No.

ROAD DEVELOPMENT AUTHORITY
MINISTRY OF HIGHWAYS AND ROAD DEVELOPMENT
THE DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA

THE DETAILED DESIGN STUDY ON THE OUTER CIRCULAR HIGHWAY TO THE CITY OF COLOMBO

FINAL REPORT
(FOR NORTHERN SECTION 1)
DESIGN STANDARDS
3 of 10

February 2008

JAPAN INTERNATIONAL COOPERATION AGENCY

Oriental Consultants Company Limited

Pacific Consultants International

SD
CR(3)
08–016

ROAD DEVELOPMENT AUTHORITY
MINISTRY OF HIGHWAYS AND ROAD DEVELOPMENT
THE DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA

THE DETAILED DESIGN STUDY ON THE OUTER CIRCULAR HIGHWAY TO THE CITY OF COLOMBO

FINAL REPORT
(FOR NORTHERN SECTION 1)
DESIGN STANDARDS
3 of 10

February 2008

JAPAN INTERNATIONAL COOPERATION AGENCY

Oriental Consultants Company Limited

Pacific Consultants International

ROAD DEVELOPMENT AUTHORITY
MINISTRY OF HIGHWAYS AND ROAD DEVELOPMENT
THE DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA

THE DETAILED DESIGN STUDY ON THE OUTER CIRCULAR HIGHWAY TO THE CITY OF COLOMBO

FINAL REPORT
(FOR NORTHERN SECTION 1)

DESIGN STANDARDS OF HIGHWAY

February 2008

JAPAN INTERNATIONAL COOPERATION AGENCY

Oriental Consultants Company Limited

Pacific Consultants International

THE OUTER CIRCULAR HIGHWAY
TO
THE CITY OF COLOMBO

DESIGN STANDARDS OF HIGHWAY

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

Design Standards of Highway

1. Introductions

1.1. Background

The JICA Study Team has studied the expected geometric design standards of OCH taking into account the following policies.

- To ensure necessary levels of safety and comfort for drivers by the provision of adequate sight distances, coefficients of friction and road space for vehicle maneuvers;
- To ensure that the road is designed economically
- To ensure uniformity of the alignment
- To determine geometric design criteria applicable for use in Sri Lanka.

From the engineer's points of view, the JICA Study Team recommended the RDA to adopt the Japanese Standards as a basic standard taking into account similar geography and topography in Sri Lanka. The RDA agreed with this recommendation however the standards should be rectified to suit for local conditions if necessary.

Various studies have been conducted, which include comparative studies on the geometric design standards adopted for highways linking to the Outer Circular Highway, such as Colombo-Katunayake Expressway and Southern Highway and other matters of influence to the Project.

The details given below in this chapter describes the analysis and various parameters involve in the geometric design of OCH.

Table 1-1 Adopted Design Standards and Major Design Elements

Project Name	Standards	Design Speed (km/h)	Number of lanes in ultimate Stage (lane width)	Max Gradient (%)	Radius (m)	Max. Super-elevation (%)	Remarks
Colombo. Katunayake Expressway	NAASRA & RDA	110	4 (3.6m)	3 (Des) 5 (Abs)	*1500* (Des) *600* (Abs)	*5*	
Outer Circular Highway	Japanese **amended to suits local conditions*	80	6 (3.5m)	4(Des) 7(Abs)	*400* (Des) *280* (Abs)	*6*	
Southern Highway	AASHTO	120	6 (3.6m)	3 (Des) *4 (Abs*)	1250 (Des) *755* (Abs)	*4* *6*	

1.2. Definition of Terms

The following technical terms as defined below are used in this Report

- **Roadway:**
  A highway cross section including shoulders, provided for vehicular use.

- **Carriageway:**
  The portion of the roadway cross section provided for the movement of vehicles, exclusive of shoulders.

- **Shoulder:**
  The portion of the roadway contiguous with carriageway for accommodation of stopped vehicles, for emergency use and also for lateral support of Sub base, base, and surface courses. Hard Shoulder is the portion to be paved or surface treated and the soft shoulder is the portion to be covered by sod or turf.

- **Marginal Strip:**
  The portion of the shoulder with the same pavement structure of the traveled way extended usually 0.25m – 0.3m. This is also the space for road marking at both ends of carriageway.

- **Median (Center Strip):**
  A cross section element provided to separate a lane by directional separation and ensure lateral clearances.

- **Traffic Lane:**
  A strip section of the carriageway (except for the service road) provided for safe and smooth traffic by directional separation of a row of vehicles.

- **Service road (Frontage Road):**
  A parallel carriageway provided to applicable sections to ensure access of vehicles to roadsides where access is prevented for the reason of embankment and cut, or other.

- **Approach road:**
  The existing roads, which are non accessible once the OCH is constructed, shall be compensated to ensure the existing access. An overpass or underpass structure shall be constructed in order to cross the OCH.

2. MAIN CARRIAGEWAY

2. Main Carriageway

2.1. Design Vehicle

The size and physical characteristic of the vehicles essentially form geometric features, such as cross section elements, widening on curve, and corner treatment at intersection, gradient, sight distance and so forth. In the feasibility study, the AASHTO design vehicles are recommended to be used for designing of OCH, since it is specified in the Geometric Design Standards produced by RDA 1998. However, it seems that the Japanese design vehicles are more appropriate to apply taking into account various usage of Japanese vehicles in Sri Lanka. In addition, the design vehicle for STDP is applied to Japanese design vehicle for the economic and practical reasons. Accordingly, the JICA Study Team recommended using the Japanese design vehicles in the study.

The figures in **Fig. 2-1** shows design vehicles are, which quoted from the Japan Road Ordinance says that the dimensions of the vehicles are conformed to the relevant laws and regulations of transportation enforced in Japan.

The design vehicles shown in **Fig. 2-1,** the semi-trailer is almost similar to AASHTO design vehicle type WB – 15. Therefore, the proposed Japanese design vehicles are appropriate for use in designing of OCH. The minimum turning path will follows AASHTO design vehicle type WB –15.

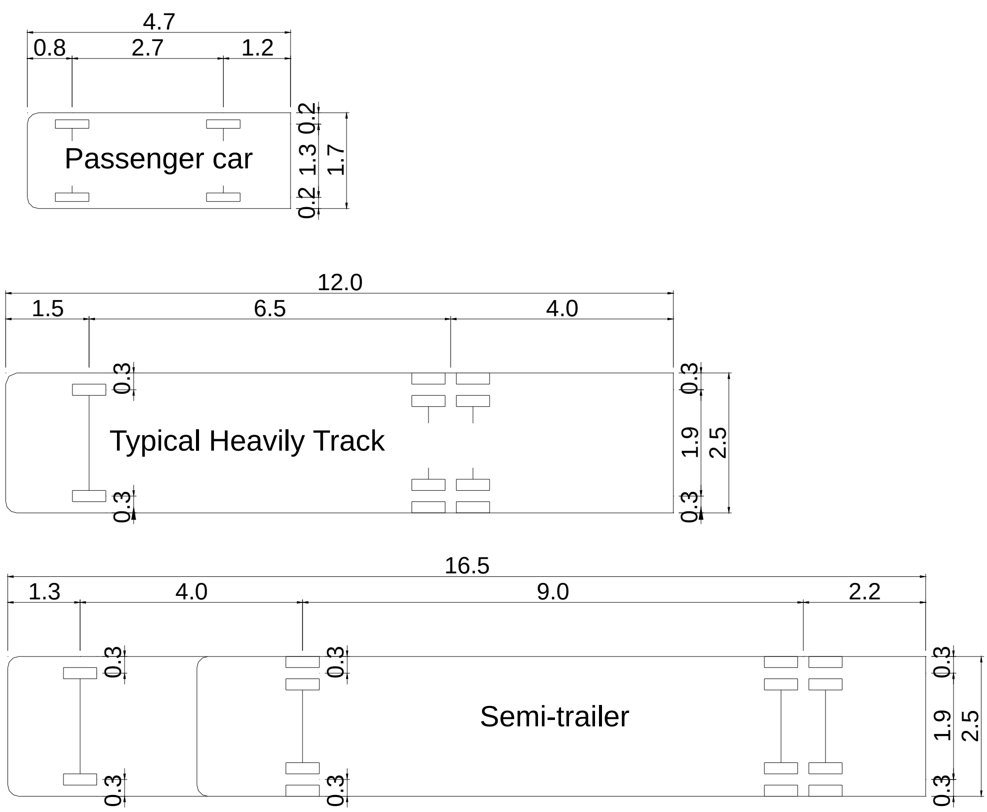
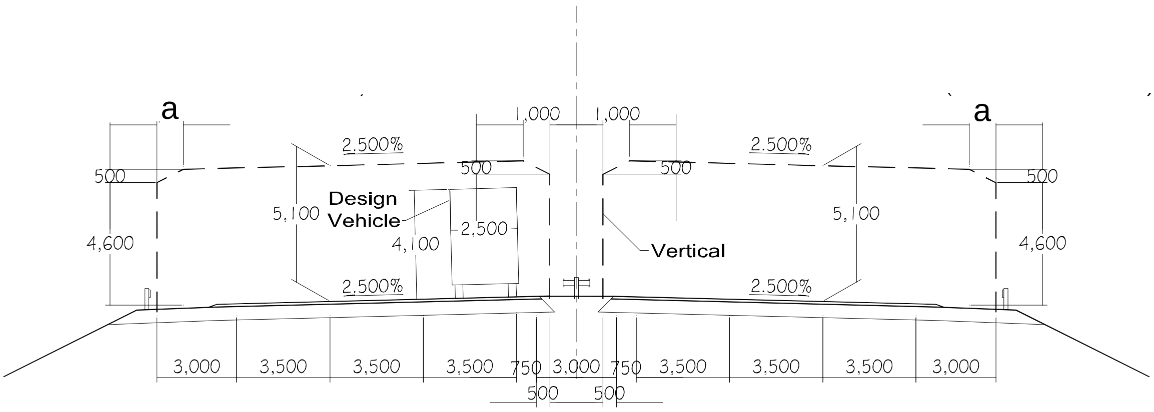


Fig. 2-1 Design Vehicles

2.2. Clearances

2.2.1. Clearances for Main Carriageway

The vertical clearance and lateral clearance for OCH given in **Fig. 2-2** as discussed with RDA. The both top side end of clearance shall be shaped same as the hunch to reduce concise the clearance by economic reasons in Japan.



a: the width varies depending on the width of shoulder but not more than 1.0m

Fig. 2-2 Vertical & Lateral Clearance

2.2.2. Clearances for National Roads and Railways

Minimum vertical clearances for the existing national roads are determined after the series of discussions with RDA in 2001. However, the RDA requested JICA Study Team to rectify the vertical clearance for C and D class roads by the letter issued on 9th September 2003.

As for the clearance of railways, it was given by the letter dated on 20th from RDA attached with the clarification from SLR (Sri Lanka Railways).

The vertical clearances applied to the study are as shown in **Table 2-1.**

The vertical clearances, which have been shown in **Table 2-1**, conform to the AASHTO standard, which requires min. clearance of 4.3m for minor roads and 4.9m for major roads. NAASRA specifies a vertical clearance of 5.5m for service wires, 5.4m preferred and 4.6m minimum clearances for highways.

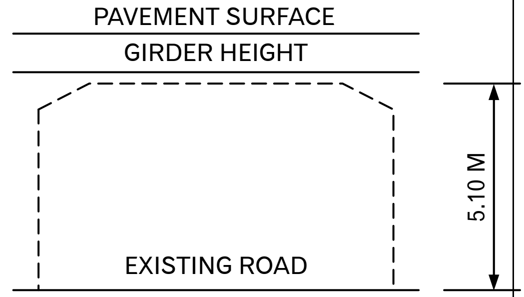
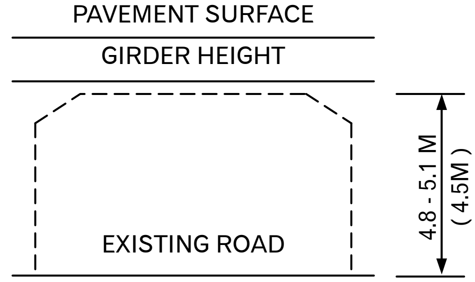
The clearances for the overhead signs and pedestrians are specified according to RDA standards as below.

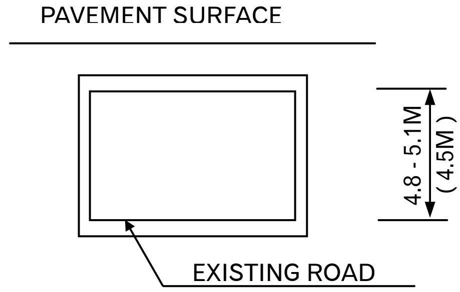
Overhead Signs : 5.7m

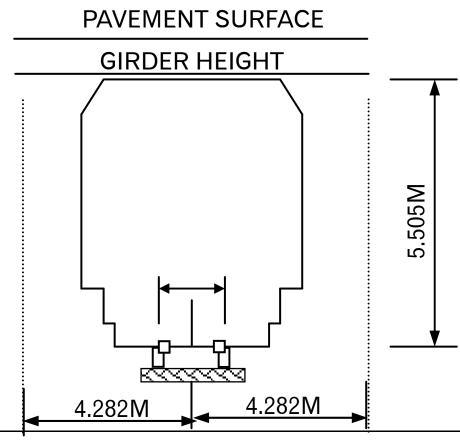
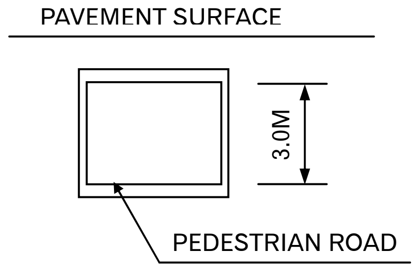
Pedestrian Bridges : 5.7m

These clearances are provided for design of national highways should be taken into consideration.

Table 2-1 Vertical Clearance

<table>
<tr><th>Type of Crossing</th><th>Classifications</th><th>Sketch</th></tr>
<tr><td>Crossing with Major Roads</td><td>- Expressways
- A3
- A1
- B214
- AB10
- B240
- A4</td><td>
</td></tr>
<tr><td>Crossing with Minor Roads</td><td>- C, D, E
- Other Provincial (Gravel) Roads</td><td>

( ): E class and Other Road)


( ): E class and Other Road)</td></tr>
<tr><td>Crossing with Railways</td><td>- Sri Lanka Railways</td><td>
</td></tr>
<tr><td>Crossing with Pedestrian Roads</td><td></td><td>
</td></tr>
</table>

2.3. Design Speed

The ranges of design speeds are provided in Japan by the different road classifications, depending mainly on terrain and traffic volume, and that the standards used should be consistent on long length of roads. Consideration should be given to the economic tradeoffs between the increased construction costs of higher standards and the saving in operation costs which result. Most of these savings in operating costs will be saving in travel time from higher speeds of travel.

According to the feasibility study, the Outer Circular Highway will serve slight different role of general expressway linking among urban cities. It should play a role of an intercity expressway in which dispersing the traffic centralizing to Colombo should be more important than the benefit of time saving. Therefore, it is important to improve the mutual accessibility between arterial roads and economic growth zones through Outer Circular Highway.

Hence, the design speed adopted for OCH is 80km/h, which the speed categorized to meet the requirements of an expressway.

Table 2-2 Adopted Design Speed of Each Highway in Sri Lanka

Expressways in Sri Lanka	Design Speed
Outer Circular Highway	80km/h
Southern Highway	120km/h
Colombo-Katunayake Expressway	110km/h

2.3.1. Restriction of Applicable Different Design Speeds

The design speed may vary within a short distance due to the changing zone, topography, planning and traffic volume in a given section.
If the design speed changes in a short distance, it is undesirable for the users in terms of traffic safety and driver’s comfort. Therefore, it is recommended that the minimum distance for a design speed should be determined and consistently maintained. The minimum distance stipulated in Japan is given in **Table 2-3** for reference.

Table 2-3 Minimum Distance for One Design Speed

	Standard	Special Case
Minimum distance for one design speed	30 – 20 km	5 km

2.3.2. Transition between Different Design Speeds

The difference in design speeds at connection points normally need to be kept within 10-20 km/h. The design speed is changed according to variations in topography and zone, or at interchanges. The transition between OCH (80km/h) and STDP (120km/h) is over 20 km/h. Section for transition (operation speed: 100km/h) will be required at the south of Kottawa Interchange when STDP starts 6-lane operation.

2.4. Cross Section Elements

The typical cross sections of OCH have been determined with the agreement of RDA. The cross section consists of lanes, center median, shoulders, etc. This cross section width has been established based on Japan Highway Design Manual in relation to design speed and traffic demand forecast. The determination of width of each element and the concept in relation to expressway standard is presented hereinafter. Note that the cross-section of the OCH will ultimately have three lanes per direction and total six lanes, while the initial OCH will have two lanes per direction and total four lanes.

2.4.1. Traffic Lane

The standard lane width for Japanese expressway is 3.5 m. When the first expressway (Meishin Expressway) in Japan was constructed, a lane width of 3.6 m was adopted in accordance with the practices in various foreign countries. Based on the lane studies done on the operation of the representative expressways, a lane width of 3.5 m has been found to be adequate to adopt as the standard width for the expressway in Japan.

The Southern Highway (AASHTO) recommends 3.60m (App. 12ft) lane widths for the main carriageway. However, the design speed adopted for Outer Circular Highway is different from that of STDP (as per section 3.2.3 Design Speed), so that it doesn’t warrant a consistent lane width of 3.60m for the OCH as the direct linking expressway.

The volume of traffic and the type of vehicle in the traffic stream are the main factors affecting pavement width. According to the traffic demand forecasts for the Outer Circular Highway, the lane widths of 3.50m to 3.65m, are desirable considering the reductions in capacity, driver comfort and safety associated with narrower lanes.

The JICA Study Team, based on the feasibility study, recommends a lane width of 3.5m to adopt at OCH for the following reasons.

- Conformity of Design Standard:
  3.50m lane width that conforms to the design standards in Japan can lead to a safe and economical design.

- Classification of the Highway:
  OCH is classified as the intercity expressway that does not require high traveling speed, as the Southern Highway.

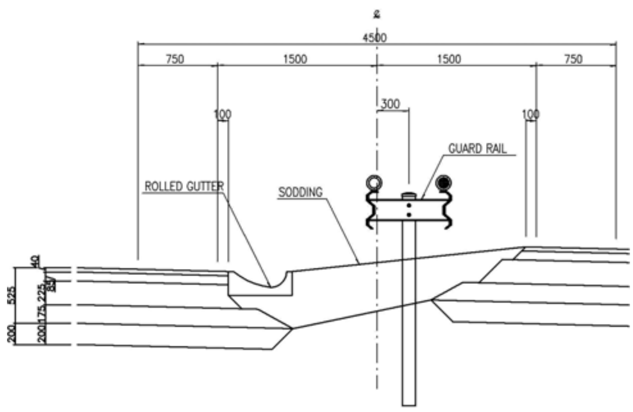
- Vehicle Speed:
  OCH is applied to different design speed of 80km/h that the required circumstance may be considered as providing acceptable levels of service.

2.4.2. Center Median

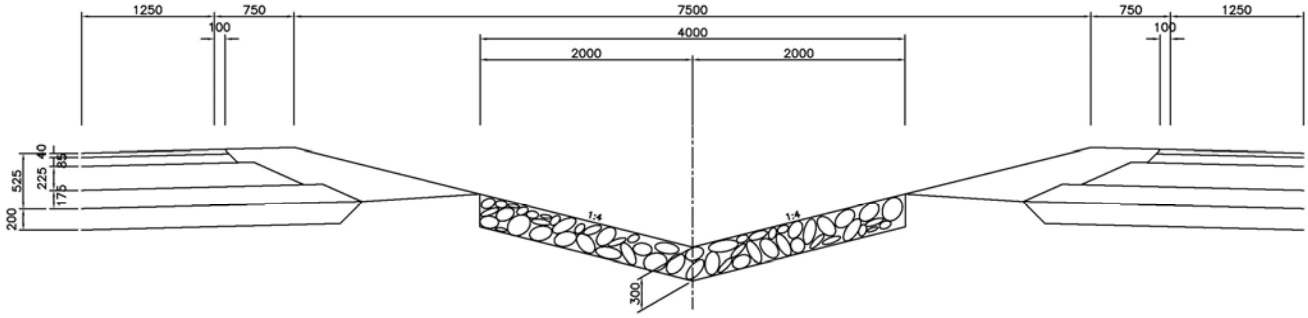
This consists of a median and a marginal strip. The median separates the two-way traffic flow to prevent turns and minimize disorder in the traffic flow in order to ensure safety environment. The center median for OCH will be provided with a guardrail to facilitate these functions at the ultimate stage of six traffic lanes. The marginal strip provided in center median has the function of maintaining the lane effect by indicating clearly the external line of the traffic line, guiding the driver's vision, increasing driver's safety and providing a lateral clearance.

In order to improve the visibility, a white marking line with 20cm wide indicating the outer line of the carriageway is commonly drawn on the marginal strip. The basic width of the center median is 3.0 m in Japan. This is sufficient to ensure the lateral clearance and it should not be affected by the installation of guardrail or by the landscape within the median.

The minimum width of the center median for expressways in Japan is 2.0 m as it requires sufficient width to accommodate any facilities installed within the median. Under clearance to the structures that NAASRA recommends lateral clearance from shoulder to guard fence to be 0.3m. This could be taken into consideration at the ultimate stage where the guard fence is to be placed in the center of the median. The center median at ultimate 6 lanes (flat type) given in **Fig. 2-3**, which has been adopted and agreed by the RDA at the feasibility study. The center median at initial 4 lanes is given in **Fig. 2-3** as well.



Ultimate 6 Lane



Initial 4 Lane

Fig. 2-3 Center Median

(1) Median Opening (Emergency Crossing)

Necessary opening of median shall be applied at required interval for the maintenance purpose, to divert the traffic during maintenance works. The location of median opening has specified in **Fig. 2-4**.

- where the alignment secure enough visibility at grade section (over 600 m radius of horizontal curve)
- before and after interchange
- standard interval is about 2 km

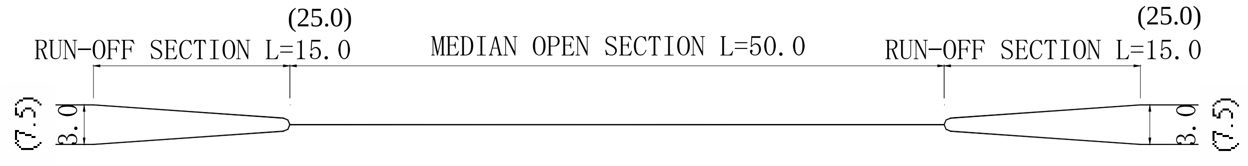


Fig. 2-4 Median Opening (Emergency Crossing)

2.4.3. Shoulder

The shoulder for the expressway should play the following roles.

- To provide the space for the treatment of traffic disturbances caused by the disabled cars.
- To secure the traffic safety and driver’s comfort by providing lateral clearance.
- To protect the carriageway.

The shoulder width should be determined taking into account the functions above. In Japan, the standard width of passenger car is 1.70m and the truck is 2.50m. Accordingly, 2.50m wide shoulder is sufficient to provide the adequate space off the traffic lanes for any kind of disabled vehicle; the width should be at least 1.70 m, assuming only passenger cars. The basic concept is that the shoulder installed to the left of the outer lane should provide space for a disabled car.

The inner shoulder is not installed in cases of standard cross section has center median. However, roads with grade separation and those where the two way directions of traffic are separated by means other than a center strip, the inner shoulder is installed. As for pavement structure of the shoulder, the surface course could eliminate to reduce the construction cost, as the vehicle is not running frequently.

The gap should be treated as tapered to the shoulder not to require suddenly drivers maneuver. Also, there is a marginal strip if 0.75m out side of traffic lane, therefore it could be affectivity on safety.

In comparison with other international standards, the 3.00m wide inner shoulder, which is applied to OCH (including marginal strip of 0.75m and stabilized median of 1.25m) is compatible to AASHTO and NAASRA.”

Table 2-4 Shoulder Width of Each Standard

		Shoulder Width	
		Inner	Outer
AASHTO		1.2 -2.4m	3.0m
NAASRA	4 Lanes	1.2 m	3.0 m
	6&8 Lanes	2.4 m	3.0 m
OCH	Initial 4 lanes	1.25 m	3.0 m
	Ultimate 6 lanes	*0.75 m (1.25m) *Marginal strip for center median ( ):lateral clearance	3.0 m

2.4.4. Verge Width

The desirable standard of 0.75 m width of outer verge specified in Japanese Standards has been adopted. Also, 0.75m width of inner verge will be adopted at the initial stage of four lanes.

2.4.5. Crossfall

The crossfall of main carriageway adopted is 2.5% at the feasibility study as the normal crossfall. To facilitate discharging run-off-water, the crossfall of 4.0% on shoulder is recommended which is steeper than normal crossfall of 2.5%. "In case that algebraic difference between superelevation of main carriageway and crossfall of shoulder is over 6%, the shoulder superelevation shall be adjusted up to the algebraic difference is 6%. However, when the superelevation of the main carriageway becomes more than 4%, it is allowed to adopt algebraic difference 8%."

Superelevation development for shoulder depending on the superelevation of main carriageway shows in **Fig. 2-5**.

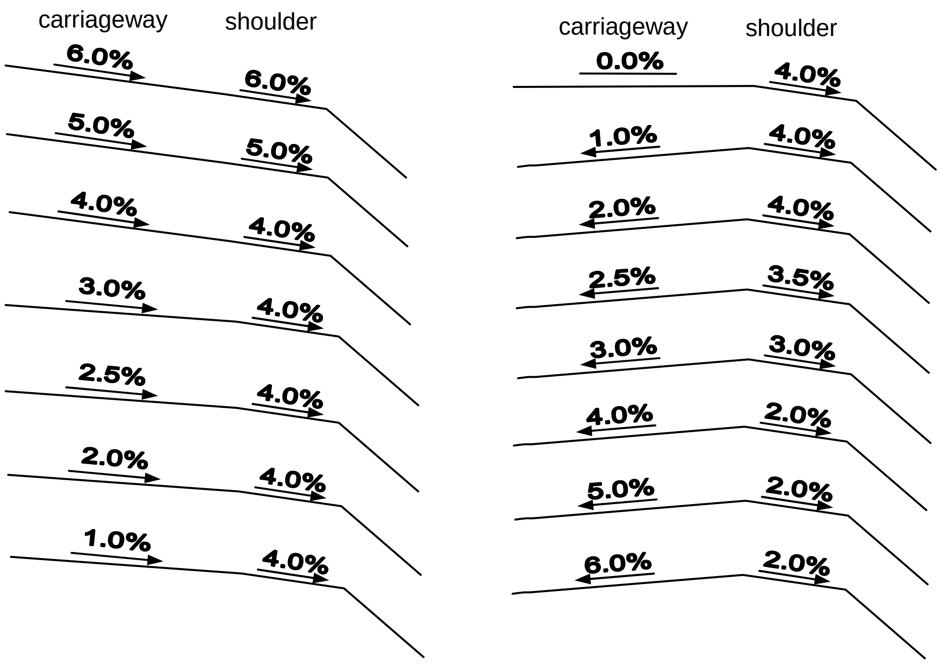


Fig. 2-5 Variation of Crossfall

2.5. Proposed Height and Axis of Rotation

The proposed height (PH) which is indicated in the design profile is the point about which the crossfall is rotated to develop the superelevation. The position of the axis of rotation on the cross section of OCH has been placed at the center of three lanes in ultimate 6 lane carriagway as shown in **Fig. 2-6** and **Fig. 2-7**.

2.6. Right of Way Setting

For the setting of the right of way, the reservation that be applied to STDP is commonly adaptable to use for OCH taking into account the consistency between the projects directly linking each other. At the normal earthworks section except the some constraints as below, 5.0 m reservations even from the edge of cut and fill slope will be kept, in order to accommodate the side storm drainage, boundary fencing and other necessary facilities. The reservation will be also used for the temporary yard at the construction stage and landscaping or other buffer zone at the operation stage.

- boundary at the high- density urban area
- specific circumstances concerning land acquisition
- reduced 2.0 m reservations at the frontage section along OCH
- extra reservation needed adjacent to high cut slope to allow for the erosion

For the approach roads crossing OCH, 5.0 m reservations will be secured at both road sides.

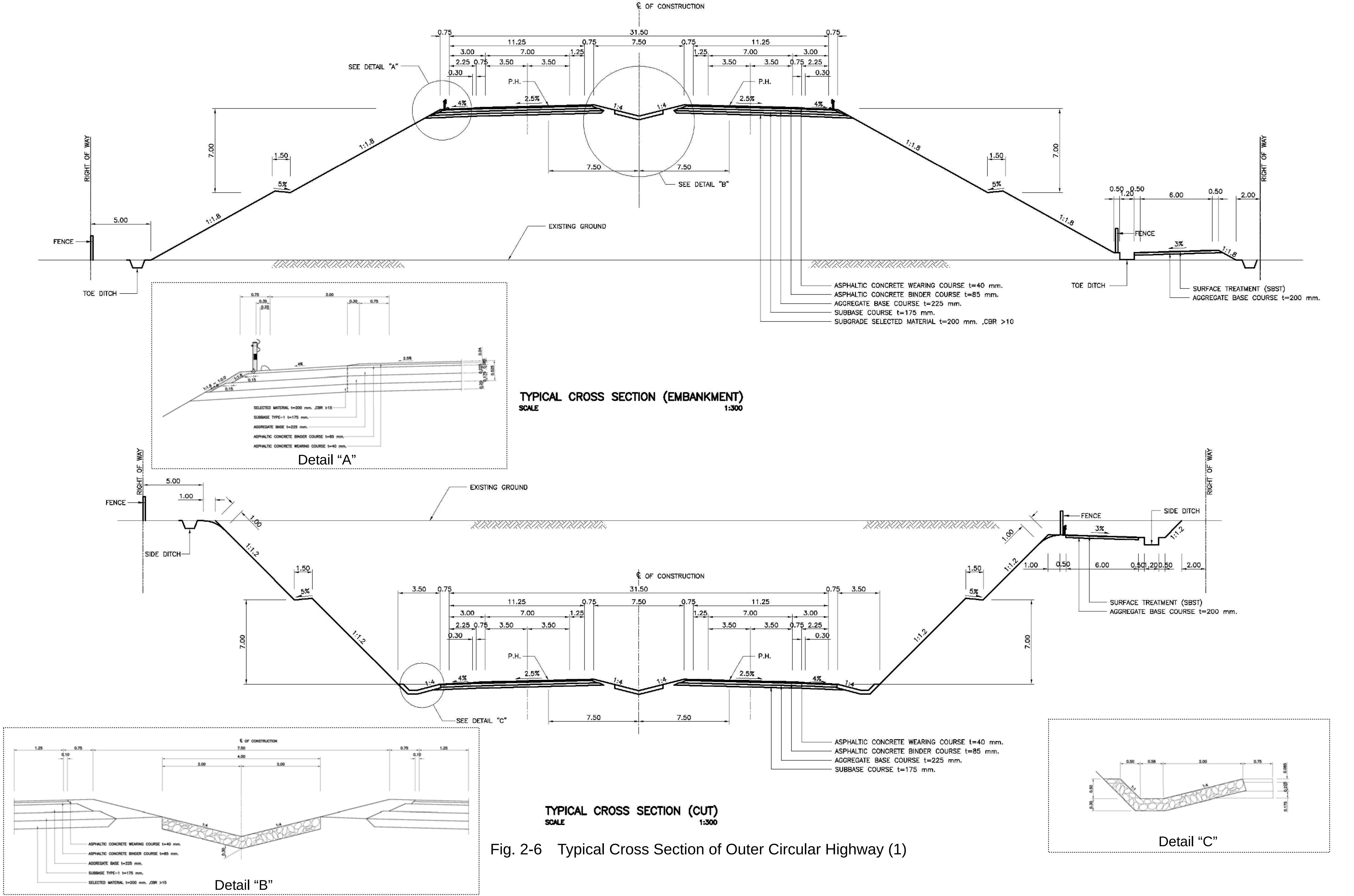


Fig. 2-6 Typical Cross Section of Outer Circular Highway (1)

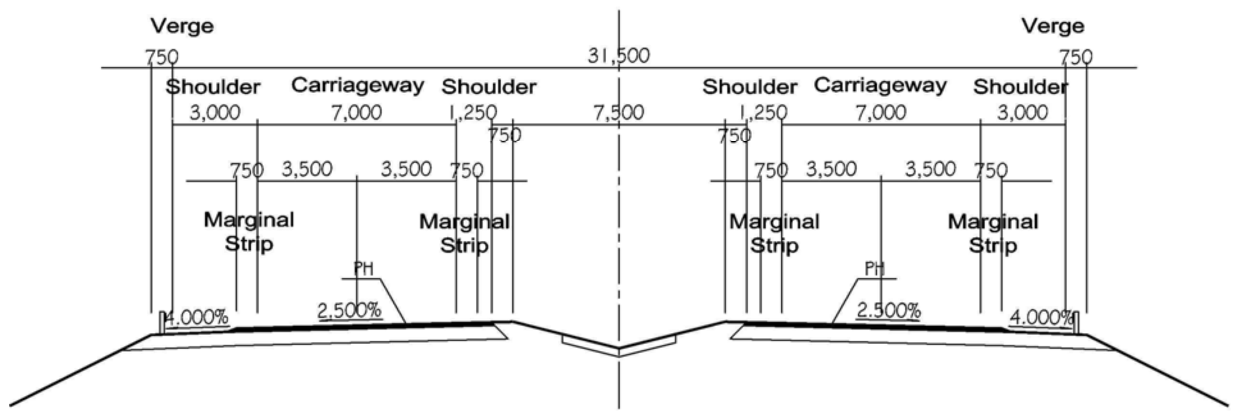
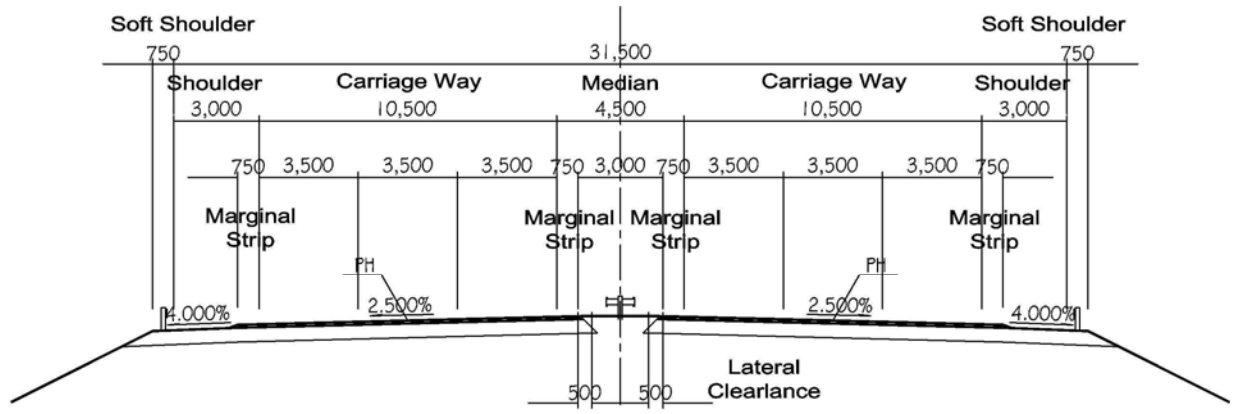
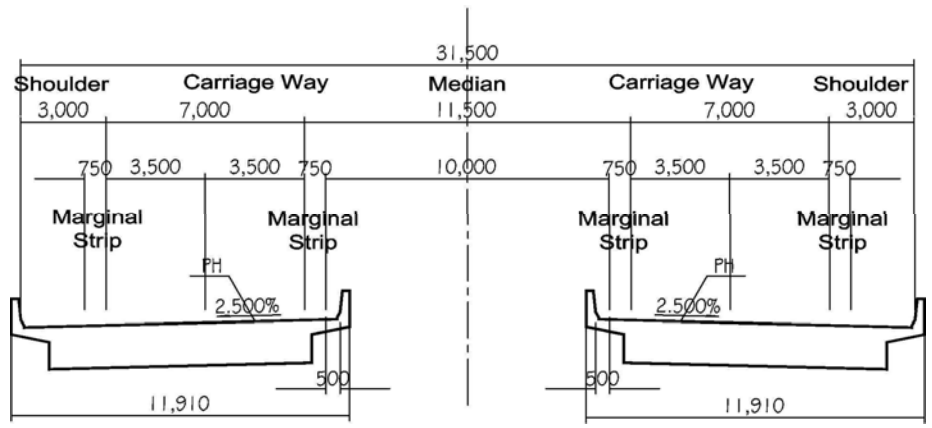
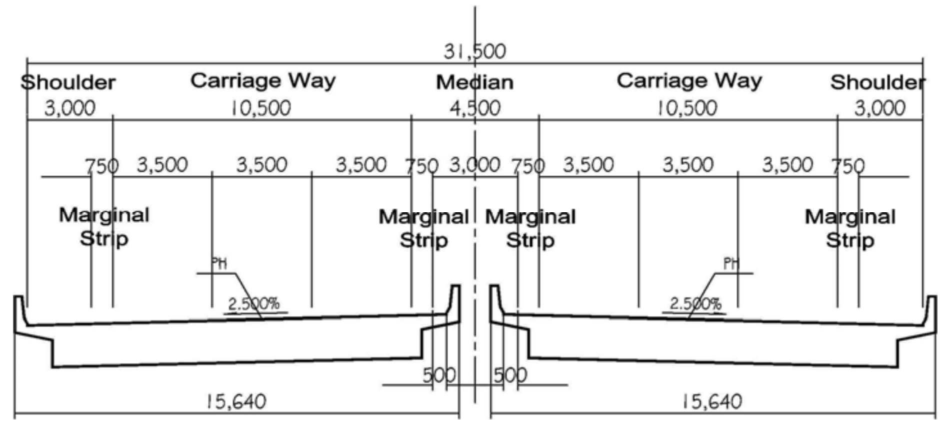
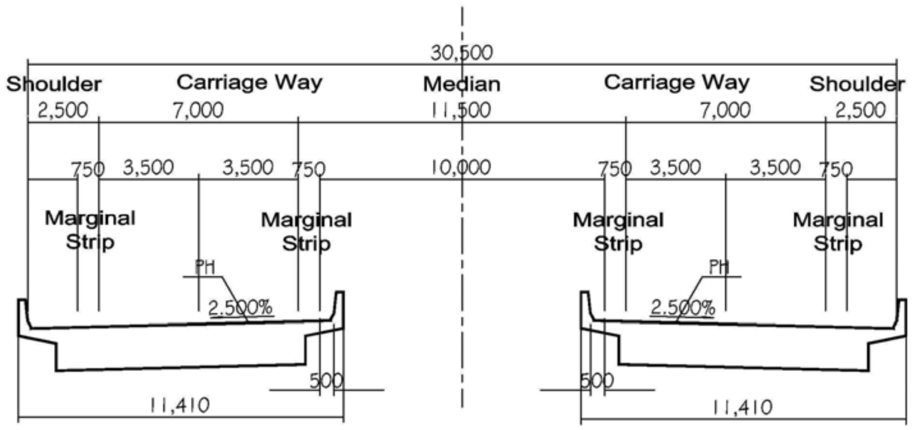
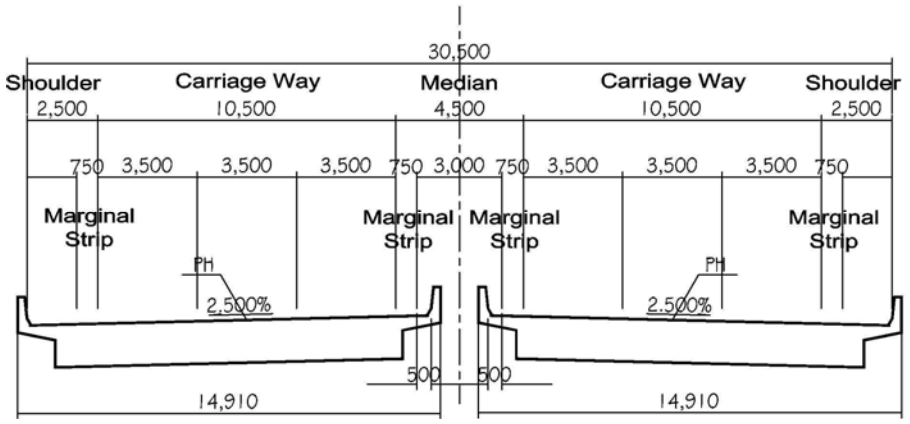
<table>
<tr><th></th><th>Initial Stage</th><th>Ultimate Stage</th></tr>
<tr><td>Unseparated Road Section</td><td></td><td></td></tr>
<tr><td>Minor Bridge</td><td></td><td></td></tr>
<tr><td>Major Bridge (Bridge Length more than 100m)</td><td></td><td></td></tr>
</table>

Fig. 2-7 Typical Cross Section of Outer Circular Highway (2)

2.7. Road Alignment

Road alignment includes horizontal and vertical alignment; the horizontal alignment is composed of straight lines, circular curves and transition curves, and the vertical alignment is composed of straight lines and vertical curves. The standard minimum values for these elements are described below.

2.7.1. Horizontal Alignment

This consists of straight lines, circular curves and spiral (clothoid) curves to be used as transition curves. The general policy for horizontal alignment design is as follows;

(a) The alignment should be suitable to the topography.
(b) The alignment should be continuous with no rapid changes.
(c) Sufficient curve length should be maintained to prevent an illusion in which the curve looks less sharp than it actually is. This is a particular problem where the radius of the curve is small.

(1) Minimum Radius of Horizontal Curve

The radius of the circular curve given is the minimum for guaranteeing driving safety and vcomfort and the applicable radius is generally far bigger than this value.
Minimum curve radius (R min.) for the Outer Circular Highway given by design speed (v= 80 km/h) can be determined using the following equation.

$R_{min} = v^2 / 127 * (e_{max} + f_{max})$
Where e_{max} … maximum superelevation
f_{max} … maximum side friction factor

Summary of the above factors are given in **Table 2-5**.

Table 2-5 Desirable Minimum Radius of Horizontal Curve

Design Speed (km/h)	80
Max. Allowable Side Friction Factor (f)	0.05
Max. Superelevation (i max) %	6.0
Desirable Minimum Radius (m)	400

Table 2-6 Minimum and Absolute Minimum Radius of Horizontal Curve

Design Speed (km/h)	80 - Expressway -		
	AASHTO	NAASRA	JAPANESE
f max	0.14	0.14	0.12
e max	8-12	6-10	10
min.Radius (m)	250	300	280 (230)

( ) means absolute value

The Japanese Standard defines lower value for maximum side friction and for the maximum super-elevation than those values given in AASHTO. However, the minimum radius of 280m is also governed by AASHTO standard. Therefore, it is safe and appropriate to adopt the Japanese Standard for the detail design of OCH.

Table 2-7 Values of Superelevation related to Horizontal Curve

Design Speed	80	Superelevation (%)
Radius of curve (m)	Less than 710	6.0
	From 710 to 790	5.5
	From 790 to 900	5.0
	From 900 to 1030	4.5
	From 1030 to 1190	4.0
	From 1190 to 1400	3.5
	From 1400 to 1680	3.0
	From 1680	2.5

The values of superelevation related to horizontal curve shown in the above table are a little more conservative than the AASHTO standard. It is slightly safer than the AASHTO. Therefore, the proposed superelevation development of horizontal curve is appropriate.

(2) Horizontal Curve Length

In order to ensure comfortable driving, the minimum horizontal curve brings into sufficient length to allow secure comfort steering for the drivers at the change of curve.

1) Minimum Horizontal Curve Length by the Required Steering Time on Curve

In Japan, the minimum required steering time on curve should be more than 6 seconds that may not incur any drivers' stress practically. Accordingly, the equation imply to the OCH design speed has given following figures.

$$L = t*v$$

Where t : the required steering time on curve (sec)
v: design speed (m/s)

2) Minimum Horizontal Curve Length in Practical Appearance

For small deflection angles, curves should be sufficiently long to avoid the appearance of a kink. Curves should be at least 150 m long for a central angle of 5° and the minimum length should be increased 30 m for each 1° decrease in the central angle. The minimum length of horizontal curve on main highways, L, should be about 3 times the design speed, or $L_{o\ min} = 3V$. On high speed controlled-access facilities that use flat curvature, a desirable minimum length of curve for aesthetic reasons would be about double the minimum length, or $L_{o\ des} = 6V$.

Therefore, $L_{o\ des}$ = 6 x 80 km/h = 480 m, $L_{o\ min}$ = 3 x 80 km / h = 240 m

Table 2-8 Minimum Horizontal Curve Length

Design Speed (km/h)		80
Minimum required steering time on curve (sec)		6
Minimum horizontal curve length by the required steering time on curve (m)	Calculated	133
	Rounded	140
Minimum horizontal curve length is Practical Appearance (m)	Desirable	480
	Minimum	240

3) Minimum Transition Curve Length

Transition curve are inserted between tangents and circular curves, or between circular curves of substantially different radius for the following reasons:

- to provide a gradual increase or decrease in the radial acceleration when a vehicle enters or leaves a circular curve.
- to provide a length over which the superelevation can be applied.
- to facilitate pavement widening on curves.
- to improve the appearance of the road by avoiding sharp discontinuities in alignment at the beginning and end of circular curve.

The type of transition curve that is normally used in practice, is Euler spiral, or clothoid. This spiral is defined by the degree of curvature at any point on the spiral being directly proportional to the distance along spiral.

In Japan, the minimum transition curve length required steering time on curve should be more than 3 seconds that may not incur any drivers' stress practically. Accordingly, the equation implying the OCH design speed has given following figures.

$$L = t*v$$

Where t : the required steering time on curve (sec)
v: design speed (m/s)

Table 2-9 Minimum Transition Curve Length

Design Speed (km/h)		80
Minimum required steering time on curve (sec)		3
Minimum transition curve length by the required steering time on curve (m)	Calculated	67
	Rounded	70

The minimum transition curve length is derived from the formula above and which conforms to that of AASHTO. Therefore, the proposed minimum transition curve length of 70m for the design speed of 80 km/hr is appropriate.

When the length of the superelevation run-off is over the value, the transition curve length shall comply with the length of the superelevation run-off.
In the actual design, the parameter of spiral curve shall be set to satisfy necessary transition curve length, but it will also determine the conditions for obtaining a visually smooth alignment (select from the range of 1/1 to 1/3, of circular curves, for parameters of spiral curve). Generally, the length becomes far longer than that of the steering time on curve.

Further to this, the length of transition curve, as determined by rotation angle and changing ratio of centrifugal acceleration will become shorter, as the radius of the circular curve becomes larger. Visually desirable transition curve length characteristically becomes longer as the circular curve radius becoming bigger.

4) Minimum Radius of Curve Omitting Transition Curve

The Japan Highway Design Manual recommends that for appearance purposes, length of transitions should be sufficient to provide a shift of 0.2 meters. If the shift is less than 0.2meters, the transition curve is omitted.

When continuing straight line and circular curve; if size of the circular curve is more than R=2000m at the design speed 80km/h shown in the **Table 2-10**, transition curve can be omitted.

$R= 1/24*L^2/S$

Where

S: Shift in meters between curve and tangent
L: Transition curve length (m)
R: Radius of circular curve (m)

Table 2-10 Minimum Radius of Circular Curve

Shift in meters between curve and tangent (m)		0.2
Minimum transition curve length (m)		70
Minimum radius of circular curve (m)	Calculated	900
	Rounded	2000

From the above, 900 m is calculated by using minimum transition curve length. However, from experience in Japan, this value of circular curve is not sufficient visually. Therefore, the desirable radius of curve is recommended to use about twice of calculated value.

2.7.2. Vertical Alignment

(1) Gradient

In Japan Highway Design Manual, it is recommended to apply the desirable maximum gradient given in **Table 2-11** as far as possible. The critical gradient is defined that a typical truck is able to climb up on that gradient with a half of design speed (App. 40km/h). In some instances, a gradient higher than the desirable maximum may be applied but the length of that segment should be limited to the specified value. In a flat area, the minimum gradient is specified at 0.5 – 0.3% to ensure drainage. The gradient of 0% could be applied in some cases but the road surface drainage must be considered carefully. It is preferable to limit the length of level gradient to be as small as possible.

Japanese standards regulate and control the absolute limit length at maximum gradient, so that no extreme difficulty is caused by the traffic where a steeper gradient than the desirable steepest gradient, is applied. When limiting gradient length, it can be eased to apply a climbing lane, however when each absolute limit length is controlled, in most cases a climbing lane is needed, except when the traffic volume is very small.

The critical lengths of gradient determined by AASHTO and RDA (not for expressway) given in **Table 2-11** as well. Length on the Japanese Standard is longer than the RDA Standard, and Length on AASHTO for the maximum speed reduction of 15 km/h is the shortest of all.

Therefore the Length on AASHTO shall be adopted for OCH. However, it will not be applied in the section through the OCH.

Table 2-11 Limit of Length to Maximum Gradient

Design Speed (km/h)	Maximum Gradient (%)	Absolute Maximum Gradient (%) ** limit of length shown in parenthesis		
		Japanese (50% speed reduction allowed)	AASHTO (15km/h speed reduction allowed)	RDA (Reference)
80	4	5 (600) 6 (500) 7 (400)	5 (210) 6 (180) 7 (80)	5 (250) 6 (200) 7 (170)

Note: Limit of length is shown in the parenthesis

(2) Minimum Vertical Curve

Vertical curves effect gradual change between tangent gradients in crest and sag curves and should result in a design that is safe, comfortable in operation, pleasing in appearance and adequate for drainage.

The major control for safe operation on crest vertical curves is provision of ample sight distance for the design speed and rider comfort, while headlight sight distance and rider comfort govern the length of a sag vertical curve.

The following equations are used for the calculation of required vertical curve length and radius of vertical curve, of which longer length is applicable.

1) Pleasing in Appearance

$$L = Vd \quad * t/3.6$$

Where
L : Vertical curve Length (m)
Vd : Design Speed
t : Minimum required time, (t= 3 sec from AASHTO).

Table 2-12 Minimum Vertical Curve Length on Crest Curve

Design Vd (km/h)	On Crest Curve Min. Vertical Curve Length (m)
80	70 (67)

2) Crest Curve (object height : 0.12 m , eye- height : 1.05m) * Recommended by RDA

$L = D^2 i / 433$ or $R = 100 * D^2 / 433$

Where
L : Vertical curve length(m)
R : Radius of vertical curve (m)
D : Sight distance(m)
i : Algebraic different in gradient (%)

The following table gives some values for the design speed of 80km/h.

Table 2-13 Minimum Curve Radius on Crest Curve

Design Speed vd (km/h)	Sight Distance (m)	On Crest Curve Min. Radius (m)	
		Calculated	Rounded
80	140	4526	4500

3) Sag Curve

The length of Sag curve is based on head light sight distance, rider comfort, drainage control and a rule of thumb for general appearance.
In general use, head light height is taken as 600mm and 1° upward divergence.

$L = D^2 I / 150+3.5*D$ or $R = 100* D^2 / 150+3.5*D$

Where L: Vertical curve length (m),
D: Sight Distance (m)
R: Radius of vertical curve (m)
I: Algebraic difference in gradient (%)

Table 2-14 Minimum Curve Radius on Sag Curve

Design Speed Vd (km/h)	Sight Distance (m)	On Sag Curve Min. Radius (m)	
		Calculated	Rounded
80	140	3062	3100

4) Definition of K-value

The parabolic vertical curves are defined by the length of curve required for a change of gradient of 1%. This constant for the parabola is K-value.

$K = L / G$ where, L = length of vertical curve (m),
G = Algebraic difference in Gradient (m per %)

The minimum K- values for the OCH for the given criteria are in **Table 2-15**.

Table 2-15 Minimum K-value

Criteria	Minimum K-value (Calculated)	Minimum K-value (Adopted)
Pleasing Appearance	17.8	17.8
Crest Curve (Stopping Sight Distance: 140m)	45.27	45
Sag Curve (Head Light Sight Distance)	30.63	31

(3) Composite Gradient

This criterion, which includes checking whether the combined gradient value, which is the value of superelevation and the gradient, is suitable or not when the section overlaps a gradient and a horizontal curve.

Table 2-16 Maximum Composite Gradient

Design Speed (km/h)	Maximum Composite Gradient (%)
80	10.5

(4) Minimum Radius without Superelevation

In accordance with AASHTO, the minimum radius without superelevation of R=3,500 m is recommended with using longitudinal friction factor f=0.04 when the 2.5%-crossfall is adopted. On the other hand, in the Japanese standard, the minimum radius of R=5,100 m is recommended with using f=0.035 taking into account of driver's comfort. Therefore, the JICA Study Team recommends to adopt R=3,500 m the minimum radius without superelevation and R=5,100 m for desirable value.

Minimum radius without superelevation on 2.5% normal crossfall should be calculated as below:

$$R = V^2 / 127 (i + f)$$

Where	v:	Design Speed	80 (km/h)
	i:	Superelevation	-2.5 (%)
	f:	Longitudinal Friction Factor	0.04 (AASHTO) 0.035 (JAPANESE)

Table 2-17 Minimum Radius without Superelevation

Design Speed (km/h)	Minimum Radius without superelevation (m)		
	Calculated	Rounded	Desirable
80	3,359	3,500	5,100

(5) Superelevation Development

In accordance with the Japanese Standards, ratio of the superelevation development should be less than 1/200 where the position of the rotation axis is at center of lane (OCH: Center of 6 lanes). Superelevation development should be done along the whole length of the transition curve, and its ration should not exceed the appearance value above.

The point of 0 % superelevation corresponds to the start of the transition (for a vehicle entering the curve) and the full superelevation for the curve (e%) is attained at the end of the transition. The superelevation development is extended back from the start at the same rotation to the point of normal cross fall on the approach tangent.

Table 2-18 Maximum Ratio for Superelevation Development

Design Speed (km/h)	Maximum Ratio for Superelevation Development
80	1/200

(6) Minimum Superelevation Development for Secure Drainage

Superelevation development at the carriageway where the superelevation becomes level should not be smaller than the values in **Table 2-19**.

Table 2-19 Minimum Superelevation Development for Secure Drainage

Number of Lane	Minimum Superelevation Development for Secure Drainage
6 Lanes	1/325

This case will be required when shifting from a straight line to a curve, or in the vicinity of a changing point of a reverse curve. **Table 2-20** shows the Standard Length to Secure Minimum Superelevation Development for Secure Drainage. Where the superelevation becomes smaller, the superelevation necessary for drainage should be secured as shown in the table, however, the section adopted must be minimized.

Table 2-20 Standard Length to Secure Min. Superelevation Development for Secure Drainage (m)

Number of Lane	Distance to outer edge	Length of Development
6 Lanes	6.00 m	100 m

Algebraic Difference of Superelevation: 0.05 (-2.5% to 2.5%)

2.8. Sight Distance

Sight distance is an important factor in highway design. Two different kinds of sight distance will be considered, stopping sight distance and passing sight distance. As the OCH is proposed to be a single way traffic highway, the passing sight distance is irrelevant.

2.8.1. Stopping Sight Distance (SSD)

Sight distance is defined as the distance along a roadway that an object of specified height is continuously visible to the driver with eye- height above the road surface. The height of 0.15 m of object height is recommended by AASHTO. The height of driver's eye ranges 1.07m to 1.2m in international standards. 1.05 m as the eye-height and 0.2m as the object height are used for our work as followed to RDA. The following table gives the eye and object height specified in other standards.

Table 2-21 Criteria of Stopping Sight Distance

Standards	AASHTO	NAASRA	RDA	JAPANESE
Driver's eye height (m)	1.07	1.15	1.05	1.20
Object Height (m)	0.15	0.2	0.2	0.10

Stopping sight distance is the sum of two distances:

The distance traversed by the vehicle from the instant that the driver sight an object necessitating a stop to the instant that break are applied (Break reaction time), and the distance required to stop the vehicle from the instant that break application begins (Breaking distance).

2.5 seconds is used for the former and the later is dependent on the initial speed and coefficient of friction between tires and pavement. The following equation is used for the calculation of stopping sight distance;

$$d = 0.278 * t*v + v^2/254f$$

Where
d: Stopping Sight distance (m)
t : Break reaction time, generally assumed to 2.5sec.
v: Initial Speed (km/h)
f :Coefficient of Friction between Tires and pavement

The minimum stopping sight distance of 140m is longer than that of AASHTO standard (av.120m.). The distance depends on driver's eye height, object height and coefficient of friction on wet condition. The minimum stopping sight distance according to the RDA standard is proposed safer than that of other standards. Therefore, the JICA Study Team recommends to adopt the sopping sight distance of 140 m in the OCH as well as that is adopted in other highways in Sri Lanka.

Stopping Sight Distances for design speed under wet conditions are shown in **Table 2-22**.

Table 2-22 Stopping Sight Distance on Wet Pavement

Design Speed (km/h)	80			
Standard	AASHTO	NAASRA	RDA	JAPANESE
Coefficient of Friction	0.30	0.43	0.3	0.31
Stopping Sight Distance	112.8 –139.4	120	140	110

2.9. Comparison of Japanese Standards and Other Standards

In order to select an optimum design standard for OCH, a comparative study among widely used standards in the world (AASHTO & NAASRA) and the Japanese standard have been carried out as shown in **Table 2-23**.

Table 2-23 shows the various parameters used for the design of highways based on standards widely practiced around the world. It is very clear from the table that the values obtained by Japanese Standard are mostly in agreement with all other standards although we find a variation in some parameters. In cases, where there are major variations in parameters, the Japanese standard gives the safest values for the design. The condition prevailing in Sri Lanka in every aspect of highway design such as the terrain, type of vehicle and the design speed mostly suits to some existing highways in Japan. Therefore, it is strongly recommended to adopt the design criteria quoted on the Japanese Standard and suitably modified to suit for the Outer Circular Highway in regard to the Sri Lankan conditions.

Table 2-23 Comparisons of Japanese Standards and Other Standards

	Item		Unit	Design Criteria											
	Design Speed		Km/h	60				80				100			
				A	N	R	J	A	N	R	J	A	N	R	J
1.	Maximum Superelevation		%	6	5 -7	6	8	**6**	5 -7	**6**	8	6	5 -7	6	8
2.	Minimum Radius		m	125	105	130 (6)	150	250	300	255 (6)	**280**	435	450	420 (6)	460
3.	Desirable Minimum Curve Radius		m	-	130	155	200	-	350	310	**400**		550	515	700
4.	Maximum Gradient		%	5	6	4	5	4	3	4	**4**	3	3	4	3
5.	Stopping Sight Distance		m	74 -85	80	85	75	112 -139	120	**140**	110	157 -205	180	205	160
6.	"K" value of Vertical Curves	Crest	m	14 -18	14	17	20	32 -49	31	**45**	45 30	62 -105	70	97	100
		Sag	m	15 -18	15	17	15	25 -32	25	**31**	30 20	37 -51	42	50	45
7.	Minimum Horizontal Curve Length		m	360 180	-	-	700/θ 100	**480** **240**	-	-	1000/θ 140	600 300	-	-	1200/θ 170
8.	Minimum Transition Curve Length		m	75	-	50	50	85	-	60	**70**	187	-	80 90	85
9.	Minimum Radius Without Transition curve		m	1700	-	-	1000 500	2100	-	-	**2000** 900	7300	-	-	3000 1500
10.	Minimum Radius Without Superelevation		M	1300	900	810	2900	**3500**		1440	5100	5000	2700	2250	7900

Where, A – AASHTO, N-NAASRA, R-RDA, J-Japan Highway's Design Manual
Figure in ( ) is superelevation %, which determines the radius of curve
Note: RDA standard is not for expressway, and given for references.

2.10. Summary of Geometric Design Criteria

Summary of geometric design criteria for OCH main carriageway is shown in **Table 2-24**.

Table 2-24 Summary of the Geometric Design Criteria for OCH Project.

<table>
<tr><th colspan="2">Item</th><th>Desirable Value</th><th>Criteria</th><th>Absolute Value</th><th>Adoption</th></tr>
<tr><td colspan="2">Design Speed</td><td colspan="4">80km/h</td></tr>
<tr><td colspan="2">Min. Radius to Horizontal Curve</td><td>400m</td><td>280m</td><td>230m</td><td>700m</td></tr>
<tr><td colspan="2">Min. Horizontal Curve Length</td><td>480m</td><td>240m</td><td></td><td>246m</td></tr>
<tr><td colspan="2">Min. Transition Curve Length</td><td></td><td>70m</td><td></td><td>229m</td></tr>
<tr><td colspan="2">Min. Radius Without Transition Curve</td><td></td><td>2000m</td><td></td><td>2000m</td></tr>
<tr><td colspan="2">Min. Radius Without Superelevation</td><td>5100m</td><td>3500m</td><td></td><td>3600m</td></tr>
<tr><td colspan="2">Max. Grade</td><td></td><td>4%</td><td>5 – 7%*</td><td>2.850%</td></tr>
<tr><td colspan="2">Min. Vertical Curve Length</td><td></td><td>70m</td><td></td><td>200m</td></tr>
<tr><td rowspan="2">Min.“K” value of Vertical Curves</td><td>Crest</td><td></td><td>45</td><td></td><td>70</td></tr>
<tr><td>Sag</td><td></td><td>31</td><td></td><td>46</td></tr>
<tr><td colspan="2">Crossfall of Carriageway</td><td colspan="4">2.5%</td></tr>
<tr><td colspan="2">Crossfall of Outer Shoulder</td><td colspan="4">4.0%</td></tr>
<tr><td colspan="2">Max. Superelevation</td><td></td><td>6%</td><td></td><td>6%</td></tr>
<tr><td colspan="2">Values of Superelevation related to Horizontal Curve (Design Speed 80km/h)</td><td colspan="4"><table>
<tr><td>Less than 710m</td><td>6.0%</td></tr>
<tr><td>710m to 790m</td><td>5.5%</td></tr>
<tr><td>790m to 900m</td><td>5.0%</td></tr>
<tr><td>900m to 1030m</td><td>4.5%</td></tr>
<tr><td>1030m to 1190m</td><td>4.0%</td></tr>
<tr><td>1190m to 1400m</td><td>3.5%</td></tr>
<tr><td>1400m to 1680m</td><td>3.0%</td></tr>
<tr><td>More than 1680m</td><td>2.5%</td></tr>
</table></td></tr>
<tr><td colspan="2">Max. Ratio for Superelevation Development</td><td></td><td>1/200</td><td></td><td>1/333</td></tr>
<tr><td colspan="2">Max. Composite Gradient</td><td></td><td>10.5%</td><td></td><td>6.014%</td></tr>
<tr><td colspan="2">Stopping Sight Distance</td><td></td><td>140m</td><td></td><td>142m**</td></tr>
<tr><td colspan="2">Traffic Lane Width</td><td colspan="4">3.5m</td></tr>
<tr><td colspan="2">Outer Shoulder Width</td><td colspan="4">3.0m</td></tr>
<tr><td colspan="2">Marginal Strip Width
(at Shoulder and Center Median)</td><td colspan="4">0.75m</td></tr>
<tr><td colspan="2">Right (Inner) Shoulder</td><td colspan="4">1.25m***</td></tr>
<tr><td colspan="2">Center Median Width</td><td colspan="4">4.5m** / 3.0m** (without marginal strip)</td></tr>
</table>

*: Limit Length is regulated **: Ultimate Stage: 6 lanes ***: Initial Stage: 4 lanes

3. INTERCHANGE AND JUNCTION

3. Interchange and Junction

3.1. General

The design criteria for Interchange and Junction for the OCH is prepared based on Design Manual of Japan Highway Public Corporation and a Policy of Geometric Design of Highways and Streets, American Association of State Highway and Transportation Officials (AASHTO).
Design Manual of Japan Highway Public Corporation classifies the connection manner of expressway and highway into the following two categories by the class of highway.

- Interchange
  A type of intersection, an expressway intersecting with a general national road or other normal road, not with another expressway, requires a grade-separated intersection used for entry and exit from the national road to the expressway.

- Junction
  An interchange connecting expressways to one another is called a junction and is distinguished from an interchange above.

3.2. Main Carriageway Alignment at Interchange and Junction

In Japan, the alignment conditions of main carriageway at interchange and junction has been specified as below to ensure sufficient driver's visibility and behavior against the traffic merging or diverging from/to ramp at IC and JCT. These applicable figures as shown in **Table 3-1** are acceptable to the OCH.

Table 3-1 Geometric Design Criteria for Main Carriageway at IC &JCT

		Design Speed of Main Carriageway (km/h)		
		120	100	80
Radius of Horizontal Curve (m)	Desirable Min.	2,000	1,500	1,100
	Absolute Min.	1,500	1,000	700
K-value for Crest Curve (m)	Desirable Min.	450	250	120
	Absolute Min.	230	150	60
K-value for Sag Curve (m)	Desirable Min.	160	120	80
	Absolute Min.	120	80	40
Gradient (%)	Desirable Min.	2	2	3
	Absolute Min.	2	3	4

3.3. Design Hourly Traffic Volume & Design Traffic Capacity for IC and JCT

3.3.1. Design Hourly Traffic Volume and Traffic Capacity

An interchange shall be designed and planned based on the design hourly traffic volume in accordance with the annual average daily traffic (AADT) in the target design fiscal year.

The hourly traffic volume obtained by formula below in principle shall be used as the design hourly traffic volume.

DHV (one direction) = AADT (both directions) x K x D

Here,	DHV (one direction):	Design hourly traffic volume by directions
	AADT (both directions):	Total annual average daily traffic of both directions (traffic volume on the planned date based on the estimated traffic volume)
	K:	Ratio of the 30^{th} highest hourly traffic volume (total of both directions) to the AADT
	D:	Ratio of the traffic volume of the heavy traffic side to the total traffic volume of both directions at the 30^{th} hour

3.3.2. Design Traffic Volume of Interchange

The traffic volume of interchange including the turning movement and throughway traffic flow are estimated based on the traffic demand forecast as given in the **Chapter2, 2.5.6 of the Basic Design Report for the Detailed Design Study on the Outer Circular Highway to the City of Colombo - Southern Section – July 2005**. It is proposed that the OCH will be completed by each section in assumed stage construction. The traffic demand forecast has been respectively estimated with the initial stage of four (4) lane operation in 2020 and with the ultimate stage of six (6) lane operation in 2027.

The design traffic volumes at each interchange were estimated and these diagrams were summarized in **Table 3-2** and **Table 3-3**.

Table 3-2 Traffic Forecast of Interchange (2020)

Connection Road			CKE	A3(1)	A3(2)	A1	B214	AB10	A4	Remarks
Traffic Volume at Connection Road	Outside from Colombo	Inbound	23,000	31,100	31,100	27,200	11,000	10,600	35,500	
			8,600	6,600	6,600	7,300	2,700	2,700	9,100	
		Outbound	23,800	41,100	41,100	22,500	7,400	7,800	24,000	
			8,500	7,800	7,800	7,400	2,100	2,400	8,300	
		TOTAL	46,800	72,200	72,200	49,700	18,400	18,400	59,500	
			17,100	14,400	14,400	14,700	4,800	5,100	17,400	
	Nose to Nose	Inbound	18,000	23,600	23,600	16,000	9,300	8,500	29,200	
			6,500	4,600	4,600	3,400	2,000	2,100	6,500	
		Outbound	20,700	32,900	32,900	20,300	5,500	7,800	19,600	
			7,200	5,800	5,800	6,400	1,400	2,400	6,300	
		TOTAL	38,700	56,500	56,500	36,300	14,800	16,300	48,800	
			13,700	10,400	10,400	9,800	3,400	4,500	12,800	
	Colombo Side	Inbound	26,400	23,600	23,600	21,800	9,300	9,600	35,100	
			8,800	4,600	4,600	5,300	2,000	2,500	8,100	
		Outbound	22,800	32,900	32,900	16,100	5,600	7,700	26,100	
			8,200	5,800	5,800	4,500	1,500	2,300	9,100	
		TOTAL	49,200	56,500	56,500	37,900	14,900	17,300	61,200	
			17,000	10,400	10,400	9,800	3,500	4,800	17,200	
Traffic Volume at 1st Interchange (OCH Side)	On-Ramp (A) to North		0			3,400	2,300		14,200	ON
			0			1,400	1,000		4,700	
	Off-Ramp (B) from North		0			2,300	2,500		9,000	OFF
			0			1,100	1,000		3,000	
	On-Ramp (C) to South		7,100		7,500	11,300		3,200	6,000	ON
			3,100		2,000	3,700		1,000	2,600	
	Off-Ramp (D) from South		11,500	8,200	0	13,400		2,300	8,600	OFF
			3,600	2,000	0	4,900		900	2,500	
	Total		18,600	8,200	7,500	30,400	4,800	5,500	37,800	
			6,700	2,000	2,000	11,100	2,000	1,900	12,800	
Traffic Volume at 2nd Interchange (Connection Road Side)	Off-Ramp (E) CMB Side to OCH		2,100						9,300	OFF
			1,000						3,700	
	On-Ramp (F) OCH to CMB Side		8,400						10,500	ON
			2,300						2,600	
	Off-Ramp (G) Outside to OCH		5,000						10,900	OFF
			2,100						3,600	
	On-Ramp (H) OCH to Outside		3,100						7,100	ON
			1,300						2,900	
	TOTAL		18,600	0	0	0	0	0	37,800	
			6,700	0	0	0	0	0	12,800	
Traffic Volume at OCH Main Carriageway	North Side	Southbound	0			14,500	23,600		24,400	
			0			5,000	7,700		7,800	
		Northbound	0			19,700	29,600		29,700	
			0			5,600	9,000		9,000	
		TOTAL	0	0	0	34,200	53,200	0	54,100	
			0	0	0	10,600	16,700	0	16,800	
	Nose to Nose	Southbound	0	7,000	7,000	12,200	21,100	21,100	15,400	
			0	3,000	3,000	3,900	6,700	6,700	4,800	
		Northbound	0	11,500	11,500	16,300	27,300	27,300	15,500	
			0	3,600	3,600	4,200	8,000	8,000	4,300	
		TOTAL	0	18,500	18,500	28,500	48,400	48,400	30,900	
			0	6,600	6,600	8,100	14,700	14,700	9,100	
	South Side	Southbound	7,100	14,500	14,500	23,500		24,300	21,400	
			3,100	5,000	5,000	7,600		7,700	7,400	
		Northbound	11,500	19,700	19,700	29,700		29,600	24,100	
			3,600	5,600	5,600	9,100		8,900	6,800	
		TOTAL	18,600	34,200	34,200	53,200	0	53,900	45,500	
			6,700	10,600	10,600	16,700	0	16,600	14,200	

Note: Upper Column: AADT
Lower Column: Daily Traffic Volume of Heavy Vehicles

Table 3-3 Traffic Forecast of Interchange (2027)

Connection Road			CKE	A3(1)	A3(2)	A1	B214	AB10	A4	Remarks
Traffic Volume at Connection Road	Outside from Colombo	Inbound	27,600	38,800	38,800	36,000	14,500	11,000	40,000	
			9,800	8,600	8,600	9,500	3,400	2,900	10,400	
		Outbound	31,700	53,100	53,100	29,400	9,900	9,100	27,800	
			10,900	10,500	10,500	9,700	2,600	2,700	9,500	
		TOTAL	59,300	91,900	91,900	65,400	24,400	20,100	67,800	
			20,700	19,100	19,100	19,200	6,000	5,600	19,900	
	Nose to Nose	Inbound	21,500	30,000	30,000	19,800	11,000	8,600	31,700	
			7,500	6,000	6,000	4,200	2,600	2,200	7,200	
		Outbound	24,300	41,900	41,900	23,500	6,900	9,100	22,600	
			8,200	7,900	7,900	7,000	1,700	2,700	7,300	
		TOTAL	45,800	71,900	71,900	43,300	17,900	17,700	54,300	
			15,700	13,900	13,900	11,200	4,300	4,900	14,500	
	Colombo Side	Inbound	30,900	30,000	30,000	25,500	11,000	9,900	39,400	
			10,000	6,000	6,000	5,800	2,600	2,800	9,400	
		Outbound	30,500	41,900	41,900	19,400	7,000	8,900	29,100	
			11,000	7,900	7,900	5,300	1,800	2,500	10,100	
		TOTAL	61,400	71,900	71,900	44,900	18,000	18,800	68,500	
			21,000	13,900	13,900	11,100	4,400	5,300	19,500	
Traffic Volume at 1st Interchange (OCH Side)	On-Ramp (A)		0			8,200	4,400		14,400	ON
			0			2,900	1,300		4,700	
	Off-Ramp (B)		0			6,800	3,700		9,300	OFF
			0			3,100	1,200		3,100	
	On-Ramp (C)		12,300		8,800	13,600		3,800	8,200	ON
			5,100		2,600	4,400		1,100	3,400	
	Off-Ramp (D)		16,800	11,200	0	14,400		3,000	11,200	OFF
			5,200	2,600	0	4,800		1,200	3,300	
	Total		29,100	11,200	8,800	43,000	8,100	6,800	43,100	
			10,300	2,600	2,600	15,200	2,500	2,300	14,500	
Traffic Volume at 2nd Interchange (Connection Road Side)	Off-Ramp (E) CMB Side to OCH		6,200						10,200	OFF
			2,800						4,000	
	On-Ramp (F) OCH to CMB Side		9,400						11,800	ON
			2,500						3,100	
	Off-Ramp (G) Outside to OCH		6,100						12,400	OFF
			2,300						4,100	
	On-Ramp (H) OCH to Outside		7,400						8,700	ON
			2,700						3,300	
	TOTAL		29,100	0	0	0	0	0	43,100	
			10,300	0	0	0	0	0	14,500	
Traffic Volume at OCH Main Carriageway	North Side	Southbound	0			20,900	27,800		27,800	
			0			7,600	8,900		8,800	
		Northbound	0			27,900	34,100		32,800	
			0			7,800	9,700		9,700	
		TOTAL	0	0	0	48,800	61,900	0	60,600	
			0	0	0	15,400	18,600	0	18,500	
	Nose to Nose	Southbound	0	12,200	12,200	14,100	24,100	24,100	18,500	
			0	5,000	5,000	4,500	7,700	7,700	5,700	
		Northbound	0	16,800	16,800	19,700	29,700	29,700	18,400	
			0	5,200	5,200	4,900	8,400	8,400	5,000	
		TOTAL	0	29,000	29,000	33,800	53,800	53,800	36,900	
			0	10,200	10,200	9,400	16,100	16,100	10,700	
	South Side	Southbound	12,300	21,000	21,000	27,700		27,900	26,700	
			5,100	7,600	7,600	8,900		8,800	9,100	
		Northbound	16,800	28,000	28,000	34,100		32,700	29,600	
			5,200	7,800	7,800	9,700		9,600	8,300	
		TOTAL	29,100	49,000	49,000	61,800	0	60,600	56,300	
			10,300	15,400	15,400	18,600	0	18,400	17,400	

Note: Upper Column: AADT
Lower Column: Daily Traffic Volume of Heavy Vehicles

3.3.3. Traffic Capacity of Ramp

(1) Design Traffic Capacity of Ramp Throughway

The design traffic flow volume of the one-lane ramp throughway shall be as follow:

1,200 pcu / hour

The reduction of traffic capacity due to the mixing of large-sized vehicles should be taken into account in order to design capacity of ramp.

The traffic capacity of the ramp shall be taken to be least amount among one of three values below:

(a) The capacity of the connecting section between the ramp and mainline
(b) The capacity of the ramp throughway
(c) The capacity of the connecting section between the ramp and connecting road

Refer to **Table 3-4** for the relations between the proportion of the heavy vehicles and the decrease in the traffic capacity.

Table 3-4 Reduction of Traffic Capacity by Large Vehicle Ratio

Large Vehicle Ratio (%)	10	20	30	40	50	60
Reduction Ratio (%)	88.0	81.0	77.0	74.0	72.0	71.0

Design validity can be checked by comparing design traffic flow volume with design capacity. However, the design capacity explained above is the capacity for offering quite high-class service to drivers, and can be considered as capacity with some margins. (c.f. Ramp's design traffic flow volume of the one-lane ramp throughway with 40 km/hour design speed in the Highway Capacity Manual is 1,900 pcu/hour) Therefore, when design traffic flow volume exceeds traffic capacity, attempts should be made first to accomplish one of or both of the followings, before considering change to the two lane ramp:

(a) Change the design target year from 20 to15 years.
(b) Change the design traffic volume from the 30th highest hourly traffic volume to the 50th highest hourly traffic volume (50th highest hourly traffic volume will be approx. 92% to 93% of the 30th highest hourly traffic volume).

In this way, if the traffic volumes obtained are within the range of the design capacity, the original design may sometimes be used as is without any modification. The review on traffic volumes for the merging and diverging points of ramps is made in respect to the 30th highest hourly traffic volume of the ramp and main carriageway. Note that the peak traffic volumes for ramps and the main carriageway do not necessarily occur at the same time. In such a case, possible capacity can be assumed to be 1.25 times merging traffic capacity.

On the other hand, if after the above the design capacity is still less than design traffic volume, both of these may need to be adjusted, unless the capacity is unreasonably smaller than traffic volume. In such a case, the design should be changed.

(2) Ramp's Merging and Diverging Section Connecting with Main Carriageway

The traffic flow volume of the merging and diverging sections of the ramp at the connecting points with mainline is influenced by the traffic capacity, number of lanes, and traffic flow volume of mainline. Also, because a weaving can occur if another on or off ramp exists in the proximity of these sections, the distance between the two ramps may greatly affect the traffic capacity.

This approach uses the formula from the Design Manual of Japan Highway Public Corporation, which seeks the relations among the traffic flow volume of No. 1 lane of mainline traffic flow volume of throughway, and the traffic flow volume of the ramp's entrance section to make them into a graph by using many different measured values.

The formulas of the six cases are shown below:

Where,
- Vr: Traffic flow volume of the ramp entrance section (veh/hour)
- Vf: Total traffic flow volume of one side of mainline (veh/hour)
- V_D: Design traffic capacity per lane of mainline (veh/hour)

(a) One-lane on ramp connected with one-direction two-lane

$Vr = 1.13V_D-154-0.3Vf$
$Vr = 2V_D-Vf$
(Select the smaller value noted above)
Providing that; Vf: 400 vehicles/hour to 3,400 vehicles/hour
Vr: 50 vehicles/hour to 1,400 vehicles/hour

(b) One-lane off ramp connected with one-direction two-lane

$Vr = 1.92V_D-317-0.66VF$
Providing that; Vf: 400 vehicles/hour to 4,200 vehicles/hour
Vr: 50 vehicles/hour to 1,500 vehicles/hour

(c) One-lane on ramp connected with one-direction three-lane

$Vr = V_D + 120 – 0.244Vf$
$Vr = 3V_D – Vf$
(Select the smaller value noted above)
Providing that; Vf: 2,400 vehicles/hour to 6,200 vehicles/hour
Vr: 100 vehicles/hour to 1,700 vehicles/hour

(d) One-lane off ramp connected with one-direction three-lane

$Vr = 2.11 V_D-203-0.488Vf$
Providing that; Vf: 1,100 vehicles/hour to 6,200 vehicles/hour
Vr: 20 vehicles/hour to 1,800 vehicles/hour

(e) Two-lane on ramp connected with one-direction three-lane

$Vr = 1.739V_D + 357 – 0.499Vf$
$Vr = 3V_D – Vf$
(Select the smaller value noted above)
Providing that; Vf: 600 vehicles/hour to 3,000 vehicles/hour
Vr: 1,100 vehicles/hour to 3,000 vehicles/hour

(f) Two-lane off ramp connected with one-direction three-lane

$Vr = 1.76V_D + 279 – 0.062Vf$
Providing that; Vf: 2,100 vehicles/hour to 6,000 vehicles/hour
Vr: 1,100 vehicles/hour to 6,000 vehicles/hour

In the above cases, no restrictions are imposed on the length of the declaration and acceleration lane of the one-lane ramp. In the case of a two-lane ramp, it is, however, desirable that the on ramp be provided with the length of a speed change lane of more than 240 m, and the off ramp with a length of more than 210 m.

Table 3-5 Ramp Capacity Analysis at Merging and Diverging Section (2020)

(One-lane Ramp Connected with One-direction Two Lanes)

Connection Road	Direction	Traffic Volume in Main Carriageway (One Direction) (veh/day) (veh/hour)	Heavy Vehicle			Design Traffic Capacity (VD)	Design Capacity		Ramp Type
			Traffic Volume (veh/day)	Proportion (%)	Adjustment Factor	(pcu/hour)	Computed (Vrc) (veh/hour)	Adopted (Vrc) (veh/hour)	
CKE(1)	CKE→ OCH(E)	0 0	0	0.0	1.00	2,200	2,332 4,400	2,332	ON
	OCH(E)→ CKE	11,500 805	3,600	31.3	0.76	1,675	2,369	2,369	OFF
	CKE→ OCH(W)	0 0	0	0.0	1.00	2,200	2,332 4,400	2,332	ON
	OCH(W) →CKE	0 0	0	0.0	1.00	2,200	3,907	3,907	OFF
CKE(2)	CKE(N) →OCH	23,000 1,610	8,600	37.4	0.73	1,601	1,695	1,695	OFF
	OCH→ CKE(N)	20,700 1,449	7,200	34.8	0.74	1,632	1,125 1,816	1,125	ON
	CKE(S)→ OCH	22,800 1,596	8,200	36.0	0.74	1,618	1,736	1,736	OFF
	OCH→ CKE(S)	18,000 1,260	6,500	36.1	0.73	1,616	1,181 1,973	1,181	ON
A3(1)	OCH(E)→ A3(N)	19,700 1,379	5,600	28.4	0.78	1,713	2,062	2,062	OFF
A3(2)	A3→ OCH(E)	7,000 490	3,000	42.9	0.70	1,540	1,395 2,590	1,395	ON
	OCH(E)→ A3(S)	19,700 1,379	5,600	28.4	0.78	1,713	2,062	2,062	OFF
A1	A1→ OCH(N)	16,300 1,141	4,200	25.8	0.80	1,749	1,378 2,358	1,378	ON
	OCH(N) →A1	14,500 1,015	5,000	34.5	0.74	1,636	2,154	2,154	OFF
	A1→ OCH(S)	12,200 854	3,900	32.0	0.76	1,667	1,397 2,480	1,397	ON
	OCH(S)→ A1	29,700 2,079	9,100	30.6	0.77	1,684	1,544	1,544	OFF
B214	B214→ OCH(N)	27,300 1,911	8,000	29.3	0.77	1,701	1,023 1,492	1,023	ON
	OCH(N) →B214	23,600 1,652	7,700	32.6	0.75	1,659	1,778	1,778	OFF
AB10	AB10→ OCH(S)	21,100 1,477	6,700	31.8	0.76	1,670	1,157 1,863	1,157	ON
	OCH(S)→ AB10	29,600 2,072	8,900	30.1	0.77	1,691	1,563	1,563	OFF
A4(1)	A4→ OCH(N)	15,500 1,085	4,300	27.7	0.78	1,722	1,369 2,359	1,369	ON
	OCH(N) →A4	24,400 1,708	7,800	32.0	0.76	1,667	1,757	1,757	OFF
	A4→ OCH(S)	15,400 1,078	4,800	31.2	0.76	1,677	1,321 2,276	1,321	ON
	OCH(S)→ A4	24,100 1,687	6,800	28.2	0.78	1,716	1,864	1,864	OFF
A4(2)	A4(E)→ OCH	35,500 2,485	9,100	25.6	0.80	1,751	1,405	1,405	OFF
	OCH→ A4(E)	19,600 1,372	6,300	32.1	0.76	1,665	1,192 1,958	1,192	ON
	A4(W)→ OCH	26,100 1,827	9,100	34.9	0.74	1,631	1,609	1,609	OFF
	OCH→ A4(W)	29,200 2,044	6,500	22.3	0.82	1,799	1,082 1,555	1,082	ON

Table 3-6 Ramp Capacity Analysis at Merging and Diverging Section (2027)

(One-lane Ramp Connected with One-direction Three Lanes)

Connection Road	Direction	Traffic Volume in Main Carriageway (One Direction) (veh/day) (veh/hour)	Heavy Vehicle Traffic Volume (veh/day)	Heavy Vehicle Proportion (%)	Heavy Vehicle Adjustment Factor	Design Traffic Capacity (VD) (pcu/hour)	Design Capacity Computed (Vrc) (veh/hour)	Design Capacity Adopted (Vrc) (veh/hour)	Ramp Type
CKE(1)	CKE→ OCH(E)	0 0	0	0.0	1.00	2,200	2,320 6,600	2,320	ON
	OCH(E)→ CKE	16,800 1,176	5,200	31.0	0.76	1,680	2,768	2,768	OFF
	CKE→ OCH(W)	0 0	0	0.0	1.00	2,200	2,320 6,600	2,320	ON
	OCH(W) →CKE	0 0	0	0.0	1.00	2,200	4,439	4,439	OFF
CKE(2)	CKE(N) →OCH	27,600 1,932	9,800	35.5	0.74	1,624	2,280	2,280	OFF
	OCH→ CKE(N)	24,300 1,701	8,200	33.7	0.75	1,645	1,350 3,234	1,350	ON
	CKE(S)→ OCH	30,500 2,135	11,000	36.1	0.73	1,617	2,167	2,167	OFF
	OCH→ CKE(S)	21,500 1,505	7,500	34.9	0.74	1,631	1,384 3,388	1,384	ON
A3(1)	OCH(E)→ A3(N)	28,000 1,960	7,800	27.9	0.78	1,721	2,471	2,471	OFF
A3(2)	A3→ OCH(E)	12,200 854	5,000	41.0	0.71	1,560	1,472 3,827	1,472	ON
	OCH(E)→ A3(S)	28,000 1,960	7,800	27.9	0.78	1,721	2,471	2,471	OFF
A1	A1→ OCH(N)	19,700 1,379	4,900	24.9	0.80	1,762	1,545 3,906	1,545	ON
	OCH(N) →A1	20,900 1,463	7,600	36.4	0.73	1,613	2,487	2,487	OFF
	A1→ OCH(S)	14,100 987	4,500	31.9	0.76	1,668	1,547 4,016	1,547	ON
	OCH(S)→ A1	34,100 2,387	9,700	28.4	0.78	1,713	2,246	2,246	OFF
B214	B214→ OCH(N)	29,700 2,079	8,400	28.3	0.78	1,715	1,328 3,066	1,328	ON
	OCH(N) →B214	27,800 1,946	8,900	32.0	0.76	1,666	2,364	2,364	OFF
AB10	AB10→ OCH(S)	24,100 1,687	7,700	32.0	0.76	1,667	1,376 3,315	1,376	ON
	OCH(S)→ AB10	32,700 2,289	9,600	29.4	0.77	1,701	2,268	2,268	OFF
A4(1)	A4→ OCH(N)	18,400 1,288	5,000	27.2	0.79	1,730	1,536 3,902	1,536	ON
	OCH(N) →A4	27,800 1,946	8,800	31.7	0.76	1,671	2,373	2,373	OFF
	A4→ OCH(S)	18,500 1,295	5,700	30.8	0.76	1,682	1,486 3,750	1,486	ON
	OCH(S)→ A4	29,600 2,072	8,300	28.0	0.78	1,718	2,411	2,411	OFF
A4(2)	A4(E)→ OCH	40,000 2,800	10,400	26.0	0.79	1,746	2,115	2,115	OFF
	OCH→ A4(E)	22,600 1,582	7,300	32.3	0.76	1,663	1,397 3,407	1,397	ON
	A4(W)→ OCH	29,100 2,037	10,100	34.7	0.74	1,633	2,249	2,249	OFF
	OCH→ A4(W)	31,700 2,219	7,200	22.7	0.81	1,793	1,371 3,159	1,371	ON

(3) Number of Lanes

The number of lanes for ramp works out according to the following equation.

Number of Lanes = Hourly Turning Movements / (Design Traffic Capacity x Reduction Ratio)

The study on number of ramp lane at initial stage is resulted as shown in **Table 3-7**.

Table 3-7 Traffic Forecast and Number of Lanes (2020)

Connecting Road	Direction	Traffic Volume (veh/day)	K Rate for 30th Highest Hourly Ttraffic Volume	Traffic Volume (veh/hour)	Large Vehicle		Traffic Capacity (Ramp) (Mer/Diver) (veh/hour)	Design Volume (veh/hour)	Number of Lane 30th Highest Traffic	Number of Lane 50th Highest Traffic
					Volume (veh/day)	Proportion (%) Factor				
CKE(1)	CKE→ OCH(E)	7,100	0.07	497	3,100	43.7 0.733	880 2,332	880	0.6 1	
	OCH(E)→ CKE	11,500	0.07	805	3,600	31.3 0.766	919 2,369	919	0.9 1	
	CKE→ OCH(W)	0	0.07	0	0	0.0 1.000	1,200 2,332	1,200	0.0 1	
	OCH(W) →CKE	0	0.07	0	0	0.0 1.000	1,200 3,907	1,200	0.0 1	
CKE(2)	CKE(N) →OCH	5,000	0.07	350	2,100	42.0 0.736	883 1,695	883	0.4 1	
	OCH→ CKE(N)	3,100	0.07	217	1,300	41.9 0.736	883 1,125	883	0.2 1	
	CKE(S)→ OCH	2,100	0.07	147	1,000	47.6 0.725	870 1,736	870	0.2 1	
	OCH→ CKE(S)	8,400	0.07	588	2,300	27.4 0.780	936 1,181	936	0.6 1	
A3(1)	OCH(E)→ A3(N)	8,200	0.07	574	2,000	24.4 0.792	950 2,062	950	0.6 1	
A3(2)	A3→ OCH(E)	7,500	0.07	525	2,000	26.7 0.783	940 1,395	940	0.6 1	
	OCH(E)→ A3(S)	0	0.07	0	0	0.0 1.000	1,200 2,062	1,200	0.0 1	
A1	A1→ OCH(N)	3,400	0.07	238	1,400	41.2 0.738	886 1,378	886	0.3 1	
	OCH(N) →A1	2,300	0.07	161	1,100	47.8 0.724	869 2,154	869	0.2 1	
	A1→ OCH(S)	11,300	0.07	791	3,700	32.7 0.762	914 1,397	914	0.9 1	
	OCH(S)→ A1	13,400	0.07	938	4,900	36.6 0.750	900 1,544	900	1.0 1	
B214	B214→ OCH(N)	2,300	0.07	161	1,000	43.5 0.733	880 1,023	880	0.2 1	
	OCH(N) →B214	2,500	0.07	175	1,000	40.0 0.740	888 1,778	888	0.2 1	
AB10	AB10→ OCH(S)	3,200	0.07	224	1,000	31.3 0.766	919 1,157	919	0.2 1	
	OCH(S)→ AB10	2,300	0.07	161	900	39.1 0.743	892 1,563	892	0.2 1	
A4(1)	A4→ OCH(N)	14,200	0.07	994	4,700	33.1 0.761	913 1,369	913	1.1 2	1.0 1
	OCH(N) →A4	9,000	0.07	630	3,000	33.3 0.760	912 1,757	912	0.7 1	
	A4→ OCH(S)	6,000	0.07	420	2,600	43.3 0.733	880 1,321	880	0.5 1	
	OCH(S)→ A4	8,600	0.07	602	2,500	29.1 0.774	929 1,864	929	0.6 1	
A4(2)	A4(E)→ OCH	10,900	0.07	763	3,600	33.0 0.761	913 1,405	913	0.8 1	
	OCH→ A4(E)	7,100	0.07	497	2,900	40.8 0.738	886 1,192	886	0.6 1	
	A4(W)→ OCH	9,300	0.07	651	3,700	39.8 0.741	889 1,609	889	0.7 1	
	OCH→ A4(W)	10,500	0.07	735	2,600	24.8 0.791	949 1,082	949	0.8 1	

Note: Capacity reduction ratio due to heavy vehicle ratio shall be considered.

The study on number of ramp lane at ultimate stage is resulted as shown in **Table 3-8**.

Table 3-8 Traffic Forecast and Number of Lane (2027)

Connecting Road	Direction	Traffic Volume (veh/day)	K Rate for 30th Highest Hourly Ttraffic Volume	Traffic Volume (veh/hour)	Large Vehicle		Traffic Capacity (Ramp) (Mer/Diver) (veh/hour)	Design Volume (veh/hour)	Number of Lane 30th Highest Traffic	Number of Lane 50th Highest Traffic
					Volume (veh/day)	Proportion (%) Factor				
CKE(1)	CKE→ OCH(E)	12,300	0.07	861	5,100	41.5 0.737	884 2,320	884	1.0 1	
	OCH(E)→ CKE	16,800	0.07	1,176	5,200	31.0 0.767	920 2,768	920	1.3 2	1.2 2
	CKE→ OCH(W)	0	0.07	0	0	0.0 1.000	1,200 2,320	1,200	0.0 1	
	OCH(W) →CKE	0	0.07	0	0	0.0 1.000	1,200 4,439	1,200	0.0 1	
CKE(2)	CKE(N) →OCH	6,100	0.07	427	2,300	37.7 0.747	896 2,280	896	0.5 1	
	OCH→ CKE(N)	7,400	0.07	518	2,700	36.5 0.751	901 1,350	901	0.6 1	
	CKE(S)→ OCH	6,200	0.07	434	2,800	45.2 0.730	876 2,167	876	0.5 1	
	OCH→ CKE(S)	9,400	0.07	658	2,500	26.6 0.784	941 1,384	941	0.7 1	
A3(1)	OCH(E)→ A3(N)	11,200	0.07	784	2,600	23.2 0.797	956 2,471	956	0.8 1	
A3(2)	A3→ OCH(E)	8,800	0.07	616	2,600	29.5 0.772	926 1,472	926	0.7 1	
	OCH(E)→ A3(S)	0	0.07	0	0	0.0 1.000	1,200 2,471	1,200	0.0 1	
A1	A1→ OCH(N)	8,200	0.07	574	2,900	35.4 0.754	905 1,545	905	0.6 1	
	OCH(N) →A1	6,800	0.07	476	3,100	45.6 0.729	875 2,487	875	0.5 1	
	A1→ OCH(S)	13,600	0.07	952	4,400	32.4 0.763	916 1,547	916	1.0 1	
	OCH(S)→ A1	14,400	0.07	1,008	4,800	33.3 0.760	912 2,246	912	1.1 2	1.0 1
B214	B214→ OCH(N)	4,400	0.07	308	1,300	29.5 0.772	926 1,328	926	0.3 1	
	OCH(N) →B214	3,700	0.07	259	1,200	32.4 0.763	916 2,364	916	0.3 1	
AB10	AB10→ OCH(S)	3,800	0.07	266	1,100	28.9 0.774	929 1,376	929	0.3 1	
	OCH(S)→ AB10	3,000	0.07	210	1,200	40.0 0.740	888 2,268	888	0.2 1	
A4(1)	A4→ OCH(N)	14,400	0.07	1,008	4,700	32.6 0.762	914 1,536	914	1.1 2	1.0 1
	OCH(N) →A4	9,300	0.07	651	3,100	33.3 0.760	912 2,373	912	0.7 1	
	A4→ OCH(S)	8,200	0.07	574	3,400	41.5 0.737	884 1,486	884	0.6 1	
	OCH(S)→ A4	11,200	0.07	784	3,300	29.5 0.772	926 2,411	926	0.8 1	
A4(2)	A4(E)→ OCH	12,400	0.07	868	4,100	33.1 0.761	913 2,115	913	1.0 1	
	OCH→ A4(E)	8,700	0.07	609	3,300	37.9 0.746	895 1,397	895	0.7 1	
	A4(W)→ OCH	10,200	0.07	714	4,000	39.2 0.742	890 2,249	890	0.8 1	
	OCH→ A4(W)	11,800	0.07	826	3,100	26.3 0.785	942 1,371	942	0.9 1	

Note: Capacity reduction ratio due to heavy vehicle ratio shall be considered.

Note that the above tables shows two lane will be required at some ramps as the results calculated by 30th highest traffic volume. However, according to the explanation at 3.3.3 (1), it is confirmed that one lane will be enough capacity for these ramps on calculation by 50th highest traffic volume without off ramp from OCH east side at the Interchange with CKE. Therefore, reconfirmation will be required at this ramp at the time of detailed design in the future.

3.4. Classification of Interchange

3.4.1. Classification of Interchange

In the Japanese standards, the interchange is classified into the standards and grades according to the classification of roads (expressways or ordinary highways, and other roads) that cross and connect with one another as shown in **Table 3-9**.

Table 3-9 Standards and Grades of Interchanges

<table>
<tr><th>Classification</th><th>Standard</th><th>Grade</th><th colspan="2">Design Speed of Ramp (km/h)</th></tr>
<tr><td>Interchange between Expressways</td><td>Type 1</td><td>-</td><td colspan="2">80 – 40</td></tr>
<tr><td rowspan="2">Interchange between Expressway and an Ordinary Highway</td><td rowspan="2">Type 2</td><td rowspan="2">Class 1
Class 2
Class 3</td><td>Expressway</td><td>Ordinary Highway</td></tr>
<tr><td>40
35
30</td><td>40 - 30
35 - 30
30</td></tr>
<tr><td>Interchange between Ordinary Highways.</td><td>Type 3</td><td>Class 1
Class 2
Class 3</td><td colspan="2">40
35
30</td></tr>
</table>

3.4.2. Classification of OCH Interchanges

According to the classification of interchange indicated above, the interchanges along with OCH are classified as shown in the **Table 3-10**.

Table 3-10 Classification Application of OCH Interchanges

Interchange	Connection Highways	Adopted Standard
Kerawalapitiya Interchage	CKE	Type 2
Wattala Interchange	A3	ditto
Kadawatha Interchange	A1	ditto
Biyagama Interchange	B214	ditto
Kaduwela Interchange	AB10	ditto
Kottawa Interchange	A4	ditto

The Type 1 interchange is composed of those that separate or merge expressways and those that connect expressways with each other via directional ramps. Previously, there was a junction connecting OCH with Colombo-Kandy Expressway is to be classified into Type 1 interchange. However, it presumes that the junction is currently being eliminated because its realization has not still arrived in governmental policy yet.
For Kerawalapitiya interchange connecting with CKE, this could be classified into Type 1 according to the table above, however there will be a toll gate exists on the ramps connecting OCH with CKE therefore the classification should be categorized into Type 2 according to the applicable lower design speed of throughway between both expressways. According, all the interchanges to be located along with OCH will be classified only for Type 2.

3.5. Design Speed of Ramp

The classification of the Type 2 interchange could be subsequently categorized into three (3) classes as shown in the **Table 3-11** according to the design speed of the main body of an interchange and the daily traffic flow volume in the initial operating year of the respective interchange.

Table 3-11 Classifications of the Type 2 Interchange

Exit and entry traffic volume in the initial operating year (Vehicle/day)	Design Speed of the Expressway (km/h)		
	120	100	80
5000 or more	Class 1	Class 1	Class 1 (Class 2)*
More than 1000 and less than 5000	Class 1 (Class 2)*	Class 1 (Class 2)*	Class 2 (Class3)*
1000 or less	Class 2 (Class 3)*	Class 2 (Class 3)*	Class 3

*Note: In the tables, the classification and figures shown in the parenthesis could be considered only when unavoidable conditions due to such restrictions as a smaller forecast traffic flow volume, the configuration of the surrounding terrain, structures and land acquisition issues etc.

The design speed of the Type 2 interchange shall be made as shown in the **Table 3-12** according to the classification of the interchange.

Table 3-12 Design Speed of the Type 2 Interchange

Class	Ramp on Expressway (Km/h)	Ramp on Ordinary Highway (km/h)			
		80	60	50	40
Class 1	40	40 (35)*	35 (30)*	35 (30)*	30
Class 2	35	35 (30)*	35 (30)*	30	30
Class 3	30	30	30	30	30

*Note: In the tables, the classification and figures shown in the parenthesis could be considered only when unavoidable conditions due to such restrictions as a smaller forecast traffic flow volume, the configuration of the surrounding terrain, structures and land acquisition issues etc.

3.6. Cross Section Elements for Interchange

Ramp cross section as shown in **Fig. 3-1** consists of inner shoulder and outer shoulder, traffic lanes, marginal strip and verge.

3.6.1. General

The ramp cross section elements are established herewith based on the following policies.

- The Cross Section of ramp consists of traffic lane, shoulder and marginal Strips.
- The numbers of traffic lanes in principle are one way or two lanes.
- The median is required if it is two-way traffic.

Table 3-13 Cross Section Elements of Ramp

Traffic Lane (m)	Shoulder including Marginal strip (m)			Total Width	
	One way One lane		One way two lanes Two way two lanes (unseparated)	One way One lane	One way two lanes Two way two lanes (unseparated)
	Left	Right			
3.50	2.50 (1.50)	1.00	0.75	7.00 (6.00)	8.50

( ): Values for Tunnel, Long Bridge

3.6.2. Center Median

The center median consists of center strip and marginal strips. The width of center strip shall be more than 1.00 meter except the figure shown in parenthesis if necessary costly structures for center strip.

Table 3-14 Width of Center Median

Center Median (m)	2.50(2.00)
Center Strip (m)	1.00(0.50)

3.6.3. Shoulder

As ramps consist of only one lane, the shoulder has the same pavement structure and elevation as the carriageway in order to accommodate semi-trailer trucks.

3.6.4. Marginal Strip

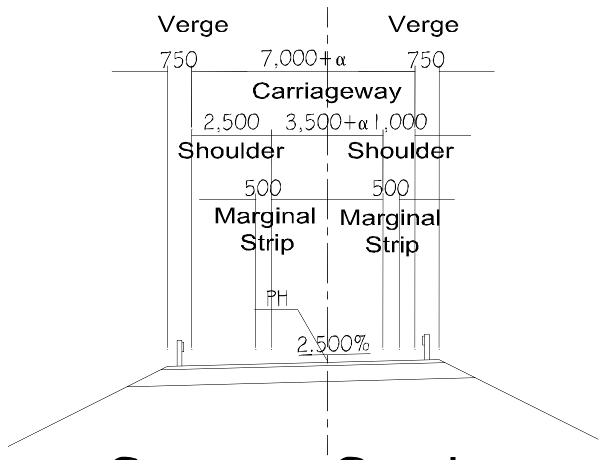
The marginal strip shall be applied in the center median and shoulder.

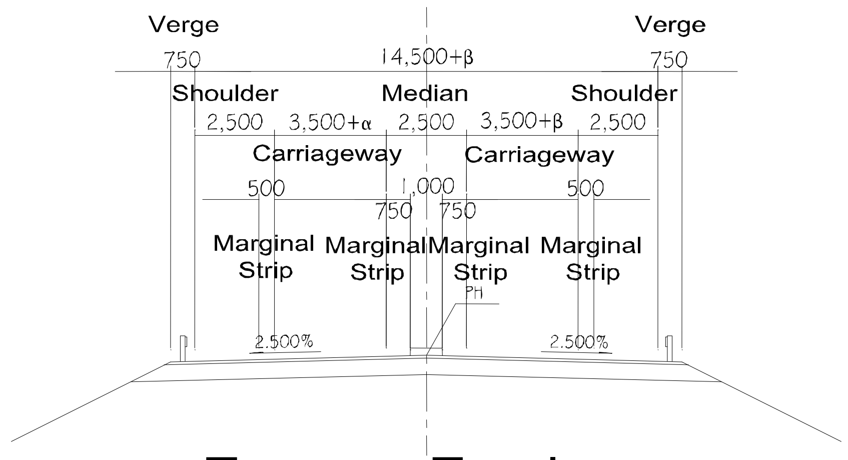
Table 3-15 Width of Marginal Strip

Position	Width (m)
Shoulder	0.50
Center Median	0.75

3.6.5. Crossfall

The crossfall adopted for carriageways is 2.5% and is the same as crossfall for the main carriageway. The crossfall for the outer shoulder of ramps shall be the same as that of the ramp carriageway so as to accommodate semi-trailer trucks.

<table>
<tr><th></th><th>Cross Section</th></tr>
<tr><td>Ramp</td><td>

One way One lane


Two way Two lane</td></tr>
</table>

Fig. 3-1 Cross Section of Ramp

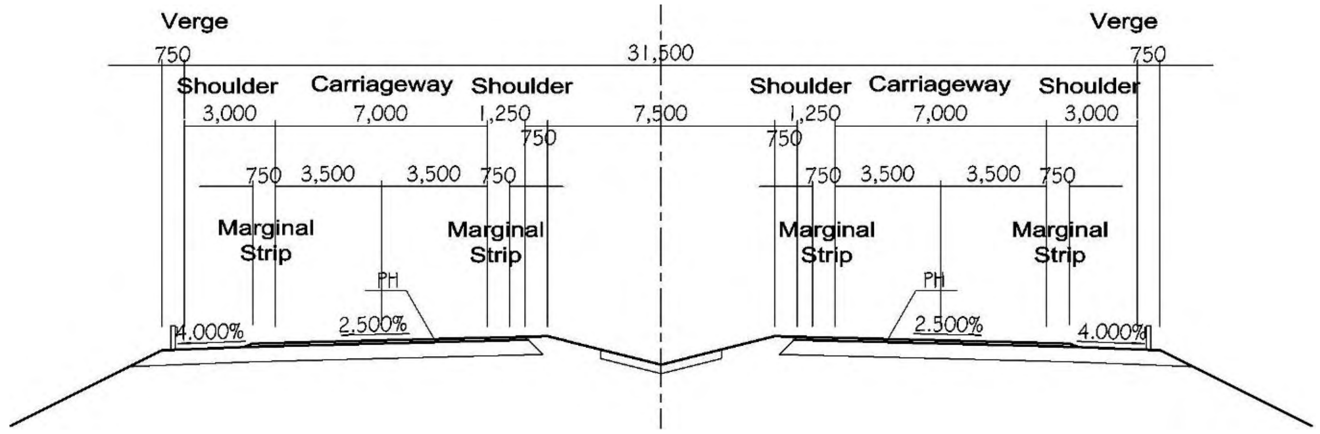
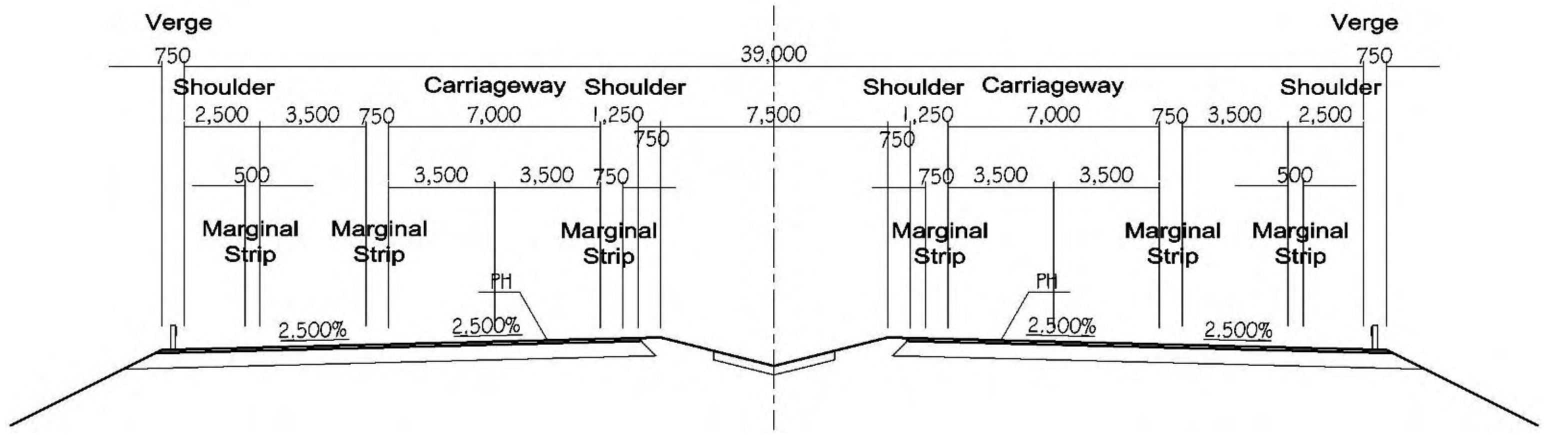
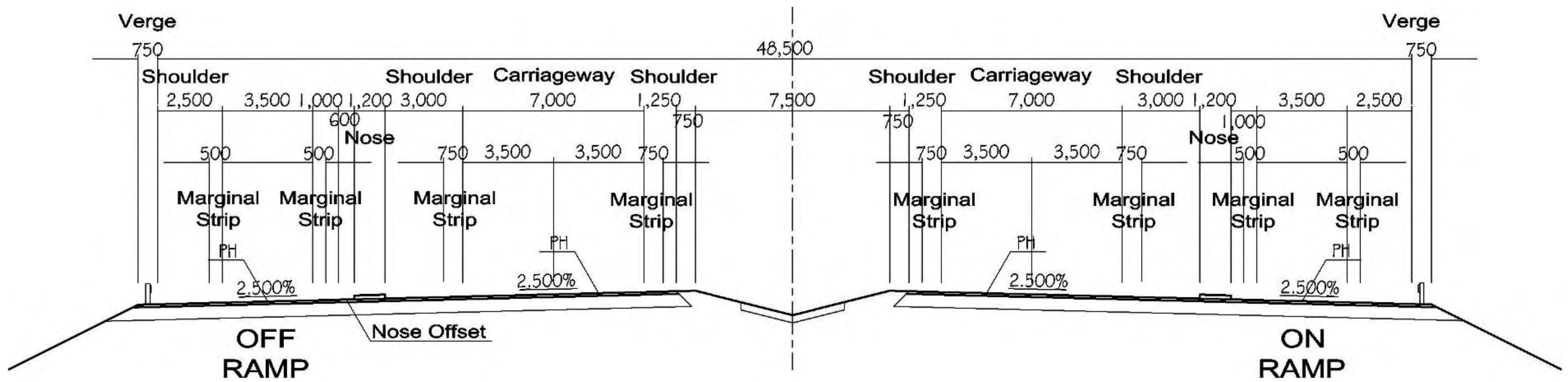
	Cross Section
Unseparated Road Section	
Section at Ramp Terminal Parallel Section	
Section at Nose	

Fig. 3-2 Cross Section of Ramp Terminal (Initial Stage)

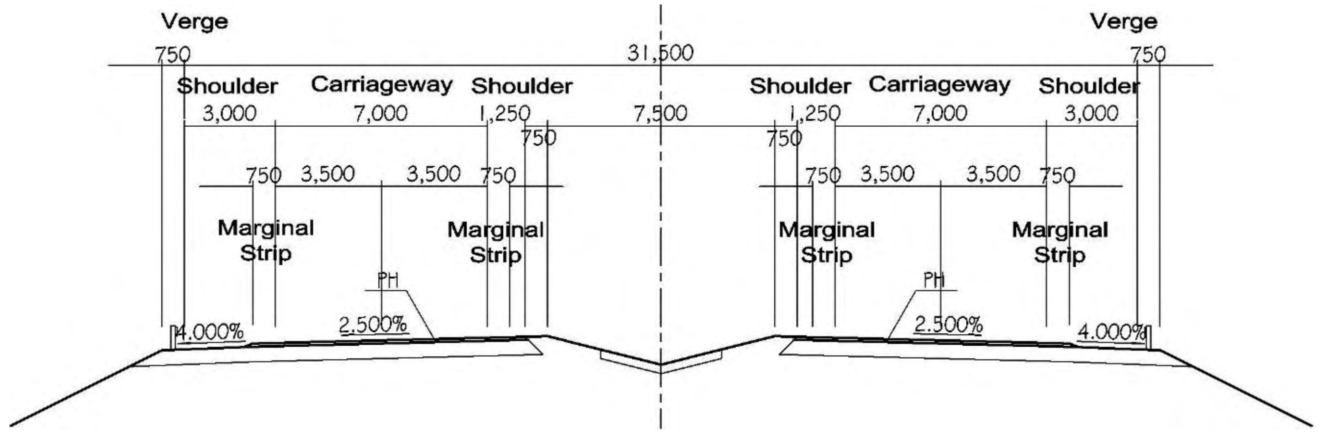
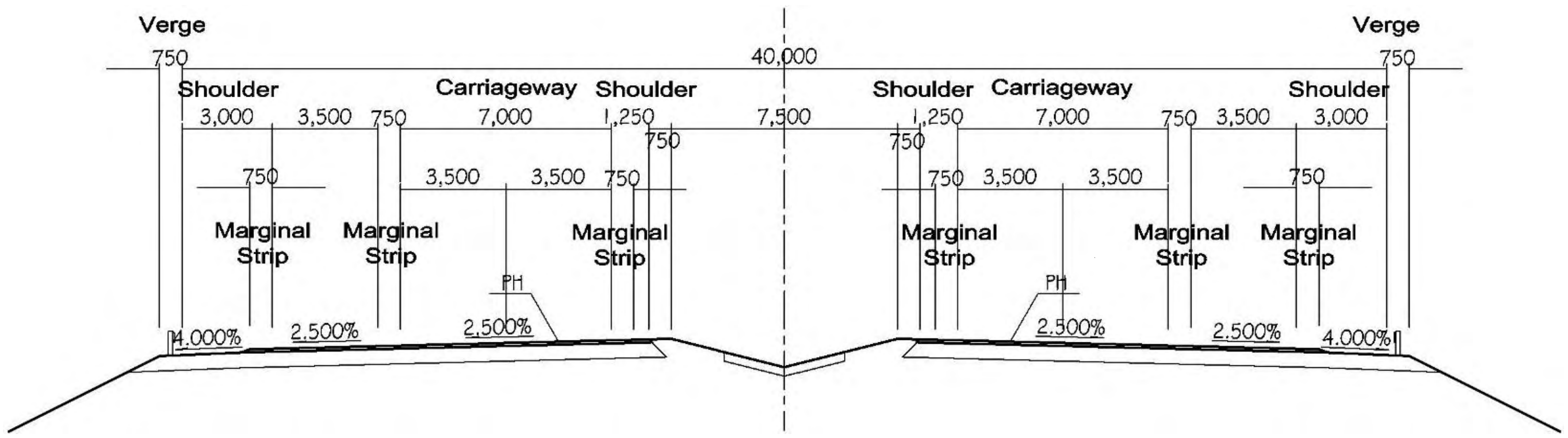
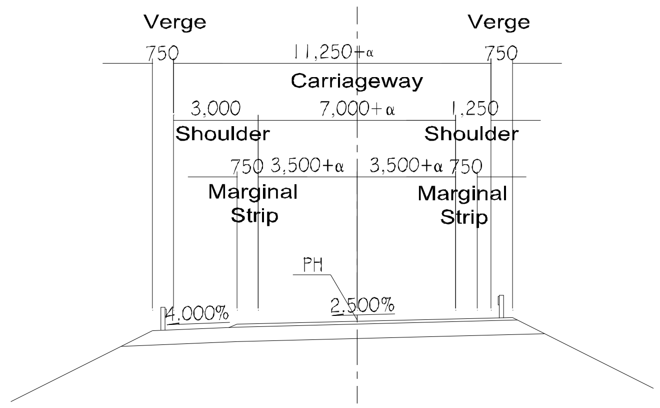
<table>
<tr><th></th><th>Cross Section</th></tr>
<tr><td>Unseparated Road Section (CKdE JCT)</td><td>
</td></tr>
<tr><td>Section at Ramp Terminal (Parallel Section)</td><td>
</td></tr>
<tr><td>Ramp (CKdE JCT)</td><td>
</td></tr>
</table>

Fig. 3-3 Cross Section of Ramp Terminal for CKdE Junction (Initial Stage)

3.7. Sight Distance of Ramp

Stopping Sight Distance is adopted from RDA Standard as follows - the minimum stopping sight distance of 140m; relates to the design speed of 80km/h, the minimum stopping sight distance of 85m; relates to the design speed of 60km/h and the minimum stopping sight distance of 45m; relates to the design speed of 40km/h.

Table 3-16 Stopping Sight Distance for Ramp

Design Speed (km/h)	Break reaction time (sec.)	Friction Factor	Calculated (m)	Sight Distance (m)
80	2.5	0.30	139.5	140
60	2.5	0.33	84.6	85
40	2.5	0.38	44.4	45

3.8. Horizontal Alignment of Ramp

3.8.1. Minimum Radius of Horizontal Curve

The minimum radius of horizontal curve is shown at the **Table 3-17**.

Table 3-17 Minimum Radius of Horizontal Curve

Interchange Type	Design Speed (km/h)	Running Speed (km/h)	Super elevation	Side Friction Factor	Calculated (m)	Rounded (m)
JCT (Type 1)	80	80	6.0	0.12	280	280
	60	60	6.0	0.13	149	150
	40	40	6.0	0.15	60	60
IC (Type 2)	40	40	6.0	0.19	50	50

3.8.2. Minimum Parameter of Transition Curve

$$A = \sqrt{(0.0215 \cdot V^3 / P)}$$

Where A: Parameter of Transition (Clothoid) Curve (m)
V: Design Speed (km/h)
P: Rate of increase of centripetal acceleration (m/s2)

Table 3-18 Minimum Parameter of Transition (Clothoid) Curve

Design Speed (km/h)	80	60	40
P (km/h)	0.60	0.90	1.15
A (m)	135	72	35
Rounded	140	70	35

3.8.3. Minimum Radius of Curve Omitting Transition Curve

$$R = \sqrt[3]{A^4 / (24*S)}$$

Where R: Radius of Curve (m)
A: Minimum parameter of Transition Curve (Clothoid Curve) (m)
S: Shift in meters between curve and tangent (m)

Table 3-19 Minimum Radius of Curve Omitting Transition Curve

Design Speed (km/h)	80	60	40
A (m)	135	72	35
S (m)	0.20	0.20	0.20
Calculated (m)	411	177	67
Rounded (m)	800	350	140

3.8.4. Minimum Radius of Curve and Minimum Parameter of Transition Curve at Exit Ramp Nose

In an exit ramp of throughway, drivers generally cannot quickly lose the sense of high-speed throughway driving and tend not to slow down completely to a speed intended by the planner. Consequently, at an exit ramp, a design that will permit a small curve radius to appear suddenly is not desirable.

Further, if the parameters of the spiral curve in the vicinity of the nose of an exit ramp when using the spiral curve as a transition curve is computed based on the relations between the distance and the required curve radius, a prescribed absolute value can be obtained. It was determined that an increased absolute value is used as a standard value.

Minimum radius of curve on exit ramp is desired at least 160 meter at IC of Outer Circular Highway. A value of 60m for transition curve at IC is desired for ramp terminals with the absolute value of 50m.

(1) Minimum Radius of Curve

Table 3-20 Minimum Radius of Curve on Exit Ramp Nose

Design Speed for Main Carriageway (km/h)	Nose Passing Speed (km/h)	Superelevation at Nose (%)	Side Friction Factor	Minimum Radius at Nose (m)	
				Calculated	Rounded
80	50	2.5	0.10	157	160

(2) Minimum Parameter of Transition Curve on Exit Curve

Table 3-21 Minimum Parameter of Transition Curve on Exit Ramp

Design Speed of Main Carriageway V (km/h)	Nose Passing Speed V (km/h)	Minimum Radius at Ramp R(m)	Average Running Speed at Minimum Curve on Ramp V1 =√ (127)(i+)R) I=0.10,f=0.10 (km/h)	Deceleration (m/s^2)	Transition Curve Length (m)	Minimum .Parameter		
						Calculated A(m)	Rounded	
							Absolute min. A(m)	Desirable Min. A(m)
80	50	50	36	1.0	46	48	50	60

3.9. Vertical Alignment of Ramp

3.9.1. Gradient

The maximum gradient for ramp shall be given the values shown in **Table 3-22**.

Table 3-22 Maximum Gradient for Ramp

Applicable	Design Speed	Gradient		
		Desirable (%)	Except. (%)	
			Up	Down
JCT (Type 1)	80	4.0	5.0	5.0
	60	5.0	6.0	6.0
IC (Type 2)	40 - 30	6.0	6.0	7.0

3.9.2. Minimum "K" Value

The minimum K-values for vertical curve shall be given to the values shown in **Table 3-23**.

Table 3-23 Minimum K-value

□ Minimum K-Values for Crest Vertical Curve				
Design Speed of Ramp (km/h)	Stopping Sight Distance (m)	Minimum K-Value		
		Calculated	Rounded	
			Absolute Min.	Desirable Min
80	140	45.3	45	68
60	85	16.7	17	25
40	45	4.7	5	7
□ Minimum K-Values for Sag Vertical Curve				
Design Speed of Ramp (km/h)	Stopping Sight Distance (m)	Minimum K-Value		
		Calculated	Rounded	
			Absolute Min.	Desirable Min
80	140	30.6	31	46
60	85	16.1	17	24
40	45	6.6	7	10

3.10. Superelevation of Ramp

The superelevation for ramp shall be broken off at 6%.

3.10.1. Superelevation on Curve

Superelevation at the curved section of ramp shall be given to the values shown in **Table 3-24** according to the ramp standard (interchange type and design speed) and the curve radius of the respective curved section.

Table 3-24 Values of Superelevation related to Horizontal Curve of Ramp

Type	JCT		IC	Superelevation (%)
Design Speed (km/h)	80	60	40 or less	
Radius of Curve (m)	Less 540	Less 330	Less 160	6
	540 670	330 420	160 210	5
	670 870	420 560	210 280	4
	870 1,150	560 740	280 360	3
	over1,150	over 740	over 360	2.5

3.10.2. Superelevation Development

Table 3-25 Superelevation Development
(Position of Rotation Axis: Center of Traffic Lane)

Ramp / Design Speed (km/h)	One-lane, one-way or separated Two-lane two-way operation	Two-lane operation Either one-way or two-way (unseparated)
80	1/250	1/200
60	1/225	1/175
40	1/150	1/150

Table 3-26 Superelevation Development
(Position of Rotation Axis: Both Edge of Center Median)

Ramp / Design Speed (km/h)	One-lane, one-way or separated Two-lane two-way operation	Two-lane operation Either one-way on two-way (unseparated)
80	1/200	1/150
60	1/150	1/125
40	1/100	1/100

3.10.3. Minimum Superelevation Development for Secure Drainage

Superelevation development at the carriageway where the superelevation becomes level should not be smaller than the values in **Table 3-27**.

Table 3-27 Minimum Superelevation Development for Secure Drainage

Kind of Ramp \ Position of Rotation Axis	Center of Traffic Lane	Both Edge of Center Median
One-lane, one-way or separated Two-lane two-way operation	1/800	1/500
Two-lane operation Either one-way on two-way (unseparated)	1/500	1/300

This case will be required when shifting from a straight line to a curve, or in the vicinity of a changing point of a reverse curve. **Table 3-28** shows the Standard Length to Secure Minimum Superelevation Development for Secure Drainage. Where the superelevation becomes smaller, the superelevation necessary for drainage should be secured as shown in the table, however, the section adopted must be minimized.

Table 3-28 Standard Length to Secure Min. Superelevation Development for Secure Drainage (m)

Kind of Ramp \ Position of Rotation Axis	Center of Traffic Lane		Both Edge of Center Median	
	Distance to outer edge	Length of Development	Distance to outer edge	Length of Development
One-lane, one-way or separated Two-lane two-way operation	2.25	90	4.50	110
Two-lane operation Either one-way on two-way (unseparated)	4.50	110	8.00	120

Algebraic Difference of Superelevation: 0.05 (-2.5% to 2.5%)

3.10.4. Adverse Crossfall

Table 3-29 Minimum Curve Radius for Section with Adverse Crossfall

Design Speed (km/h)	80	60	40
Min. Radius of Curve (m)	3,500	1,900	900

3.10.5. Traveled Way Widening on Curves

Two lane ramps in the table means unseparated two lane ramp. It should be adopted to the value of “one way one lane ramp” for both lane of separated two way two lane ramp. We will rectify the **Table 3-30**.

Table 3-30 Traveled Way Widening on Curves at Interchange

One way one lane ramp Width 7.00m (tangent) Two way two lanes ramp (separated) Width 14.50m (tangent)		One way two lanes ramp Two way two lanes ramp (unseparated) Width: 8.50m (tangent)	
Radius of Curve(m)	Widening / 1 lane(m)	Radius of Curve(m)	Widening / 2 lane(m)
More than 15 and less than 21	2.75	More than 15 and less than 21	3.75
Less than 23	2.50		
Less than 25	2.25	Less than 22	3.25
Less than 27	2.00	Less than 23	3.00
Less than 29	1.75	Less than 24	2.75
Less than 32	1.50	Less than 25	2.50
Less than 36	1.25	Less than 26	2.25
Less than 42	1.00	Less than 27	2.00
Less than 48	0.75	Less than 29	1.75
Less than 58	0.50	Less than 31	1.50
Less than 72	0.25	Less than 33	1.25
72 or more	0	Less than 36	1.00
		Less than 39	0.75
		Less than 43	0.50
		Less than 47	0.25
		47 or more	0

3.10.6. Composite Gradient

This criteria, which includes checking whether the combined gradient value, which is the value of superelevation and the gradient, is suitable or not when the section overlaps a gradient and horizontal curve.

Table 3-31 Composite Gradient of Ramp

Type	JCT		IC
Design Speed of Ramp (km/h)	80	60	40
Maximum (%)	10.5	10.5	11.0

3.11. Ramp Terminals

Sufficient distance from the nose to the structure shall be maintained since the vehicles pass through the diverging nose at high speed without speed reduction in a deceleration lane in cases where the loop is at off-ramp side. Good forward visibility is maintained throughout the loop in order that drivers may see the loop.

Entry and Exit ramp terminals have been shown in **Fig. 3-4** representing length of deceleration lane and acceleration lane, taper length, exit angle and entrance angle.

3.11.1. Deceleration Lane

There are two typical types of deceleration lane. One is the parallel and the other is the direct system. The parallel system has a starting point with an appropriate exit angle and a fixed width up to the nose. A heavy emphasis is placed on the starting point of parallel's deceleration lane compared with direct system. Although the tapered section of parallel system coincides with a vehicle's traveling locus, a vehicle must travel the S-letter shape to use the total length of a deceleration lane. Several investigations revealed that generally drivers attempting to exit favor a direct type of outflow and do not travel the S-letter shape.

3.11.2. Acceleration Lane

As the volume of traffic increases, the drivers may come to use the S-letter traveling over the total length of an acceleration lane while looking for a chance to enter a throughway. Also, generally, an acceleration lane is longer than a deceleration lane, and if the taper type is adopted, the taper may become narrow and longer making it difficult to be installed. Therefore, it was determined that the parallel system be used for the acceleration lane.

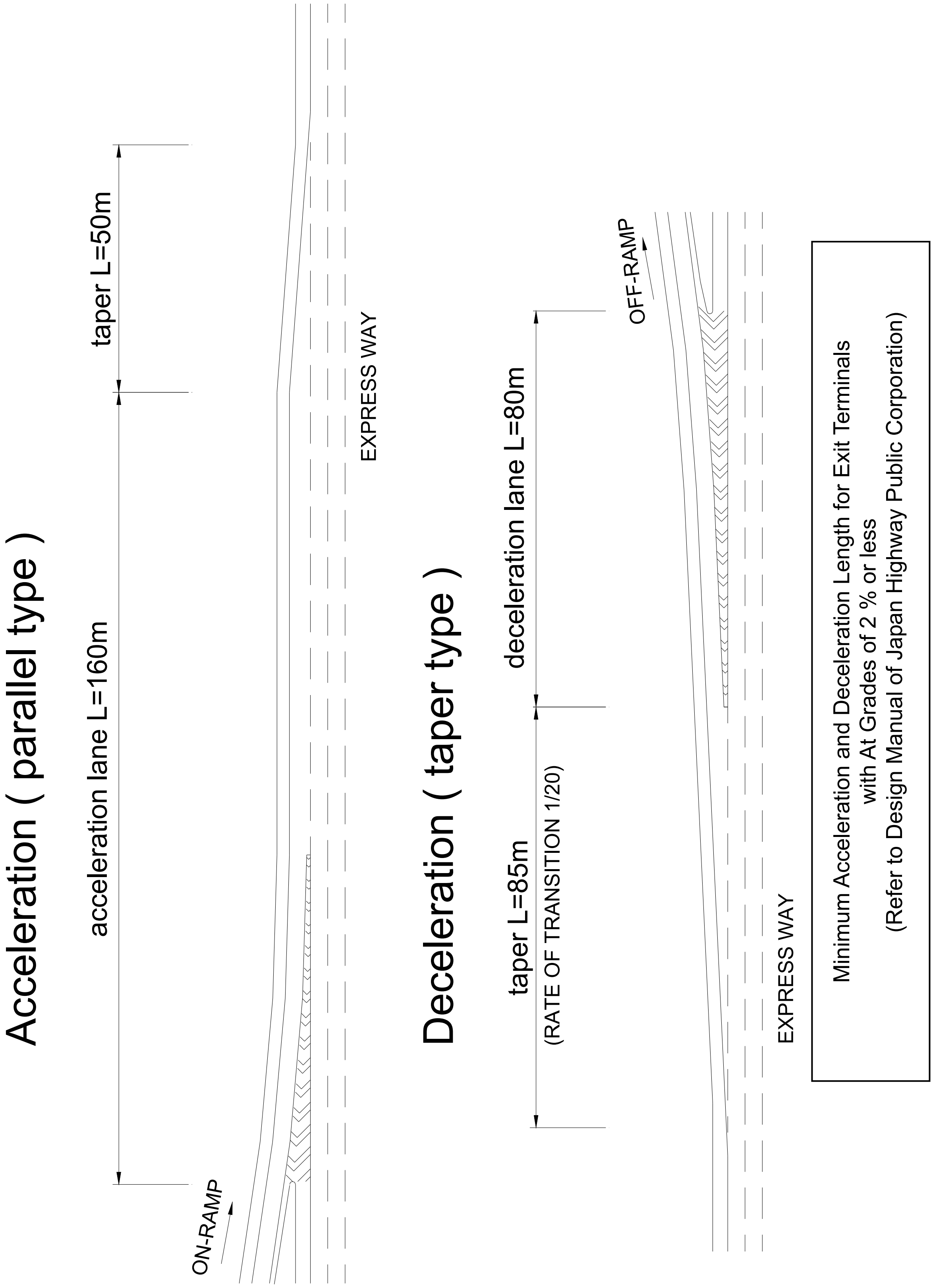


Fig. 3-4 Acceleration and Deceleration Length

Table 3-32 Length of Speed Change Lane and Exit, Entrance Angle

Design Speed (km/h)		120	100	80
Length of Deceleration Lane	One lane Two lanes	100 150	90 130	80 110
Length of Acceleration Lane	One lane Two lanes	200 300	180 260	160 220
Taper Length for Parallel Type Speed Change Lane	One lane	70	60	50
Exit Angle for Taper Type Deceleration Lane	One lane Two lanes	1/25		1/20
Entrance Angle for Taper Type Acceleration Lane	One lane Two lanes	1/40		130

Table 3-33 Geometric Design Criteria for Ramp Terminals

Item	unit	Adopted Value
Maximum K-value at Ramp Nose Crest Sag		 15(10) 17(12)
Min. Vertical Curve Length at Ramp Nose	m	60(40)
Min. Parameter of Transition Curve at Exit Ramp Nose	m	60(50)
Min. Radius of Curve at Exit Ramp Nose	m	160
Length of Deceleration Lane	m	80(1-Lane) / 110(2-lane)
Taper Length for Parallel Type	m	50
Length of Acceleration Lane	m	160(1-Lane) / 220(2-lane)
Taper Length for Parallel Type	m	50
Exit Angle for Taper Type Deceleration Lane		1/20
Entrance Angle for Taper Type Acceleration Lane		1/30

( ) is Absolute Value   Design Speed of Main Carriageway: 80km/h

3.12. Summary of Interchange Design Criteria

Summary of geometric design criteria for Interchanges are shown in **Table 3-34**.

Table 3-34 Summary of Geometric Design Criteria for Interchange

Interchange Standard				Type 2, Class 1			Remarks
Design Speed of Ramp (Main Carriageway)				V = 40 km/ h (80km/h)			
Elements				Reference Value			
				Criteria	Absolute Value	Adoption	
Horizontal Alignment	Min. Radius of Curve		m	50	40	50	
	Min. Parameter of Spiral Curve		m	35*	-	45	*Min. length 35m
	Min. Radius of Curve Omitting Transition Curve		m	140	-	180	
	Min. Parameter at Exit Ramp Nose		m	60	50	60	
	Min. Radius of Curve at Exit Ramp Nose		m	160	-	300	
	Minimum Curve Radius without Superelevation		m	900	-	1000	Normal Crossfall 2.5%
Vertical Alignment	Max. Gradient	Expressway Side	%	6	Up Slope 6 Down Slope 7	5.918	
		National Highway Side	%	6	Up Slope 7 Down Slope 8	3.282	
	Min. K-Value of Vertical Curve	Crest	m	7	5	7.8	
		Sag	m	10	7	10	
	Min. Vertical Curve Length		m	40	35	40	
	Min. K-Value at Ramp Nose	Crest	m	15	10	15.1	
		Sag	m	17	12	-	
		Curve length	m	60	40	60	
Normal Crossfall			%	2.5			
Crossfall of Outer Shoulder			%	2.5			
Max. Superelevation			%	6.0		6	
Max. Composite Gradient			%	11.0		8.339	
Stopping Sight Distance			m	45		45	
Traffic Lane Width			m	3.5			
Outer Shoulder Width			m	2.5			
Marginal Strip Width (at Shoulder and Center Median)			m	0.5: Shoulder / 0.75: Median			
Right (Inner) Shoulder			m	1.0			
Center Median Width			m	2.5 / 1.0 (without marginal strip)			

Note: The setting of the maximum vertical gradient for ramps (including loop portions) must be considered maximum composite gradient.

3.13. Planning of Interchange and Junction

The locations of interchange should be normally determined based on road users' demand. The expressway, which is limited access controlled roads, is able to provide high-speed and comfortable uninterrupted travel. But if there are many access points, it would be adversely for user's comfort to keep smooth traffic flow and may be impaired to effectiveness for expressway. For the reason above, it is not desirable to plan the interchanges too close to one another. However, if the expressway is situated in the outskirts of major city and the number of interchanges is not sufficient, the traffic may be congested by the vehicles centralizing to the major city. As a solution, the distance between interchanges is likely to keep short to increase the diversification effect. The location of interchange has been decided by the following criteria recommended in the Japan Highway Design Manual.

(a) Crossing or nearest points of important arterial roads such as national highways.
(b) Surrounding of cities with a population of more than 30,000, or a location to provide an interchange for areas with a populations of 50,000 to 100,000.
(c) Crossing or nearest points of major roads to important ports, airports, material transport facilities and internationally known sightseeing areas.
(d) Locations to maintain the distance between interchange between the minimum 4km and the maximum 30km.

The relation between city population and the standard number of interchanges according to the above standard is as shown in the **Table 3-35**.

Table 3-35 Standard Number of Interchanges

Population	Standard Number of Interchanges
Less than 100 thousand	1
Less than 100 thousand ～ 300 thousand	1 – 2
Less than 300 thousand ～ 500 thousand	2 – 3
Over 500 thousand	3

The standard distance between interchanges for each area is as shown in the **Table 3-36**.

Table 3-36 Standard Distance between Interchanges

Area	Standard Distance
Within a City	5km – 10km
City Outskirts	**15km – 25km**
Between Cities	20km – 30km

3.13.1. Application of Interchange Type

There are major considerations in application of interchange type. As given in the AASHTO, the type of interchanges should be determined by the number of intersection legs, the expected volumes of through and turning movements, topography, culture, design controls, proper signing, and the designer's initiative. The types of interchanges as shown in the **Table 3-37** and **Table 3-38** are widely adopted in common pattern.

Table 3-37 Applicable 3 – leg Interchange Types for Outer Circular Highway

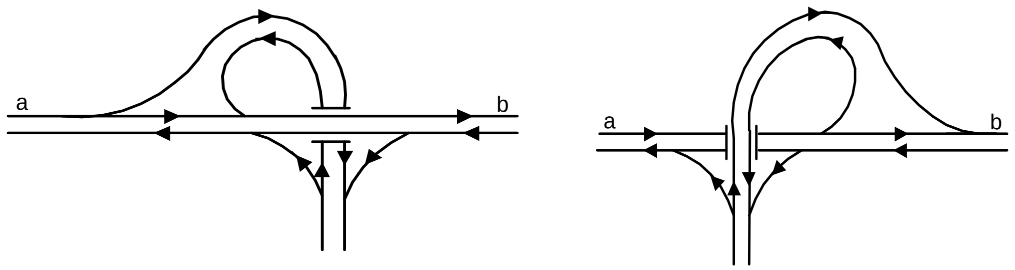
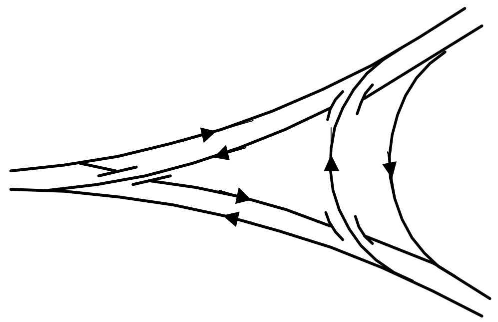
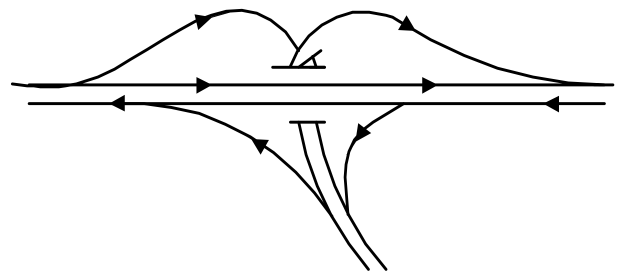
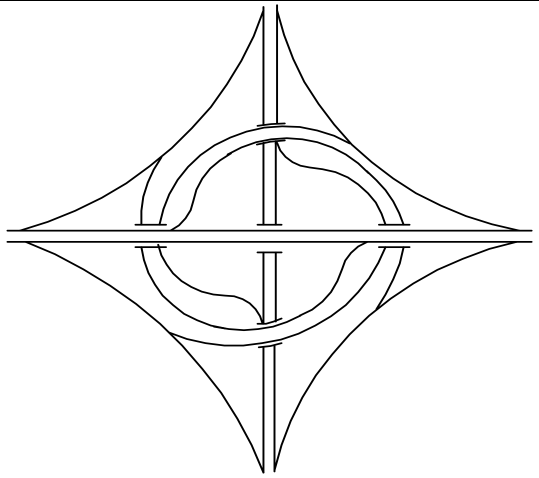
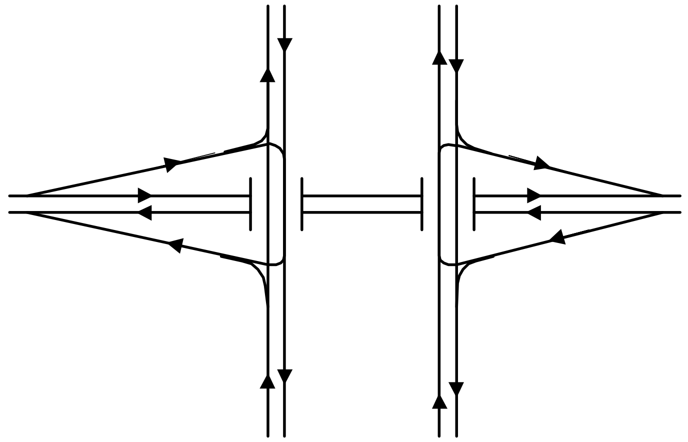
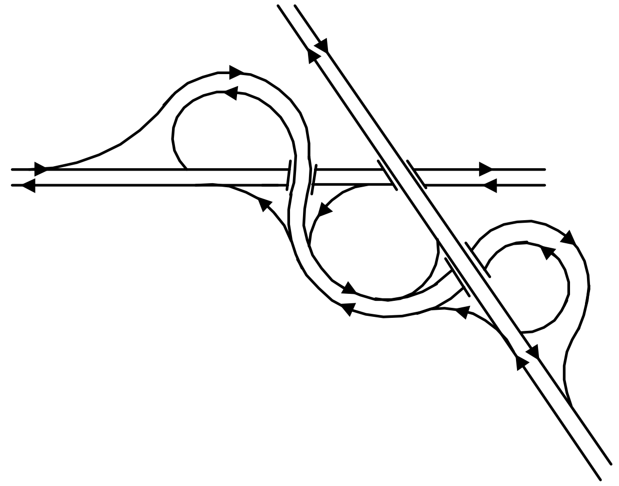
Type	**Trumpet Type**	**Direct Y Type**	**Semi direct Y Type**
Sketch			
Geometry	This interchange type comprises of a directional loop ramp and a complete loop ramp for right turning traffic. A directional loop ramp serves higher traffic volume than a complete loop ramp. There are cross-over bridges located at the intersecting road. Left turning traffic is at-grade level. Turning radius for each ramp depends on traffic forecast and design speed. Normally, skewed crossing is more desirable than right angle since the skewed crossing has a somewhat shorter travel distance and flatter turning radius for heavier turning traffic volume. The transition spirals provide for a smooth speed change and steering maneuver both into the loop and on to the expressway.	This interchange type comprises of three directional ramps serving righting turning traffic with three level of structure. Three left turning traffic ramps are located at-grade level. Horizontal turning radius for all turning traffic is corresponding to each other. At least two-vertical curve alignments are employed for second and third levels. Directional right turning ramps will serves high traffic volume.	This interchange type comprises of two-directional loop ramps looked like a double jug-handle pattern. Three-separated bridges for right turning traffic will cross over the intersecting road and between the right turning ramps. Two vertical curves alignment are employed for both directional loop ramps.
Operation	All right turning traffic from and to the connecting road will be maneuvered along the loop ramps with the appropriate speed of 40-60 km/hr and vertical alignment for crossing over bridges, Left turning traffic from and to the connecting road will be maneuvered along a directional ramps. No weaving traffic is involved. Toll plaza is suitable to be located at main carriageway.	This interchange type is suitable for a junction of two freeways with high traffic volume and high speed (Approx. 80-100 km/hr) for all directions and required high level of service. All right turning traffic will be maneuvered along directional ramps and two vertical alignments for second and third level. Left turning traffic mostly will be maneuvered along at-grade directional ramps. No weaving traffic is involved. Toll plaza can be located at any leg of the interchange.	This interchange type is suitable for serving high right turning traffic volume from both intersecting road and the expressway with the appropriate speed of 60 km/hr. All right turning traffic will be maneuvered along directional loop ramps and two vertical alignment for second level of two-separated bridges. Left turning traffic will be maneuvered along at-grade directional ramp. Toll plaza is suitable to be located at main carriageway.
Stage Construction	It is not so difficult to modify for stage construction. It is very simple to arrange for stage construction as long as land acquisition provided for final stage construction. Land acquisition cost would be the highest comparing to the others.	It is very difficult to modify for stage construction since the area is quite limited and the constraint of elevated structure.	It is less difficult to modify for stage construction since the area is not quite limited and locations of three bridges are separated.
Construction Cost / Land acquisition cost	Only one location of two-level structure is required. The construction cost would be less comparing to the others. The area for loop ramps is the most critical to the land acquisition.	Since the complex three-level structure, normally it has been found that the construction cost is high. This type of interchange requires very little right of way, therefore, land acquisition cost is quite less comparing to the others.	The construction cost for three bridges is rather as high as the Y-type interchange. The land cost is moderate comparing to the others.
Maintenance Cost	Maintenance cost for structure is quite less, but for loop ramps and surrounding areas, maintenance cost is rather high, since the area is wider than the others.	Maintenance cost for structure is quite high because of the complex three-level structure. But, maintenance cost for at grade roads and surrounding area is low since the area is less.	Maintenance cost for structure is rather high because of three bridges located separately. Maintenance cost for at-grade roads and surrounding area is rather high but still less than trumpet type since the area is less.
Future Expansion / Modification	It is difficult to modify in the future, unless otherwise, future expansion plan must be taken into account.	It is the most difficult to modify in the future expansion since the structure is complicated and the area is quite limited. The future expansion will be done properly if future expansion plan is taken into account.	It is more difficult than the trumpet type to modify in the future expansion since the area is rather limited. The future expansion will be done properly if future expansion plan is taken into account.

Table 3-38 Applicable 4 – leg Interchange Types for Outer Circular Highway

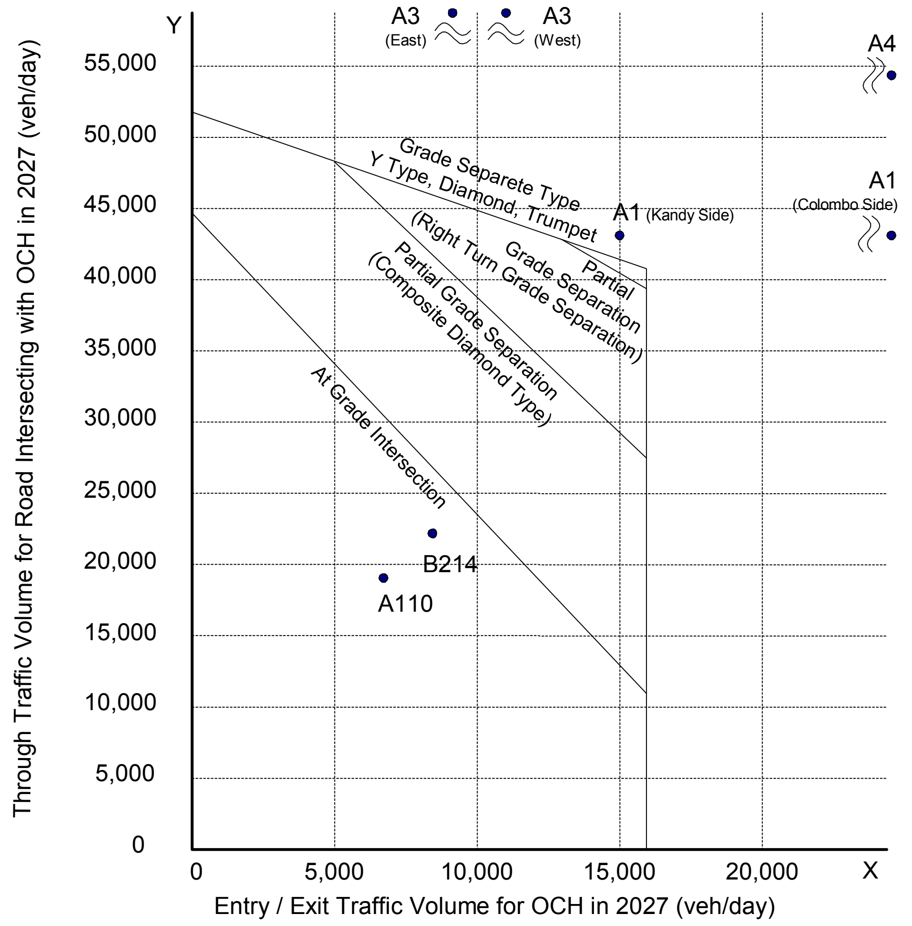
Type	**Semi Directional Type**	**Diamond Type (Split-type)**	**Double Trumpet Type**
Sketch			
Geometry	This interchange type comprises of all right turning directional loop ramps with elevated structure of 2 and 2 ½ level. A through traffic is at-grade and another through traffic will be on an elevated structure of 2 and 2 ½ level. All left turning traffic is mostly at-grade level. Vertical curve alignments are employed for all right turning traffic and a through traffic.	Diamond interchange has been introduced for two different types, which are conventional type and split type. For conventional type, it comprises of on-off ramps from the expressway for both directions connecting to two-way intersecting road at-grade level. Mainline of the expressway will cross over the intersection road. Right turning traffic and left turning traffic from both directions of the expressway and intersecting road will form two – separate at-grade intersections. For split type, there are two separate two-way intersecting roads. On-off ramps from each side of the expressway will join to each two-way intersecting roads at-grade level. Mainline of the expressway will cross over both two-way intersecting roads. Right turning traffic and left turning from a two-way intersecting road and on-off ramps from each side of the expressway will from two-separated at-grade intersections.	This interchange type comprises of a pair of trumpet interchanges, one on the expressway and one on the intersecting road, which are connected to each other with a ramp highway. The length of the connecting road way depends on the distances between the trumpet interchanges and the intersecting point. At the middle of ramp highway will be utilized for toll plaza. For each trumpet interchange, a directional loop ramp will be provided for high right turning traffic volume and a complete loop ramp will be provided for less right turning traffic volume. Left turning traffic ramps will be corresponding to right turning traffic ramps. Vertical curve alignment will be applied at the ramps intersecting the expressway and the intersecting road.
Operation	This interchange type is suitable for serving high right turning traffic volume from all legs with the speed of 60 km/hr. All right turning traffic will be maneuvered along directional loop ramps and alignment from at-grade level to second level and second and half level. Left turning traffic will be maneuvered along at-grade directional ramps. No weaving traffic is involved. Toll plaza has to be located at 2 locations on the opposite legs for a toll expressway.	For conventional type, each at-grade intersection will create 3 conflict points. If there are a lot of right turning traffic, there will be a lot of traffic problems. Traffic signals for each intersection should be provided. For split type, each at-grade intersection will create only 1 conflict point. It is not necessary to provide traffic signal. Toll plaza has to be provided for each side of the on-off ramps of the expressway.	Turning traffic for this interchange has to be maneuvered by following traffic signs carefully. Some turning traffic have to use right turning loop ramp with left turning ramps, right turning loop ramp with another right turning loop ramp, left turning ramp with another left turning ramp, and left turning ramp with right turning loop ramp. There is weaving traffic on the ramp highway, normally, weaving length would not be less than 150m. However, it depends on weaving traffic volume as well. At this location, toll plaza is suitable to be provided.
Stage Construction	It is not so difficult to modify for stage construction as long as the right of way for final stage development is provided. The areas inside left turning ramps are sufficient for final stage development.	This interchange type is not necessary to be provided for stage construction since it is the minimum requirement for design of the expressway.	It is so difficult to modify for stage construction. It is very simple to arrange for stage construction as long as land acquisition provided for final stage construction.
Construction Cost / Land acquisition cost	Since 7 bridges will be constructed at separated locations, the construction cost for structure is rather high even though the structure height is only 2 to 2 ½ level. The land cost is higher than the diamond type, but less than the double trumpet type.	The construction cost for this interchange type is minimum. It is required minimum right of way. There is no construction cost for structure. The bridges crossing the intersecting road are included in the mainline of the expressway.	There are only two locations for elevated structure for right turning ramps. The construction cost would be less comparing to the directional type. However, the area for loop ramps of both trumpet type including the ramp highway is the most critical to the land acquisition. Therefore, the land acquisition cost is the highest.
Maintenance Cost	Maintenance cost for structure is rather high because seven bridges are located separately. Maintenance cost for at-grade ramps and surrounding area is rather high but still less than the double trumpet type since the area is less.	There are only maintenance cost for only on-off ramps, at-grade intersections and traffic signals. Surround areas for this interchange type is very little. Therefore, the maintenance coat is very little comparing to the others.	Maintenance cost for structure is quite less, but for loop ramps highway and surrounding areas, maintenance cost is very high.
Future Expansion / Modification	Not applicable	Not applicable	Not applicable

3.13.2. Selection of Configuration

The configuration of an interchange should be normally decided by the expected through traffic volumes of connection road and the expected entry and exit traffic volume of the interchange. As a reference, the criterion to select interchange configuration is shown in **Fig. 3-5** (Source: Japan Highway Design Manual).

However, if a plotted point falls outside of the outer most solid line, as most of the OCH interchanges do, it does not mean that it is impossible to manage traffic, as there is a margin of safety for these calculations. In such a case, the capacity of each interchange should be confirmed by via another method.

It is obviously that grade separation is more costly than at grade intersection. So, it shall be carefully considered in forecasting of the traffic demand and all the turning movements before the grade-separated intersection is adopted. At the selection of the type, it is important to minimize the number of structure for possible cost-saving of the construction of interchange. The standpoints commonly used in Japan, as far as possible, adopts at grade intersection due to the cost effectiveness, in case of linking four lanes.



Note: The figure shown is applicable to two-lane ramp corresponding to the four lane-intersecting road.

Fig. 3-5 Criterion for Selection of Interchange Configuration

3.14. Toll Collection Facilities

RDA was decided that OCH and STDP shall be toll road and will adopt the Interchange toll collection (closed collection) with variable rate for travel distance, in February 2007. Following to the decision, it is required to install toll plaza on every ramps (on and off) of OCH.
For standardisation of toll facilities, it is expected to establish the unity design standard by the RDA. Before that, following standard based in the Japanese Standard is used tentatively.

3.14.1. Determination of Number of Lanes at Toll Gate

The number of lanes at a toll gate is determined based on the traffic volume (vehicle entry interval), average service time and level of service (judged by average queue) as shown in **Table 3-39**. Note that at least 2 lanes shall be provided in each direction to secure reserve lane, even if only 1 lane is required by the above.

(1) Design Hourly Traffic Volume (DHV)

Design hourly traffic volume shall be 30th highest hourly traffic volume. 30th highest hourly traffic volume is calculated from annual average daily traffic volume (AADT) using the formula below:

$$DHV = AADT \times K \times D$$

Here,	DHV (one direction):	Design hourly traffic volume by directions
	AADT (both directions):	Total annual average daily traffic of both directions (traffic volume on the planned date based on the estimated traffic volume)
	K:	Ratio of the 30th highest hourly traffic volume (total of both directions) to the AADT
	D:	Ratio of the traffic volume of the heavy traffic side to the total traffic volume of both directions at the 30th hour

K-value and D-value as above may be established with reference to actual data measured in the design area or an area with similar conditions.

(2) Average Service Time

Average service time of 6 seconds at entry and 14 seconds at exit in case of variable rate system, and 8 seconds in case of flat rate system are used to calculate the number of lanes. In a section where different values are expected, other average service times may be used.

(3) Level of Service

Level of Service shall be judged by average queue and it shall be 1.0 vehicle. Other values up to 3.0 vehicles may be used if there are topographical or other difficulties.

Table 3-39 Possible Traffic Capacity and Service Time at the Toll Gate (veh/hour)

Number of Lanes \ Average Queue \ Service Time (Sec.)	6		8		10		14		18		20	
	1.0	3.0	1.0	3.0	1.0	3.0	1.0	3.0	1.0	3.0	1.0	3.0
1	300	450	230	340	180	270	130	190	100	150	90	140
2	850	1,040	640	780	510	620	360	440	280	350	250	310
3	1,420	1,630	1,070	1,230	850	980	610	700	480	550	430	490
4	2,000	2,230	1,500	1,670	1,200	1,340	860	960	670	740	600	670
5	2,590	2,830	1,940	2,120	1,550	1,700	1,110	1,210	860	940	780	850
6	3,180	3,430	2,380	2,570	1,910	2,060	1,360	1,470	1,060	1,140	950	1,030
7	3,770	4,020	2,830	3,020	2,260	2,410	1,620	1,720	1,260	1,340	1,130	1,210
8	4,360	4,630	3,270	3,470	2,620	2,780	1,870	1,980	1,450	1,540	1,310	1,390
9	4,960	5,220	3,720	3,920	2,980	3,130	2,130	2,240	1,650	1,740	1,490	1,570
10	5,560	5,820	4,170	4,370	3,330	3,490	2,380	2,490	1,850	1,940	1,670	1,750
11	6,150	6,420	4,610	4,820	3,690	3,850	2,640	2,750	2,050	2,140	1,850	1,930
12	6,740	7,020	5,050	5,270	4,040	4,210	2,890	3,010	2,250	2,340	2,020	2,110
13	7,340	7,620	5,510	5,720	4,400	4,570	3,150	3,270	2,450	2,540	2,200	2,290
14	7,940	8,220	5,945	6,170	4,760	4,930	3,400	3,520	2,650	2,740	2,380	2,470
15	8,530	8,820	6,400	6,620	5,120	5,290	3,660	3,780	2,840	2,940	2,560	2,650

According to the above, the calculated number of lanes at the toll gates for each interchanges are resulted as shown in **Table 3-40**.

Table 3-40 Number of Lanes at the Toll Gate

IC		AADT (veh/day)	K Rate for 30th Highest Hourly Ttraffic Volume	DHV (veh/hour)	Service Time (sec)	Average Queue (veh/lane)	Handling Capacity (veh/hour)		No. of Lanes for Toll Gate	
							No. of Lanes	Capacity (veh/hour)	Calculated	Adopted
CKE	OCH→CKE	16,800	0.07	1,176	14	1.0	6	1,360	5.2	6
	CKE→OCH	12,300	0.07	861	14	1.0	4	860	4.0	4
A3	OFF	11,200	0.07	784	14	1.0	4	860	3.6	4
	ON	0	0.07	0	14	1.0	2	360	0.0	2
	ON	8,800	0.07	616	6	1.0	2	850	1.4	2
A1	OCH(N)→A1	6,800	0.07	476	14	1.0	3	610	2.3	3
	A1→OCH(S)	13,600	0.07	952	6	1.0	3	1,420	2.0	3
	OCH(S)→A1	14,400	0.07	1,008	14	1.0	5	1,110	4.5	5
	A1→OCH(N)	8,200	0.07	574	6	1.0	2	850	1.4	2
B214	OFF	3,700	0.07	259	14	1.0	2	360	1.4	2
	ON	4,400	0.07	308	6	1.0	2	850	0.7	2
AB10	OFF	3,000	0.07	210	14	1.0	2	360	1.2	2
	ON	3,800	0.07	266	6	1.0	2	850	0.6	2
A4	OCH→A4	20,500	0.07	1,435	14	1.0	7	1,620	6.2	7
	A4→OCH	22,600	0.07	1,582	6	1.0	4	2,000	3.2	4

3.14.2. Design Criteria for Toll Plaza Facility

(1) Horizontal Alignment

Straight line is desirable for horizontal alignment at an area installing the toll plaza. Otherwise, it shall be the same criteria with main carriageway at the area of interchange, in case of main carriageway toll gates, and minimum radius of 200m is adopted for interchange toll gates.

Table 3-41 Minimum Radius of Horizontal Curve at Toll Plaza

Toll Gate Type	Desirable Alignment	Desirable Minimum (m)	Absolute Minimum (m)	Remarks
Main Carriageway	Straight	1,100	700	Criteria for Main Carriageway at the Area of Interchange
Interchange	Straight	-	200	

(2) K-value

K-value for vertical curve at an area installing the toll plaza shall be the same criteria with main carriageway at the area of interchange in case of main carriageway toll gates. In case of interchange toll gate, minimum K-value shall be 8 (Absolute value shall be 7) for crest curve, for sag curve, it shall be the same criteria with ramp throughway.

Table 3-42 Minimum K-value at Toll Plaza

Toll Gate Type		Desirable Minimum	Absolute Minimum	Remarks
Main Carriageway (Design Speed 80km/h)	Crest	120	60	Criteria for Main Carriageway at the Area of Interchange
	Sag	80	40	
Interchange	Crest	8	7	
	Sag	10	7	

(3) Gradient

Maximum gradient at the toll gate shall be less than 2.0% and less than 3.0% absolutely. The extent of each gradient is at least 50m on each side of the center line of the toll gate, and at least 100m for the toll gate especially on main carriageway with design speed of 80km/h or over.

Table 3-43 Minimum Gradient at Toll Plaza

Toll Gate Type		Desirable Minimum	Absolute Minimum	Remarks
Main Carriageway (Design Speed 80km/h)	Crest	120	60	Criteria for Main Carriageway at the Area of Interchange
	Sag	80	40	
Interchange	Crest	8	7	
	Sag	10	7	

(4) Crossfall or Superelevation

Crossfall or superelevation at the toll plaza shall be 2.0 %.

(5) Pavement Structure

Pavement structure at toll plaza shall be cement concrete pavement. The extent of paving on each side of the center line of the toll gate is shown in **Table 3-44**.

Table 3-44 Extent of Cement Concrete Pavement at Toll Plaza (L_0)

Toll System	Interchange Toll Gate		Main Carriageway Toll Gate
	National Highway Side	Expressway Side	
Magnetic Card	35m	30m	50m
Others	30m	30m	

(6) Taper for Transition of Width at Toll Plaza

The width for the area of cement concrete pavement as above shall be same width as that in the center of toll gate. Taper for transit section from above area to typical width of ramp shall be planed as shown in **Fig. 3-6**, with special consideration to aesthetic point of view.

Taper ratio shall be less than 1/3 in terms of S/L as shown in **Fig. 3-6**, with consideration to prevent interference to traffic.

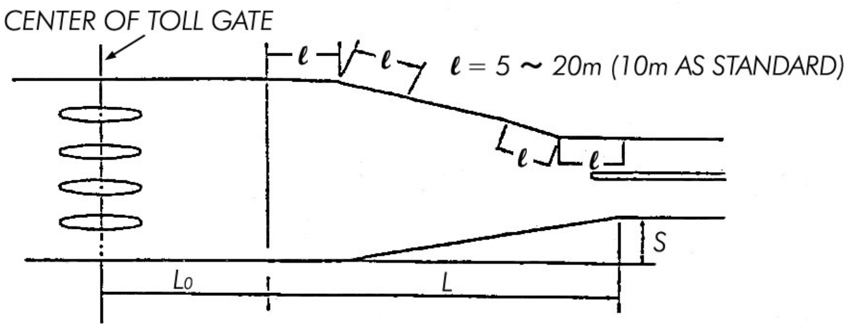


Fig. 3-6 Taper for Transition of Width at Toll Plaza

(7) Distance from Center of Toll Gate to End of Median or Diversion Point at Ramp

At main carriageway toll gates, sufficient distance shall be provided between the center of the toll gate and the end of the median, with consideration to reversible lanes, so as to ensure undisturbed traffic.

On the other hand, at interchange toll gates, distance between the center of the toll gate and the diversion point at the ramp shall be at least 75 m.

4. EARTHWORK

4. Earthwork

4.1. General

The design standards of earthworks hereunder has been prepared based on the Japanese design standards taking into account local practice particularly other relevant expressway projects in Sri Lanka.

4.2. Excavation (Cutting)

4.2.1. Cut Slope

The residual soil (namely "Laterite") is able to observe almost in project area as shown in **Fig. 4-1** Geological Distribution Map. In the hills along the OCH, there are cuts and quarries where bedrock outcrops. According to the soil investigation survey, the ground is generally covered with reddish-colored laterite (weathered soil) to a thickness of several to more than 10 m. Bedrocks distributed in the projected area are confirmed.

From the properties of Laterite, the cut slope ratio generally applied in Sri Lanka is 1: 1 unless the material will stand at a steeper slope. The cut slope ratio has been recommended based on the Japanese standards shall be 1: 1.2 generally considering the maintenance works at the operation of expressway.

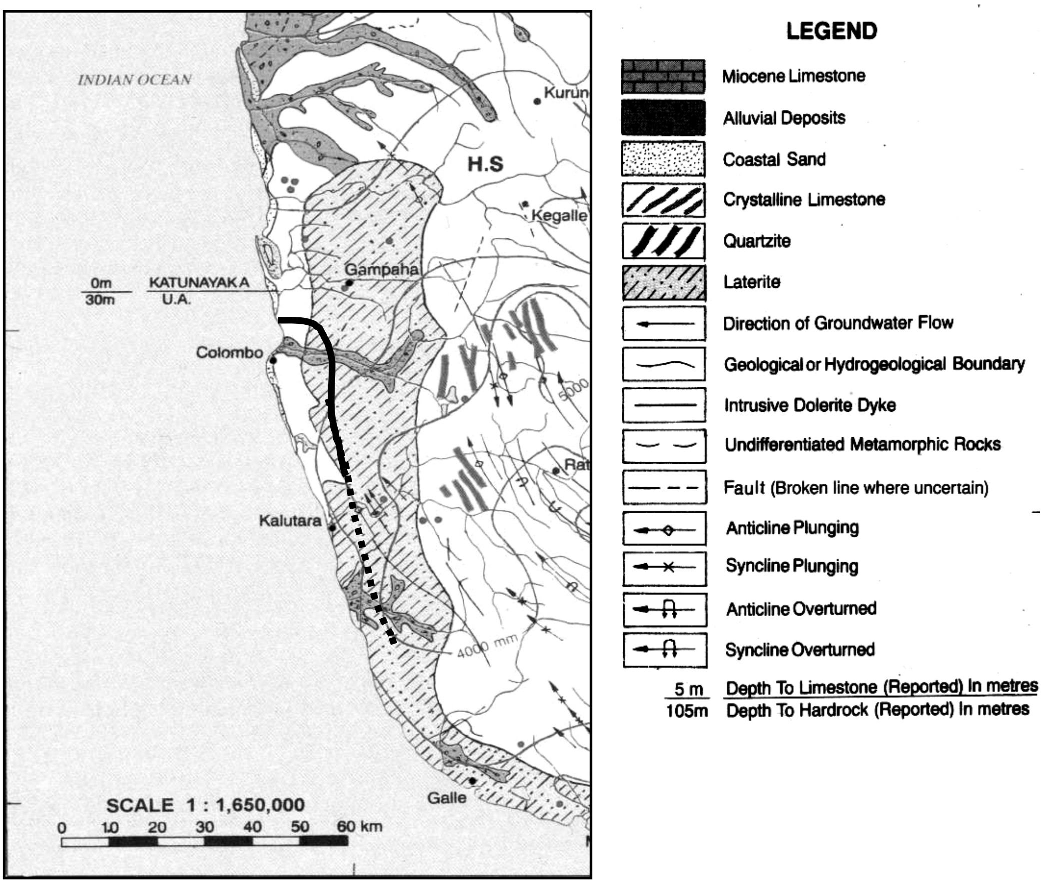


Fig. 4-1 Geological Distribution Map

The design standard of cutting slope based on the soil conditions has been decided as shown in **Table 4-1**.

Table 4-1 Standards of Cutting Slope

In situ Soil	Height	Slope Ratio	Berm*	
			Height	Width
Hard Rock	-	1: 0.3-0.5	7m	1.5 m
Soft Rock	-	1: 0.8-1.0		
Covered Soil (Laterite)	5m or less	1: 1.0-1.2		
	5 - 10m	1: 1.0-1.2		

Note: Berm will be installed when the cutting height is more than 10m.

JICA Study Team decided that the cut slope for the D/D section should be 1:1.2, since the area (Southern Section) consists mostly of residual soil and also because there is a need to reduce the volume of borrow embankments.

4.2.2. Cut Slope Treatment (Rounding)

The top of cut slopes shall be rounded in order to prevent the erosion except cutting solid rock. The amount of rounding depends on the material depth of rock if any, and the natural contour of the ground. The 1.0-meter rounding indicated in **Fig. 3.2.12** is the typical treatment.

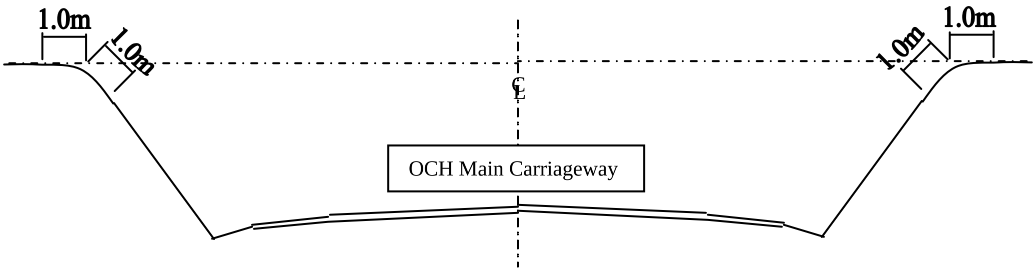


Fig. 3.2.12 Rounding of Tops of Cut Slope

4.2.3. Berm

Where cuts exceed 10 meter (vertical height), the berm must be provided at 7 meter from the bottom of slope in order to secure sufficient stability of slope. The berm should be sloped to form a valley along the center so that storm water can be collected and drained off toward the side of the carriageway through vertical drains then discharged to projected down stream. The berm width should not be less than 1.5 meter with minimum gradient 0.3% for drainage role.

4.3. Embankments (Filling)

The provision of the standards for embankment slope and the berm based on Japanese standards are given in **Table 4-2**.

Table 4-2 Standards of Filling Slope

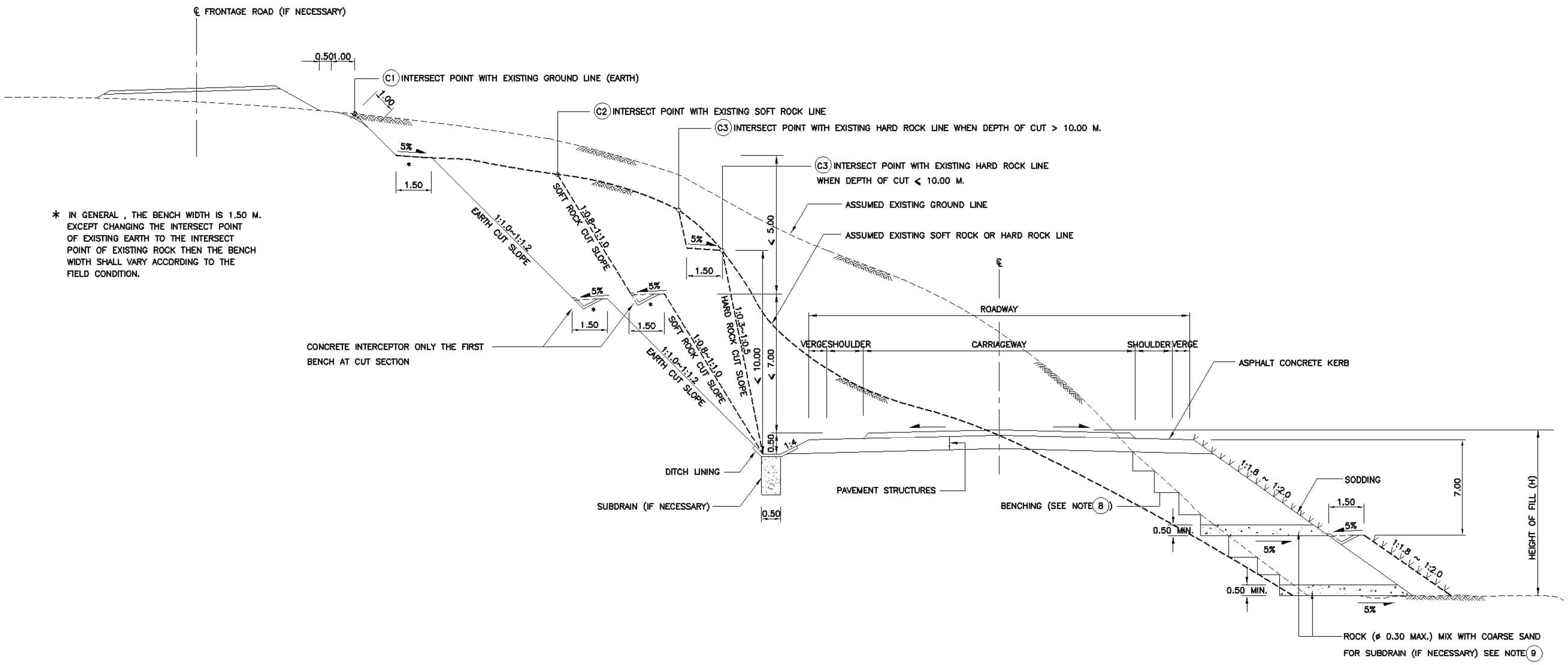
Material	Height	Slope Ratio	Berm*	
			Height	Width
Sandy Soil (Classified Material)	3m or less	1: 1.8-2.0	-	-
	3m – 6m	1: 1.8-2.0	-	-
	6m or more	1: 1.8-2.0	Every 7m	1.5m

Note: Berm will be installed when the filling height is more than 10m.

JICA Study Team decided that the filling slope for the D/D section should be 1:1.8, because of the experience of Japanese expressway based on the Japanese standard and requirement to reduce the volume of borrow embankments. This ratio has been also confirmed by calculation of slope stability through the study for soft soil countermeasures.

4.4. Standard for Earthworks

The standard earthworks cross - section is shown in **Fig. 4-2**.



NOTES :

1. DIMENSIONS ARE IN METERS UNLESS OTHERWISE INDICATED.
2. THE CROSS – SECTION OF CUT AND FILL HEREIN SHALL BE APPLIED ONLY WHEN THE DEPTH OF CUT IS OVER 7.50 METERS FROM THE BOTTOM OF THE SIDE DITCH , AND HILL SIDE FILL ALSO.
3. PAVEMENT STRUCTURES AND OTHER DETAILS WHICH ARE NOT SPECIFIED IN THIS DRAWING SHALL BE REFERED TO THAT IN THE TYPICAL CROSS – SECTION DRAWING.
4. THE PROCESS OF RIPPING AND EXPOSING THE CUT MATERIALS SHALL BE MEASURED AS FOLLOWS :
   - 4.1 IN CASE OF SOIL WITHOUT ANY ROCKS APPEAR ABOVE THE GROUND SURFACE , THE POINT C1 IN THE DRAWING WILL BE THE INTERSECTING POINT BETWEEN THE SLOPE OF CUT AND THE EXISTING GROUND LINE SO THE EXCAVATION SHALL START FROM THIS POINT.
   - 4.2 AFTER THE EXCAVATION AS INDICATED IN SECTION 4.1 FOR A DISTANCE AND THE SOFT ROCK OR HARD ROCK WAS FOUND , THEN THE TOP POINT OF THE SLOPE SHALL BE CHANGED FROM POINT C1 TO POINT C2 OR C3 AS INDICATED ON THE DRAWING THE BERM WIDTH OF BENCHING SHALL BE DIRECTED BY THE ENGINEER THE STABILITY OF THE CUT SLOPE SHOULD BE CAREFULLY CONSIDERED AND THE UNSUITABLE TOP SOIL MATERIALS SHALL BE REMOVED.
   - 4.3 THE CLASSIFICATION OF SOIL , SOFT ROCK OR HARD ROCK SHALL BE CONSIDERED IN ACCORDANCE WITH THE TECHNICAL SPECIFICATION AND ALSO SHALL BE DIRECTED BY THE ENGINEER.
   - 4.4 THE QUANTITIES SHALL BE CALCULATED FROM THE CROSS – SECTION , SEPARATELY FOR SOIL , SOFT ROCK OR HARD ROCK. IN CASE OF MIXED MATERIALS FOR EACH CROSS – SECTION, THEN THE NEGOTIATION BETWEEN THE OWNER AND THE CONTRACTOR SHOULD BE ARRANGED.
5. CONCRETE INTERCEPTOR ON CUT SLOPE SHALL BE CONSTRUCTED ON SILTY SAND , GRAVEL LATERITE , SOFT ROCK OR SHALE , BUT BE NOT NECESSARY ON SOLID ROCK AREA.
6. THE LONGITUDINAL SLOPE OF CONCRETE INTERCEPTOR IN NOTE 5 SHALL NOT BE LESS THAN 0.3 PERCENT.
7. THE INTERVAL OF 0.5 cm. WIDE MORTARED JOINT SHALL NOT BE MORE THAN 15.0 m. FOR THE CONCRETE INTERCEPTOR.
8. BENCHING SHALL BE REQUIRED ON EXISTING GROUND SLOPE OR EXISTING ROADBED IN THE PORTION OF EMBANKMENT. THE NUMBER OF STEPS FOR BENCHING DEPENDS UPON THE HEIGHT OF SLOPE. THE HEIGHT OF EACH STEP SHALL BE DIRECTED BY THE ENGINEER , AND THE WIDTH SHALL BE PERMITED FOR COMPACTION EQUIPMENT , AND THE DENSITY OF THE COMPACTED MATERIAL SHALL NOT BE LESS THAN 95 PERCENT OF STANDARD PROCTOR.
9. BEFORE CONSTRUCTING PAVEMENT STRUCTURES, IF GROUND WATER SEEPAGE APPEARS ON CUT SLOPE OR HILL SIDE FILL AND SEEMS TO DAMAGE THE ROADWAY, THE SUBDRAIN AS SHOWN ON THE DRAWING SHALL BE APPLIED.
10. CONCRETE FOR INTERCEPTORS AND DITCH LIVING SAHALL BE GRADE 15.

Fig. 4-2 Standard Earthworks Cross Section

5. APPROACH ROADS AND FRONTAGE ROADS

5. Approach Roads and Frontage Roads

5.1. General

The main object in designing the approach road is due to some of the existing roads being non-accessible once the Outer Circular Highway to the City of Colombo (OCH) is constructed. It would therefore be necessary to modify the plan and profile of the affected roads in order that the OCH could be crossed by constructing either overpasses or underpasses.

Every effort has been made to have the crossing points basically fixed. However, if any problems are envisaged due to the topography, land acquisition, road side condition or existing residences being affected, it may become necessary to change the crossing point from the existing roads.

This section discusses the design standards to be applied for the design of the approach roads including the widening of existing National Highway Sections. These design standards shall be based on Geometric Design Standards of Roads, published by the Road Development Authority in 1998.

5.2. Classification of Roads

The classifications of roads together with corresponding standards and specifications have been used, as stipulated in the design guide, criteria and standards of the Road Development Authority (RDA) of Sri Lanka.

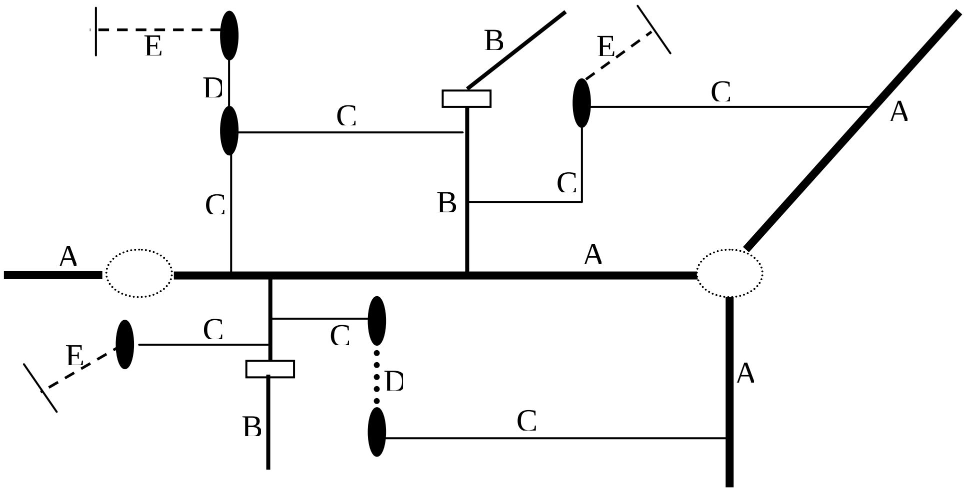
The ‘A’, ‘B’, ‘C’, ‘D’ and ‘E’ class roads are classified as follows. (Refer **Fig. 5-1**)

‘A’ Class Roads – Roads connecting the district centers with national capital city or roads connecting one district with another. Also included are roads serving densely populated corridors for which consideration of transport demand and usage is of great importance.

‘B’ Class Roads – Roads connecting towns or population centers with the district centers or with each other.

‘C’ Class Roads – All roads (other than ‘A’ and ‘B’ class roads) that have at least one terminal point connected to either an ‘A’ or a ‘B’ class road.

‘D’ and ‘E’ Class Roads – All residual roads that do not get into any of the classes mentioned above. These are local roads that provide access to settlements and villages.



Notations:

-Provincial/ District Centre

-Residential/ Industrial area, town centre

-Village

A, B, C, D, E -Road Class

Fig. 5-1 Road Network Diagram

5.3. Design Speed

The Design speed based on the highway classification shall be applied as shown on **Table 5-1**.

Table 5-1 Relationship of Design Speed Related with the Highway Classification, Terrain and Design Volume

Type of Road	Road Class	Terrain	Design Volume	Design Speed(km/h)	
				Rural	Urban
R5	D,E	F	< 300	50	40
R4	C,D	F	300 – 18,000	60	50
R3	A,B	F	18,000 – 25,000	70	60
R2	A,B	F	25,000 – 40,000	80	70
R1	A	F	40,000 – 72,000	80	70
R0	A	F	72,000 – 108,000	80	70

Note: Type of Road depends on the ranges of traffic volume in terms of PCU/day.
PCU – Passenger Car Units
F-Flat Terrain

5.4. Typical Cross Section

According to Geometric Design Standards of Roads of RDA, there are six types of road cross sections, R0 to R5 based on the Average Daily Traffic (ADT). The cross section types R0, R1, R2 and R3 may be used Class ‘A’ roads, type R2, R3 and R4 may be used for Class ‘B’ roads, types R4 and R5 may be used for Class ‘C’ roads and type R5 for Class ‘D’ and ‘E’ roads. The selection of the cross section type depends on the traffic volume ranges in terms of PCU/day.

Type R0 is used for 6-lane divided highways. The types R1 and R2 are for the 4-lane divided highway, but the selection of type R1 is for the highway, where the necessity for upgrading to a 6-lane facility is expected in the near future. Otherwise, it is most always type R2, subject to the following conditions:

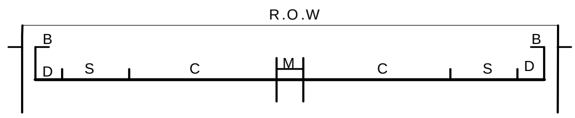
(1) The 4-lane undivided road is more hazardous than the divided road and has to be avoided as far as possible.
(2) The type R2 may be undivided if the length of the road is not more than 3 kms.
(3) The design speed to be limited to 40km/h in urban areas where the available R.O.W is very much limited.

Table 5-2 and **Fig. 5-2** shows width of the typical cross section element based on RDA Geometric Design Standards of Roads.

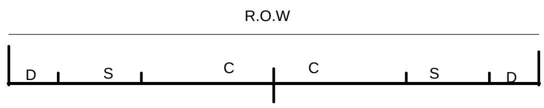
Table 5-2 Width of the Typical Cross Section Element

Type of Cross Section	Berm (m)	Drain (m)	Shoulder (m)	Carriageway (m)	Median (m)	R.O.W (m)
R0	-	0.9x2	3.0x2	10.5x2	1.2	30.0
R1	1.0 x 2 (0.0 min)	1.5x2 (0.9min)	3.0x2 (2.4min)	7.4x2	1.2	27.0
R2	0.6 x 2 (0.0 min)	0.9x2	3.0x2 (2.4 min)	7.4x2 (7.0 min)	1.2	25.0
R3	--	0.9x2	3.0x2	3.7x2	--	15.2
R4	1.2x2	0.9x2	2.4x2	3.1x2	--	15.2
R5	--	0.9x2	2.4x2	3.5	--	10.1

Note: (i) A bicycle lane of width 1.5m is included in the shoulder adjacent to the carriageway for Class ‘A’ and ‘B’ type roads.
(ii) The absolute minimum lane width for the carriageway is 3.1m and that of the shoulder is 1.8m.



(a) Typical Cross Section with Median
(Applicable to types R0,R1 & R2)



(b) Typical Cross Section without Median
(Applicable to type R3,R4+R5)

M = *Median*, C = *Carriageway*, S = *Shoulder*, D = *Drain*, B = *Berm*

Fig. 5-2 Typical Cross Section

5.5. Type of Pavement

Considering the case of construction and maintenance, the economical and the functional viewpoints *flexible type pavement* is selected for improving existing roads.

5.6. Crossfall

The recommended crossfalls for the carriageway and shoulder based on the RDA Geometric Design Standards will be used in **Table 5-3**.

Table 5-3 Recommended Crossfalls on Straight

	Type of Surface	Crossfall
Carriageway	Portland Cement Concrete	2.0%
	Asphalt Pavement	2.5%
	Surface Seals	3.0%
	Unsealed Gravel	4.0%
Shoulder	Type of Shoulder	Crossfall
	Bitumen or other all weather surface	3 – 4%
	Gravel	4 – 5%

5.7. Sight Distance

It is necessary for a driver to see sufficiently ahead to enable him to assess developing situations and take appropriate action. The common occurrences that arise are:

(i) Stop when approaching an obstacle
(ii) Decision on overtaking
(iii) An assessment of the action to be taken at an intersection

Accordingly, the driver may require a distance where he could safely come to a stop or may require a distance to overtake another vehicle safely.

5.7.1. Constants used for the Design of Sight Distance

The values specified in the Geometric Design Standards of RDA shall be used for constants for the design of Sight Distance.

▪ **Driver eye height**	
Passenger car	= 1.05 m
Commercial vehicle	= 1.8 m
▪ **Object cut-off height above road surface**	
Stationary object	= 0.20 m
Approaching Vehicle	= 1.15 m
Vehicle Tail height / Stop light	= 0.6 m
Height of Head Light	= 0.75 m
Upward Divergence Angle	= 1.0 deg
Vertical clearance	= 5.20 m

5.7.2. Stopping Sight Distance

Stopping Sight Distance (SSD), which is the distance a driver, travelling at the design speed needs to stop, has two components.

- The distance travelled during the total reaction time of 2.5 seconds (based on the RDA Standards)
- The distance travelled during the braking time.

The stopping distance therefore, is the sum of the distance the vehicle travels during a total reaction time (2.5seconds) and the braking distance.

The following equation can be used for the calculation of SSD:

$$\boxed{D = 0.694 * V + 0.00394 * V^2 / F} \quad \text{------------ (1)}$$

where D … SSD (m)
V … Design Speed (kmph)
F … Coefficient of Longitudinal Friction

Note: - The Coefficient of Longitudinal Friction 'F' varies with speed tyre pressure, tyre condition, type of pavement and whether the surface is dry or wet.

The tabulation below gives the desired Stopping Sight Distance for the respective design speeds.

Table 5-4 Stopping Sight Distance

Design Speed	Friction Coefficient on Wet Pavement	Stopping Sight Distance (m)	
(km/h)		Calculated	Rounded
70	0.31	110.84	115
60	0.33	84.62	85
50	0.35	62.84	65
40	0.38	44.36	45

5.7.3. Over Taking Sight Distance

The distance the driver needs to see ahead to safely overtake the vehicle moving ahead of him at constant speed is called the Overtaking Sight Distance (OSD).

The cross- section type used for the design of Class 'A' and Class'B-2'type roads are divided – 4 lane. Hence Over taking Sight Distance does not come into effect, as there is no opposing traffic stream.

However for the Class'B-3' and 'C' type roads which has no centre median, overtaking sight distance has to be considered.

A vehicle overtaking on a two-lane, two-way road has to encroach on to the other lane, which may at times be occupied by traffic in the opposite direction. As such, an overtaking driver requires sufficient visibility ahead of himself to ensure that there is enough gap length in the opposite traffic stream for him to safely complete the overtaking operation.

Since the safe Overtaking Sight Distance depends on many variables, the following assumption should be considered:

- (i) Only one vehicle is overtaken at a time
- (ii) The overtaking vehicle trails the overtaken vehicle as it enters the overtaking section
- (iii) The overtaken vehicle travels uniformly at one step lower than the design speed.
- (iv) Overtaking maneuver is accomplished by accelerating in the early part of the maneuver up to reaching the design speed and continuing at the same speed to complete the overtaking maneuver.

The overtaking Sight Distance for the various design speeds shall be as per Table 4.2 of the RDA Geometric Design Standards.

5.7.4. Application of Sight Distance Standards

Safe Overtaking Sight Distances are considerably longer than safe Stopping Sight Distance. As such it is not usually economical to provide OSD for the entire length of the design trace.

The absolute minimum sight distance that should be provided is the Stopping Sight Distance.

5.8. Horizontal Alignment

A horizontal alignment of a road is normally a series of straights and circular curves connected by transition curves indicating the path of the road in plan. The adaptation of superelevation also needs to be considered depending on the radius, of the curve and speed of the vehicle.

5.8.1. Maximum Superelevation

The superelevation to be adopted is selected primarily on the basis of safety. However, other factors such as comfort and appearance could also be considered.

Hence, a maximum superelevation of 6% is considered as given in Table 5.1 of RDA Geometric Design Standards.

It is important to note that existing road widening is to be carried out along a strip where ribbon development has taken place and consideration must be given to the comfort of pedestrians and cyclists as well as to traffic safety and car parking.

5.8.2. Minimum Superelevation

It is recommended that a minimum superelevation equal to normal cross fall, which is 2.5%, is considered for larger radii, where a smaller superelevation is sufficient for stability.

5.8.3. Maximum Side Friction Factor

The side friction factor is selected based on Table 5.2 of RDA Geometric Standards, which specifies 0.15 and 0.16 for the design speeds of 70 kmph and 60 kmph and 0.17 and 0.19 for design speeds of 50kmph and 40 kmph respectively, for bituminous roads.

5.8.4. Minimum Curve Radius

Minimum curve radius (R_{min}) for a given design speed (V km/h) can be determined using the following equation:-

$$\boxed{R_{min} = V^2 / 127 * (e_{max} + f_{max})} \quad \text{------------} \quad (2)$$

where e_{max} … maximum superelevation
f_{max} … maximum side friction factor

Summary of the above factors are given in **Table 5-5**.

Table 5-5 Minimum Curve Radius

Design Speed (km/h)	70	60	50	40
Max. Allowable Side Friction Factor (f_{max})	0.15	0.16	0.17	0.19
Max. Superelevation (e_{max}) %	6.0	6.0	6.0	6.0
Minimum Radius (m)	185	130	90	55

5.8.5. Adverse Crossfall

The minimum radius of curves with adverse crossfall of 2.5 % for roads in both built up and open areas are given in **Table 5-6**.

Table 5-6 Minimum Radii with Adverse Crossfall

Design Speed (kmph)	Minimum Radii for Adverse Crossfall (m)	
	Open	Built-up
70	1105	860
60	810	630
50	565	440
40	360	280

5.8.6. Transition Curves

Transition curves are inserted between tangents and circular curves, between two tangents (without circular curves), between two similar curves or between two reverse curves, especially where the curve radii are quite small.

The transition curve is used for the following reasons: -

(i) to provide a gradual increase or decrease in the radial acceleration when a vehicle enters or leaves a circular curve
(ii) to provide a length over which the superelevation development can be applied.
(iii) to improve the appearance of the road by avoiding sharp discontinuities in alignment at the circular curves.

Type of transition curve used is the clothoid (spiral curve). This is defined by the degree of curvature at any point being directly proportional to the distance along the curve.

Calculation of spiral length is determined using relative gradient method for lower design speeds (i.e. < 80 kmph) and using rate of pavement rotations method for higher design speeds (i.e. > = 80 kmph)

5.8.7. Relative Gradient Method

The minimum length of superelevation development from this method can be calculated from the expression,

$$\boxed{L_e \quad = W(e + n) / G_r} \text{ ------------------ (3)}$$

where L_e … Length of superelevation development (m)
W … Lane width (m)
e … Superelevation (%)
n … Normal crossfall (%)
G_r … Relative Gradient (%)

G_r should not exceed the values of maximum relative gradient which is given in **Table 5-7**.

Table 5-7 Maximum Relative Gradient

Design Speed (kmph)	Maximum Relative Gradient (%)		
	1-lane	2-Lane	> 2-Lane
70	0.56	0.84	1.12
60	0.63	0.95	1.26
50	0.71	1.07	1.42
40	0.83	1.25	1.66

If the transition curves are adopted, the length of superelevation development (Le) has to be contained fully within the length of spiral (Ls). Therefore the maximum value of Ls is taken as Le.

5.8.8. Minimum Length of Spiral Curve ($L_{S(min)}$)

This is considered as the distance travelled in 2 sec.

$$\boxed{L_{s(min)} = 2\,V/ 3.6 = 0.556V} \qquad \text{---------------- (4)}$$

where V … Design speed (kmph)

For design speed of 70 km/h, $L_{s(min)}$ = 0.556 * 70 = 38.92 m

For design speed of 60 km/h, $L_{s(min)}$ = 0.556 * 60 = 33.36 m

For design speed of 50 km/h, $L_{s(min)}$ = 0.556*50 = 27.8m

For design speed of 40 km/h, $L_{s(min)}$ = 0.556*40 = 22.24m

It is therefore recommended that the following minimum lengths of spiral be considered for the respective design speeds (rounded off to a multiple of 10m) in **Table 5-8**.

Table 5-8 Minimum Length of Spiral

Design speed (kmph)	70	60	50	40
Minimum length of spiral (m)	60	50	40	30

5.8.9. Selection of Appropriate Design Curves

If the length of circular curve is less than 25 m, it is recommended to use full spiral curve. If the shift of the circular curve is less than 100 mm a full circle curve could be adopted.

Again if the superelevation required is less than minimum of 4.0 % or 1.5 n (where n is normal crossfall – i.e. 2.5 %) the full circular curve could be selected. Otherwise the spiral-circle-spiral should be selected.

2/3 of the Superelevation Development occurs prior to the tangent point and 1/3 of the Superelevation Development is within the circular curve. The length of the Superelevation Development (Le) is generally rounded to the next higher multiple of 3.

5.8.10. Pavement Widening on Horizontal Curves

Pavements are widened on some curves to maintain lateral clearance between vehicles equal to the clearance available on straight section of road.

The amount of widening required depends on:

(i) The radius of the curve
(ii) Length and width of the vehicle
(iii) Lateral clearance between two vehicles
(iv) Width of the lane on the straight

Minimum radius recommended for design speeds of 70kmph and 60kmph are 185m and 130m and for design speeds of 50kmph and 40kmph are 90m and 55m respectively.

Pavement widening for design speed of 70kmh and minimum radius of 185 meters for Class ‘A’ roads can be disregarded, since the minimum widening required would be less than 0.6 meter. (Reference RDA Geometric Design Standards, Clause 5.7.1)

Pavement widening for single lane carriageway widths may be ignored.

The values for curve widening for the assumed design condition of an SU vehicle and 2-Lane highways are given in **Table 5-9**.

Table 5-9 Design Values for Pavement Widening on Curves

Radius of Curve (m)	Carriageway width 7.4 m and design speed = 70 km/h (m)	Carriageway width 7.0 m and design speed = 60 km/h (m)	Carriageway width 7.0 m and design speed = 50 km/h (m)
300	0.1	0.4	0.3
250	0.2	0.5	0.4
200	0.3	0.6	0.5
150	0.4	0.7	0.6
140	-	0.7	0.7
130	-	0.8	0.7
120	-	0.9	0.8
110	-	-	0.8
100	-	-	0.9
90	-	-	1.0

5.9. Vertical Alignment

The longitudinal profile of a road consists of a series of straight gradients and vertical curve. The vertical curves, in addition to smoothening the passage of a vehicle from one gradient to another, also increases the sight distance over crests at the junction of the gradients.

5.9.1. General Maximum Gradient

The maximum gradients vary with the class of the road, speed and topography.

The following maximum gradients are recommended based on RDA Geometric Design Standards.

Table 5-10 Maximum Gradients

Class of Road		A	B	C	D	E
Terrain Type	Flat	4%	5%	7%	9%	9%
	Rolling	6%	7%	9%	10%	10%
	Mountainous	8%	9%	10%	10%	10%

5.9.2. Minimum Gradient

Minimum gradient is mainly dependent upon drainage.

In urban areas where pavements are kerbed, longitudinal gradient of kerb and channel should not be flatter than 0.3%. In rural area a minimum gradient of 0.5% will be kept.

Note: If the road gradient is flatter than 0.5%, then the drains must be graded separately from the road center line to obtain a minimum of 0.5% slope.

5.9.3. Critical Length of Gradients

However, in very exceptional cases where the general maximum gradient are not practical and generally on less important roads, the length of steep gradient need to be limited to maintain the quality of service of the road and this is known as the Critical Length of Gradients. **Table 5-11** shows the Critical Length of Grades.

Table 5-11 Critical Length of Grades

Grades %	Critical length (m)
3	480
4	330
5	250
6	200
7	170
8	150
9	140
10	135
12	120

5.9.4. Vertical Curves

The type of vertical curve traditionally used is the simple parabola, which gives a constant rate of change of curvature and hence constant visibility, along its length.

(1) Length of Vertical Curves

Vertical curves are introduced between two consecutive gradients in order to increase sight distance across the junction of gradients and provide comfortable riding from one gradient to another. The main parameter of definition for the vertical curve is its length.

(2) Crest Vertical Curves

On crest curves, the driver's sight line is obstructed by the vertical geometry of the road. The minimum length may be fixed either by sight distance, riding comfort or approximately by appearance requirements.

(3) Minimum Length of Crest Vertical Curve (L_v) for Sight Distance (S) Requirements.

L_v is calculated using the following equations.

Case 1: S < Lv

$$\boxed{Lv = A\,S^2 / 200(\,h_1^{1/2} + h_2^{1/2}\,)^2 = A\,S^2 / 433} \quad \text{-----------------} \quad (5.a)$$

Case 2: S > Lv

$$\boxed{Lv = 2S - 200(\,h_1^{1/2} + h_2^{1/2}\,)^2/A = 2S - 433/ A} \quad \text{-----------------} \quad (5.b)$$

Where Lv … length of vertical curve (m)
S … Sight distance (m)
A … Algebraic difference in gradients (%)

h_1 … driver's eye height (m) (=1.05m)
h_2 … Object height (m) (=0.20m)

In summary,

S < Lv occurs when $AS>200\,(h_1^{\,1/2} + h2^{\,1/2})^{\,2}$, i.e. AS > 433
S > Lv occurs when $AS<200\,(h_1^{\,1/2} + h2^{\,1/2})^{\,2}$, i.e. AS < 433

The minimum length of the crest vertical curves based on the Sight Distance Criteria shall be as per Table 6.3 of the RDA Geometric Design Standards.

(4) Minimum Length of Crest Vertical Curve (L_v) for Appearance Criterion

For appearance criterion, minimum length of vertical curve is calculated using following equation.

$$\boxed{Lv = V_d * t / 3.6} \quad \text{-----------------} \quad (6)$$

where Lv … Vertical curve length (m)
V_d … Design speed (kmph)
t … Minimum required time (t = 3 sec)

Table 5-12 gives the minimum vertical curve lengths for crest curves for appearance criterion.

Table 5-12 Minimum Vertical Curve Lengths for Crest Curves for Appearance Criterion

Design speed V_d (kmph)	Minimum vertical curve length on crest curves (m)
70	60
60	50
50	50
40	40

(5) Length of Vertical Curve for Comfort Criterion

To minimize the discomfort felt by a human when passing from one grade to another, value of the vertical acceleration generated on the vertical curve should be less than 0.05g (where g is the acceleration due to gravity) based on RDA Geometric Standards. On low standard roads and at intersections a limit of 0.10 g may be used.

The following equation is used for calculating the length of vertical curve to satisfy the comfort criterion.

$$\boxed{a = V^2 A / 100 Lv} \quad \text{----------------} \quad (7)$$

Where
- a … vertical component of acceleration (m / s^2)
- V… speed of the vehicle (m / s)
- A… Algebraic difference in gradients (%)
- Lv… Length of vertical curve (m)

The recommended design value for a = 0.03g, where g = 9.81 m/sec2 as per RDA Geometric Design Standards.

Table 5-13 gives the length of vertical curves (m) for different design speeds and 1 % algebraic difference in gradients based on comfort criterion for vertical acceleration of 0.03 g.

Table 5-13 Minimum Vertical Curve Length based on Comfort Criterion.

Design Speed (kmph)	Length of Vertical Curve in Meters for 1% Algebraic Difference in Gradients (K-value) based on Comfort Criterion for Vertical Acceleration of 0.03g
70	13
60	9.4
50	6.5
40	4.2

(6) Length of Sag Vertical Curve for Head Light Criterion

During day light hours, it is assumed that adequate sight distance is available on sag curves.

However, on unlit roads at night, the sight distance available may be limited by head light reach.

Expression for the vertical curve length (Lv) required to satisfy head light requirements in terms of the required stopping sight distance, S (m) and change in grades A (%) are calculated using the following equations.

Case 1: S < Lv

$$Lv = \frac{S^2*A}{200(h+S \tan q)} = \frac{S^2*A}{150+3.49*S} \quad \text{----------} \quad (8.a)$$

Case 2: S > Lv

$$Lv = 2S-200(h + S \tan q)/A = 2S - (150+3.49*S)/A \quad \text{--------} \quad (8.b)$$

Where q … Upward divergence angle (= 1.0 degree)
h … Head Light height (= 0.75 m)
S … Sight Distance
L_v … Length of sag vertical curve
A … Algebraic difference in grade (%)

In Summary,
$S < L_v$ occurs when AS > 200/h + Stanq), i.e. AS 7u> 150+3.49*S
$S > L_v$ occurs when AS < 200/h + Stanq), i.e. AS < 150 +3.49*S

Table 5-14 Minimum Sag Vertical Curve Length based on Headlight Sight Distance Criterion

Design Speed (kmph)	Minimum Length of Vertical Sag Curves based on Head Light Criterion
70	25
60	17
50	12
40	7.3

(7) Over Head Obstructions

Over head obstructions such as road or rail overpasses or even overhanging trees may limit the sight distance available on Sag Vertical Curves. With minimum Over head 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 overtaking provision.

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

$$L_v = \frac{S^2 * A}{200 * ((H - h_1)^{1/2} + (H - h_2)^{1/2}\ )^2} = \frac{S^2 * A}{3105} \qquad \text{---------(9)}$$

where : L_v … length of vertical curve (m)
A … algebraic difference in gradients (%)
H … height of obstruction (= 5.10m)
h_1(=1.80m) and h_2 (=0.60m) are eye height and object cutoff height respectively.

(8) Drainage considerations

Longitudinal grade on kerbed pavements should at least be 0.30% for satisfactory drainage. On vertical curves, grades less than 0.30% near the apex of the crest curves or near the lowest point of the sag curves may be found for long vertical curves.

5.10. Road Alignment Harmonization

This it is a very important aspect for the success of road geometric design, and hence its design standards.

Rationalization of the following requirements with the proposed design standards needs to be considered:

i. General controls for Horizontal Alignment
ii. General controls for Vertical Alignment
iii. Combination of Horizontal and Vertical Alignment
iv. Alignment Co-ordination in Design

5.11. Summary of Geometric Design Criteria

The recommended geometric design standards for approach roads for the OCH determined by the discussion with RDA are shown below on **Table 5-15**.

Table 5-15 Summary of Geometric Design Criteria

Item		Unit	Contents of Geometric Design Criteria								Remarks (RDA Standards Reference
			A			B		C	D,E		
Road Class		-	A-0 (6-lane)	A-1 (4-lane)	A-2 (4-lane)	B-2 (4-lane)	B-3 (2-lane)	C (2-lane)	D (S-lane)	E (S-lane)	-
Type of Road		-	R0	R1	R2	R2	R3	R4	R5		Clause 3.1
Terrain			Flat	Flat /Rolling	Flat /Rolling	Flat /Rolling	Flat /Rolling	Flat /Rolling /M	Flat /Rolling /Mountainous		-
Designed Traffic Volume		PCU /day	72,000-108,000	40,000-72,000	25,000-40,000	25,000-40,000	18,000-25,000	300-18,000	<300		Table 2.6
Design Speed	Rural	km/h	80	80(70)	80(70)	80(70)	70(60)	60(40)	50(30)		Clause 2.5 / Table 2.6
	Urban	km/h	70	70	70	70	60	50(40)	40(30)		
Lane Width		m	10.5x2 3.5/lane	7.4x2 3.7/lane	7.4x2 3.7/lane	7.0x2 3.5/lane	3.5x2 3.5/lane	3.5x2 3.5/lane	3.5x1 3.5/lane		Table 3.3 -modified through discussion
Cycle Lane		m	1.5x2	1.5x2	1.5x2	1.5x2	1.5x2	-	-		Included to outer shoulder width
Outer Shoulder Width (Cycle lane included)		m	3.0x2	3.0x2	3.0x2	3.0x2	3.0x2	1.5x2 (2.4x2)	1.5x2 (1.8x2)	1.2x2 (1.8x2)	Primary width considering existing conditions for C,D,E class road
Median Width Including Inner Shoulder Width		m	1.20	1.20	1.20	1.20	-	-	-		Table 3.3
Drain (minimum)		m	0.90x2	0.90x2	0.90x2	0.90x2	0.90x2	0.90x2	0.90x2		Table 3.3
R.O.W. (Drain Widths Excluded)		m	28.2 (30.0)	22.0 (23.8)	22.0 (23.8)	21.2 (23.0)	13.0 (14.8)	10.0 (11.8)	8.3 (10.1)		Future width for C,D,E class road
Crossfall of Carriageway		%	2.5	2.5	2.5	2.5	2.5	3.0	3.0		Table 3.1
Crossfall of Outer Shoulder		%	4.0	4.0	4.0	4.0	4.0	3.0	3.0		Table 3.2
Minimum Radius		m	255 (185)	255 (185)	255 (185)	255 (185)	185 (130)	130 (55)	90 (30)		Clause 5.2.4/ Table 5.3
Minimum Radii with Adverse Crossfall of 2.5%	Open	m	1440 (1105)	1440 (1105)	1440 (1105)	1440 (1105)	1105 (810)	810 (565)	360 (205)		Clause 5.2.5/ Table 5.4
	Built up	m	1120 (860)	1120 (860)	1120 (860)	1120 (860)	630 (440)	630 (280)	280 (160)		
Maximum Gradient		%	4	4(6)	4(6)	5 (7)	5 (7)	7(10)	9(10)		Clause 6.2.1/ Table 6.1
Minimum Gradient	Rural	%	0.5	0.5	0.5	0.5	0.5	0.5	0.5		Clause 6.2.3
	Urban	%	0.3	0.3	0.3	0.3	0.3	0.3	0.3		
Stopping Sight Distance (SSD)		m	140(115)	140(115)	140(115)	140(115)	115(85)	85(45)	65(30)		Clause 4.2/ Table 4.2
Minimum Vertical Curve Length (for appearance criteria)		m	70(60)	70(60)	70(60)	70(60)	60(50)	50(40)	50(30)		Clause 6.3.2.2/ Table 6.4
Maximum Superelevation on Curvature		%	6								Clause 5.2.1/ Table 5.1

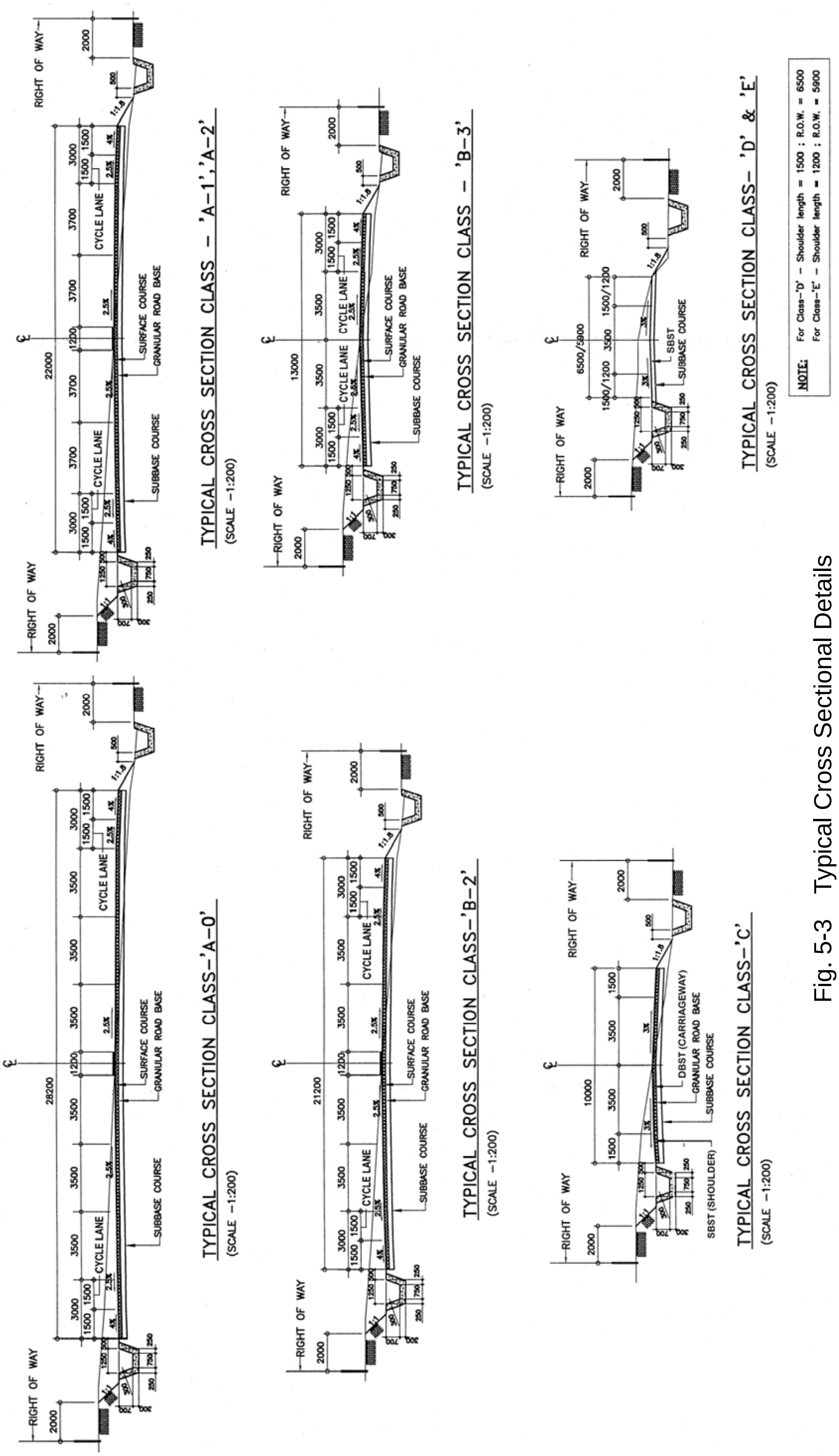


Fig. 5-3 Typical Cross Sectional Details

ROAD DEVELOPMENT AUTHORITY
MINISTRY OF HIGHWAYS AND ROAD DEVELOPMENT
THE DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA

THE DETAILED DESIGN STUDY ON THE OUTER CIRCULAR HIGHWAY TO THE CITY OF COLOMBO

FINAL REPORT
(FOR NORTHERN SECTION 1)

DESIGN STANDARDS OF STRUCTURE

February 2008

JAPAN INTERNATIONAL COOPERATION AGENCY

Oriental Consultants Company Limited

Pacific Consultants International

THE OUTER CIRCULAR HIGHWAY
TO
THE CITY OF COLOMBO

DESIGN STANDARDS OF STRUCTURE

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. GENERAL

2. APPLICABLE DESIGN STANDARD

3. GEOMETRIC DESIGN STANDARD

Design Standards of Structure

1. General

The design criteria to be used shall be based on RDA bridge design practice and the results of meetings between RDA and the Study Team. Where the specification is not sufficient then the Japanese Design Specification will be applied. This design criteria covers the following aspect of design:

1) Applicable Design Standards
2) Geometric Design Standards
3) Design Loads
4) Properties of Materials
5) Detailing
6) Design of Bridges

2. Applicable Design Standard

Design standards in OCH project are listed below.

Main standards from RDA
- ✓ Geometric Design Standards of Roads (1998)
- ✓ Bridge Design Manual (1997)
- ✓ Standard Specifications for Construction, and maintenance of Roads and Bridges (1989)
- ✓ Bridge Construction Manual (1997)

Sub standards for reference
- ✓ British Standard BS 5400 (1978 – 2000), 8002 (1994), 8004 (1986), 8110 (1985 – 1997)
- ✓ Design Manual for Roads and Bridges, British Standards Institutions (BSI)
- ✓ Specification of Highway Bridges (Japan Road Association, 2002)

3. Geometric Design Standard

3.1. Road Classification

The bridges in OCH are categorized according to the road classification given in "Geometric Design Standards of Roads" and Design Live Loads are different for the classification.

Table 3-1 Road Classification

	Road Classification	Design Live Load
Highway bridges	A – class road	HA and HB Live load
Overpass bridges	A, B – class roads	HA and HB Live load
	C, D, E – class roads	HA Live load

3.2. Bridge Width

Effective road width on the bridge is following:

Table 3-2 Road Width for Highway Bridges

(unit: mm)

	The 1st Stage (4 Lane Operation)	Final Stage (6 Lane Operation)
Viaduct Section	10,750 = 1,250 + 2@3,500 + 2,500	14,250 = 1,250 + 3@3,500 +2,500
Kelani River Section	10,750 = 1,250 + 2@3,500 + 2,500	14,250 = 1,250 + 3@3,500 +2,500

Table 3-3 Road Width for Ramp Bridges (1-Way 1-Direction)

(unit: mm)

	One Time Construction (1 Lane Operation)
A1 & B214 Interchange	7,000

Table 3-4 Road Width for Overpass Bridges

(unit: mm)

Road Class	Carriageway Width	Cycle Lane Width	Footway	Total Width
B2	15,200 (including center median 1,200)	1,500 * 2	1,500 * 2	21,200
B3	7,000	1,500 * 2	1,500 * 2	13,000
C	7,000	---	1,500 * 2	10,000
D	3,500	---	1,500 * 2	6,500
E	3,500	---	1,200 * 2	5,900

Note: Road width may be changed by the future plan of local road.

3.2.1. Highway Bridges

Highway bridges shall be widened inward at the final construction stage, that is same as earthwork (embankment and cut) section. The concept of the road widening is as follows:

Superstructure : 2 (two) lanes carriageway per each direction as the minimum required initially will be constructed at the 1st stage, and expanded inward the additional member at the final stage.

In case of being difficult for future expansion structurally, full width of structure (completed section) shall be constructed at the beginning (i.e. Pre-stressed concrete (PC) box girder is to be constructed with full width, and Pre-stressed concrete (PC) I girder is to be constructed with minimum required width at the 1st stage).

Substructure : One time construction shall be applied (full width to be constructed at the beginning) because of difficulty for future expansion.

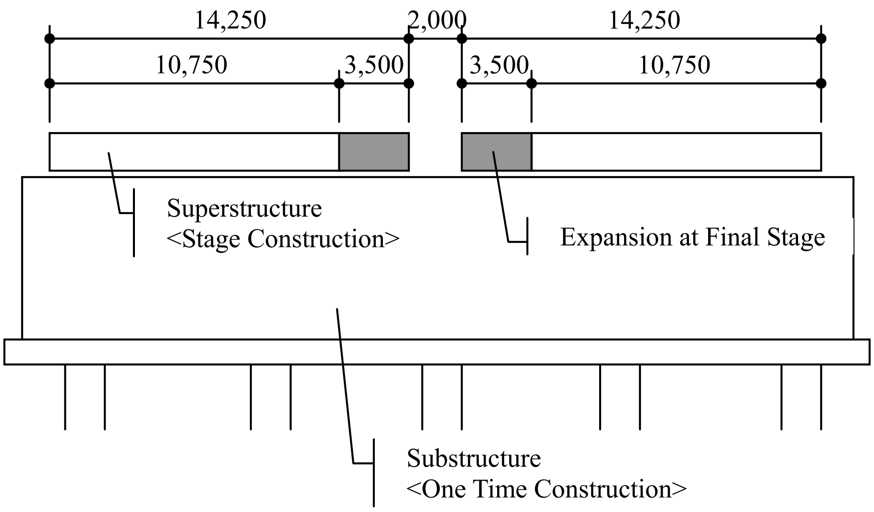


Fig. 3-1 Road Width Widening in the Structures (Effective Width)

In Northern Section 1 of OCH Project, PC-I Girder will be used as superstructure type for highway, ramp and overpass bridges. PC-I Girder can be added expansion members at the final stage (see **Fig. 3-2**).

The following subjects shall be considered in the design and/or construction methodology to join the member between old (the 1st stage) and new (the final stage):

- Required lap splice length of reinforcing bar shall be considered and pre-installed,
- Old and new girders shall be combined by cross beams completely,
- Differential camber between old and new girders shall be adjusted, and
- The effect by the creep/shrinkage of the new concrete shall be considered.

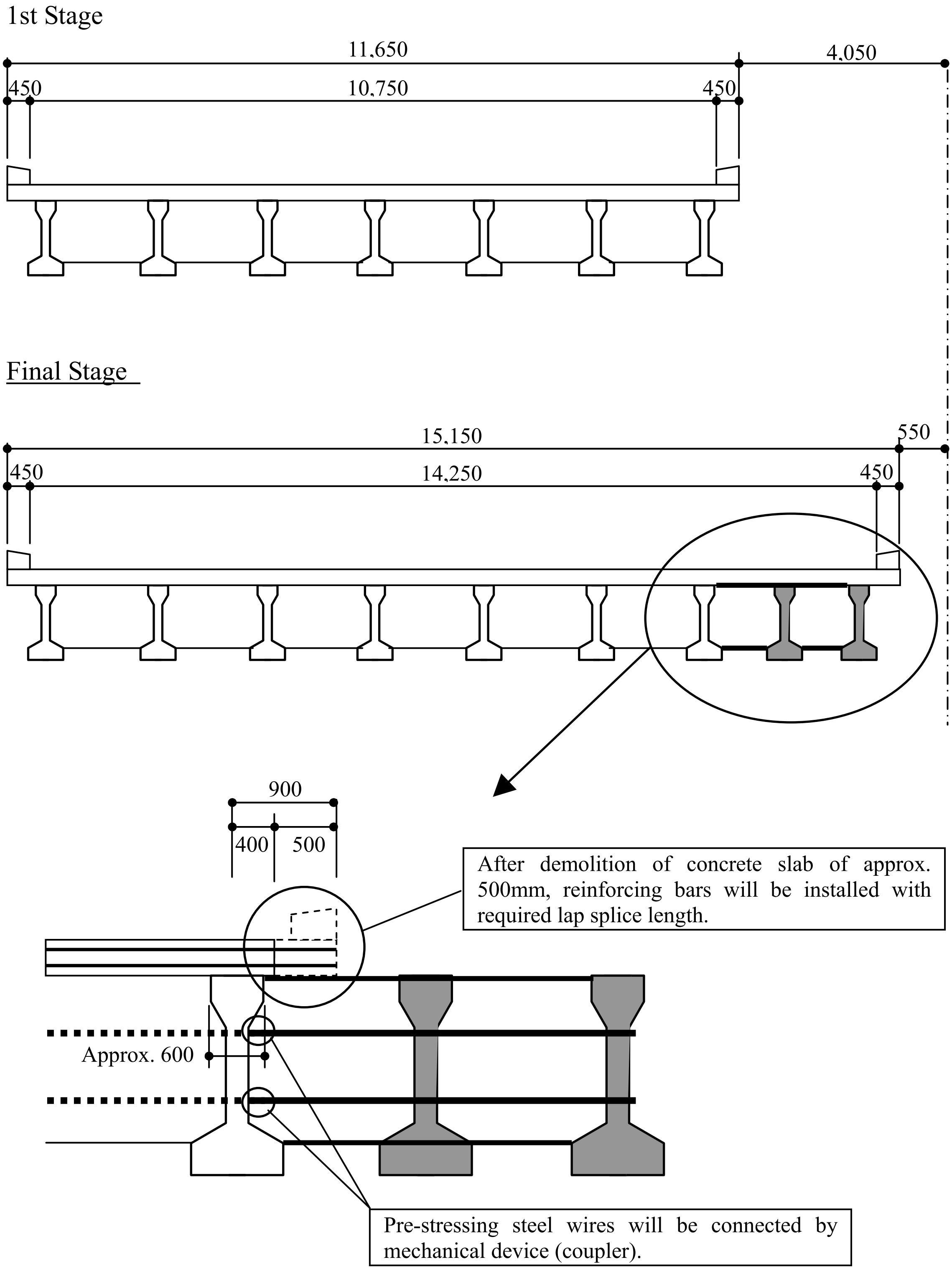
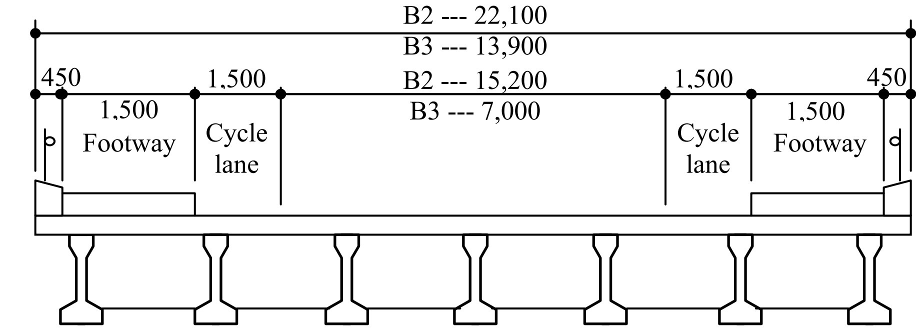


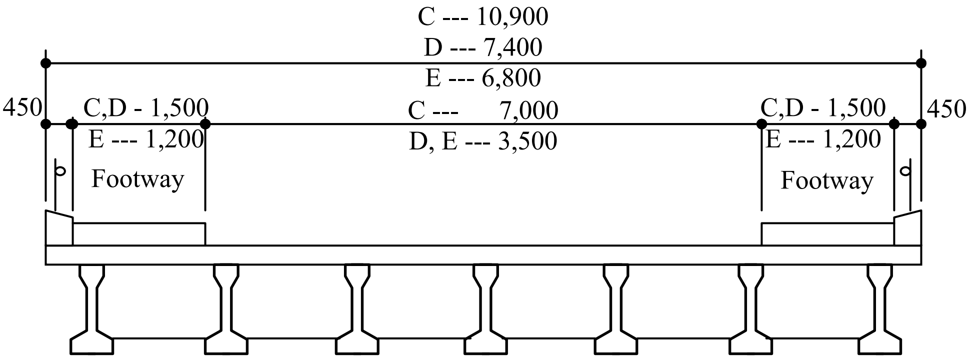
Fig. 3-2 Stage Construction of PC-I Girder

3.2.2. Overpass Bridges

Overpass bridges shall be provided with footways for pedestrian safety.



B2, B3 Class Road



C, D, E Class Road

Fig. 3-3 Cross Section of Overpass Bridge

3.2.3. Ramp Bridges

Both A1 and B214 Interchange ramps at bridge section has 1 lane. Typical cross section is shown in **Fig. 3-4**.

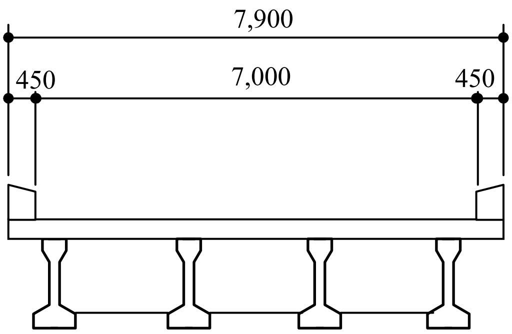


Fig. 3-4 Typical Cross Section of Ramp Bridge (1-Lane)

3.3. Bridge Clearance

3.3.1. Minimum vertical clearance

Table 3-5 Minimum Vertical Clearance

(unit: mm)

Bridges	Crossing Road	Vertical Clearance	Remarks
Overpass Bridges	OCH Road	5,100	
Highway Bridges	Railway	5,487 above rail level	N/A for OCH-N1
	Class A, B	5,100	
	Class C, D	5,100 (4,800)	
	Class E and less	4,500	
	Pedestrian Road	3,000	

Although no crossing to railway in OCH-N1, according to Detailed Design of OCH-S Project, the required vertical and horizontal clearances were decided based on Sri Lanka Railways, confirmed letter dated on September 5, 2004. And these clearances are as follows;

- Vertical clearance H=5,487mm (18ft)
- Horizontal clearance L=7,620mm (25ft)

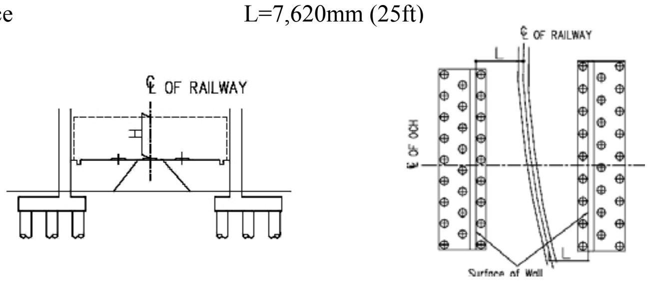


Fig. 3-5 Clearance for Railway

3.3.2. Horizontal Clearance for Main Highway

Horizontal clearance at outer shoulder side of the main highway shall be taken 3 m for the emergency cases.

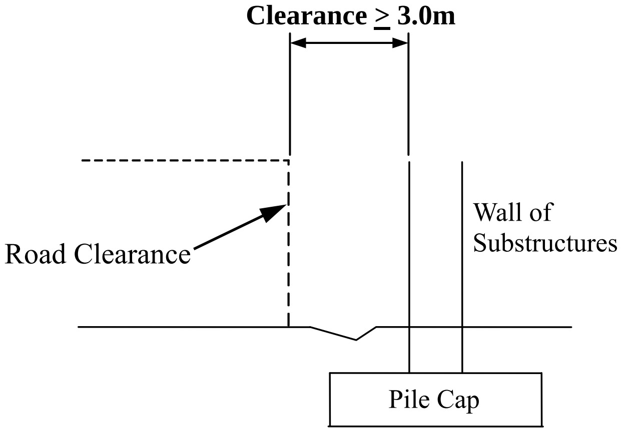


Fig. 3-6 Horizontal Clearance for Main Highway

3.3.3. Navigation Clearance and Free Board

1) Kelani Ganga (STA. 16+320) ---- ---- ---- ---- ---- ---- ---- ---- ---- ----

✓ Height : 4.75 m from A.W.L. (Annual Water Level)
✓ Width : 6.00 m
✓ Free Board : 1.20 m above H.F.L. (High Flood Level)
Where;
Annual Water Level : + 4.00m from M.S.L. (Mean Sea Level)
High Flood Level : + 8.50m from M.S.L.
✓ Specified Impact : Not specified in RDA, it will be considered "Floating Debris and Log Impact" discussed on 4.13.

Required elevation at bottom of upper-structure (PC I-Girder)
- Due to navigation clearance
  E.L.1 = + 4.00 + 4.75 = + 8.75m M.S.L.
- Due to high flood control
  E.L.2 = + 8.50 + 1.20 = + 9.70m M.S.L.
- Therefore, E.L.2 = + 9.70m M.S.L. governed

2) Mudun Ela (STA. 15+900) ---- ---- ---- ---- ---- ---- ---- ---- ---- ----

✓ Free Board : 0.60 m above H.F.L. (High Flood Level)
Where;
Annual Water Level : + 3.80m from M.S.L. (Mean Sea Level)
High Flood Level : + 8.20m from M.S.L.

Required elevation at bottom of upper-structure (PC I-Girder)
- Due to high flood control
  E.L. = + 8.20 + 0.60 = + 8.80m M.S.L.

3.3.4. Minimum Bridge Span in the River

The span length in river area at the normal time shall be considered the smooth water flowing. Existing bridge named Kaduwela Bridge which is located 1.6 km upstream direction from the planning OCH crossing over point for Kelani River, has 23.19m (76 feet 1 inch).

Therefore, at least the 23.19m length span (pier to pier distance) for this Project, shall be adopted in order not to obstruct the water flowing.

As the reference, Japanese code for river structure says that "minimum bridge span on the river measured perpendicular to water flow (see **Fig. 3-7**) is specified to avoid damages to bridges by floating logs and debris." And the minimum span length may be calculated as following formula:

$L = 20 + 0.005\ Q$ Q: discharge (m^3/sec.)

Discharge of Kelani River of 100 years return period is 3,000 m^3/sec.

L = 20 + 0.005 * 3,000 = 35.0m

Thus, the Standard Span Length (minimum span length) for Kelani River Crossing Bridge shall be 35m.

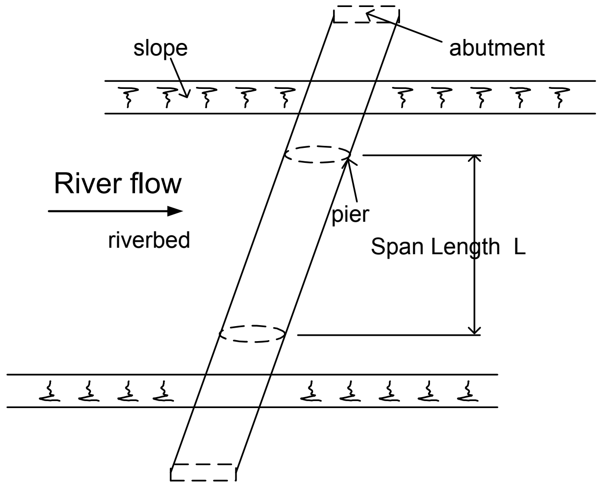


Fig. 3-7 Measurement of Minimum Span Length

4. DESIGN LOAD

4. Design Load

Design loads shall be defined by both "Bridge Design Manual (RDA, 1997)" and "BS 5400 Part-2 (1978)".

Permanent Loads

1. Dead Load / Superimposed Dead Load
2. Earth Pressure
3. Shrinkage and Creep
4. Differential Settlement
5. Water Current
6. Buoyancy

Transient Loads

7. Main Live Load
8. Sub Live Load (Centrifugal Load, Longitudinal Load, Skidding, Vehicle Collision)
9. Footway and Cycle Track Live Load
10. Wind Load
11. Temperature
12. Erection Load
13. Floating Debris and Log Impact

Notes: 1.Earthquake Effect is not considered in Sri Lanka
2.Combination of loads are made by BS 5400 Part-2

4.1. Dead Load / Superimposed Dead Load

Table 4-1 Unit Weight of Materials

Category	Item	Unit	Value	Remarks
Dead Load	Reinforced Concrete	kN/m^3	25.0	
	Pre-stressed Concrete	kN/m^3	25.0	
	Plane Concrete	kN/m^3	23.5	
	Asphalt Pavement	kN/m^3	23.0	
	Steel	kN/m^3	78.5	
	Compact Sand	kN/m^3	19.0	
	Loose Sand	kN/m^3	16.0	
Superimposed Dead Load	Pavement	mm	50	$1.15 kN/m^2$
	Bridge Parapet	kN/m	10.12	Shown in Fig.4.1
	Handrail	kN/m	1.00	
	Public Utilities	kN/m	None	
	Others	kN/m	None	

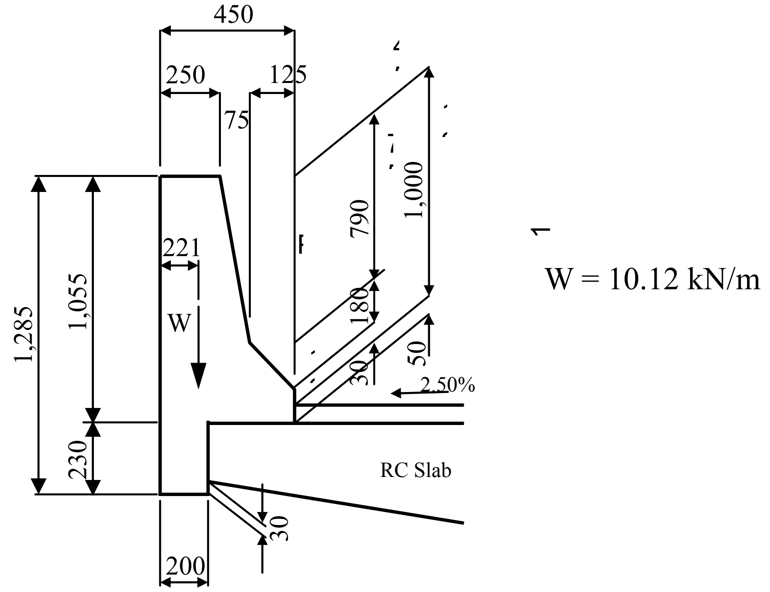


Fig. 4-1 Bridge Parapet

4.2. Earth Pressure

The earth pressure acting on the abutment shall be considered by only active earth pressure, not include the resistance by passive earth pressure.

Coefficient of earth pressure Ka is calculated as follows:

$Ka = (1 - \sin\theta) / (1 + \sin\theta)$

Where, θ: friction angle of back fill soil = 30 degree

Thus, Ka = 0.333

The effect of live load surcharge shall be considered below:

HA Live Load	10.0 kN/m^2
HB Live Load (30 units)	12.5 kN/m^2

4.3. Creep and Shrinkage

Effects of creep and shrinkage depend on the behavior of the concrete.
Those effects shall be conformed by BS 5400: Part 4 Appendix C or BS 8110: Part 2.

Effect of Creep

Creep strain in concrete Δcc is calculated as follows.

$\Delta cc = (fc / E\,c) * \varphi$

Where, fc: stress due to permanent force
E: modulus of elasticity of concrete
φ: creep coefficient of concrete

Effect of Shrinkage

Coefficient of shrinkage for concrete will be taken as 0.0002

4.4. Differential Settlement

Differential settlement shall be disregarded for structural design because the pile end reaches into the stiff stratum or spread foundation embedded into hard rock. But special considerations are needed at the connection between the structures and embankments.

4.5. Water Current

Horizontal force due to water current shall be calculated the following formula by Bridge Design Manual, RDA:

$$P = K * W * V^2 / (2g)$$
$$= 52 * K * V^2$$

Where, P : intensity of pressure due to the water current (N/m^2)
W : unit weight of water (N/m^3)
V : velocity of current at the point where the pressure intensity is being calculated (m/sec)
g : acceleration of gravity (m/sec^2)
K : a constant depending on the shape of pier as follows:

Type of Pier	K
Square ended pier	1.50
Circular piers or semi circular cutwaters	0.66
Trianglular cutwaters	0.50 to 0.90
Trestle type piers	1.25

4.6. Buoyancy

The effect of buoyancy in design, the deepest water level at the area shall be considered.

4.7. Live Load

The following loads given in BS5400 Part 2 are used for the bridge design based on Bridge Design Manual, RDA.

- All bridges should be designed to resist the effect of HA loading specified in the relevant code. HA loading is a formula loading – normal traffic, and including impact.

- Bridges should be able to resist the effect of 30 units of HB loading for A & B class of roads. And always the HB vehicle is to straddle two notional lane widths. HB loading is an abnormal vehicle unit loading including impact.

4.7.1. HA Load

Three kinds of loads are considered in design for HA load.

- Uniformly distributed load (UDL) : Intensity vary by loading length

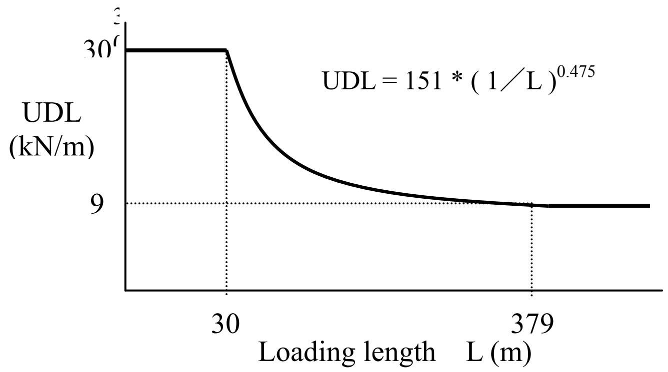


Fig. 4-2 Loading Curve for HA UDL

- Knife edge load (KEL) : 120 kN / lane
- Single wheel load : 100 kN at the most severe position

For Single wheel load, uniformly distributed over a square contact area of 300 mm sides and the effective pressure of 1.1 N/mm2 shall be considered. And the dispersal at a spread-to-depth ratio of 1 horizontally to 2 vertically for asphalt and similar surfacing, 1horizontally to 1 vertically for structural concrete slab will be taken.

4.7.2. HB Load

30 units of HB loads should be applied in design. **Fig. 4-3** shows the plan and axle arrangement for one unit of nominal HB loading.

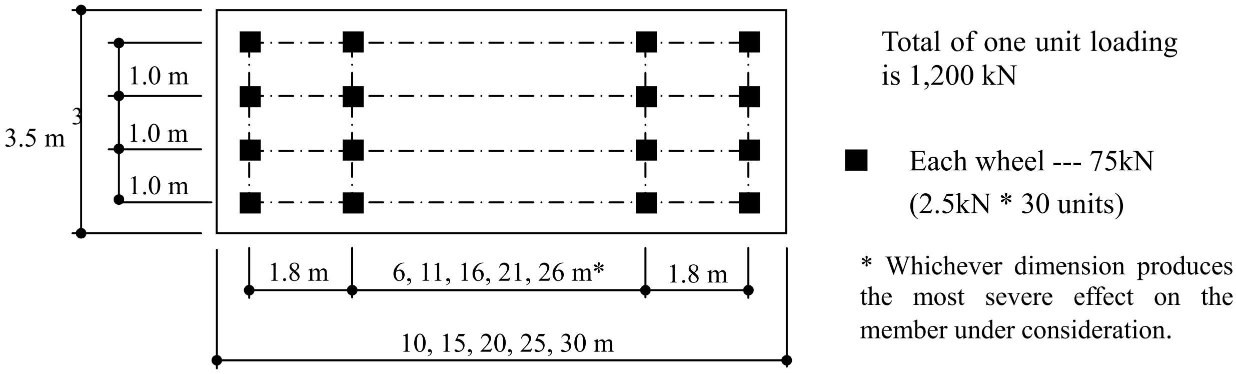


Fig. 4-3 Dimension of HB Vehicle

The overall length of one unit shall be taken as 10, 15, 20, 25, 30 m for inner axle spacings of 6, 11, 16, 21, 26 m respectively, and the effect of the most severe case shall be adopted.

For HB loading, uniformly distributed over a square contact area of 260 mm sides and the effective pressure of 1.1 N/mm2 shall be considered. And the dispersal at a spread-to-depth ratio of 1 horizontally to 2 vertically for asphalt and similar surfacing, 1horizontally to 1 vertically for structural concrete slab will be taken.

4.7.3. Notional lanes

Carriageway width is the length between raised curbs and notional lane. The number of notional lanes shall be given the following **Table 4-2**.

Table 4-2 Notional Lane

Carriageway Width (m)	Number of Notional Lanes
4.6 up to and including 7.6	2
above 7.6 up to and including 11.4	3
above 11.4 up to and including 15.2	4
above 15.2 up to and including 19.0	5
above 19.0 up to and including 22.8	6

For this Project, the following carriageway widths, 13.5m for major bridges and 14.75m for minor bridges are used.

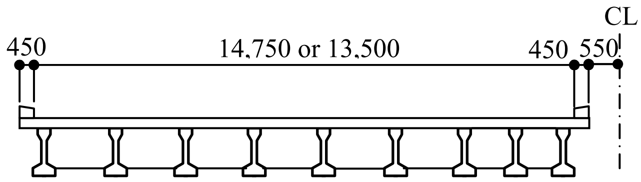


Fig. 4-4 Carriageway Widths

The notional widths are calculated below;

- Major bridge (Kelani river crossing over bridge)

  w = 13.5 m / 4 notional lanes = 3.375 m

- Minor bridge (A1 IC over bridge, Mudun Ela crossing over bridge)

  w = 14.75 m / 4 notional lanes = 3.688 m

4.7.4. Application of Type HA & HB Loading

Type HA loading

Type HA UDL and KEL loads shall be applied to two notional lanes in the appropriate parts of the influence line for the element or member under consideration and one-third type HA UDL and KEL loads shall be similarly applied to all other notional lanes. The KEL shall be applied at one point only in the loaded length of each notional lane.

Fig. 4-5 shows the application of HA loading:

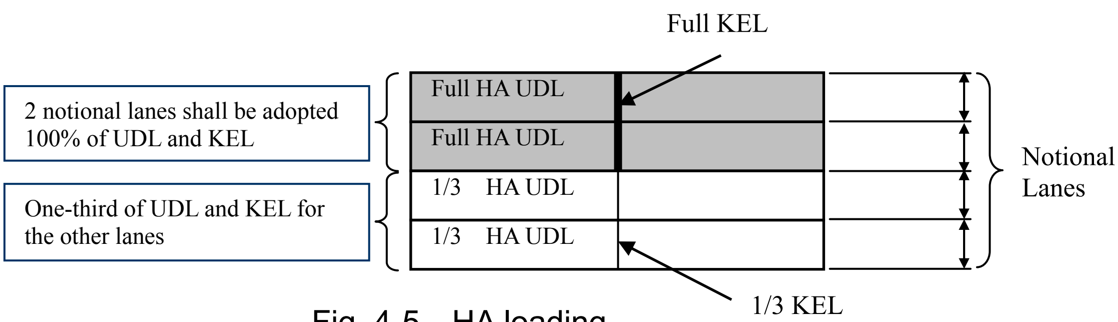


Fig. 4-5 HA loading

Type HB and HA loading combined

HB vehicle shall be positioned to straddle two notional lanes and no other primary live loading shall be considered for 25 m in front of, to 25 m behind. And HA loading shall be associated.
Fig. 4-6 illustrates type HB loading in combination with type HA loading for this Project.

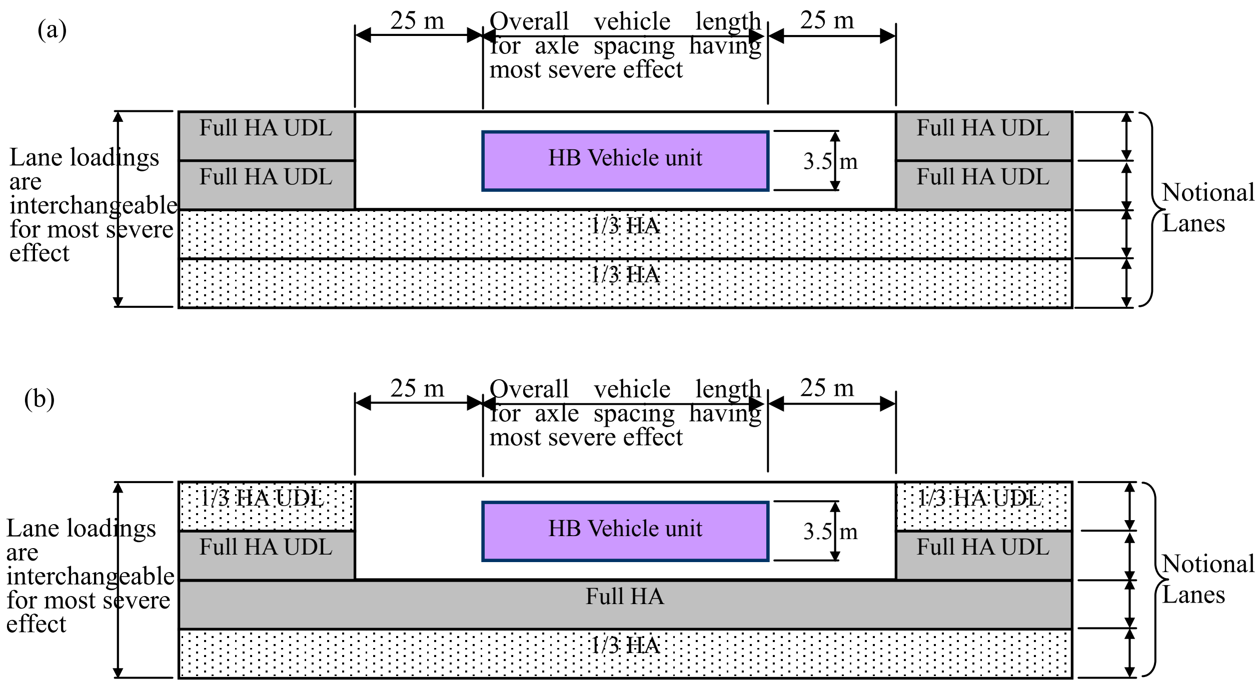


Fig. 4-6 Type HA and HB Loading Combination

The bridges in OCH Project are categorized in the road classification from "Geometric Design Standards of Roads" and Design Live Loads to be used are different for the classification.

Table 4-3 Live Load to be used for Each Road Classification

	Road Classification	Design Live Load
Highway Bridges	A – Class Road	HA and HB Live Load
Overpass Bridges	A, B – Class Road	HA and HB Live Load
	C, D, E – Class Road	HA Live Load

4.8. Sub Live Load

4.8.1. Centrifugal Load

The nominal centrifugal load Fc and associated vertical load Vc shall be taken for curved bridge and structures.

$Fc = 30{,}000 / (r + 150)$ kN
$Vc = 300$ kN
Where, r : the radius of curvature of the lane (m)

4.8.2. Longitudinal Load

The longitudinal force resulting from traction or braking shall be taken.

HAPa = 200 + 8 * L ($\leq$ 700) kN
HBPb = 25% of HB vertical load kN
Where, L : the loading length (m)

4.8.3. Skidding Load

Horizontal load of 250 kN due to skidding shall be taken in design with HA load.

4.8.4. Vehicle Collision Loads

1) Collision with parapets

Four wheels of 25 units of HB loading (250 kN = (2.5 kN x 25 units x 4 wheels)) shall be considered in any position.

2) Collision with bridge supports

For the vehicle collision load on supports shall be as follows:

Table 4-4 Collision Loads on Supports of Bridges over Highways

	Load nominal to the carriageway below	Load parallel to the carriageway below	Point of application on bridge support
Load transmitted from guardrail	150 kN	50 kN	Any one bracket attachment point, or for free standing fences, any one point 0.75 m above carriageway level
Residual load above guardrail	100 kN	100 kN	At the most severe point between 1 m and 3 m above carriageway level

4.9. Footway and Cycle Track Live Load

It varies according to loaded length and intensity of UDL in HA load.

$L \leq 30$ m $\quad q = 5.0$ kN/m^2
$L > 30$ m $\quad q = 5.0 * (UDL / 30)$ kN/m^2

If bridge supports footway, the live load intensity shall be 80% of the load above.

4.10. Wind Load

Wind load P shall be given the following formula according to BS 5400 Part-2:

$P = q * A * C_D$ (N)

Where; q : dynamic pressure head (N/m2) = $0.613 * v_c^2 m$
v_c : maximum wind gust speed (m/s)
A : solid area (m^2)
C_D : drag coefficient

4.10.1. Maximum Wind Gust Speed v_c

The maximum wind gust speed shall be taken as:

$v_c = v * K1 * S1 * S2$

where; v : mean hourly wind speed (m/s) = 38.0 m/s ~ "Zone 3", see **Table 4-6**
K1 : wind coefficient related to the return period = 1.00 ~ "120 years"
S1 : funneling factor = 1.0 ~ "General", see Table 4.7
S2 : gust factor; i.e.: maximum height is 15 m and 40 m length ~ "1.59"

Thus, maximum wind gust speed for this Project is;

$v_c = 38.0 * 1.00 * 1.00 * 1.59 = 60.4$ m/s

Table 4-5 Mean Hourly Wind Speed

Zone	Mean Hourly Wind Speed
1	53.5 m/s
2	47.0
3	38.0

Table 4-6 Wind Coefficient K1

Coefficient K1	Return Period	Application	Remarks
1.00	120 years	Highway, Railway, Foot/Cycle Track Bridges	
0.94	50	Foot/Cycle Track Bridges	Subject to RDA agreement
0.85	10	During Erection	As the corresponding return period

Table 4-7 Funneling Factor S1

Description	Factor S1
General	1.0
Valleys where local funneling of the wind occurs, or where a bridge is sited to the lee of a range of hills causing local acceleration of wind	greater than 1.1

Table 4-8 Funneling Factor S2

Height above Ground Level	Horizontal Wind Loaded Length (m)								
	20 or less	40	60	100	200	400	600	1,000	2,000
5	1.47	1.43	1.40	1.35	1.27	1.19	1.15	1.10	1.06
10	1.56	1.53	1.49	1.45	1.37	1.29	1.25	1.21	1.16
15	1.62	1.59	1.56	1.51	1.43	1.35	1.31	1.27	1.23
20	1.66	1.63	1.60	1.56	1.48	1.40	1.36	1.32	1.28
30	1.73	1.70	1.67	1.63	1.56	1.48	1.44	1.40	1.35
40	1.77	1.74	1.72	1.68	1.61	1.54	1.50	1.46	1.41
50	1.81	1.78	1.76	1.72	1.66	1.59	1.55	1.51	1.46
60	1.84	1.81	1.79	1.76	1.69	1.62	1.58	1.54	1.50
80	1.88	1.86	1.84	1.81	1.74	1.68	1.64	1.60	1.56
100	1.92	1.90	1.88	1.84	1.78	1.72	1.68	1.65	1.60
150	1.99	1.97	1.95	1.92	1.86	1.80	1.77	1.74	1.70
200	2.04	2.02	2.01	1.98	1.92	1.87	1.84	1.80	1.77

4.10.2. Nominal Wind Load

The nominal wind load Pt or P_L (N) shall be taken as acting at the centroids of the appropriate areas and horizontally unless local conditions change the direction of the wind, and shall be derived from:

$$\text{Pt or } P_L = q * A * C_D = 0.613 * v_c^2 * A * C_D$$

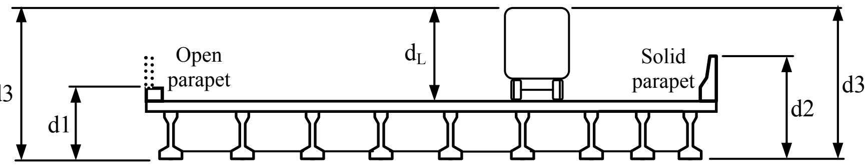
Where; v_c : maximum wind gust speed (N/s)

A : solid area (m2) ~ see **Table 4-9** for superstructure

* parapet and safety fence, piers shall be derived for the solid area in normal projected elevation

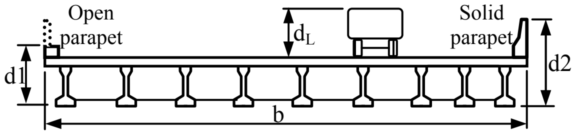
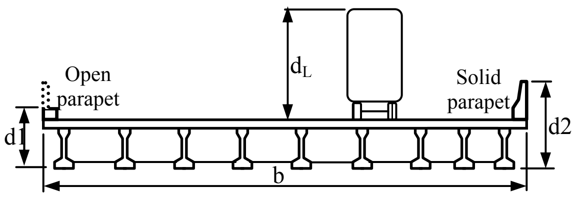
C_D : drag coefficient, ratio b/d ~ see **Table 4-10**

Table 4-9 Depth d to be used in Deriving Area A



Parapet	Unloaded Bridge	Live Loaded Bridge
Open	d = d1	d = d3
Solid	d = d2	d = d2 or d3 whichever is greater
dL = 2.5 m above the highway carriageway, or 3.7 m above the rail level, or 1.25 m above footway or cycle track		

Table 4-10 Depth d to be used in Deriving C_D

	Parapet	Superstructures without live load	Superstructures with live load
(a) Superstructures where the depth of the superstructure (d1 or d2) exceeds d_L 	Open	d = d1	d = d1
	Solid	d = d2	d = d2
(b) Superstructures where the depth of the superstructure (d1 or d2) is less than d_L 	Open	d = d1	d = d_L
	Solid	d = d2	d = d_L

* Drag coefficient $\mathbf{C_D}$ for parapet and pier shall be conformed to Table 8 and 9, BS 5400 Part 2.

4.10.3. Nominal Vertical Wind Load

An upward or downward nominal vertical wind load Pv (N), acting at the centroids of the appropriate areas, for all superstructures shall be derived the following formula;

Pt or $P_L = q * A * C_D = 0.613 * v_c^2 * A * C_D$

Where; v_c : maximum wind gust speed (N/s)

A : area in plan (m2)

C_D : lift coefficient as derived from Figure 4.7 based on superelevation of superstructure

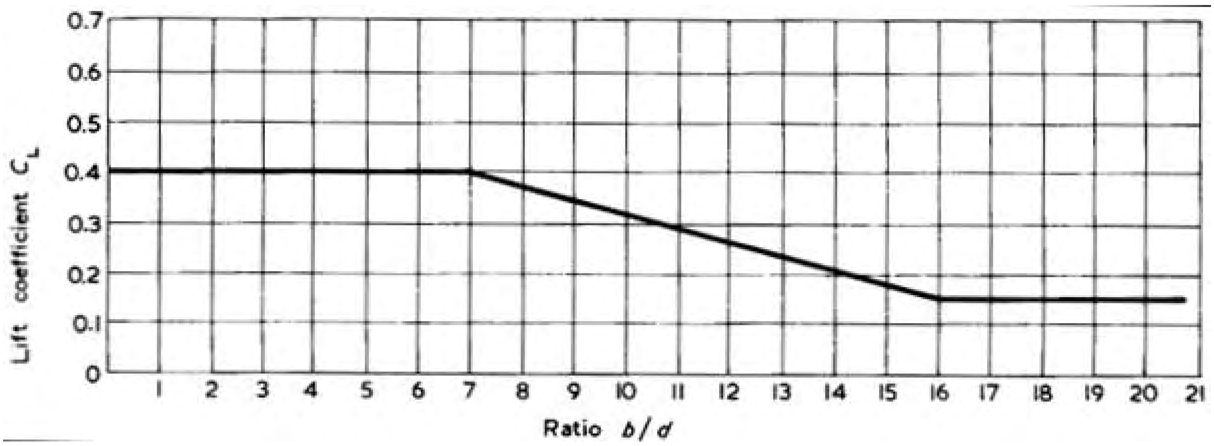


Fig. 4-7 Lift Coefficient

4.10.4. Load Combination

The wind loads Pt, P_L and Pv shall be considered the combination of following 4 cases;

a) Pt alone
b) Pt in combination with +/- Pv
c) P_L alone
d) 0.5*Pt in combination with P_L +/- 0.5*Pv

4.11. Temperature

1) Effective Bridge Temperature

Effective Bridge Temperature shall be considered by RDA bridge design manual in continuous bridges for calculation of temperature stress.

Based on the data issued by Department of Meteorology on January, 2007, the maximum and minimum shade air temperature for 30 year averages in Colombo are:
- Maximum temperature 31.8 degree (^oC)
- Minimum temperature 22.3 degree (^oC)

As required maximum/minimum shade air temperatures for the bridge design, a 120 years return period shall be considered, and the following values are applied.

- Maximum shade air temperature 35 degree (^oC)
- Minimum shade air temperature 20 degree (^oC)
- Mean temperature of 27.5 degree (^oC) is applied, and plus/minus (+/-) 7.5 degrees temperature change is considered.

2) Frictional Bearing Resistance Force by Temperature Change

As the consideration of horizontal force due to temperature change in OCH Project, 15% of dead load reaction from superstructure will be adopted for the substructure design (longitudinal direction only).

The “15%” is the minimum friction coefficient of the elastomeric bearings (by Japanese design standard).

4.12. Erection Load

Erection load shall be considered in the design depending on the bridge type and the erection method (i.e. cantilevered erection method of box girder).

4.13. Floating Debris and log Impact

1) Floating Debris

Where debris is likely, allowance shall be made for the force exerted by a minimum depth of 1.2 m debris. The length of the debris applied to any one pier shall be one half of the sum of the adjacent spans with maximum 22.0 m where the deck is not submerged.

For debris the formula for water current shall be used the value of the constant K = 1.0.

2) Log Impact

Impact force shall be calculated by RDA manual

$$P = 0.1 * W * V$$

Where, P : collision force (kN)
W : weight of drifting item (kN) ~ assumed 20 kN
V : surface velocity of the water (m/s)

4.14. Earthquake

As this is not relevant to Sri Lanka, no effect is considered.

4.15. Combination of Loads

Three principal combinations (Combination-1~3) and two secondary combinations (Combination-4 & 5) of loads are specified in **Table 4-13** based on BS 5400 Part 2. The five combinations of loads are to be considered in the design.

Table 4-11 Loads to be taken in each combination with appropriate γ fL

Clause No.	Load		Limit State	γ fL to be considered in combination 1	2	3	4	5
5.1	**Dead:**	Steel	ULS [a]	1.05	1.05	1.05	1.05	1.05
			SLS	1.00	1.00	1.00	1.00	1.00
		Concrete	ULS [a]	1.15	1.15	1.15	1.15	1.15
			SLS	1.00	1.00	1.00	1.00	1.00
5.2	**Superimposed dead**		ULS [b]	1.75	1.75	1.75	1.75	1.75
			SLS [b]	1.20	1.20	1.20	1.20	1.20
5.1.2.2 **5.2.2.2**	Reduced load factor for dead and superimposed dead load where this has a more severe total effect		ULS	1.00	1.00	1.00	1.00	1.00
5.3	**Wind:**	During erection	ULS		1.10			
			SLS		1.00			
		With dead plus superimposed dead load only, and for members primarily resisting wind loads	ULS		1.40			
			SLS		1.00			
		With dead plus superimposed dead plus other appropriate combination 2 loads	ULS		1.10			
			SLS		1.00			
		Relieving effect of wind	ULS		1.00			
			SLS		1.00			
5.4	**Temperature:**	Restraint to movement, except frictional	ULS			1.30		
			SLS			1.00		
		Frictional bearing restraint	ULS					1.30
			SLS					1.00
		Effect of temperature difference	ULS			1.00		
			SLS			0.80		
5.6	**Differential settlement**		ULS	To be assessed and agreed between the engineer and the appropriate authority				
5.7	**Exceptional loads**		SLS					
5.8	**Earth Pressure:**	Retained full and/or live load surcharge	ULS	1.50	1.50	1.50	1.50	1.50
			SLS	1.00	1.00	1.00	1.00	1.00
		Relieving effect	ULS	1.00	1.00	1.00	1.00	1.00
5.9	**Erection:**	Temporary loads	ULS		1.15	1.15		
6.2	**Highway bridges live loading:**	HA alone	ULS	1.50	1.25	1.25		
			SLS	1.20	1.00	1.00		
6.3		HA with HB or HB alone	ULS	1.30	1.10	1.10		
			SLS	1.10	1.00	1.00		
6.5	**Centrifugal load and associated primary live load**		ULS	Each secondary live load shall be considered separately together with the other combination 4 loads as appropriate			1.50	
			SLS				1.00	
6.6	**Longitudinal load:**	HA and associated primary live load	ULS				1.25	
			SLS				1.00	
		HB and associated primary live load	ULS				1.10	
			SLS				1.00	
6.7	**Accident skidding load and associated primary live load**		ULS				1.25	
			SLS				1.00	
6.8	**Vehicle collision load with bridge parapets and associated primary live load**		ULS				1.25	
			SLS				1.00	
6.9	**Vehicle collision load with bridge supports** [c]		ULS				1.25	
			SLS				1.00	
7	**Foot/cycle track bridges:** Live load and parapet load		ULS	1.50	1.25	1.25	1.25	
			SLS	1.00	1.00	1.00	1.00	
8	**Railway bridges:**	Type RU and RL primary and secondary live loading	ULS	1.40	1.20	1.20		
			SLS	1.10	1.00	1.00		

Notes:
- For loads arising from creep and shrinkage, or from welding and lack of fit, see BS5400 Parts 3, 4 and 5, as appropriate.
- ULS: ultimate limit state, SLS: serviceability limit state

a γfl shall be increased to at least 1.10 and 1.20 for steel and concrete respectively to compensate for inaccuracies when dead loads are not accurately assessed.

b γfl may be reduced to 1.2 and 1.0 for the ULS and SLS respectively subject to approval of the appropriate authority (see 5.2.2.1).

c This is the only secondary live load to be considered for foot cycle track load bridges.

5. PROPERTIES OF MATERIALS

5. Properties of Materials

5.1. Concrete

Concrete for bridge and the relative structures shall be used depend on the classifications, shown in **Table 5-1** below;

Table 5-1 Concrete Strength (Cube Strength) & Elastic Modulus

Classification	Strength (N/mm^2)		Elastic Modulus (kN/mm^2)	
	At transfer	Characteristic Strength	At transfer	Characteristic Strength
PC Girder	36	50	29.8	34.0
Crossbeam	24	35	25.8	29.5
RC Slab, RC Panel	---	35	---	29.5
Abutment, Pier, Bored Pile, Approach Slab	---	30	---	28.0
Environmentally, the structures belonged in “Very severe” or “Extreme” (see Note)	28	40	27.2	31.0

Poisson’s Ratio: 0.20

Temperature Coefficient: $12 * 10^{-6}$

Stress-Strain Curve for Design: BS 5400 Part 4: Figure 1

Note: When the environmental category is ranked in “Extreme” or “Very severe”, minimum concrete strength of 40 N/mm2 shall be used for Crossbeam, RC Slab, RC Panel, Abutment, Pier, Bored Pile and Approach Slab.

5.2. Steel

Reinforcing steel bars shall be used Grade 460 with yield strength of 460 N/mm2, and classification of Type-2 for deformed bars. The elastic modulus is 200 N/mm2.
Applicable diameter and the area, perimeter, mass of bars are shown in **Table 5-2**.

Table 5-2 Deformed Reinforcing Steel Bars (Type-2)

Nominal Size (mm)	Cross Sectional Area (mm2)	Effective Perimeter (mm)	Mass (kg/m)
8	50.3	25.1	0.395
10	78.5	31.4	0.616
12	113.1	37.7	0.888
16	201.1	50.2	1.579
20	314.2	62.8	2.466
25	490.9	78.5	3.854
32	804.2	100.5	6.313
40	1,256.6	125.6	9.864

Notes:
1. Preferred sizes only are shown
2. Effective Perimeter is; (3.14 * nominal size)
3. Reinforcing steel bars shall be conformed BS 4449
4. The maximum length is 12 m

Prestressing tendons to be used for the Project shall be conformed to BS 4486 and 5896, and the properties are as follows:

Table 5-3 Prestressing Tendons

Classification	Nominal Tensile Strength (N/mm2)	Yield Strength (N/mm2)	Elastic Modulus (N/mm2)
12S12.7 9S12.7	1,860	1,670	200
1S21.8	1,860	1,670	200

Notes:
1. 12.7 mm formed tendon is using 7-wire, and 21.8 mm is 19-wire
2. Low relaxation – Relaxation Class 2 shall be used

Both of reinforcing steel bars and prestressing tendons shall be taken;

Poisson's Ratio: 0.30, Temperature Coefficient: $12 * 10^{-6}$.

And Stress-Strain Curves for Design are referred to: BS 5400 Part 4: Figure 2 for reinforcing bars and Figure.3 for prestressing tendons.

5.3. Backfill Material

As the backfill material shall be used "Granular Soil Material", and the required properties are;

Internal friction of angle	$\phi = 30$ degree
Unit weight	$\gamma = 19$ kN/m^3
Cohesion	c = 0 kN/m^2

6. DETAILS

6. Details

6.1. Embedment Depth of Pile-cap below Planned Ground Level and Riverbed

6.1.1. Pile-cap constructed on the Ground

Pile-cap shall be embedded 0.50 m depth and more from the Planned Ground Level to the top of pile-cap, and 0.30 m depth and more from the Existing Ground Level to the bottom of pile-cap for design of this Project.

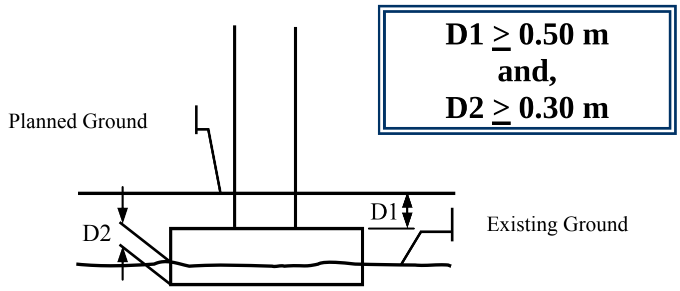


Fig. 6-1 Embedment Depth of Pile-cap on the Ground

6.1.2. In the River – current river area

Top of the pile-cap shall be embedded at least 200cm in consideration of the scouring effect.

Major river - Kelani River in this study section was computed the scouring depth of 550cm by the hydraulic analysis. The effective area by the scouring is protected

Therefore, 200 cm depth or deeper from the riverbed in the current river area (river water flowing area at current time) shall be taken for design of this Project.

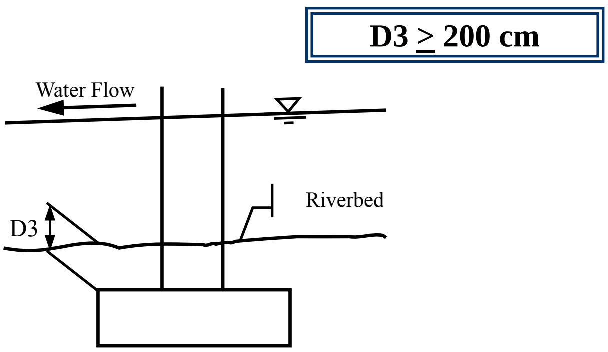


Fig. 6-2 Embedment Depth of Pile-cap in the River

6.2. Pile Arrangement

Distance between the edge of pile-cap and the edge of the nearest pile shall be at least 150 mm. And the distance between two adjacent center of piles shall be at least 2.50D for End Bearing Pile, and 3.00D for Friction Pile.

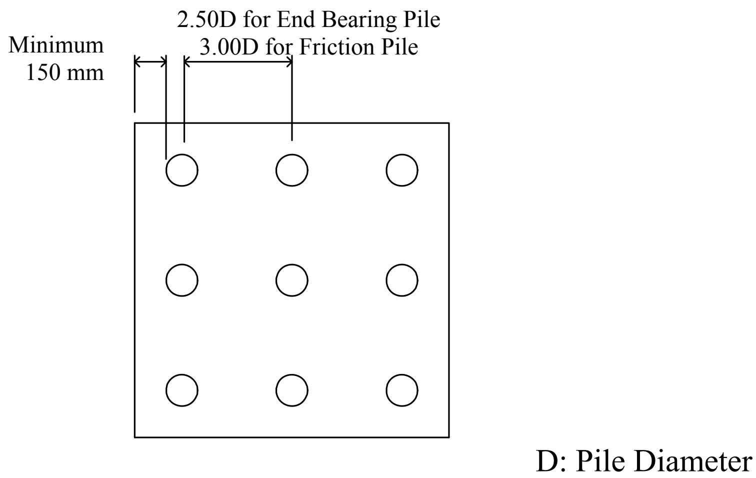


Fig. 6-3 Pile Arrangement

6.3. Reinforcing Bar Arrangement

6.3.1. Minimum Required Concrete Clear Cover

Based on BS 5400-4, the nominal cover to reinforcement under particular condition of exposure is prescribed (**Table 6-1**).

Depending on the member and its situation, and RDA practice, the minimum clear cover to reinforcement shall be conformed **Table 6-2**.

6.3.2. Space between Adjacent Reinforcing Bars

According to the code requirements, minimum and maximum spaces are maximum aggregate size plus 5 mm and 300 mm respectively. Actual practice in Sri Lanka, the minimum spacing (bar center-to-center) is 100 mm.

Table 6-1 Nominal Cover to Reinforcement under particular condition of exposure

Environment	Example	Nominal Cover (mm) a Concrete Grade (N/mm2) 25	30	40	55
Extreme Concrete surface exposed to: Abrasive action by sea water or Water with a pH < 4.5	Marine structures Parts of structure in contact with moorland water	b	b	c 65	55
Very severe Concrete surfaces directly affected by: de-icing salts or Sea water spray	Walls and structure supports adjacent to the carriageway Parapet edge beams Concrete sdjacent to the sea	b	d	c 50	40
Severe Concrete surfaces exposed to: Driving rain or Alternate wetting and drying	Walls and structure supports remote from the carriageway Bridge deck soffits Buried parts of structures	b	c 45	35	30
Moderate Concrete surfaces above ground level and fully sheltered against all of the following: Rain, De-icing, Sea water spray Concrete surfaces permanently saturated by water with a pH > 4.5	Surface protected by bridge deck water proofing or by permanent formwork Interior surface of pedestrian subways, voided superstructures or cellular abutments Concrete permanently under water	45	35	30	25

a Actual cover may be up to 5 mm less than nominal cover
b Concrete grade not permitted
c Air entrained concrete should be specified where the surface is liable to freezing whilst wet
d For parapet beams only grade 30 concrete is permitted provided it is air entrained and the nominal cover is 60 mm

6.3.3. Minimum Area of Main Reinforcement in Members

Minimum and maximum area of main reinforcement (G460) depending on member shall be conformed the following Table:

Table 6-2 Minimum Clear Cover to Reinforcement of members

Member	Clear Cover (mm)			
	Concrete Grade	Environment Severe	Concrete Grade	Environment Very Severe / Extreme
PC Box Girder, PC I-Girder	50	30	50	Very Severe: 40 Extreme: 55
RC Slab, Cross Beam, Steps	35	40	40	40
Parapet	30	50	40	50
Abutment Walls , Piers	30	50	40	50
Pile-Cap, Footing (except Bottom)	30	50	40	50
Pile Cap, Footing (Bottom)	30	75	40	75
Cast-in-situ RC Piles	30	75	40	75

Table 6-3 Minimum and Maximum Reinforcement (G460)

Member	**Minimum Ratio**	**Maximum Ratio**
Slab, Girder, Cross Beam	0.15 % of ba*d	4 % of gross sectional area of concrete
Column	1.0 % of cross sectional area or 0.15*N/fy, whichever is the lesser * minimum number shall be 4 for rectangular, 6 for circular section **Note: Based on RDA practice, at least the ratio of 1.0 % of cross sectional area shall be required.**	6 % in vertically cast columns 8 % in horizontally cast columns 10 % at laps in both above types
Wall	0.40 % of gross sectional area of concrete	4 % of gross sectional area of concrete

Where,
- ba : breadth of section, or average breadth excluding the compression flange for non-rectangular sections
- d : effective depth to tension reinforcement
- N : ultimate axial load
- Fy : characteristic strength of reinforcement

6.3.4. Lap Length/Anchorage Length and the other details

Lap length/anchorage length and the other details are described in "Road and Drainage Structures", and shall be conformed.

7. DESIGN REQUIREMENTS

7. Design Requirements

7.1. Design Class for Pre-stressed Concrete Structure

Pre-stressed concrete is classified by flexural tensile limitations, the categories are as follows;

Class 1:no tensile stress permitted;

Class 2:tensile stress permitted, in accordance with **Table 7-1**, but no visible cracking;

Class 3:tensile stress permitted, in accordance with **Table 7-2**, but with design crack widths limited to the values of **Table 7-5**.

Category Class 2 is applied for design in the Project.

Table 7-1 Flexural Tensile Stresses for Class 2 Members
(Serviceability Limit State: Cracking)

	Allowable Stress for Concrete Grade (N/mm2)			
	30	40	50	60
Pre-tensioned members	---	2.9	3.2	3.5
Post-tensioned members	2.1	2.3	2.55	2.8

Table 7-2 Hypothetical Flexural Tensile Stresses for Class 3 Members

	Limiting Crack Width (mm)	Stress for Concrete Grade (N/mm2)		
		30	40	50 and over
a) Pre-tensioned tendons	0.1 0.15 0.25	--- --- ---	4.1 4.5 5.5	4.8 5.3 6.3
b) Grouted post-tensioned tendons	0.1 0.15 0.25	--- 3.5 4.1	4.1 4.5 5.5	4.8 5.3 6.3
c) Pre-tensioned tendons distributed in the tensile zone and positioned close to the tension faces of the concrete	0.1 0.15 0.25	--- --- ---	5.3 5.8 6.8	6.3 6.8 7.8

7.2. Design Method

Bridges shall be designed by checking the formula below for two limit states, namely "Ultimate Limit State (ULS)" and "Serviceability Limit State (SLS)".

$R^* \geq S^*$

R* = f unction (fk / γm) ; Design Resistance
S* = γf3 * effect (γfL * Qk) ; Design Load Effect

Fk : nominal strength of the material (by available material)
Qk : nominal load (by BS5400 Part-2)
γ m : partial safety factor for strength (by BS5400 Part-4)
γ f3 : partial inaccurate factor (by BS5400 Part-4)
γ fL : partial load factor (by BS5400 Part-2)

7.3. Partial Inaccurate Factor γ f3

The partial inaccurate factor shall be used the following values:

γ f3 = 1.00 (SLS)
1.10 (ULS) --- except that where plastic methods are used for the analysis of the structure, γ f3 should be taken as 1.15

7.4. Partial Safety Factor for Strength γ m

The values of γm for both serviceability and ultimate limit state (SLS & ULS) is shown in **Table 7-3** below:

Table 7-3 Partial Safety Factor for Strength

Material	Type of stress	Serviceability Limit		Ultimate Limit
		RC	PC	RC, PC
Concrete	Triangular or near-triangular compressive stress distribution	1.00	1.25	1.50
	Uniform or near-uniform compressive stress distribution	1.33	1.67	1.50
	Tension	Not applicable	1.25 (pre-tensioned) 1.55 (post-tensioned)	1.50
Reinforcement	Compression Tension	1.00	Not applicable	1.15
Pre-stressing Tendons	Tension	Not applicable	Not required	1.15

7.5. Stress Limitation for the serviceability limit state (SLS)

Both reinforced and prestressed concrete, the compressive and tensile stress limitations are summarized in **Table 7-4** below:

Table 7-4 Stress Limitation

Material	Type of stress	Serviceability Limit		At transfer
		RC	PC	PC
Concrete	Triangular or near-triangular compressive stress distribution	0.50 fcu	0.40 fcu	0.50 fci but ≤ 0.40 fcu
	Uniform or near-uniform compressive stress distribution	0.38 fcu	0.30 fcu	0.40 fci but ≤ 0.30 fcu
	Tension	-	0.45 fcu ^$^{0.5}$ (pre-tensioned) 0.36 fcu ^$^{0.5}$ (post-tensioned)	1.0
Reinforce-Ment	Compression Tension	0.75 fy	Not applicable	-
PC Strand	Tension	Not applicable	Not required	0.75 fpy (pre-tensioned) 0.70 fpy (post-tensioned)

Note. fcu : characteristic strength of concrete at serviceability limit state (N/mm^2)
fci : characteristic strength of concrete at transfer (N/mm^2)
fy : characteristic strength of reinforcement (N/mm^2)
fpy : characteristic strength of PC strand (N/mm^2)

7.6. Crack Width

Crack width must be checked for serviceability limit states. OCH Project is located in inland area so that most of the structures shall be conformed the design crack width categorized in environment “Severe”. However, Kelani River Bridge affected by seawater due to backward flow and the bridges located in affective area by acid soils (i.e. peaty soil), shall be considered environment categories “Severe” and/or “Extreme”.

Table 7-5 Design Crack Widths

Environment	Examples	Design Crack Width (mm)
Extreme Concrete surfaces exposed to: abrasive action by sea water or water with a pH ≤ 4.5	 Marine structures Parts of structure in contact with moorland water	0.10
Very severe Concrete surfaces directly affected by: de-icing salts or sea water spray	 Walls and structure supports adjacent to the carriageway Parapet edge beams Concrete adjacent to the sea	0.15
Severe Concrete surfaces exposed to: driving rain or alternate wetting and drying	 Walls and structure supports remote to the carriageway Bridge deck soffits Buried parts of structures	0.25
Moderate Concrete surfaces above ground level and fully sheltered against all of the following: rain, de-icing salts, sea water spray Concrete surfaces permanently saturated by water with a pH > 4.5	 Surface protected by bridge deck water-proofing or by permanent formwork Interior surface of pedestrian subways, voided superstructures or cellular abutments. Concrete permanently under water	0.25

7.7. Load Distribution for PC-I Girder

Design of PC-I girder requires consideration of load distribution by cross beams. Design method analyzed load distribution, such as Influence Line Modeing Method, Guyon-Massonnet Theory or Grid Modeling Analysis by computer program shall be applied to calculate load distribution.

7.8. Calculation of Pre-stressing Force

7.8.1. Loss of Pre-stress due to Friction

$$P x = P i * e^{-(\mu \alpha + \lambda x)}$$

Where,	P x	:	Tension force of PC Strand at Design Section
	P I	:	Jacking Force
	μ	:	Friction Coefficient per 1 Radian Angle Change
	α	:	Angle Change (radian)
	λ	:	Friction Coefficient per 1 m long
	x	:	Distance between Jacking Point to Design Section (m)

Table 7-6 Friction Coefficient

	λ	μ
PC Wire	0.004	0.30
PC Strand	0.004	0.30
PC Bar	0.003	0.30

7.8.2. Loss of Prestress due to Slip

Loss of prestress due to slip ΔP is calculated as follows.

i) In case No Friction between Sheath and Strand exists

$$\Delta P = Ep * Ap * \Delta l / l$$

Where,	l	:	Length of PC Strand
	Δ l	:	Slip
	Ap	:	Area of PC Strand
	Ep	:	Young's Modulus of PC Strand

ii) In case Friction between Sheath and Strand exists

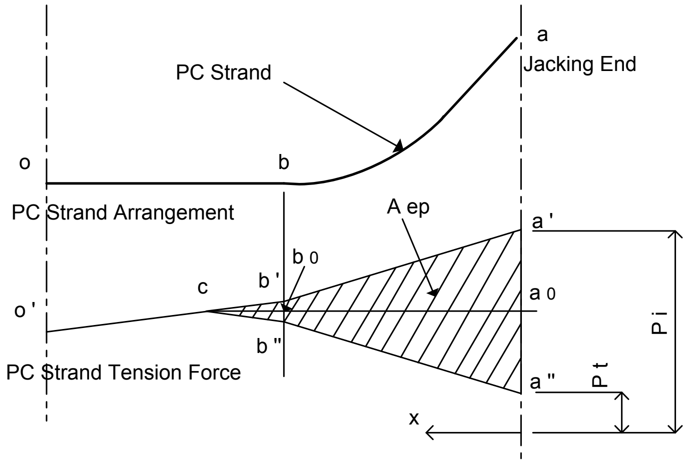


Fig. 7.1 Loss of Prestress due to Slip

$\Delta l =$ Aep / (Ap * Ep)

Where, Aep : Area as shown in **Fig. 7-1**

7.8.3. Loss of Prestress due to Elastic Deformation

Loss of prestress due to elastic deformation in post-tensioned method $\Delta\ \sigma p$ is calculated as follows:

$$\Delta\ \sigma p = (1/2) * n * \sigma_{cpg} * (N-1) / N$$

Where, n : Ratio of Young's Modulus n = Es / Ec

Ep : Young's Modulus of PC Strand

Ec : Young's Modulus of Concrete at transfer

σ_{cpg} : Concrete Stress due to Prestress immediately after prestressing at the centroid of PC Strands

N : Number of PC Strand (Number of Jacking)

7.8.4. Loss of Pre-stress due to Creep and Shrinkage

Loss of pre-stress due to creep and shrinkage Δp is calculated as follows.

$$\Delta p = \frac{\alpha e * fco * \phi ti + \Delta cst * Es}{1 + \rho * \alpha e * (1 + as^2 / i^2) * (1 + \eta \phi ti)}$$

Where,

fco	:	Stress in concrete at the level of the tendon due to initial prestress and dead load
Δcst	:	Shrinkage at time t
ϕti	:	Creep coefficient at time t for a load applied at time i
ρ	:	Geometrical ratio of reinforcement = As/bd
αe	:	Modular ratio
as	:	Distance of the centroid of the steel from the centroid of the net concrete section
I	:	Radius of gyration
Es	:	Elastic modulus of steel
η	:	relaxation coefficient

7.8.5. Loss of Prestress due to Relaxation

Loss of pre-stress due to creep and shrinkage Δr is calculated as follows.

$$\Delta r = \gamma * fpt$$

Where,

fpt	:	Tensile Stress of PC Strand immediately after pre-stressing
γ	:	Ratio of Relaxation (0.05)

7.9. Design of Member

7.9.1. Serviceability Limit State

The following two points are to be checked for the serviceability limit state.

- ✓ Design Crack Width
- ✓ Stress Limitation

7.9.2. Ultimate Limit State

a) Resistant Moment

For flexure members, the bending moments shall be such that:

$$M \leq Mr$$

Where, Mr : ultimate resistant moment

b) Resistant Shear Force

For shear force, shear reinforcement shall be provided in accordance with Clause 6.3.4.4 of BS 5400 Part. 4.

Minimum shear reinforcement should be provided in the form of links such that:

$$(Asv / sv) * (0.87 * fyu / b) = 0.40 \text{ N/mm}^2$$

Where, fyu : characteristic strength of link reinforcement but not greater than 460 kN/mm^2

Asv : total cross sectional area of the leg of the links

sv : link spacing along the length of beam

When the shear stress, v, due to the ultimate loads exceeds vc, the shear reinforcement provided should be such that:

$$Asv \geq b * sv * (v + 0.4 - vc) / (0.87 * fyu)$$

Where links are used, the area of longitudinal steel in the tensile zone should be such that:

As >= V / (2 * 0.87 * fy)

Where, As : are of effectively anchored longitudinal tensile reinforcement and prestressing tendons (excluding debonded tendons) additional to that required at the ultimate limit state for other purposes

fy : characteristic strength of the longitudinal reinforcement and prestressing tendons but not greater than 460 kN/mm^2

7.10. Stability Analysis of Pile Foundation

As the method of stability analysis for the pile foundation, the Displacement Method or Frame Modeling Method is adopted.

The examination for stability of pile is as the following 3 points:

- --Axial compressive force due to the pile bearing capacity,
- --Axial pull-out force due to the pile friction capacity, and
- --Horizontal displacement.

7.10.1. Ultimate Bearing Capacity

The ultimate bearing capacity is calculated based on the soil tests results, the formula (conformed in BS 8004) is shown below:

Q = f * As + Ab * (q + po) - P

Where, Q : Ultimate bearing capacity (kN)

As : Surface of the pile shaft (m^2)

Ab : The plain area of the pile shaft (m^2)

f : Average skin friction or adhesion per unit area of the shaft at the condition of full mobilization of frictional resistance (kN/m^2)

q : The ultimate value of the resistance per unit area of base due to the shearing stress of the soil (kN/m^2)

po : The effective pressure of the overburden at the level of the base (kN/m^2)

Refer to Japanese Code;

f * As = Σ fi * U * li

Where, fi : skin friction (kN/m^2)

Sand fi = 5 * N (fi $\leq$ 200)

Clay fi = 10 * N (fi $\leq$ 150)

N $\leq$ 2 shall be neglected the skin friction

N : N-value

7.10.2. Allowable Capacity and Displacement (refer to Japanese Code)

a) Allowable Bearing Capacity

$$Ra = 1/n * (Ru - Ws) + Ws - W$$

Where,

Ra	:	allowable bearing capacity of a pile (kN)
n	:	safety factor (=2.5) Safety factor 2.5 is normally used for verification of foundation in case of standard condition, therefore n = 2.5 is used for this design
Ws	:	effective weight of soil to be replaced with a pile
W	:	effective weight of a pile in the ground

b) Allowable Uplift Capacity

$$Pa = 1/n * Pu$$

Where,

Pa	:	allowable uplift capacity of a pile (kN)
n	:	safety factor (=6) Safety factor 6 is normally used for verification of foundation in case of standard condition, therefore n = 6 is used for this design
Pu	:	ultimate uplift resistance of a pile Pu = f * As

c) Allowable horizontal displacement

Horizontal displacement at the top of a pile shall be checked not to give adverse effect to superstructure and to avoid plasticity of ground in front of pile. The allowable horizontal displacement shall generally be less than 1 % of pile diameter or 15mm, whichever bigger in order to assure the safety against lateral force.

7.11. Stability Analysis of Spread Foundation

Sliding, overturning and bearing failure shall be checked as stability calculation for spread type foundation.

7.11.1. Sliding

Shear resisting for sliding shall be followed the following formula;

$$Hu = (C_B * Ae + V * \tan \phi_B)$$

Where;
- Hu : Shear resistance between the foundation and embedded soil (kN)
- C_B : Cohesion between the foundation and embedded soil (kN/m2)
- Ae : Eeffective loading area (m2)
- V : Working vertical load (kN)
- ϕ_B : Friction angle between the foundation and embedded soil (degree)

* Factor of safety shall be taken 1.5 or greater

7.11.2. Bearing Failure

Ultimate bearing capacity for the embedded soil of foundation is calculated by Terzaghi Theory below;

$$Qu = Ae (\alpha * k * c * Nc + k * q * Nq + 0.5 * \gamma 1 * \beta * Be * N\gamma) / F.S.$$

Where;
- Qu : Ultimate bearing capacity (kN)
- Ae : Effective loaded area (m2)
- α,β : Shape factor of the foundation
- k : Rate of increase for embedded effects
- c : Cohesion of the embedded soil (kN/m2)
- q : Applied load (kN/m2), $q = \gamma 2 * Df$
- Df : Effective embedded depth (m)
- Be : Effective loading width of the foundation, for which eccentricity of loads is considered, Be = B – 2 eB
- B : Width of the foundation
- eB : Eccentricity of loads (m)
- $\gamma 1,\gamma 2$: Unit weight of bearing soil layer and embedded soil layer (kN/m3)
- Nc, Nq, Ng : Coefficientof bearing capacity considering inclination of loads

* Factor of safety shall be taken 3 or greater

8. STEEL BRIDGE

8. Steel Bridge

8.1. General

In this Claus 8, steel structures for highway bridges and ramp bridges, mainly the superstructure of OCH-N1 Section are applicable otherwise conform to Clause 1 to 7.

And steel bridge design may be conformed to either British Standards or Japan Road Association (JRA) Standards for OCH-N1 Project.

8.2. Geometric Design Standard

8.2.1. Staged Construction for Highway Bridge

As per mentioned in the Clause 3.2.1, highway bridges shall be applied the staged construction method for superstructure to consider the minimum requirement at the 1st stage, and one time construction method for substructure to consider the difficulty of future expansion.

In case of being difficult for future expansion structurally, full width of structure (completed section) shall be constructed at the beginning (i.e. Steel box girder is to be constructed with full width, and Steel I girder is to be constructed with minimum required width at the 1st stage).

In OCH-N1, Steel I Girder will be used as superstructure type for highway bridges, and it can be added expansion members at the final stage (see Fig. 8.1).

The following subjects shall be considered in the design and/or construction methodology to join the member between old (the 1st stage) and new (the final stage):

- Old and new girders shall be combined by cross beams completely, and the connection plates for it shall be pre-fabricated/pre-installed,
- Required lap splice length of reinforcing bar for concrete deck slab shall be considered and pre-installed, and,
- The effect by the creep/shrinkage of the new concrete (deck slab) shall be considered.

8.2.2. Typical Section of Ramp Bridges

Box type girder was selected for ramp bridges in the A1 interchange. The typical cross section is shown in Fig. 8.2 below:

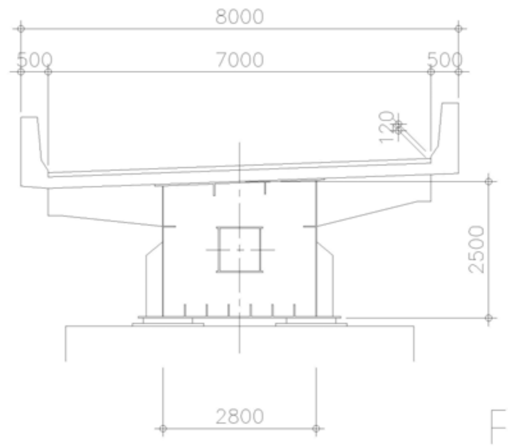


Fig. 8.2 Typical Cross Section of Ramp Bridge (1-Lane)

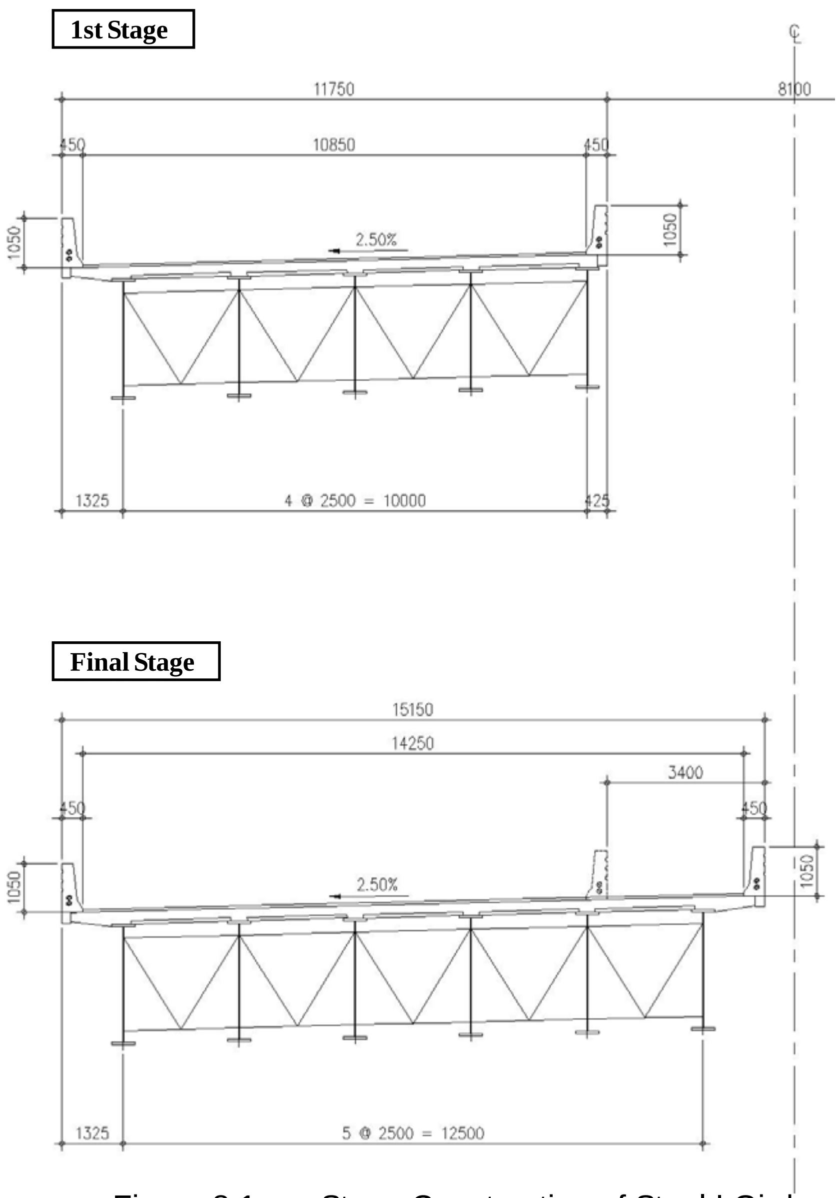


Figure 8.1 Stage Construction of Steel I Girder

8.3. Properties of Materials

8.3.1. Structural Steel

Structural steel shall comply with BS Code or equivalent standards including JRA Specification.

Strength of structural steels (at the thickness is 16mm, BS EN 10025 & JIS 3101, 3106 & 3114) for bridges are shown in Tables 8.1 to 8.5.

And the following properties of steel should be assumed in design:

- Modulus of Elasticity : $E = 205,000\ N/mm^2$
- Shear Modulus : $G = 80,000\ N/mm^2$
- Poisson's Ratio : $\nu = 0.3$
- Coefficient of Thermal Expansion : $\alpha = 12 \times 10^{-6}\ /^{o}C$

Table 8.1 Non-Alloy Structural Steels
(BS EN 10025 Part 2)

BS EN 10025: 2004			Previous Code and the Grade	
Grade	Yield (Reh)	Tensile (Rm)	BS EN 10025: 1993	BS 4360: 1990
	Strength @16mm Thick. (MPa)			
S185	185	290/510	S185	---
S235	235	360/510	S235	40A
S235JR			S235JRG1/G2	40B
S235J0			S235J0	40C
S235J2			S235J2G3/G4	40D
S275	275	410/560	S275	43A
S275JR			S275JR	43B
S275J0			S275J0	43C
S275J2			S275J2G3/G4	43D
S355	355	470/630	S355	50A
S355JR			S355JR	50B
S355J0			S355J0	50C
S355J2			S355J2G3/G4	50D
S355K2			S355K2G3/G4	50DD
E295	295	470/610	E295	---
S335	335	570/710	S335	---
E360	360	650/830	E360	---

Table 8.2 Normalised/Normalised Rolled Weldable Fine Grain Structural Steels (BS EN 10025 Part 3)

BS EN 10025: 2004			Previous Code and the Grade	
Grade	Yield (Reh)	Tensile (Rm)	BS EN 10113 Part 2: 1993	BS 4360: 1990
	Strength @16mm Thick. (MPa)			
S275N	275	370/510	S275N	43DD
S275NL			S275NL	43EE
S355N	355	470/630	S355N	50
S355NL			S355NL	50EE
S420N	420	520/680	S420N	---
S420NL			S420NL	---
S460N	460	550/720	S460N	55C
S460NL			S460NL	55EE

Table 8.3 Thermomechanically Rolled Weldable Fine Grain Structural Steels (BS EN 10025 Part 4)

BS EN 10025: 2004			Previous Code and the Grade	
Grade	Yield (Reh)	Tensile (Rm)	BS EN 10113 Part 3: 1993	BS 4360: 1990
	Strength @16mm Thick. (MPa)			
S275M	275	370/510	S275M	---
S275ML			S275ML	---
S355M	355	470/630	S355M	---
S355ML			S355ML	---
S420M	420	520/680	S420M	---
S420ML			S420ML	---
S460M	460	550/720	S460M	---
S460ML			S460ML	---

Table 8.4 Structural Steels with Improved Atmospheric Corrosion Resistance - Also Known as Weathering Steels (BS EN 10025 Part 5)

BS EN 10025: 2004			Previous Code and the Grade	
Grade	Yield (Reh)	Tensile (Rm)	BS EN 10113 Part 3: 1993	BS 4360: 1990
	Strength @16mm Thick. (MPa)			
S235J0W	235	360/510	S235J0W	---
S235J2W			S235J2W	---
S355J0WP	355	470/630	S355J0WP	WR50A
S355J2WP			S355J2WP	---
S355J0W	355	470/630	S355J0W	WR50B
S355J2W			S355J2W	WR50C
S355K2W			S355K2W	WR50D

Table 8.5 Flat Products of High Yield Strength Structural Steels in the Quenched and Tempered Condition (BS EN 10025 Part 6)

BS EN 10025: 2004			Previous Code and the Grade	
Grade	Yield (Reh)	Tensile (Rm)	BS EN 10025: 1993	BS 4360: 1990
	Strength @16mm Thick. (MPa)			
S460Q	460	550/720	S460Q	---
S460QL			S460QL	---
S460Q1			S460Q1	55F
S500Q	500	590/770	S500Q	---
S500QL			S500QL	---
S500Q1			S500Q1	---
S550Q	550	640/820	S550Q	---
S550QL			S550QL	---
S550Q1			S550Q1	---
S620Q	620	700/890	S620Q	---
S620QL			S620QL	---
S620Q1			S620Q1	---
S690Q	690	770/940	S690Q	---
S690QL			S690QL	---
S690Q1			S690Q1	---
S890Q	890	940/1,100	S890Q	---
S890QL			S890QL	---
S890Q1			S890Q1	---
S960Q	960	980/1,150	S960Q	---
S960QL			S960QL	---

Table 8.6 Rolled Steels for General Structure (JIS G 3101) Rolled Steels for Welded Structure (JIS G 3106) Hot-Rolled Atmospheric Corrosion Resisting Steels For Welded Structure (JIS G 3114)

JIS G 3101, 3106, 3114			Similar/equivalent Code
Grade	Yield (Reh)	Tensile (Rm)	BS EN 10025: 2004
	Strength @16mm Thick. (MPa)		
SS400 (JIS G3101)	245	400/510	Part 2: S235/S235JR/S235J0/S235J2, S275/S275R/S275J0/S275J2
SM400 (JIS G3106)	245	400/510	Part 3: S275N/S275NL Part 4: S275M/S275ML
SMA400W (JIS G3114)	245	400/540	Part 5: S235J0W/S235J2W
SM490 (JIS G3106)	325	490/610	Part 3: S355N/S355NL Part 4: S355M/S355ML
SM490Y (JIS G3106)	365	490/610	
SMA490W (JIS G3114)	365	490/610	Part 5: S355J0WP/S355J2WP
SM520 (JIS G3106)	365	520/640	Part 3: S420N/S420NL Part 4: S420M/S420ML
SM570 (JIS G3106)	460	570/720	Part 3: S460N/S460NL Part 4: S460M/S460ML
SMA570W (JIS G3114)	460	570/720	Part 6: S460Q/S460QL/S460QL1

8.3.2. High Strength Friction Grip Bolts, Nuts and Washers

High strength friction grip bolts and associated nuts and washers (HSFG Bolts) shall comply with BS Code and/or JRA, JSS Specification.

HSFG Bolt (BS 4395, BS 4604 and JIS B1186/JRA/JSS) for bridges are as follows;

Table 8.7-1 Properties of High Strength Friction Grip Bolts and associated Nuts and Washers (General Grade - HSFG Bolts, BS 4395-1)

Nominal Size and Tread Diameter	Area	Strength					
d (mm)	As (mm2)	Ultimate Load (kN, <N/mm2>)		Yield Load (kN, <N/mm2>)		Proof Load (kN, <N/mm2>)	
M12	84.3	69.6		53.5		49.4	
M16	157	130		99.7		92.1	
M20	245	203	827	155	635	144	587
M22	303	250		192		177	
M24	353	292		225		207	
M27	459	333		259		234	
M30	561	406	725	313	558	286	512
M36	817	591		445		418	

Table 8.7-2 Properties of High Strength Friction Grip Bolts and associated Nuts and Washers (Higher Grade Bolts and Nuts and General Grade Washers - HSFG Bolts, BS 4395-2)

Nominal Size and Tread Diameter	Area	Strength					
d (mm)	As (mm2)	Ultimate Load (kN, <N/mm2>)		Yield Load (kN, <N/mm2>)		Proof Load (kN, <N/mm2>)	
M16	157	154.1		138.7		122.2	
M20	245	240.0		216		190.4	
M22	303	296.5		266		235.5	
M24	353	346	981	312	882	274.6	776
M27	459	450		406		356	
M30	561	550		495		435	
M33	694	680		612		540	

Table 8.8 Properties of High Strength Hexagon Bolt, Hexagon Nut and Plain Washers for Friction Grip Joints (JIS B 1186 and JRA, JSS II09-1996)

Grade	Nominal Size d (mm)	Area As (mm2)	Minimum Tensile Strength (N/mm2)	Yield Strength (N/mm2)	Specification
F8T	M20	245	640	800/ 1,000	JIS B 1186
	M22	303			
	M24	353			
F10T	M20	245	900	1,000/ 1,200	JIS B 1186
	M22	303			
	M24	353			
S10T	M20	245	900	1,000/ 1,200	JRA JSS II09-1996
	M22	303			
	M24	353			

8.4. Details

8.4.1. Steel Girder Type

Both composite and non-composite between steel girder and concrete deck slab types are applicable for the superstructures. In the OCH-N1 Project, non-composite girder type was adopted by the following reasons:

- ✧ Maintenance and repair of girders/slab for composite type are complicated and very difficult, and needed to close the traffic during the works, not to affect by vehicle's impact and vibration.
- ✧ Accurate quality control shall be needed during construction of composite type, and more severe accuracy is needed for main highway expansion in future.

8.4.2. Thickness of Steel Plates

Technically, 400mm thickness of steel plate can be produced and it is mentioned on BS. However, such thickness of steel is not required for the bridge fabrications.

In the OCH-N1 Project, the **maximum 100mm thickness** is adopted from the previous practices of the steel bridge constructions. Actually, thickness up to 40mm will be enough and used for the steel I or box girder bridges in this Project.

And the minimum thickness is adopted **8mm** based on JRA as the steel girder bridge, and minimum thickness of sole plate at bearing support points is adopted 22mm or greater in consideration of the purpose.

8.5. Design Requirements

8.5.1. Design Method

Bridges shall be designed by checking the formula below for two limit states, namely "Ultimate Limit State (ULS)" and "Serviceability Limit State (SLS)".

$R^* \geq S^*$

R* = f unction (fk / γm) ; Design Resistance

S* = γf3 * effect (γfL * Qk) ; Design Load Effect

Fk	:	nominal strength of the material	(by available material)
Qk	:	nominal load	(by BS5400 Part-2)
γ m	:	partial safety factor for strength	(by BS5400 Part-3)
γ f3	:	partial inaccurate factor	(by BS5400 Part-3)
γ fL	:	partial load factor	(by BS5400 Part-2)

8.5.2. Partial Inaccurate Factor γf3

The partial inaccurate factor shall be used the following values:

γ f3 = 1.00 (SLS)
1.10 (ULS)

8.5.3. Partial Safety Factor for Strength γm

The values of γm for both serviceability and ultimate limit state (SLS & ULS) is shown in Table 8.9 below:

Table 8.9 Partial Safety Factor for Strength

a) Ultimate Limit State	
The Value of γm for the ultimate limit state should be taken as **1.05**, except in the following cases for which the appropriate value of γm is given.	
Structural component and behaviour	**γm**
Strength of longitudinal stiffeners	1.20 (fibre in compression) 1.05 (fibre in tension)
Buckling resistance of stiffeners	1.20
Fasteners in tension	1.20
Fasteners in shear	1.10
Friction capacity of HSFG bolts	1.30
Welds	1.20
b) Serviceability Limit State	
The Value of γm for the serviceability limit state should be taken as **1.00**, except in the following cases for which the appropriate value of γm is given.	
Structural component and behaviour	**γm**
Friction capacity of HSFG bolts	1.20

8.5.4. Global Analysis for Load Effects

The global analysis of the bridge structure should be in accordance with BS 5400 Part 1 using an elastic method to determine load effects. And the sectional properties to be used in this analysis should generally be calculated for the gross cross-section assuming the specified sizes.

8.5.5. Stress Analysis

Longitudinal Stress in Beams: The distribution of longitudinal stress between the flanges and web or webs of a beam may be calculated on the assumption that plane sections remain plane, but using the effective widths of flanges and the effective thickness of a deep web in accordance with BS 5400 Part 3.

Shear Stresses: Shear force due to vertical loads will induce a shear stress in webs. The design values of the shear stress in webs of rolled or fabricated (I, box and channel, etc.) may be calculated in accordance with BS 5400 Part 3.

Distortion and Warping Stresses in Box Girders: Stresses in a box girder due to transverse bending of the walls of the box torsional and distortional warping should be calculated by linear elastic analysis.

8.5.6. Design of Beams

Beams are defined as members with solid webs or with openings, subjected primary to bending, including members of rolled and hollow section, plate girders and box girders. The following need to be considered:

- ✓ Material strength
- ✓ Limitation of shape
- ✓ Limitation of strength
- ✓ Effective section
- ✓ Slenderness (resistance for buckling, effective length of members)
- ✓ Compression/ tension member design
- ✓ Combined effects of bending and shear
- ✓ Fatigue (conform to BS 5400 Part 10)
- ✓ Support settlement
- ✓ Creep and shrinkage of concrete
- ✓ Positive/negative temperature differences
- ✓ Pre-camber

Design of steel girder requires consideration of load distribution by cross beams. Design method analyzed load distribution, such as Influence Line Modeling Method or Grid Modeling Analysis by computer program shall be applied to calculate load distribution.

8.5.7. Strength of HSFG Bolts acting in Friction

Both BS and JRA of strength of high strength friction grip bolts are shown in bellow:

- BS 5400 Part 3 Clause 14.5.4

Friction capacity PD of a HSFG bolt should be taken as:

PD = kh * Fv * μ * N / (γm * γf3)

where:

kh: a factor depend on the oversized and slotted holes
Fv: pre-stress load
μ: slip factor
N: number of friction interfaces

- JRA

Allowable friction capacity Pa of a HSFG bolt by JRA is shown below:

Pa = 1 / ν * N = 1 / ν * μ * (α * σy * Ae)

where:

N: design axial force of bolt = μ * (α * σy * Ae)
ν: factor of safety = 1.7
μ: slip facto = 0.4
α: rate for yield strength of bolt = 0.75
σy: yield strength of bolt
Ae: effective area of bolt

Table 8.10 Allowable Friction Capacity of HSFG Bolt
(JIS B 1186 and JRA, JSS II09-1996)

Grade	Nominal Diameter	ν	μ	α	σy (N/mm2)	Ae (mm2)	N (kN)	Pa (kN)
S10T F10T	M20	1.7	0.4	0.75	900	245	165	38.8
	M22					303	205	48.2
	M24					353	238	56.0

9. BOX CULVERT

9. Box Culvert

9.1. Introduction

This clause describes culvert structure with box section. Box culverts comprise under-pass for the minor crossing roads and drainage opening.

9.2. Basic Policy

9.2.1. Dimensions

Under-pass

Generally, box culvert is applicable when its inner width is 14 m or smaller. In case of exceeding 14 m in inner width, bridge structure is deemed more economical.

Drainage culvert

Type of the drainage culvert which transfers water across the road shall be selected by drainage capacity calculation.

If the discharge volume is relatively small, pipe culvert is adopted, otherwise box culvert is required.

9.2.2. Consideration of the Future 6 Lane

Culvert shall be designed for the future 6-lane condition. Especially in structural design, cover height at the 6-lane stage shall be taken into account.

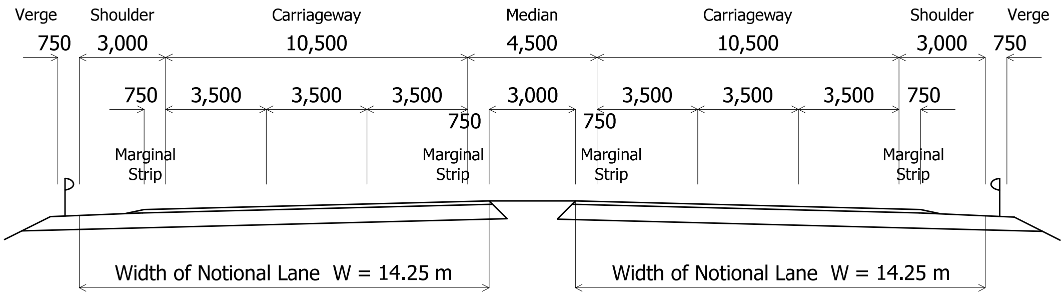


Fig. 9-1 Notional Lanes applying Specified Live Loads for 6-Lane

Number of notional lanes is to be 4 lanes for one side (one notional lane width of 3.5625 m) in accordance with 3.2.9.3 of BS 5400-2.

9.3. Design Standards

The design standards to be used for box culvert design are as follows:

- British Standard “BS 5400”;
- British Standard “BS 8110”;
- “BD 31/01: The Design of Buried Concrete Box and Portal Frame Structures”, Design Manual for Roads and Bridges;
- “BD 37/01: “Loads for Highway Bridges”, Design Manual for Roads and Bridges

9.4. Design Principles

Limit state principles are to be adopted for design of structural elements and the foundations. Both Ultimate Limit State (ULS) and Serviceability Limit State (SLS) are considered.

The design life of buried concrete box structures shall be 120 years.

9.5. Loads, Load Combinations and Partial Safety Factors

Load combinations to be considered are Combination-1, Combination-3 and Combination-4.

Combination-1	:	Permanent loads, Vertical live loads and Horizontal live load surcharge
Combination-3	:	Combination-1 plus temperature effects
Combination-4	:	Permanent loads, Horizontal live load surcharge plus Traction

For structure element design, loads applied simultaneously in any load combination and values of partial safety factor γ_{fL} are as shown in **Table 9-1** and the partial safety factors γ_{f3} at SLS and ULS are respectively 1.00 and 1.10, except 1.00 shall be taken at ULS for all relieving effects.

Table 9-1 Loads, Load Combinations and Values of γ_{fL} for the Design of Structure Members

LOADS		Limit State	γ_{fL} for Combinations 1	3	4
PERMANENT LOADS					
Weight of concrete	3.1.1	ULS	1.15*	1.15*	1.15*
		SLS	1.00	1.00	1.00
Superimposed pavement construction within 200mm from the surface	3.1.2	ULS	1.75	1.75	1.75
		SLS	1.20	1.20	1.20
Superimposed fill including pavement in excess of 200 mm	3.1.2	ULS	1.20	1.20	1.20
		SLS	1.00	1.00	1.00
Horizontal earth pressure (using default earth pressure coefficients)	3.1.3	ULS	1.50	1.50	1.50
		SLS	1.00	1.00	1.00
Horizontal earth pressure (using earth pressure coefficients in accordance with BS8002)	3.1.3	ULS	1.20	1.20	1.20
		SLS	1.00	1.00	1.00
Hydrostatic pressure and buoyancy	3.1.4	ULS	1.10	1.10	1.10
		SLS	1.10	1.10	1.10
Settlement	3.1.5	ULS	1.20	1.20	1.20
		SLS	1.00	1.00	1.00
LIVE LOADS					
Vehicle live loads					
HA carriageway loading	3.2.1	ULS	1.50	1.25	
		SLS	1.20	1.00	
HB carriageway loading	3.2.1	ULS	1.30	1.10	
		SLS	1.10	1.00	
Footway and cycle track loads	3.2.2	ULS	1.50	1.25	
		SLS	1.00	1.00	
Accidental wheel loading	3.2.3	ULS	1.50		
		SLS	1.20		
Construction traffic	3.2.5	ULS	1.15	1.15	
		SLS	1.00	1.00	
Horizontal pressure due to live load surcharge	3.2.6	ULS	1.50	1.50	1.50
		SLS	1.00	1.00	1.00
HA traction	3.2.7	ULS			1.25
		SLS			1.00
HB traction	3.2.7	ULS			1.10
		SLS			1.00
Temperature range	3.2.8	ULS		N/A	
		SLS		1.00	
Differential temperature	3.2.8	ULS		1.00	
		SLS		0.80	
Parapet collision	3.2.9	In accordance with BD37			
Skidding	3.2.10	ULS			1.25
		SLS			1.00
Centrifugal load	3.2.11	ULS			1.50
		SLS			1.00

Note: For applied loads causing a relieving effect on the element under consideration, the value of γ_{fL} shall be taken as 1.00.

For foundation design, the loads to be applied simultaneously for checking sliding and bearing pressure together with partial safety factors γ_{fL} and γ_{f3} are in **Table 9-2**.

Table 9-2 Loads, Load Combinations and Values of γ_{fL} and γ_{f3} for the Design of Foundation

LOADS		Limit State	Partial Safety Factor γ_{fL}	Partial Safety Factor γ_{f3}
Sliding	Dead load	ULS	1.00	1.00
	Minimum superimposed dead load	ULS	1.00	1.00
	Buoyancy	ULS	1.10	1.10
	Traction	ULS	1.25 (HA) 1.10 (HB)	1.10
	Vertical live load associated with traction	ULS	1.00	1.00
	Disturbing earth pressure (active)	ULS	1.50	1.10
	Disturbing live load surcharge (active)	ULS	1.50	1.10
	Relieving earth pressure (passive)	ULS	1.00	1.00
Bearing Pressure	Dead load	SLS	1.00	1.00
	Maximum superimposed dead load	SLS	1.00	1.00
	Maximum horizontal earth pressure on both sides of the box structure	SLS	1.00	1.00
	Hydrostatic pressure and buoyancy	SLS	1.00	1.00
	Vertical live load	SLS	1.00	1.00
	Live load surcharge on one side of the box	SLS	1.00	1.00

9.6. Loads

The following nominal loads shall be used in the box culvert design:

Permanent Loads

- Dead loads
- Superimposed dead loads
- Horizontal Earth Pressure
- Hydrostatic Pressure and Buoyancy

Vertical Live Loads

- HA or HB loads on carriageway

Horizontal Live Loads

- Live load surcharge
- Traction

Others

- Temperature effects

The earthquake effect shall not be considered in the Project site.

9.6.1. Dead Loads

The nominal dead load consists of the weight of the materials and parts of the structure that are structural elements excluding superimposed materials.

The following densities specified in **Table 9-3** for each material are used for dead loads.

Table 9-3 Densities

Material	Density (kN/m^3)	Material	Density (kN/m^3)
Surfacing Material (Pavement)	23.0	Reinforced Concrete	25.0
Fill Material (Compacted)	19.0	Plain Concrete	23.5
Fill Material (Loose)	16.0	Water	10.0

9.6.2. Superimposed Dead Loads

The nominal superimposed dead loads consist of <u>the weights of surfacing material and fill material</u> above the structure and shall be applied as a uniformly distributed load.

In the design, <u>pavement material for top 200 mm thickness and compacted fill material in excess of 200 mm from the road surface</u> are considered.

It is said that the superimposed dead loads are to be increased where consolidation or settlement of the fill adjacent to a buried structure is expected. In consideration of these effects, the following two (2) cases shall be considered in this Project.

- Maximum superimposed dead load intensity = $1.15\ \gamma\ H$ (H<8m)
- Minimum superimposed dead load intensity = $\gamma\ H$

where:

γ : bulk density of compacted fill or road construction materials
H : height of cover from the top of the structure to the finished surface level.

9.6.3. Load Effects due to Temperature

Buried structures to satisfy the following conditions have no temperature effect:

(i) cover (H) > 2.0 m and $X_{clear} < 0.20\ L_t$
(ii) overall length of structure (L_L) $\leq$ 3.0 m

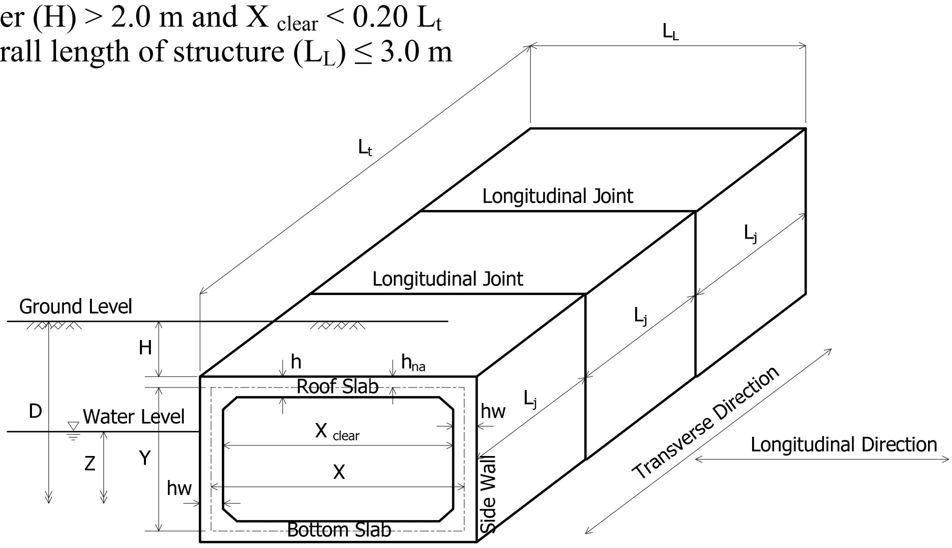


Fig. 9-2 Symbols of Typical Box Structure

In all other buried structures, "Temperature Range" and "Differential Temperature" shall be considered. The coefficient of thermal expansion (α) shall be taken as $12 * 10^{-6}$ per °C for concrete structure.

Temperature Range

Shade air temperature in Project site is 20 °C in minimum and 30 °C in maximum from Isotherms, Department of Meteorology. Temperature variation in concrete structure above the ground surface is 28 ± 3 °C by "Bridge Design Manual, RDA".

These variations in mean temperature shall be considered at SLS, and shall be given to the roof slab only for box culvert structure.

In consideration of the above and "BD37/01, Design Manual for Roads and Bridges", maximum and minimum effective temperature (T_{max} and T_{min}) are determined in **Table 9-4**.

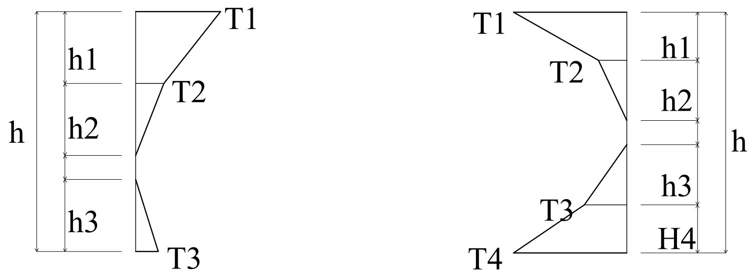
Differential Temperature

The effects of temperature gradients within a section shall be applied to the roof slab only. Resulting flexure on other members due to this shall be considered.

Both positive and negative temperature differences are described in **Tables 9-4** and 9-**5** together with **Fig. 9-3**.

Table 9-4 Load Effect due to Temperature

Span to Width Ratio	Cover	Minimum & Maximum Effective Temperature		Differential Temperature	
X_{clear} / L_t	H (m)	T_{min}	T_{max}	Temperature Difference	Reduction factor η
≥ 0.20	All depths	7 °C	13 °C	Table 3.1.5	N/A
< 0.20	H ≤ 2.00	7 °C	13 °C	Table 3.1.5	0.00
	2.00 < H	Temperature effects may be neglected			



Positive Temperature Difference | Negative Temperature Difference

h1 = 0.30h	but ≤ 0.15m	h1 = 0.20h	but ≤ 0.25m
h2 = 0.30h	but 0.10m≤, ≤ 0.25m	h2 = 0.25h	but ≤ 0.20m
h3 = 0.30h		h3 = 0.25h	but ≤ 0.20m
		h4 = 0.20h	but ≤ 0.25m

Fig. 9-3 Temperature Difference (Figure 9, Group 4 in BD37/01, Table 24)

Table 9-5 Values of Temperature Difference

Depth of slab h (m)	Surfacing thickness H (m)	Positive temperature difference			Reverse temperature difference			
		T1 (°C)	T2 (°C)	T3 (°C)	T1 (°C)	T2 (°C)	T3 (°C)	T4 (°C)
< 0.20	Unsurfaced	12.0	5.0	0.1	4.7	1.7	0.0	0.7
	Waterproofed	19.5	8.5	0.0	4.7	1.7	0.0	0.7
	50	13.2	4.9	0.3	3.1	1.0	0.2	1.2
	100	8.5	3.5	0.5	2.0	0.5	0.5	1.5
	150	5.6	2.5	0.2	1.1	0.3	0.7	1.7
	200 ≤	3.7	2.0	0.5	0.5	0.2	1.0	1.8
0.40	Unsurfaced	15.2	4.4	1.2	9.0	3.5	0.4	2.9
	Waterproofed	23.6	6.5	1.0	9.0	3.5	0.4	2.9
	50	17.2	4.6	1.4	6.4	2.3	0.6	3.2
	100	12.0	3.0	1.5	4.5	1.4	1.0	3.5
	150	8.5	2.0	1.2	3.2	0.9	1.4	3.8
	200 ≤	6.2	1.3	1.0	2.2	0.5	1.9	4.0
0.60	Unsurfaced	15.2	4.0	1.4	11.8	4.0	0.9	4.6
	Waterproofed	23.6	6.0	1.4	11.8	4.0	0.9	4.6
	50	17.6	4.0	1.8	8.7	2.7	1.2	4.9
	100	13.0	3.0	2.0	6.5	1.8	1.5	5.0
	150	9.7	2.2	1.7	4.9	1.1	1.7	5.1
	200 ≤	7.2	1.5	1.5	3.6	0.6	1.9	5.1
0.80	Unsurfaced	15.4	4.0	2.0	12.8	3.3	0.9	5.6
	Waterproofed	23.6	5.0	1.4	12.8	3.3	0.9	5.6
	50	17.8	4.0	2.1	9.8	2.4	1.2	5.8
	100	13.5	3.0	2.5	7.6	1.7	1.5	6.0
	150	10.0	2.5	2.0	5.8	1.3	1.7	6.2
	200 ≤	7.5	2.1	1.5	4.5	1.0	1.9	6.0
1.00	Unsurfaced	15.4	4.0	2.0	13.4	3.0	0.9	6.4
	Waterproofed	23.6	5.0	1.4	13.4	3.0	0.9	6.4
	50	17.8	4.0	2.1	10.3	2.1	1.2	6.3
	100	13.5	3.0	2.5	8.0	1.5	1.5	6.3
	150	10.0	2.5	2.0	6.2	1.1	1.7	6.2
	200 ≤	7.5	2.1	1.5	4.8	0.9	1.9	5.8
> 1.50	Unsurfaced	15.4	4.0	2.0	13.7	1.0	0.6	6.7
	Waterproofed	23.6	5.0	1.4	13.7	1.0	0.6	6.7
	50	17.8	4.0	2.1	10.6	0.7	0.8	6.6
	100	13.5	3.0	2.5	8.4	0.5	1.0	6.5
	150	10.0	2.5	2.0	6.5	0.4	1.1	6.2
	200 ≤	7.5	2.1	1.5	5.0	0.3	1.2	5.6

9.6.4. Earth Pressures

The nominal horizontal earth pressures on side walls of box culvert shall be taken as follows:

Values of Earth Pressure Coefficients (BD 31/01)

K_a	:	Coefficient of active earth pressure	=	0.33
K_{min}	:	Minimum credible coefficient for balanced earth pressure	=	0.20
K_o	:	Coefficient of lateral earth pressure at rest	=	0.60

Loading Conditions

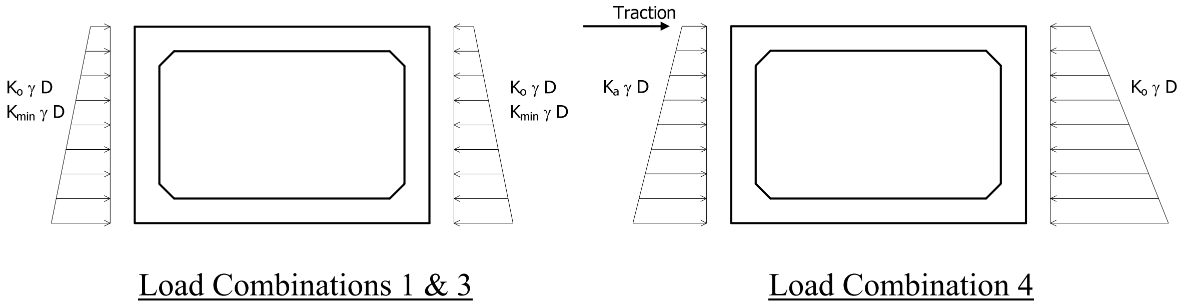


Fig. 9-4 Loading Conditions on Earth Pressure

where:

γ = bulk density of compacted fill or road construction materials (kN/m^3)
D = depth at considered point from the highway profile (m)

The coefficient of earth pressure for wing wall design, the value K_a shall be used.

9.6.5. Hydrostatic Pressure

The increase in pressure on the back of the walls due to hydrostatic pressure at a depth of Z in meter below water level shall be taken as 10Z (1-K) in kN/m^2.

where:

K = earth pressure coefficient to be used for a given load in the design

Since project area is flooded in the rainy season, design water level in the design shall be as follows:

High Water Level: Top of box culvert roof slab or 1 m below the finished road level, which is higher

Low Water Level: Box culver base level

9.6.6. Vertical Live Load

HA UDL/KEL load and 30 units of HB load are considered on box structure under OCH main carriageway including the ramp way. HA single wheel load shall also be taken into account instead of HA UDL/KEL load.

For crossing minor roads, only HA load is considered for class C/D roads.

HA UDL/KEL load

Where the depth of cover (H) is 0.60 m or less, HA UDL/KEL without dispersion through the fill shall be considered.

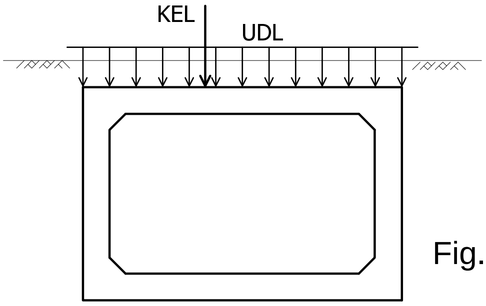
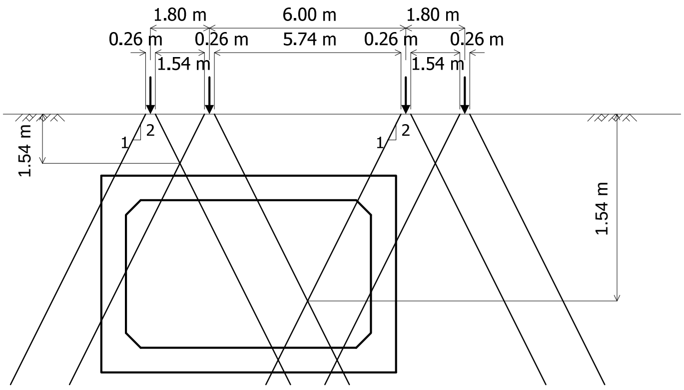


Fig. 8-5 HA UDL/KEL Application

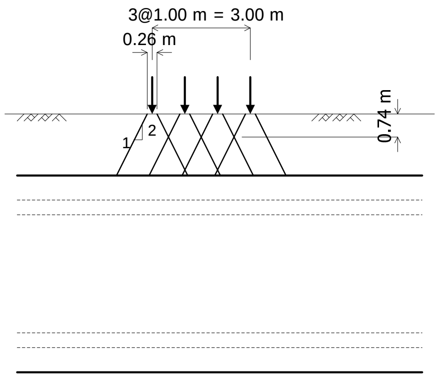
UDL = 30 kN /notional lane width /m

KEL = 120 kN/ notional lane width

For cover depth exceeding 0.60 m, 30 Units of HB loading (75 kN wheel load; 300 kN axle load) shall be used instead of HA UDL/KEL with the dispersion both longitudinally and transversely from the limits of the wheel contact area at ground level to the level of the top of the roof at a slope of 2 vertically to 1 horizontally. The wheel contact area shall be of square shape with 260 mm sides, and effective pressure is 1.1 N/mm^2.



Longitudinal Direction



Transverse Direction

Fig. 9-6 HB Loading Application

HA single wheel load

HA single wheel load of 100 kN shall be considered. This load is dispersed both longitudinally and transversely from the limits of the wheel contact area at ground level to the level of the top of the roof at a slope of 2 vertically to 1 horizontally. The wheel contact area shall be of square shape with 300 mm sides, and effective pressure is 1.1 N/mm^2.

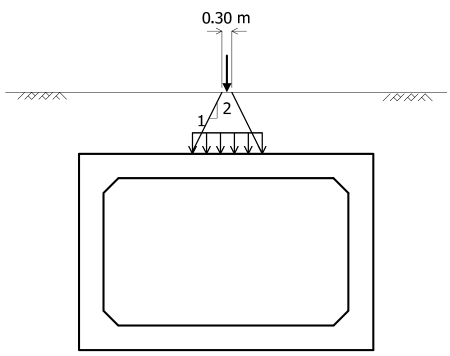


Fig. 8-7 HA Single Wheel Load Application

HB Loading

30 units of HB load shall be considered with the dispersion as described in the above HA UDL/KEL load.

In the "Bridge Design Manual 1997; RDA", it is mentioned that HB vehicle is always to be straddle two notional lane width.

Load Combination

The most onerous effect shall be considered among the two (2) combinations of "HA UDL/KEL + 30 units of HB load" and "HA single wheel load + 30 units of HB load".

It is noted that two (2) HB vehicles are loaded in parallel when the soil cover depth exceeds 0.60 m.

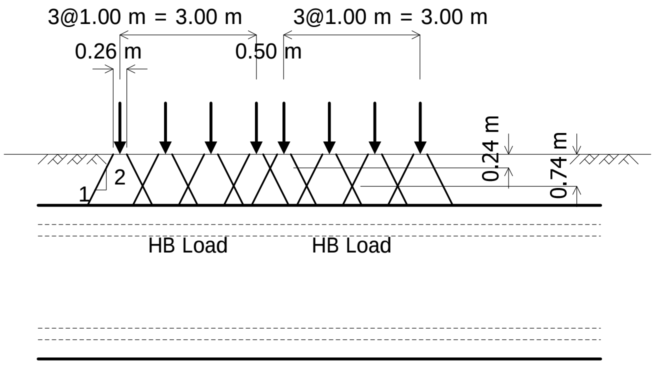


Fig. 9-8 Two (2) HB Vehicles Loaded in Parallel

9.6.7. Live Load Surcharge

A horizontal live load surcharge shall be applied in conjunction with all vertical live loads. The nominal uniform horizontal pressure (p_{sc}) to be applied to the external walls of the structure shall be determined from the following equation:

$$p_{sc} = K * v_{sc}$$

where:

K : nominal earth pressure coefficient
v_{sc} : vertical surcharge pressure applied to the external walls (kN/m^2)

Vertical Live Load	v_{sc}	
HA Loading	10.0	kN/m^2
30 Units of HB	12.5	kN/m^2
Accidental Wheel	10.0	kN/m^2
Construction	10.0	kN/m^2

The same value of nominal live load surcharge with the same partial safety factor γ_{fL} and γ_{f3} shall be applied simultaneously to both external walls except for Combination-4 and calculation of maximum bearing pressure.

The values of γ_{fL} and γ_{f3} are the same with those of earth pressure. Detail of application of γ_{fL} and γ_{f3} are described in **Fig. 3.3.26**.

9.6.8. Traction

The traction force shall be applied directly to the roof of the structure over the following widths measured perpendicular to the direction of traffic.

HA traction : Notional carriageway lane width (3.5625 m)
HB traction : A width equal to 3.00 m + C (0.26 m) = 3.26 m

All traction forces shall be multiplied by Kt in consideration of cover (H).

$$Kt = (L_L - H) / (L_L - 0.6) \quad \text{but} \quad 0 \leq Kt \leq 1.0$$

The intensity of traction forces are as follows:

Nominal traction load for Type HA : $8\ L + 200\ kN \leq 700\ kN$
L = loaded length = L_L (m)
Nominal traction load for Type HB : 25% of total nominal HB load (equally distributed between 8 wheels of 2 axles)

Nominal HA or HB load shall be considered to act with traction load as appropriate.

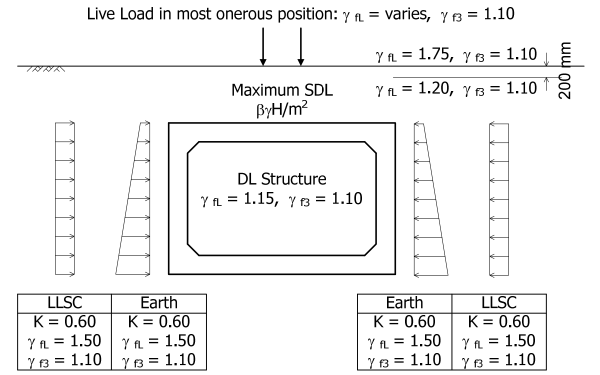


Diagram A/1a

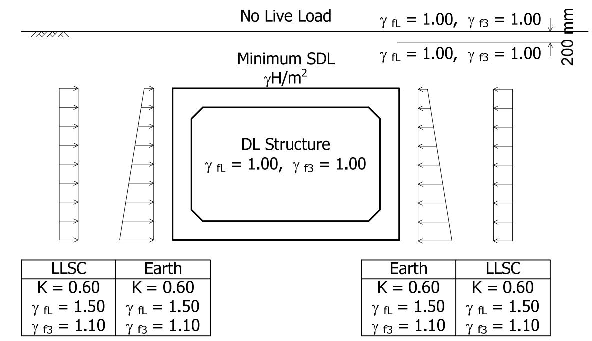


Diagram A/2a

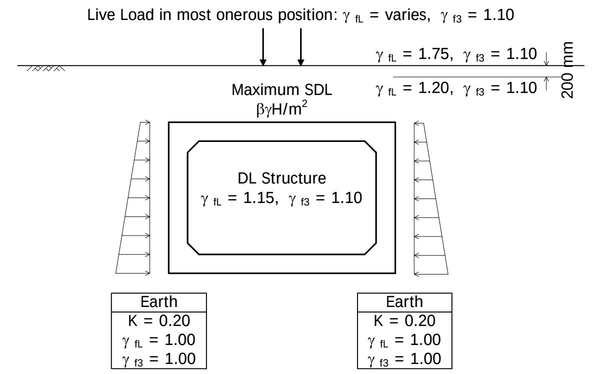


Diagram A/3a

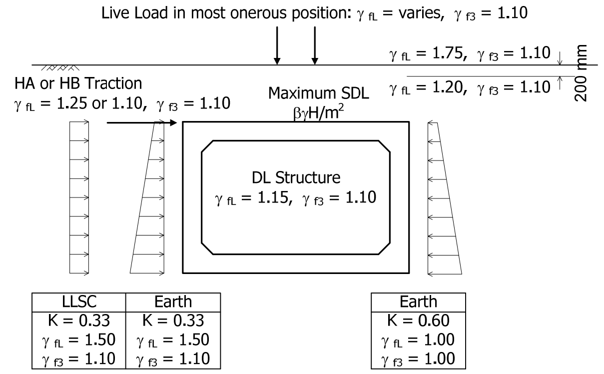


Diagram A/4a

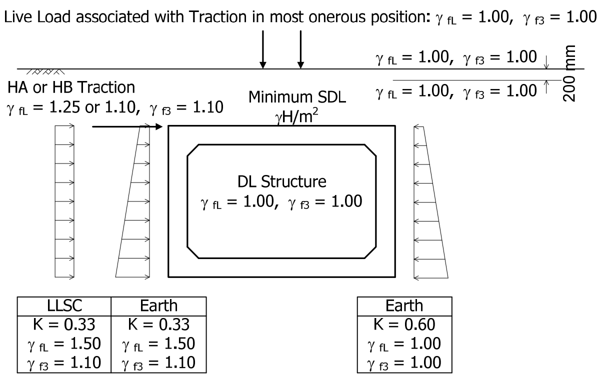


Diagram A/5a

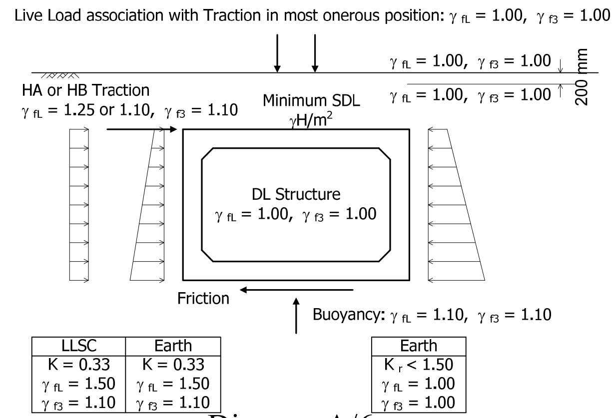


Diagram A/6a

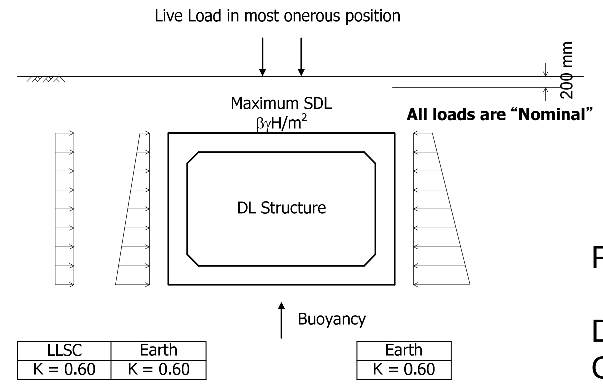


Diagram A/7a

Fig. 9-9

Diagram showing Load Cases to be Considered for Earth Pressure

9.7. Materials

9.7.1. Concrete

Concrete used for box structure shall be of Grade 30 normal-weight concrete with characteristic cube strength (fcu) of 30 MPa at 28 days and its modulus of elasticity (Ec) is 28 kN/mm^2.

Poisson's ratio is taken as 0.20 and coefficient of thermal expansion is 12 * 10^{-6}.

According to the experience/practice in Sri Lanka, maximum size of coarse aggregate is 20 mm.

9.7.2. Reinforcing Steel Bars

Reinforcing bar shall be of Grade 460 with yield strength (fy) of 460 MPa and be of deformed bar Type- 2 for main bars and round bar for ties. Preferred nominal diameters together with cross sectional area and mass used are shown in Table 3.1.6. Modulus of elasticity (Es) is 200 kN/mm^2.

1. Strength Requirement

Requirement	Unit	Grade 250	Grade 460
Yield Strength	MPa	250	460
Bar Type		Not Applicable in OCH	Deformed Bar, Type-2

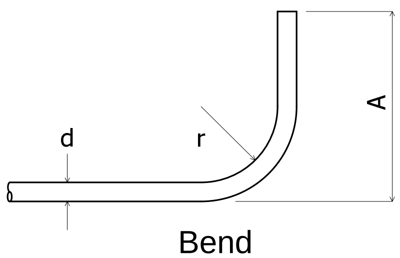
Bond Classification Requirement

Bond classification specificed in 5.8.6 IN BS5400-4:1990 shall be "Type-2". To ensure this classification, performance test described in Annex-D of BS4449:1997 shall be conducted.

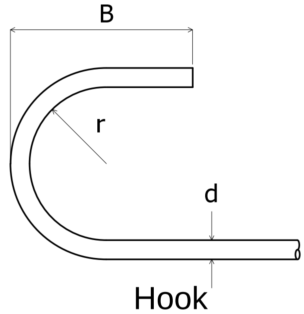
Table 9-6 Reinforcing Steel for Deformed Bar (Type-2)

Bar Designation	Nominal Diameter (mm)	Nominal Mass (kg/m)	Peripheral Length (mm)	Cross Sectional Area (mm^2)
8	8.0	0.395	25.1	50.3
10	10.0	0.616	31.4	78.5
12	12.0	0.888	37.7	113.1
16	16.0	1.579	50.3	201.1
20	20.0	2.466	62.8	314.2
25	25.0	3.854	78.5	490.9
32	32.0	6.313	100.5	804.2

2. Bend and Hook [BS 8666-2000]



Bend



Hook

<u>Minimum bend and hook dimensions are in accordance with Table-3 of BS 4466-1989.</u>

Table 9-7 Minimum bend and hook dimension

Bar Designation d	Grade 460, Type-2							
	Bend				Hook			
	Radius		Extension		Radius		Extension	
	r	r/d	A	A/d	r	r/d	B	B/d
8	16	2.0	115	14.4	16	2.0	115	14.4
10	20	2.0	120	12.0	20	2.0	120	12.0
12	24	2.0	125	10.4	24	2.0	125	10.4
16	32	2.0	130	8.1	32	2.0	130	8.1
20	70	3.5	100	5.0	70	3.5	100	5.0
25	87	3.5	240	9.6	87	3.5	240	9.6
32	112	3.5	305	9.5	112	3.5	305	9.5

3. Anchorage length in tension/compression

$$L\,db = As \times \underbrace{0.87 \times Fy}_{\text{(Design Strength)}} / \phi / t\,c$$

where:

As = area oh reinforcing bar (mm2)
Fy = specified yield strength of reinforcing bars 460 (MPa)
τ c = ultimate anchorahe bond stress as specified below (MPa)
ϕ = peripheral length of bar (mm)

Table 9-8 Ultimate Anchorage Bond Stress τ c (N/mm2)

Bar Type	Concrete Grade			
	20	25	30	40 or more
Deformed Bar Type-2 in Tension	2.20	2.50	2.80	3.30
Deformed Bar Type-2 in Compression	2.70	3.10	3.50	4.10

Table 9-9 Effectove Anchorage Length Ldb (mm)

Bar Designation			Concrete Grade							
			20		25		30		40 or more	
Deformed Bar, Type-2	8	Tension	370	(46d)	330	(41d)	290	(36d)	250	(31d)
		Compression	300	(38d)	260	(33d)	230	(29d)	200	(25d)
	10	Tension	460	(46d)	400	(40d)	360	(36d)	310	(31d)
		Compression	380	(38d)	330	(33d)	290	(29d)	250	(25d)
	12	Tension	550	(46d)	490	(41d)	430	(36d)	370	(31d)
		Compression	450	(38d)	390	(33d)	350	(29d)	300	(25d)
	16	Tension	730	(46d)	650	(41d)	580	(36d)	490	(31d)
		Compression	600	(38d)	520	(33d)	460	(29d)	400	(25d)
	20	Tension	910	(46d)	810	(41d)	720	(36d)	610	(31d)
		Compression	750	(38d)	650	(33d)	580	(29d)	490	(25d)
	25	Tension	1140	(46d)	1010	(40d)	900	(36d)	760	(30d)
		Compression	930	(37d)	810	(32d)	720	(29d)	620	(25d)
	32	Tension	1460	(46d)	1290	(40d)	1150	(36d)	980	(31d)
		Compression	1190	(37d)	1040	(33d)	920	(29d)	790	(25d)

4. Lap Lengths

When bars are lapped, the length of the lap shall be at least equal the anchorage length required to develop the stress in the smaller of the two bars lapped.

However, the following requirements are also to be considered.

- 25 times the smaller bar diameter plus 150 mm in tension.
- 20 times the smaller bar diameter plus 150 mm in compression.

Table 9-10 Lap Length (mm)

Bar Designation			Concrete Grade							
			20		25		30		40 or more	
Deformed Bar, Type-2	8	Tension	370	(46d)	350	(44d)	350	(44d)	350	(44d)
		Compression	310	(39d)	310	(39d)	310	(39d)	310	(39d)
	10	Tension	460	(46d)	400	(40d)	400	(40d)	400	(40d)
		Compression	380	(38d)	350	(35d)	350	(35d)	350	(35d)
	12	Tension	550	(46d)	490	(41d)	450	(38d)	450	(38d)
		Compression	450	(38d)	390	(33d)	390	(33d)	390	(33d)
	16	Tension	730	(46d)	650	(41d)	580	(36d)	550	(34d)
		Compression	600	(38d)	520	(33d)	470	(29d)	470	(29d)
	20	Tension	910	(46d)	810	(41d)	720	(36d)	650	(33d)
		Compression	750	(38d)	650	(33d)	580	(29d)	550	(28d)
	25	Tension	1140	(46d)	1010	(40d)	900	(36d)	780	(31d)
		Compression	930	(37d)	810	(32d)	720	(29d)	650	(26d)

	32	Tension	1460	(46d)	1290	(40d)	115 0	(36d)	980	(31d)
		Compression	1190	(37d)	1040	(33d)	920	(29d)	790	(25d)

In addition, the lap length above shall be increased by the following factors:

Table 9-11 Factor to be increased

	Description	m
a)	the nominal cover to lapped bars from top of the section is less than twice the bar size.	1.40
b)	clear distance between the lap and another pair of lapped bars is less than 150 mm.	
c)	a cornar bar is being lapped and the nominal cover to either face is less than the twice the bar size.	
-	both a) and b) are satisfied	2.00
-	both a) and c) are satisfied	

Table 9-12 Reinforcement Requirments [BS 5400-4:1990, 5.8.4 - 5.8.5]

Requirement	Beam/Slab	Column	Wall
Minimum area of main reinforcement	- $0.0015\ b_a\ d$ (Grade 460) $0.0025\ b_a\ d$ (Grade 250) b_a : width of section d : effective depth to tension reinforcement	- Minimum number of longitudinal bars is 4 in rectangular column - Minimum number of longitudinal bars is 6 in circular column - Diameter shall not be smaller than 12mm - 0.01 of cross-sectional area	- 0.004 of gross cross-sectional area
Minimum area of secondary reinforcement	Solid Slab - $0.0012\ b_t\ d$ (Grade 460) $0.0015\ b_t\ d$ (Grade 250) b_a : width of section d : effective depth to tension reinforcement - Diameter shall be not less than 1/4 of main bar with horizontal spacing not exceeding 300 mm.		- $0.0012\ b_t\ d$ (Grade 460) $0.0015\ b_t\ d$ (Grade 250) b_a : width of section d : effective depth to tension reinforcement - Diameter shall be not less than 1/4 of main bar with horizontal spacing not exceeding 300 mm.
Maximum area of reinforcement	- 0.04 of gross cross-sectional area for both tension and compression reinforcement.	- 0.06 of gross cross-sectional area for vertically cast - 0.08 of gross cross-sectional area for horizontally cast	- 0.04 of gross cross-sectional area.
Minimum center-to-center Spacing	- 100 mm (RDA Practice)		

Requirement	Beam/Slab	Column	Wall
Minimum area of link or tie (main bar required to resist compression)	- Link/tie shall have the bar size shall be 1/4 of the largest compression bar size at a maximum spacing of 12 times the smallest compression bar size. - When the reinforcement percentage of compression face exceeds 1%, link/tie shall have at least 6mm or 1/4 of the largest bar size, whichever is the greater, and be provided through the thickness of the member. The spacing shall not exceed twice the member thickness and shall not be greater than 16 times of bar size in compression size. - The spacing of links shall not exceed 0.75 times the effective depth of the beam, nor shall the lateral spacing of individual legs of the links exceed this - In all beams, shear reinforceemnt shall be provided through the span.	- Link/tie shall have the bar size shall be 1/4 of the largest compression bar size at a maximum spacing of 12 times the smallest compression bar size. - When the reinforcement percentage of compression face exceeds 1%, link/tie shall have at least 6mm or 1/4 of the largest bar size, whichever is the greater, and be provided through the thickness of the member. The spacing shall not exceed twice the member thickness and shall not be greater than 16 times of bar size in compression size.	- N/A
Reinforcement for Shrinkage and Temperature	- Area of reinforcement As for shrinkage and temperature shall be as follows: As ? kr * (Ac - 0.5 * Acor) where: kr : 0.005 for Grade 460 reinforcement, 0.006 for Grade 250 reinforcement Ac : area of gross concrete section at right angle to the direction of restraint. Acor : area of core of the concrete section Ac, namely the portion of the section more than 250 mm away from all concrete surface - Reinforcement shall be distributed uniformly around the perimeter of the concrete section and spaced at not more than 150 mm.		

Concrete Cover to Reinforcement

Concrete nominal cover to the outermost reinforcement surface shall be as shown in **Table 9-13** from BS5400-4:1990 together with the following modifications:

- Nominal cover to reinforcement for cast in-situ box culvert shall be increased by 10 mm to the values in Table 3.1.7.
- Where the concrete is cast directly against the ground (as opposed to on blinding), the nominal cover shall be increased to 40 mm to the values in **Table 9-13**.
- For cast in-situ concrete, where the surface is subject to flowing water, the nominal cover shall be increased by a further 10 mm to allow for erosion.

However, from the RDA practice, 75 mm clear cover shall be applied conservatively for all elements on both road and drainage culverts.

Table 9-13 Nominal cover to reinforcement under particular conditions of exposure (Cast-in-place structures)

Environment	Examples	Nominal cover [a] (mm) Concrete grade (MPa) 25	30	40	50 and over
Extreme Concrete surfaces exposed to: abrasive action by sea water or water with a pH ≦ 4.5	Marine structures Parts of structure in contact with moorland water	[b]	[b]	65 [c]	55
Very severe Concrete surfaces directly affected by: de-icing salts or sea water spray	Walls and structure supports adjacent to the carriageway Parapet edge beams Concrete adjacent to the sea	[b]	[d]	50 [c]	40
Severe Concrete surfaces exposed to: driving rain or alternate wetting and drying	Walls and structure supports remote from the carriageway Bridge deck soffits Buried parts of structures	[b]	45 [c]	35	30
Moderate Concrete surfaces above ground level and fully sheltered against all of the followings: rain, de-icing salts, sea water spray Concrete surfaces permanently saturated by water with a pH > 4.5	Surface protected by bridge deck water-proofing or by permanent formwork Interior surface of pedestrian subways, voided superstructures or cellular abutments Concrete permanently under water	45	35	30	25

a Actual cover may be up to 5 mm less than nominal cover (see Part 7)
b Concrete grade not permitted.
c Air entrained concrete should be specified where the surface is liable to freezing whilst wet (see Part 7)
d For parapet beams only grade 30 concrete is permitted provided that it is air entrained and the nominal cover is 60 mm.

9.7.3. Values of γ m

Values of γ m for serviceability limit state (SLS) and ultimate limit state (ULS) are listed in **Table 9-14**.

Table 9-14 Values of r m

rial	of stress	SLS	ULS
rete	ngular or near-triangular pressive stress distribution due to bending)	1.00	1.50
	orm or near uniform pressive stress distribution due to axial loading)	1.33	1.50
	ion	N/A	N/A
forcement	pression	1.00	1.15
	ion	1.00	1.15

9.7.4. Backfill Material

On both sides of the wall, granular backfill material shall be used. Granular backfill material has the following properties:

Soil Unit Weight: γ = 19 kN/m^3
Angle of Internal Friction: ϕ = 30 degree

9.8. Design Requirements

9.8.1. Structural Elements

The checking items for both limit states are:

SLS - Crack width is within the requirement limit
- Stress limitation
- Defection of roof slab

ULS - Rupture

Crack Width

Cracking of concrete should not adversely affect the appearance or durability of the structure. Under the severe condition, crack width of box culvert shall be equal to or less than 0.25 mm from BS5400-4:1990.

Stress Limitation

To prevent unacceptable deformations occurring, compressive stresses in concrete and stress in steel calculated by linear elastic analysis shall be limited to the values in **Table 9-15**.

Table 9-15 Stress Limitations for Serviceability Limit State

Material	Type of stress	Reinforced Concrete
Concrete	Triangular or near-triangular compressive stress distribution (e.g. due to bending)	0.50 Fcu
	Uniform or near uniform compressive stress distribution (e.g. due to axial loading)	0.38 Fcu
Reinforcement	Compression	0.75 Fy
	Tension	0.75 Fy

<u>Deflection on Roof Slab</u>

Under the serviceability limit state, deflection on roof slab shall not affect to the required vertical clearance in the box structure.

Provision of additional vertical clearance in determining the box structure dimensions may be one of the countermeasures.

<u>Rupture</u>

The assessment of the structure under the design loads should ensure that prior collapse does not occur as a result of rupture of one or more critical sections or buckling. The following equation shall be satisfied:

$$S^* \leq R^*$$

where:

Q* : Design loads expressed as the product of nominal loads (Q_k) and the partial safety factor γ_{fL}

S* : Design load effects expressed as the effects of the product of the design loads (Q*) and the partial safety factor γ_{f3}

R* : Design resistance expressed as the nominal strength of the component (f_k) divided by the partial safety factor γ_m, = function (f_k / γ_m)

f_k : Nominal strength of material

9.8.2. Foundation

The structure as a whole may have the possibility to fail due to overloading of soil-structure interface or excessive soil deformation, even though structure elements themselves are designed to their requirements. In order to prevent such failures, two situations of “Sliding” and “Bearing Failure” shall be investigated.

Limit states to be investigated for both failure modes are as follows:

Failure Mode	SLS (Serviceability Limit State)	ULS (Ultimate Limit State)
Sliding	-	To be studied
Bearing Failure	To be studied	-

Sliding

The following relationship shall be satisfied for the box culvert structure:

(Traction + Active Earth Pressure) * γ_{fL} * γ_{f3} < (Passive Earth Pressure + F_R)

where:

F_R	=	V_{tot} * tan δ_b
V_{tot}	=	total applied vertical force at base of culvert due to permanent loads less any uplift due to Combination 4 loading and buoyancy.
δ_b	=	design angle of base friction (tan δ_b = 0.75 tan ϕ')
ϕ'	=	effective internal friction angle

Bearing Capacity

Ultimate bearing capacity q_{ult} on foundation ground is calculated by Terzaghi theory (refer to Foundation Analysis & Design, 5th Edition, Joseph E. Bowels). Developed pressure at base of culvert shall not exceed the value q_{ult} / SF.

q_{ult} = c' * Nc + γ1' * z * Nq + 0.5 * γ2' * B * Nγ

where:

c'	=	cohesive strength of foundation soil	
z	=	soil cover depth from the base of culvert	
B	=	culvert width	
γ1'	=	unit weight of soil above the base of culvert (soil cover)	
γ2'	=	unit weight of foundation soil	
Nc	=	$(N_q-1) * \cot\phi$	: Bearing Capacity Factor
Nq	=	$a^2 / 2 / \cos^2 (45 + \phi / 2)$	: Bearing Capacity Factor
a	=	$\exp [(0.75 \pi - \phi / 2) \tan\phi]$	
Nγ	=	$\tan\phi / 2 * (Kp\gamma / \cos\phi - 1)$	: Bearing Capacity Factor
Kpγ	=	Refer to the above book (Joseph E. Bowels)	
ϕ	=	internal friction angle of foundation soil	

SF = 3 (Safety Factor)

9.9. Design Method for Box Culvert

9.9.1. Box Structure

Design of box culvert is based on a meter width, and 2D computer analysis approach is adopted.

9.9.2. Parallel Wing Wall

Active earth pressure (including live load surcharge) acting to the wing wall was analyzed by the Sliding Plate Method. Passive earth pressure was not taken into account.

Segments are assumed whose width is ⊿x (10cm was adopted in the design). And, earth pressure for the each segment is calculated (see **Fig. 3.3.27**).

Moment force at the joint is estimated as a sum of the earth pressure of each segment multiplied by each lever arm length.

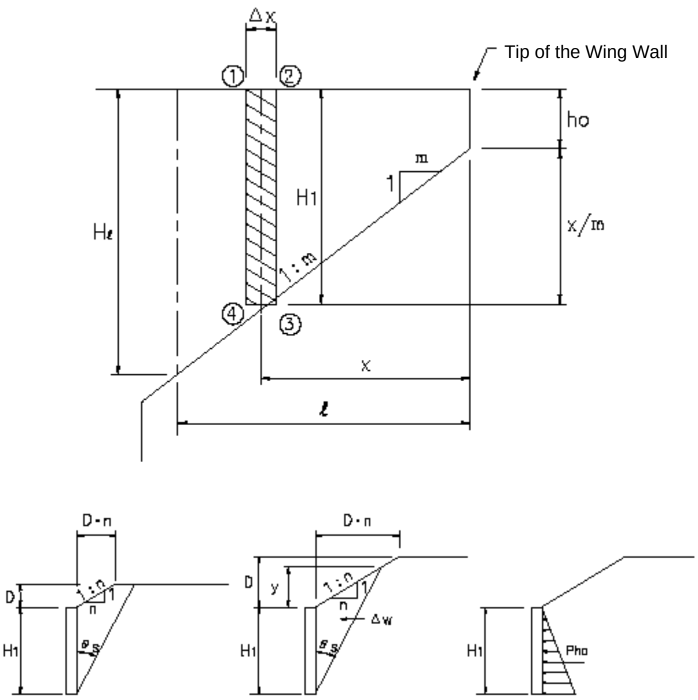


Fig. 9-10 Segment of Wing Wall

10. PIPE CULVERT

10. Pipe Culvert

10.1. Required Dimensions

Inner diameters of pipe culverts were determined in terms of drainage design as mentioned in “Chapter-4 Drainage”.

Wall thickness is in general determined by the internal diameter of pipe.

10.2. Design Criteria

10.2.1. Structural Design Standard and Manual

- British Standard BS 5911;
- Design Manual for Roads and Bridges (British Highway Agency, 1989);
- A Design Manual for Small Bridges (Transport and Road Research Laboratory Overseas Unit; UK);
- A Guide to Design Loadings for Buried Rigid Pipes (Transport and Road Research Laboratory Overseas Unit; UK)

11. RETAINING WALL

11. Retaining Wall

11.1. Design Standard

- British Standard: “BS 5400: Part4” and “BS 8110”
- British Standard: BS 8002
- British Standard: BS 8004
- Design Manual for Roads and Bridges: British Highway Agency, 1989

11.2. Design Principles

The structure and surrounding soil are designed to perform satisfactorily for both the ultimate and serviceability limit.

Ultimate Limit State

This limit state corresponds with the failure of structural elements or the foundation beneath the footing base, and is as defined in BS 5400 (Part 4) for concrete walls.

Serviceability Limit State

This limit state corresponds with the overall stability and the acceptable limits of cracking as described in BS 5400 (Part 4) for concrete walls.

Partial Safety Factors for Loads

The partial safety factors γ_{fL} and γ_{f3} are set based on BS 5400 Part2.

11.3. Loads

11.3.1. Permanent Load

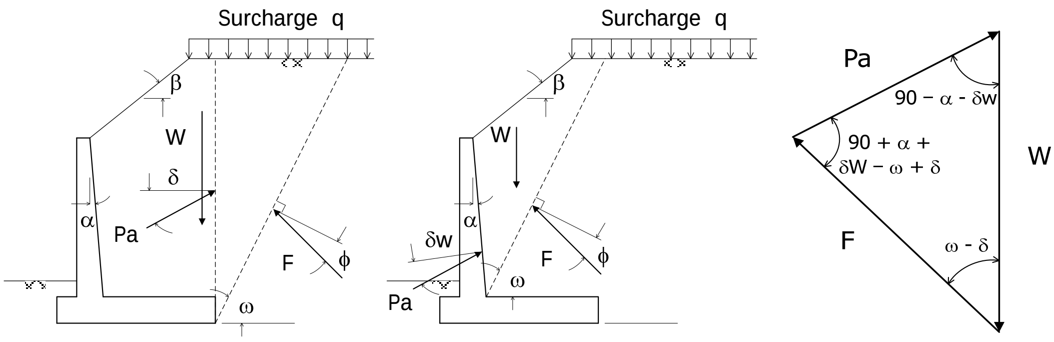
The following unit weights of materials are used for permanent load calculation:

Reinforced concrete	:	γc	=	25.0 kN/m^3
Compacted soil	:	γs	=	19.0 kN/m^3
Water	:	γw	=	10.0 kN/m^3

11.3.2. Earth Pressure

Sliding Plate Method by the graphical procedure is adopted in order to calculate earth pressure, since the retained ground surface is not flat (irregular).

It is to find out the maximum active earth pressure as the sliding plate angle (ω) from the horizontal line varies.



Stability Check | Structure Design | Load Equilibrium

Fig. 11-1 Graphical Determination of Active Earth Pressure for Cohesionless Soils

where:

α	:	angle of wall slope (deg)
β	:	slope angle of backfill (deg)
δ	:	wall friction angle for stability calculation (deg) = ϕ (deg)
ϕ	:	internal friction angle of backfill (deg)
δw	:	wall friction angle for structural element calculation (deg) = 0 (deg)
ω	:	assumed sliding plate angle from the horizontal surface (deg)

11.3.3. Live Load Surcharge

Distributed horizontal/vertical surcharge force due to HA live load is considered in the design.

$$q = 10 \text{ kN/m}^2$$

11.3.4. Effect of Water (Buoyancy)

Conditions with and without water are considered in confirming stability and in carrying out stress analysis. The water level adopted is both the annual flood level (HWL) and footing base (LHL).

11.4. Design Condition

11.4.1. Back Fill

Soil Unit Weight:	γ	=	19 kN/m^3
Angle of Internal Friction:	ϕ	=	30 degree
Cohesive Strength	c	=	0 kN/m^2

11.4.2. Material Properties

<u>Gravity Concrete Retaining Wall</u>

Characteristic Strength of Wall Concrete:	fcu	=	15 MPa [1:3:6 (40)]
Characteristic Strength of Base Concrete:	fcu	=	25 MPa [1:2:4 (20)]
Tension Limit in Wall Concrete:	fct	=	0.24 MPa

Reinforced Concrete Retaining Wall

Characteristic Strength of Concrete:	fcu	=	30 MPa
Characteristic Strength of Reinforcement:	fy	=	460 MPa

Cover to Steel: 75mm for bottom of base, 50mm for rests of members
Maximum Allowable Crack Width: 0.25mm

11.5. Design Requirements

11.5.1. Structural Elements

The checking items for both limit states are:

SLS	-	Crack width is within the requirement limit
	-	Stress limitation
	-	Horizontal displacement
ULS	-	Rupture

Details are described in Section 3.1 (Box Culvert).

11.5.2. Foundation

The structure as a whole may have the possibility to fail due to overloading of soil-structure interface or excessive soil deformation, even though structure elements themselves are designed to their requirements.

(1) Spread Foundation

In order to prevent such failures, three situations of “Sliding Failure”, “Bearing Failure” and “Overturning” shall be investigated at serviceability limit state (SLS).
Sliding

Developed horizontal force on the foundation base shall not exceed the base resistance to sliding divided by the safety factor (SF) of 1.5.

Base resistance to sliding (H ult) shall be calculated as follows:

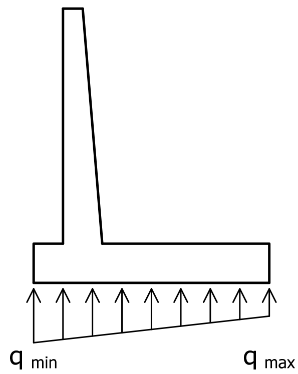
$$H\ ult = A * \sigma' * \tan \delta b$$

where:

A	=	area of base
σ'	=	effective mean normal pressure on the base due to permanent loads less buoyancy
δ_b	=	design angle of base friction ($\tan \delta_b = 0.75 \tan \phi'$)
ϕ'	=	effective internal friction angle

Bearing Failure

Bearing pressure developed shall not exceed the ultimate bearing capacity (q_{ult}) divided by the safety factor (SF) of 3. Ultimate bearing capacity shall be calculated by Terzaghi theory in consideration of inclination of the forces on the foundation.



$$q\ max,\ q\ min \leq q_{ult} / SF$$

$$q_{ult} = c' * Nc + \gamma1' * z * Nq + 0.5 * \gamma2' * B * N\gamma$$

where:

c' = cohesive strength of foundation soil

z = soil cover depth from the base of culvert

B = culvert width

γ1' = unit weight of soil above the base of culvert (soil cover)

γ2' = unit weight of foundation soil

Nc = Bearing Capacity Factor in Appendix - 1

Nq = Bearing Capacity Factor in Appendix - 1

Nγ = Bearing Capacity Factor in Appendix - 1

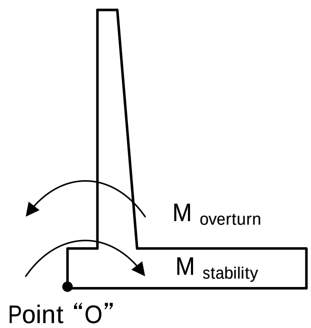
φ = internal friction angle of foundation soil

SF = 3 (Safety Factor)

tan θ = H / V (H and V are horizontal force and vertical force respectively applied on the base at SLS)

Rotation

The following relationship between Stability Moment ($M_{stabislity}$) and Overturning Moment ($M_{overturn}$) at original point "O" shall be satisfied:



$$M_{stability} / M_{overturn} \geq SF$$

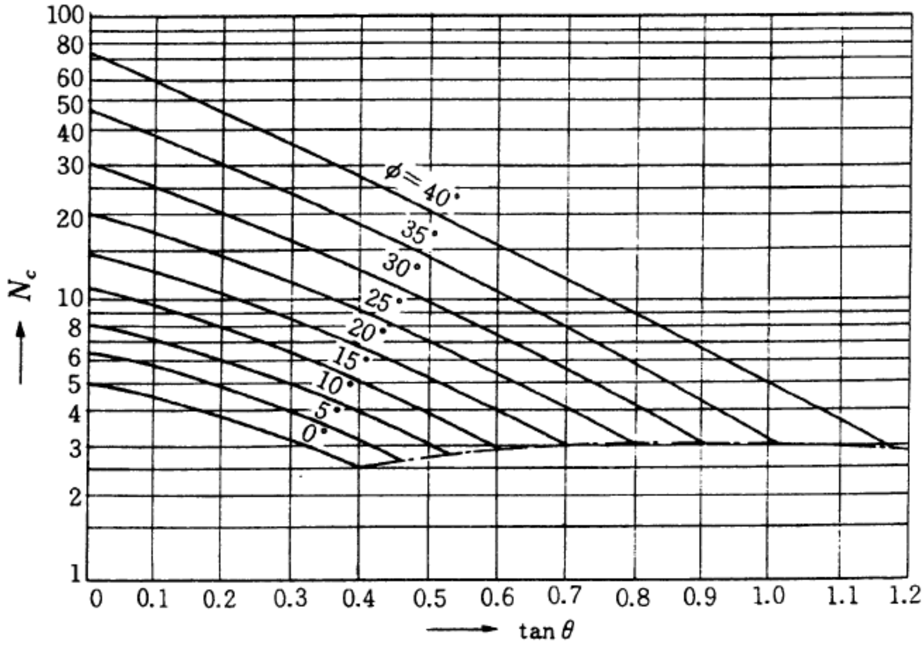
where:

SF = Safety Factor 1.50

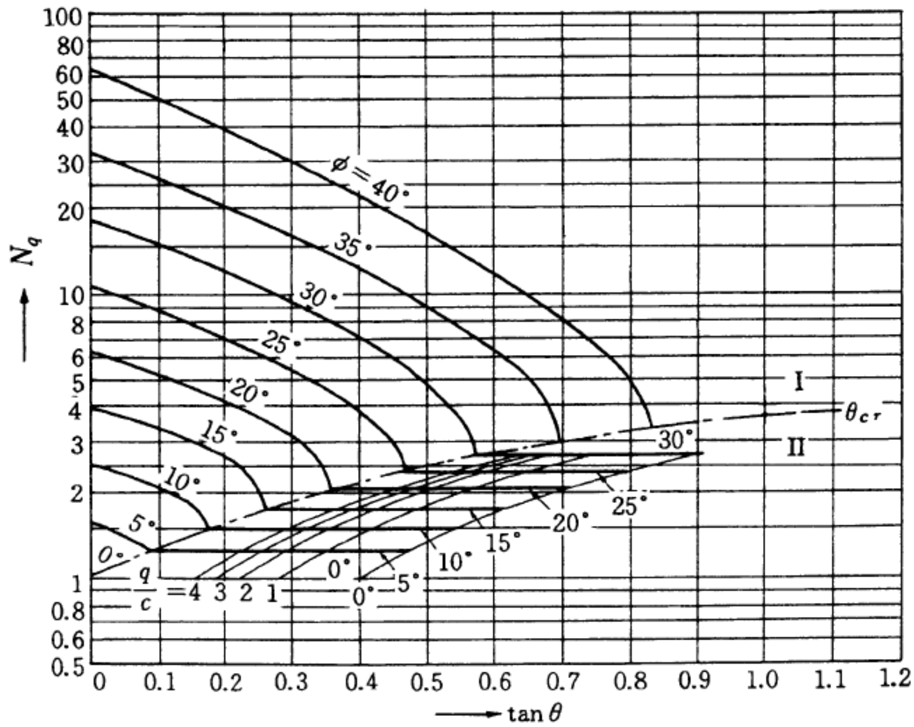
(2) Pile Foundation

Design requirements for pile foundation are described in "Bridge Design".

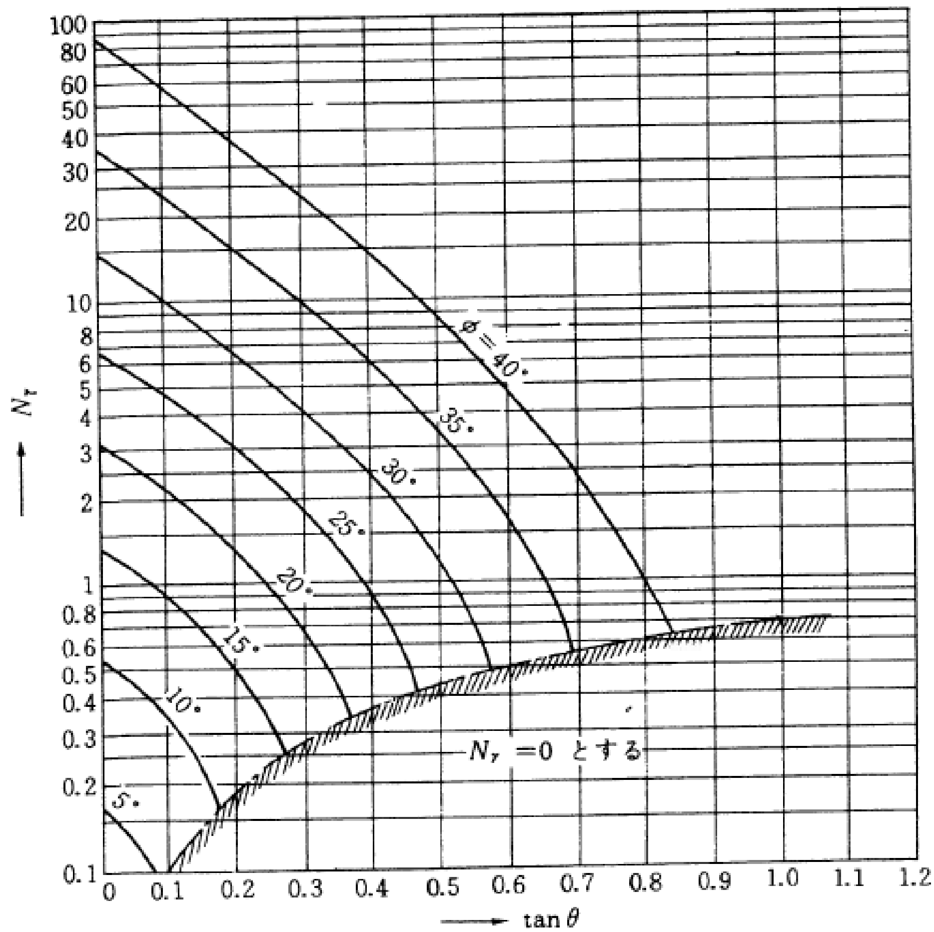
APPENDIX - 1 Bearing Capacity Factor Diagram by Terzaghi Theory in consideration of Inclination and Eccentricity of Load (Association of Japan Highway)



Bearing Capacity Factor Nc



Bearing Capacity Factor Nq



Bearing Capacity Factor Nγ

12. DRAINAGE

12. Drainage

12.1. Design Return Period

The recommended design storm return periods are given below. Also in OCH (Southern Section), the same design standard was applied with the consent of Road Development Authority.

Table 12-1 Design Storm Return Period

Type	Return Period (Year)
Bridge for Main River (Kelani)	100
Bridge for River tributary	50
Drainage Culvert for OCH, Ramps & Access Roads	50
Drainage Culvert for Crossing Minor Roads	10
Road Side Ditch/Canal*	10
Road Surface Drainage for OCH	10

*Side ditches are provided where necessary

12.2. Design Discharge

In discharge calculations, Rational method is used for minor sub catchments and Hydrological Modeling (HEC-HMS and HEC-RAS) is used for major sub catchments with waterway networks.

12.2.1 Hydrological Modeling for Waterway Networks

The Hydrologic Modeling System of Hydrologic Engineering Center (HEC-HMS), US Army Corps of Engineers, USA is used for modeling the catchment of waterway networks, to determine the discharges at required locations. The transform method used to compute direct runoff from excess precipitation is the SCS unit hydrograph technique. However, for long duration storms, which consist of several peaks, a conceptual kinematic wave model was applied.

12.2.2 Rational Method

The design discharge Q_d is calculated using rational formula as given below.

$$Q_d = \frac{1}{3.6 \times 10^6} C \cdot I \cdot A$$

where C is Coefficient of runoff
I is Intensity of storm (mm/hr)
A is Catchments area (m2)

(1) Intensity of rainfall

The intensity of rainfall derived at Colombo Meteorological Station analyzing the data from 1951 to 2000, is used in OCH design.

Table 12-2 Intensity of rainfall (unit:mm/hr)

Return Period	Duration						
(Years)	15min	30min	60min	90min	120min	150min	180min
1.002	44	37	31	23	17	13	10
1.50	83	75	52	40	32	27	23
2	93	85	58	44	36	31	26
3	**105**	**97**	**64**	**50**	**41**	**35**	**31**
5	118	110	72	56	46	40	35
10	**135**	**126**	**81**	**63**	**52**	**46**	**41**
20	151	142	90	70	59	52	46
25	157	147	93	72	61	53	48
30	161	151	95	74	62	55	50
50	**172**	**162**	**101**	**79**	**67**	**59**	**53**
70	180	170	106	83	70	62	56
100	188	178	110	86	73	65	59

(2) Time concentration

Time of concentration (*Tc*) for a catchment is the time taken for a drop of water from the hydrologically most remote of the catchment to reach the point of inlet without undue delay. The average velocity of flow in the natural water course is estimates based on the gradient of the stream as given in **Table 12-3**.

Table 12-3 Average velocity to calculate Tc

Average Gradient	**Average Velocity**	
	Ft./s	M/s
0 to <1	1.5	0.46
1 to <2	2.0	0.61
2 to <4	3.0	0.91
4 to <6	4.0	1.22
=>6	5.0	1.52

Source: Design of Irrigation Headwork

The time of concentration *Tc* is calculated as given below

$$T_c = \frac{L}{V \times 60} + 15\,\min utes$$

where L is the length of the longest water course in meters
V is the average velocity as determined earlier in meters/sec

15 minutes is added as overland flow to allow for the initial time which is the time taken for a drop of water to travel in the flat surface before it reaches a well defined water course.

(3) Coefficient of Run-off

The run-off coefficient is calculated using **Table 12-4**. The calculated run-off coefficient is increased by 10% considering future urbanization. Further, SLLRDC recommended values are also taken into consideration.

Table 12-4 Runoff coefficient

Symbol	Feature	Description		Contributory Factor
Cs	Average slope of Catchment	< 3.5%	flat	0.05
		3.5% - 10%	flat to moderate	0.10
		10% - 25%	rolling	0.15
		25% - 35%	hilly	0.20
		> 35%	mountainous	0.25
Cp	Permeability of soil	Well drained soil e.g sand and gravel		0.05
		Fair drained soil e.g sand and gravel with fines		0.10
		Poorly drained soil e.g silt		0.15
		Imprevious soil e.g clay, organic silt and clay		0.25
		Water-logged black cotton soil		0.50
		Rock		0.40
Cv	Vegetation	Dense forest / thick bush		0.05
		Sparse forest / dense grass		0.10
		grassland / scrub		0.15
		cultivation		0.20
		sparse grassland		0.25
		barren		0.30

Note: 1. For contoured cultivated land C=0.6×(Cs + Cp + Cv)
2. For lakes, swamps and reservoirs C=1.0
3. For road surface and embankment/cut slope C=0.9

12.3. Channel / Culvert Design

In designing the channel dimensions, Manning's Formula is used for minor sub catchments and Hydraulic Modeling (HEC-RAS) is used for major sub catchments with waterway networks.

12.3.1. Hydraulic Modeling for Waterway Networks

The River Analysis System of Hydrologic Engineering Center (HEC-RAS) contains 1-dimensional hydraulic analysis and is used to compute the hydraulic parameters and water surface profiles. In the modeling, peak discharges computed by HEC-HMS modeling is used for upstream boundary conditions and lateral flow discharges. (Details are given in Appendix A.7)

The HEC-RAS model was applied for major sub catchments to compute the local water level of waterway and to determine the required dimensions of drainage provisions while minimizing the backwater effects, taking into the consideration the comments made by Sri Lanka Land Reclamation and Development Corporation, (SLLRDC).

12.3.2. Manning's Equation

The flow rate at any point along the channel is calculated using the Manning's resistance equation:

$$Q_a = \frac{1}{n} A R^{2/3} S^{1/2}$$

where Q_a is the discharge
n is the Manning roughness coefficient
A is the cross-sectional area of flow
R is the hydraulic radius (A/P)
P is the wetted perimeter
S is the longitudinal gradient of the channel

(1) Manning Roughness Coefficient

(a) Roughness coefficient for culvert barrel

Table 12-5 Roughness *n* for culvert [1]

Type of barrel	Range	Adopted
Concrete pipe (Good joints, smooth finished walls)	0.011～0.013	0.013
Concrete box (Good joints, smooth finished walls)	0.012～0.015	0.015

(b) Roughness coefficient for natural channel

Table 12-6 Roughness *n* for natural channel [1]

Type of channel	Range	Adopted
Natural streams (top width at flood stage < 30m)		
Clean, straight, full stage, no rifts or deep pools	0.025～0.033	0.030
Same as above but more stones and weeds	0.030～0.040	0.035
Clean, widning, some pools and shoals	0.033～0.045	0.040
Same as above but some weeds and stones	0.035～0.050	0.045
Same as sbove butlower stages, more ineffective slopes sections	0.040～0.055	0.048
Same as above but more stones	0.045～0.060	0.050
Sluggish reaches. Weedy deep pools	0.050～0.080	0.070
Very weedy reaches,deep pools, or floodways with heavy stand of timber and underbrush	0.075～0.150	0.100
Flood plains		
Cultivated area (Mature field crops)	0.03～0.05	0.040
Brush (Light brush and trees in summer)	0.04～0.08	0.060
(Medium to dense brush in summer)	0.07～0.16	0.100

[1]Source: Culvert Design Guide (Construction Industry Research and Information association (London))

(c) Roughness coefficient for new channel

Table 12-7 Roughness *n* for new channel

Type of material	Range	Adopted
Concrete (smooth)	0.011～0.015	0.015
Asphalt (smooth)	0.013	0.013
Stone masonry	0.017～0.030	0.025
Earth ditch	0.016～0.025	0.022

Source: Guideline for Drainage Facilities (Japan Road association)

(d) Freeboard

Generally, minimum freeboard adopted in culvert design is 0.5m. However, in case of main rivers or tributaries, following dimensions were applied.

Table 12-8 Free Board

Design discharges Qd (m^3/s)	<200	>=200 <500	>=500 <2,000	>=2,000 <5,000	>=5,000 <10,000	>=10,000
Freeboard (m)	0.6	0.8	1	1.2	1.5	2

Source: River Design Standard in Japan

12.4. Design Procedure for Drainage Culverts

The design procedure adopted according to the “Culvert Design guide (Construction Industry Research and Information association (London))” and free flow design is basically used.

12.4.1. Flow chart for design of drainage culverts

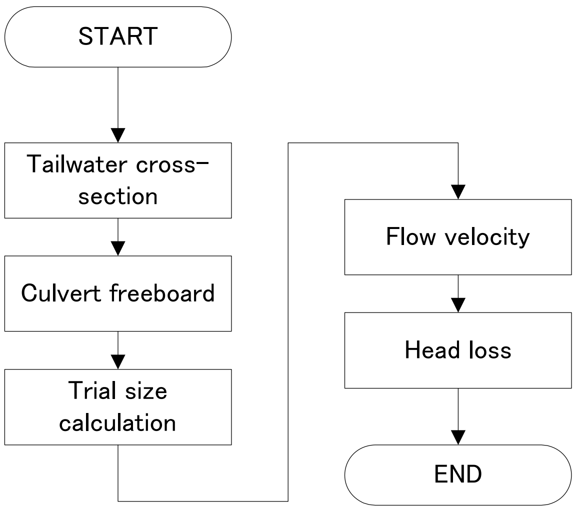


Fig.12-1 Flow chart of design procedure for drainage culverts

12.4.2. Culvert freeboard (for free flow design)

Freeboard for Box culverts adopted is 0.5m.

12.4.3. Trial size calculation

The initial trial size was selected according to the cross-sectional area and slope of the existing channel. Then, in order to minimize the impact on existing watercourse, tail water depth, headloss and standard freeboard is used in design.

12.4.4. Flow velocity in the culvert barrel

The maximum velocity in the culvert barrel can be approximated as given below.

$$V = Q / A$$

Velocity exceeding 2.0m/s will require special attention for detailing the outlet to avoid scour in the channel. If V is less than 0.75m/s the trial size should ideally be reduced until V is greater than 0.75m/s.

12.4.5. Head loss through culvert

The head loss (h_1) that occurs in the culvert inlet, barrel and outlet can be approximated for this design using:

$$h_1 = \frac{n^2 Q^2 L P^{4/3}}{A^{10/3}} + 1.5 \frac{V^2}{2g}$$

where n is the Manning's roughness for the proposed culvert barrel
L is the length of the culvert
P is the wetted perimeter based in the depth of flow at the culvert outlet
A is the cross-sectional area of flow in the barrel at the outlet

The approximate headwater elevation is calculated using:

$$HWL = TWL + h_1$$

where HWL is the headwater elevation at the inlet
TWL is the tail-water elevation ($= TW + IL_0$)

In this design, the maximum permissible HWL is taken as the existing HFL. If the calculated HWL is greater than HFL, then larger size culvert is selected for the design.

12.4.6. Dimensions of Drainage Provisions

Drainage cross-sectional dimensions should be decided to satisfy the following relation.

$$Q_a \geqq Q_d$$

where Q_a is design flow rate (80% of full-flow)
Q_d is discharge in m3/s