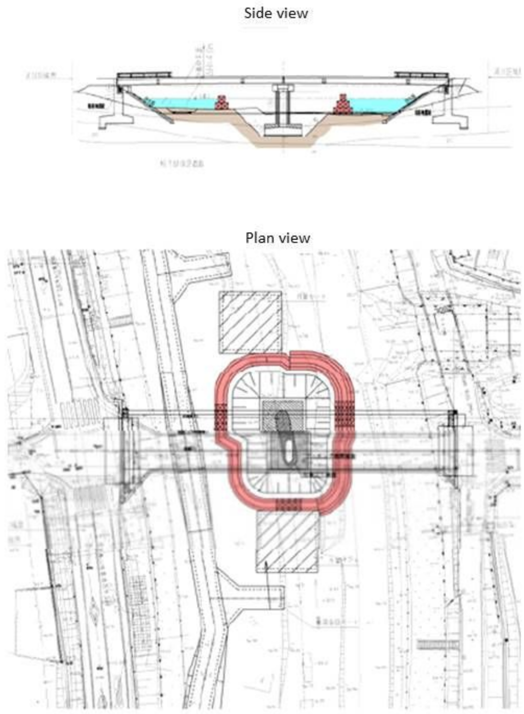
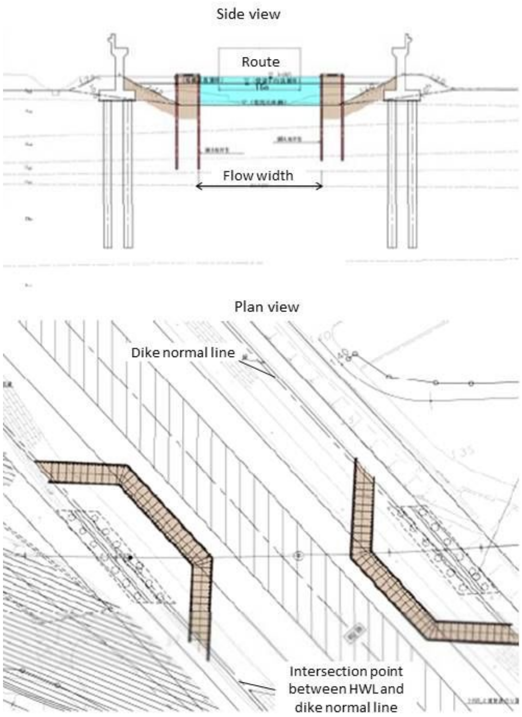
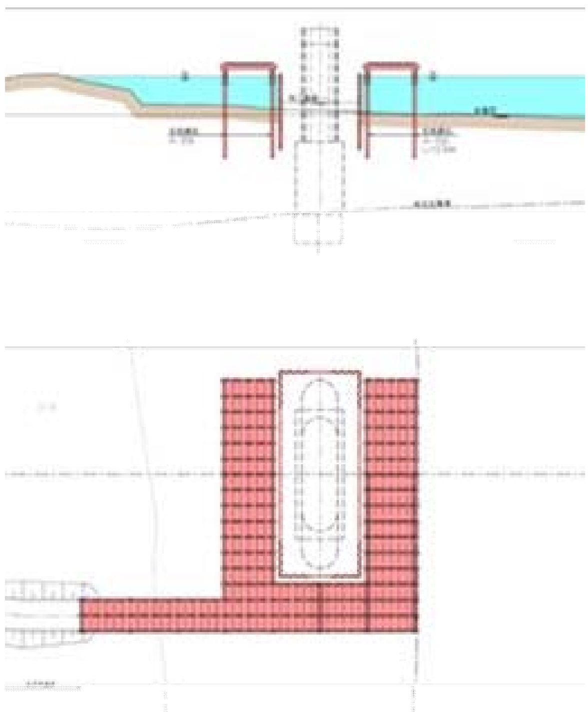
TYPE	FIGURE	ADVANTAGES	DISADVANTAGES	TYPE	FIGURE	ADVANTAGES	DISADVANTAGES
SAND BAG		EASY TO INSTALL		MOBILE TYPE		POSSIBLE TO REUSE	FAST FLOWING
EARTH EMBANKMENT		FAST FLOWING	SPACE LIMITATION	DOUBLE SHEET PILE		IN CASE OF DEEP WATER DEPTH AND GOOD FOUNDATION	
EARTH EMBANKMENT		SLOWLY FLOW SHALLOW POND	EXISTENCE OF SOFT SOIL LAYER BELOW	ARC		IN CASE OF DEEP WATER DEPTH AND STIFF ABUTMENT	WIDE TEMPORARY STRUT
EARTH EMBANKMENT & SHEET PILE			DEEP WATER DEPTH	BOX FRAME		IN CASE OF ROCK FOUNDATION AND THERE IS POSSIBILITY OF FLOOD	OTHER THAN BEDROCK
DOUBLE SHEET PILE			DEEP WATER DEPTH	BOX FRAME		BEDROCK	WITHOUT COUNTERMEASURE TO PREVENT IMMERSION OF SOFT FOUNDATION
SHEET PILE WITH SUPPORTING STRUTS		ESPECIALLY, IT CAN PREVENT WATER LEAKAGE WHEN SHEET PILE HAS BEEN INSTALLED IN CLAY	DEEP WATER DEPTH	CELL		IN CASE OF BEDROCK OR STIFF FOUNDATION AND DEEP WATER DEPTH	IN CASE OF UNABLE TO REDUCE UNDERGROUND WATER PRESSURE, THE INSTALLATION WILL BE DIFFICULT, AND CAUSES DAMAGE TO CELL
RING BEAM METHOD		IF THE AREA IS SMALL, EMBEDMENT LENGTH IS ALSO SHALLOW	LARGE DIAMETER AND DEEP WATER DEPTH	CONCRETE BLOCK		IN CASE OF BEDROCK AND FAST FLOWING	SOFT FOUNDATION
BUTTRESS		IN CASE OF REQUER WORKING	LARGE DIAMETER AND DEEP WATER DEPTH	CAISSON		DEEP WATER DEPTH & TEMPORARY STRUT TO BE REMAINED IN PERMANENT STRUCTURE	

Figure 6.4.1 Type of Temporary Cofferdam

Reference examples of cofferdam are also shown before for your information.

Table 6.4.1 Examples of Temporary Cofferdam

(1) Double sheet pile cofferdam	(2) Single sheet pile cofferdam
 	 

(3) Cofferdam with sand bags	(4) Watercourse change with sand bags
	
(5) Double wall dike	**(6) Jetty for the work**
	

6.4.4 Measures Against Water Level Rising and Localized Scouring Due to Presence of Cofferdams During Construction

When the temporary structures, such as cofferdams, etc. which are larger in scale than other normal river structures, are left in the river channel in the course of construction, the measures against water level rising and localized scouring with temporary structures are necessary.

In particular, when the temporary structures are to be left in the river channel during flood season by necessity due to issues related to construction, the effects of the temporary structures left behind on the water level and flow regime, and on the river bed, as well as on the dike and bank and existing structures must be reviewed thoroughly by means of the numerical analysis of hydraulic model experiment. On the basis of review result, the measures must be taken against water-level rising and localized scouring.

Even when the temporary structures are allowed to be left in the river channel only during the no-flood season and when the impediment ratio of river flow is high, the necessity of the measures should be reviewed at least through numerical analysis of the water-level increase for the flood expected during the no-flood season.

As a rule, the temporary structures are allowed to be left in the river channel only during the no-flood season. When the shape of temporary structures is to be selected, due care must be taken to ensure prevention of adverse effects of water-level rising in proportion to the flow possible during this season.

6.5 FUNDAMENTALS OF THE BRIDGE STRUCTURE

This section summarizes the fundamentals concerning the bridge structure and type, abutment and pier type and their foundation.

6.5.1 Bridge Structure

A bridge structure is divided into a superstructure and a substructure.

(1) Superstructure

Linear portion provided in such a manner as to cross the target river or road, and the vehicles and pedestrians can pass over the bridge by passing on top of or inside the superstructure.

Various types of superstructure have been proposed according to the span length, and the superstructure is a bridge component governing most the bridge appearance. For the girder bridge or truss bridge, the main girder carrying mainly the load and slabs forming the road surface to carry vehicles and pedestrians directly are principal members. In the suspension bridge and cable-stayed bridge, the main tower and cable are also included in the superstructure. Also included in the superstructure are the bridge railings and vehicle guard fence, which are to prevent departure or falling of vehicles and pedestrians from the bridge, adjuncts such as lighting poles, etc., bearing for connection to the substructure, and expansion joint at the boundary between the road and bridge.

The bridge is also classified into three types according to where the vehicles, railway and pedestrians pass: the deck bridge, half-through bridge, and through bridge.

(2) Substructure

The substructure functions to support the superstructure and to transmit the load to the ground. The superstructure is supported by the bearings provided on the abutments and piers. The members provided on both bridge ends are called abutments and the members provided in-between are called piers. The foundation functions to transmit the load of the bridge as a whole including abutments and piers to the ground and available in various types according to the bridge type, load magnitude, and ground condition.

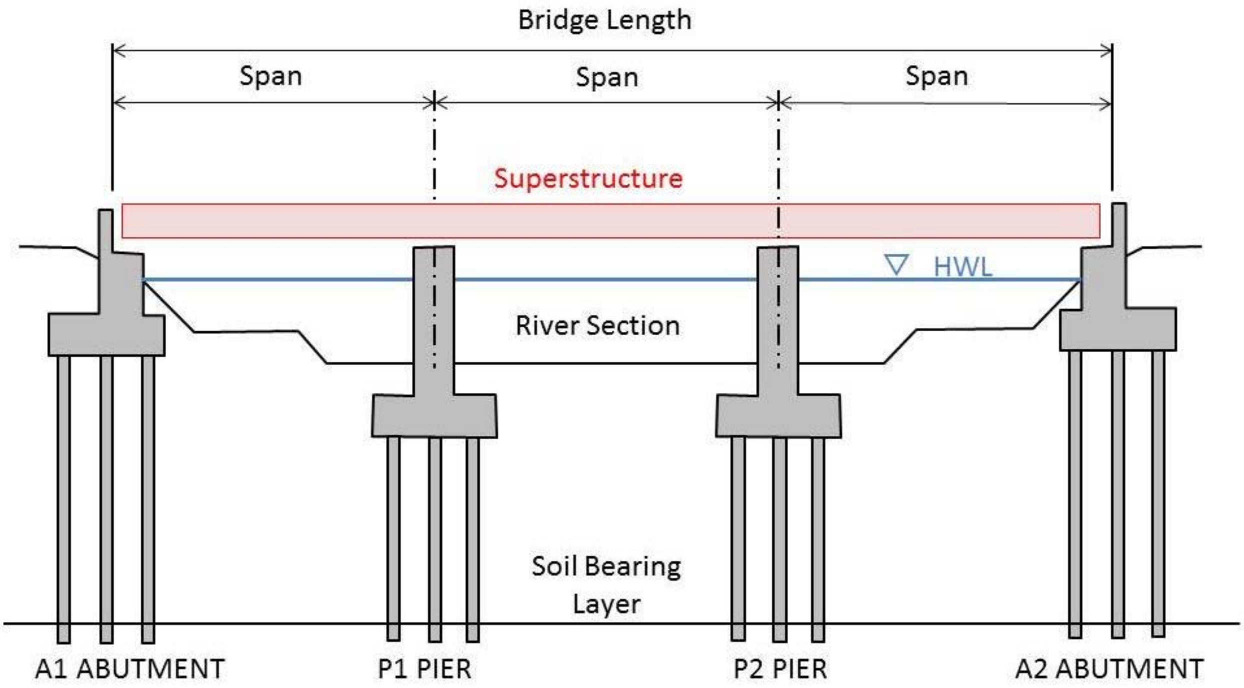


Figure 6.5.1 Outline Drawing of River Bridge

6.5.2 Bridge Types

(1) Classification by material

The classification of bridge types according to the material of principal members is described below.

1) **Steel bridge**

This is a bridge whose superstructure is made from steel.

Steel is higher in specific strength than concrete. Steel can reduce weight of the superstructure, and thus used for bridges with long spans.

2) **Concrete bridge**

This is a bridge whose superstructure is made from concrete.

Concrete has lower tensile strength relative to compressive strength, so that reinforced concrete (RC) in which steel bears the tensile stress and prestressed concrete in which PC steel provide the compressive force beforehand to negate the tensile stress are used. Recent concrete bridges are mostly prestressed concrete bridges, excluding arch bridges and substantially small bridges.

3) **Composite bridge**

This is a bridge that employs a composite structure or mixed structure of dissimilar materials and members. Generally, this refers to the superstructure in which the steel members are combined with the concrete members.

The types frequently used recently are as follows:

- Steel composite girder bridge: Bridge comprising prestressed concrete slabs and steel girders

- Corrugated steel web bridge: Bridge in which corrugated steel plates are used on the web members of prestressed concrete box girder bridge
- Steel composite truss bridge: Truss bridge in which decks and lower slabs, which are made from concrete, are combined with steel diagonal members.

Regarding the mixed structure, there are bridges that consist of steel girders for a part of multi-span and other concrete girders. The composite bridges using relatively new materials, such as carbon fiber and glass fiber, are proposed in addition to those using steel, concrete, wood and stone.

4) **Others**

Other bridge materials are wood, soil, and stone.

(2) Classification by the location of the road surface

The bridge is classified into three types of the deck, half-through, and through bridges according to the location where the vehicles, railway, and pedestrians pass.

- Deck bridge:Bridge where the road (railway) passes through the upper portion of the bridge structure (for example, the truss bridge)
- Half-through bridge:Bridge where the road (railway) passes through the middle portion of the bridge structure (for example, the prestressed rib arch bridge)
- Through bridge:Bridge where the road (railway) passes through the lower portion of the bridge structure (for example, Langer bridge)

(3) Classification by structure

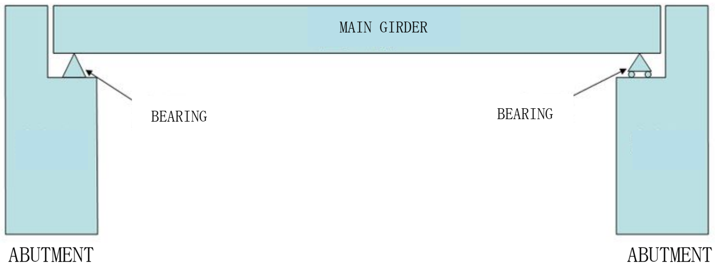
Classification of bridges by structure is as follows.

1) **Girder bridge**

This is a bridge in which passage is made on or inside the girders erected horizontally on two or more supports. The girder bridge is of a structure in which the main girders withstand the bending moment and shear stress. Girder bridges using main girders of mainly steel, concrete, and wood, are classified into the simple girder and continuous girder bridges according to the superstructure. The simple girder bridge consists of erecting individual girders on each span, with the span length being about 50 m or less. On the other hand, the continuous girder bridge has the girders for two or more spans continuously and is advantageous in that the girder height can be lower than the simple girder bridge if the span length is the same. The continuous girder bridge can also have the longer span length, 100 m or more in the case of prestressed concrete continuous girder bridge type.

Recently the continuous girder bridge is preferred generally because the simple girder bridge structure always requires joints between girders. These joints not only worsen the traveling performance of vehicles, but also become sources of noise and vibration. They tend to be structural weak points with water leakage, etc.

The shape of cross section of main girders include I box and T sections. In the case of steel bridge, the type with steel plates welded into an I section for reinforcement is called the plate girder bridge and the type with steel plates welded into a box section is called the box girder bridge. In the case of prestressed concrete bridge, the bridge with a relatively short span applies the T-section bridge type while bridges with longer spans apply the box girder type.



2) **Truss bridge**

This bridge is of a truss structure in which rod members are assembled into a triangle with the panel points connected with pins. Truss members are subject to the axial force (compression or tensile force) only. Materials used often are steel and wood. According to the truss members' arrangement, the truss structure is classified into the parallel-chord Warren truss, curved-chord Warren truss, Warren truss with vertical member, Pratt truss, Howe truss, K truss, etc.

3) **Arch bridge**

This is a bridge using an upward segment (arch), with the arch (arch rib) subject to large compression force and relatively small bending moment and shearing stress. In addition to concrete and steel or wood, stones were preferred in the pre-modern times.

4) **Rigid frame bridge**

This bridge employs the framework (Rahmen) of rigidly connected piers and main girders. ("Rahmen" is a German term). The members are subject to the axial force, shearing force and bending moment and are made from concrete or steel. When viewed from the structural dynamics, the rigid frame is a statically indeterminate structure in which the number of unknown reaction forces is larger than the number of equilibrium of force equations. This structure is considered highly earthquake resistant because considerable deformation of a certain member under extremely large load will not lead to collapse of bridge and the superstructure will not slide down as there is no bearing on piers.

5) **Suspension bridge**

This bridge has girders or slabs suspended from the flexible members with high tensile strength, such as cable, rope, net, etc. Large-scale suspension bridges in post-modern times are of a structure in which the girders to be used as passage are suspended from the cable stretched between two or more main towers provided between large-mass anchorages on both banks. In this suspension bridge, the girders as passages have the rigidity enough to maintain the bridge shape, so that they are called stiffening girders. The cable is subject to tensile force while main towers are subject to compression force. The cable is made mainly from high-strength steel while towers are made mainly from steel or concrete. There are also the stressed ribbon bridge with slabs suspended directly from the abutment or the self-anchored suspension bridge that connect the cable to both girder ends without using anchorages.

6) **Cable-stayed bridge**

This bridge is of a structure in which the cables stretched diagonally from main towers are directly connected to girders for support. Similarly to the case of suspension bridge, the cables are made from steel while main towers and girders are made from steel and concrete. The cable is subject to tensile force, the main tower is subject to compression force, and the girders are subject to bending moment, shearing stress and axial forces. The suspension bridge and the cable-stayed bridge differ greatly as follows. The cable-stayed bridge has the towers and girders connected directly by cable while the suspension bridge has girders suspended with hanger ropes suspended from the main cable stretched between towers. In addition, the cable-stayed bridge does not require anchorages. On the other hand, the girders are subject to compression force because they are pulled toward towers. If the span is assumed to be the same, the tower is slightly taller in the case of cable-stayed bridge.

7) **Extradosed bridge**

The extradosed bridge has the main tower provided to the ordinary girder bridge and has the outer cable arranged, as diagonal member, on top of the main girder. These members are to support the main girder. Though this is similar to prestressed-concrete cable-stayed bridge, the behavior of this type is much closer to the ordinary girder bridge than the suspension type and has higher girder rigidity than the case of cable-stayed bridge. The apparent differences are characterized by low main towers and nearly horizontal diagonal members. Reduction of the angle of diagonal members enables suppression of the stress amplitude of these members caused by fluctuating load (mainly, traffic load). This enables cost reduction by reducing materials because their fatigue strength is higher than that of cable-stayed bridge and the tension of diagonal members can be set higher. Generally, with the span length of 200 m or less, this type is often more advantageous than the cable-stayed bridge in terms of cost.

6.5.3 Type and Features of Abutments and Piers

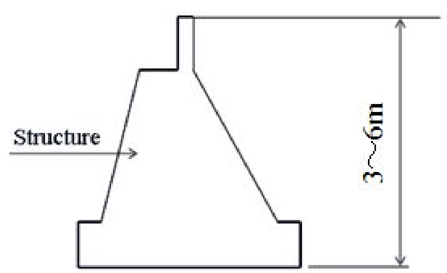
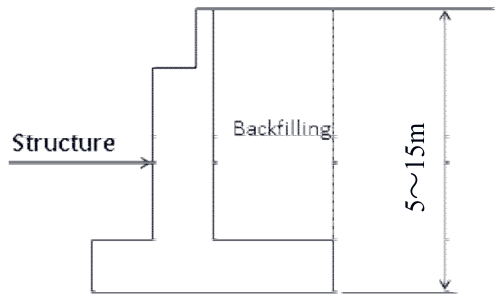
(1) Abutment

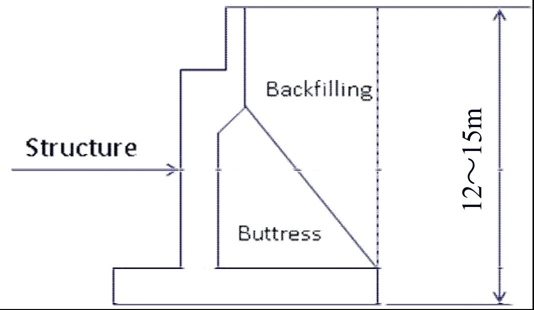
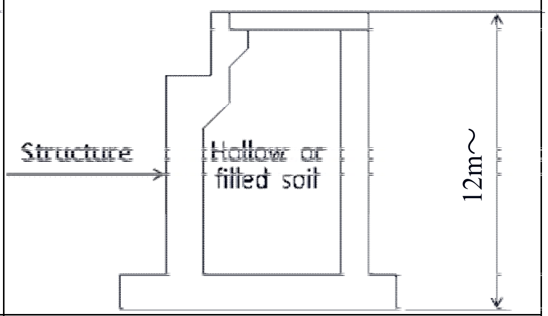
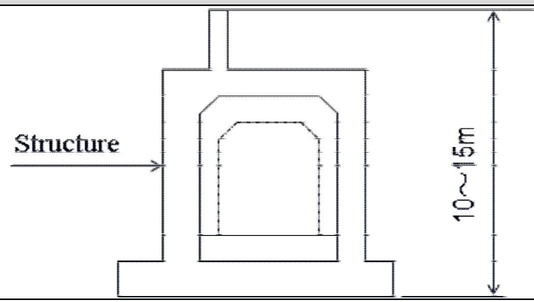
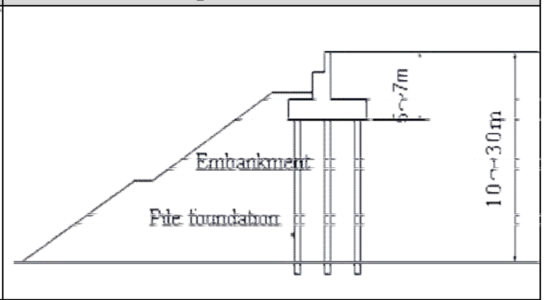
The abutment is located on both ends of bridge, which is a structure to support the superstructure and at the same time to connect to the access road. Because of the necessity to have the function not only to carry the superstructure, but also to resist the stress on back of wall, the abutment consists generally of footing and structures and is quite similar to the retaining walls structurally.

Accordingly, the abutment type must be appropriate to the superstructure type, load, topography, geology, and other conditions and must be superior in workability and economic efficiency. The abutment must also be structurally stable. Representative abutment types are as follows:

- Gravity type abutment
- Inverted T type abutment
- Buttress type abutment
- Box type abutment
- Rigid-frame type abutment
- Spreading foundation

Table 6.5.1 Outline of the abutment type

Type	Gravity type abutment	Inverted T-type abutment
Illustration		
Features	◆ Plain concrete made structure resists external forces such as soil pressure, etc. by means of its own weight ◆ Applicable to the satisfactory bearing stratum because of its large weight	◆ Economical and easy back-filling work because of small structure dead weight and the stability retained with soil weight ◆ Applicable to ordinary ground conditions with the height of around

	◆ Applicable to height of around 5 to 6 m	12 to 15m
Type	Buttress type abutment	Box type abutment
Illustration		
Features	◆ Difficulties in terms of bar arrangement and concrete placement for buttress and backfilling method ◆ Recently not often employed because of difficulties in construction ⇒ Inverted T-type abutment, box type abutment employed instead	◆ Modified buttress type abutment, in which the abutment as a whole is constructed as a multi-room box shape and top slabs are placed on the box. ◆ Inside the box : Generally, the box is filled with filling soil to increase the slide resistance in the case of spread foundation and the box is left empty to alleviate the load on piles in the case of pile foundation
Type	Rigid-frame abutment	Spread foundation
Illustration		
Features	◆ When there is an intersecting road at the abutment location, this type is employed if allowing the intersecting road through the rigid-frame abutment proves advantageous.	◆ Abutment would be extremely large when it is provided on the high embankment. ◆ Small abutment type (spread foundation) supported by pile foundation is advantageous in this case

(2) Piers

The pier is a structure located between abutments in two-span or more bridges, which supports the superstructure and transmits load to the foundation ground.

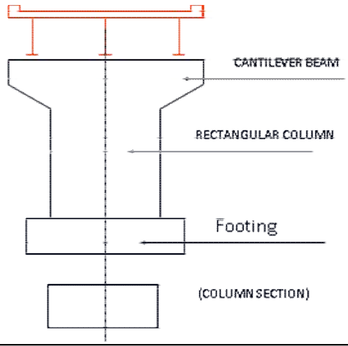
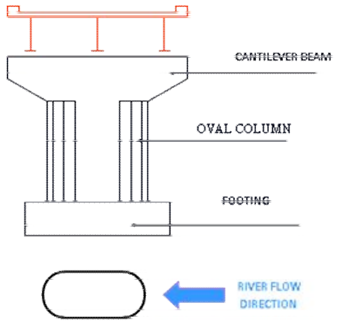
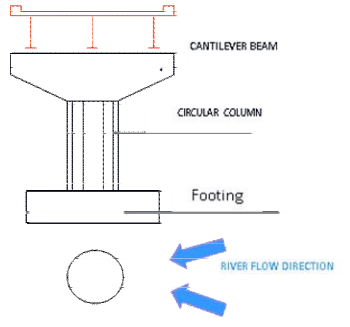
Different from the abutment, the pier is not subject to the stress on back of wall. Because the pier divides the space under the bridge, it is subject to restrictions by flowing water in the case of bridges crossing a river. The location and the number of piers require due review, together with the superstructure type while taking into account the dike, bank, and existing structure.

The representative pier types to be located in the river are as follows:

- Pier with overhanging beam (rectangular)
- Pier with overhanging beam (oval)

- Pier with overhanging beam (column)

Table 6.5.2 Outline of the pier types

Type	Pier with overhanging beam (rectangular)	Pier with overhanging beam (oval)	Pier with overhanging beam (column)
Illustration			
Features	◆ Most common pier type ◆ Rational type, in which the column is provided with the minimum section necessary for the structure and only the portion to support the superstructure is over-hanged and enlarged.	◆ Used when the bridge is constructed inside the river ◆ Column provided in a direction parallel to the water flow and shaped as oval	◆ Employed when the column width is restricted or when the flow direction is not fixed, such as in the river merging point.

6.5.4 Pier Foundation

(1) Foundation types and features

The foundation is a structure to support safely the bridge on the ground. The performances required of the foundation are the “allowable bearing capacity” and “allowable settlement”, which are generically termed as the allowable bearing capacity. It is essential to employ the safest and most economical foundation type while taking into account the superstructure conditions, ground conditions, and work conditions. In principle, it is not allowed to use dissimilar types for one unit of foundation.

For the foundation, the following types are mainly used and an appropriate one must be selected while taking into account the ground conditions, the nature of structure, construction conditions, etc. In Japan, for example, when a bridge is to cross over a river, the foundations used mainly are pile foundation, caisson foundation, and steel-pipe sheet pile because the embanked rivers in Japan run down mostly through the alluvial plain and the foundation ground is deep in many cases.

- Relatively shallow bearing stratum (or when soil improvement is included)
- Spread foundation :Footing foundation, raft foundation
- Deep bearing stratum
- Pile foundation : Prefabricated pile, cast-in-place pile, caisson type pile
- Caisson foundation : Opened caisson, pneumatic caisson
- Steel-pipe sheet pile foundation : Well type, leg type
- Combined foundation
- Piled raft foundation (Footing foundation combined with pile foundation)