4.8. Frontage Road

Frontage roads are used to control access to the arterial to function as a street facility serving adjoining property and to maintain circulation of traffic on each side of the arterial. They segregate local traffic from the higher speed through traffic and intercept driveways of residences and commercial establishments along the highway.

In urban areas, a minimum spacing of about 50m between the arterial and the frontage road is desirable.

Since all frontage roads on this project are Class ‘D’ and ‘E’ type roads, a design width of 6.0 meter is considered and exceptionally 4.0 meter is applied in case of securing the same capacity of existing road and avoiding land acquisition on the resident area for the frontage road.

Details of the frontage roads are as shown in **Table 4.8.1**.

Table 4.8.1 List of Frontage Roads

No.	L/R Side	Station		Distance	Width
		From	To		
1	R	00+136(R1)	00+363.785(R2)	227.785m	6.0m
2	R	08+970	09+080	110m	6.0m
3	L	08+808.486	09+020	211.514m	6.0m
4	L	09+550	09+700	150m	6.0m
5	L	09+945	10+222	277m	6.0m
6	R	09+960	10+185	225m	6.0m
7	R	10+200	10+420	220m	4.0m
8	L	10+918	11+030	112m	6.0m
9	R	11+260	11+610	350m	6.0m
10	L	12+490	12+520	30m	6.0m
11	R	12+520	12+725	205m	6.0m
12	L	12+590	12+870	280m	6.0m

4.9. Earthwork

(1) General

The design standards of earthworks including the slope ratio of cut and embankment hereunder have been prepared taking into account of the result of the geotechnical survey and the Japanese Standard. Also, the local practice particularly other relevant expressway projects in Sri Lanka is referred.

(2) Excavation (Cutting)

1) Cut Slope

The residual soil (namely "Laterite") is able to be observed almost in project area as shown in **Fig. 4.9.1**. In the hills along the OCH, there are cuts and quarries where the bedrock outcrops. According to the soil investigation survey, the ground is generally covered by the reddish-colored laterite (weathered soil) with 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.

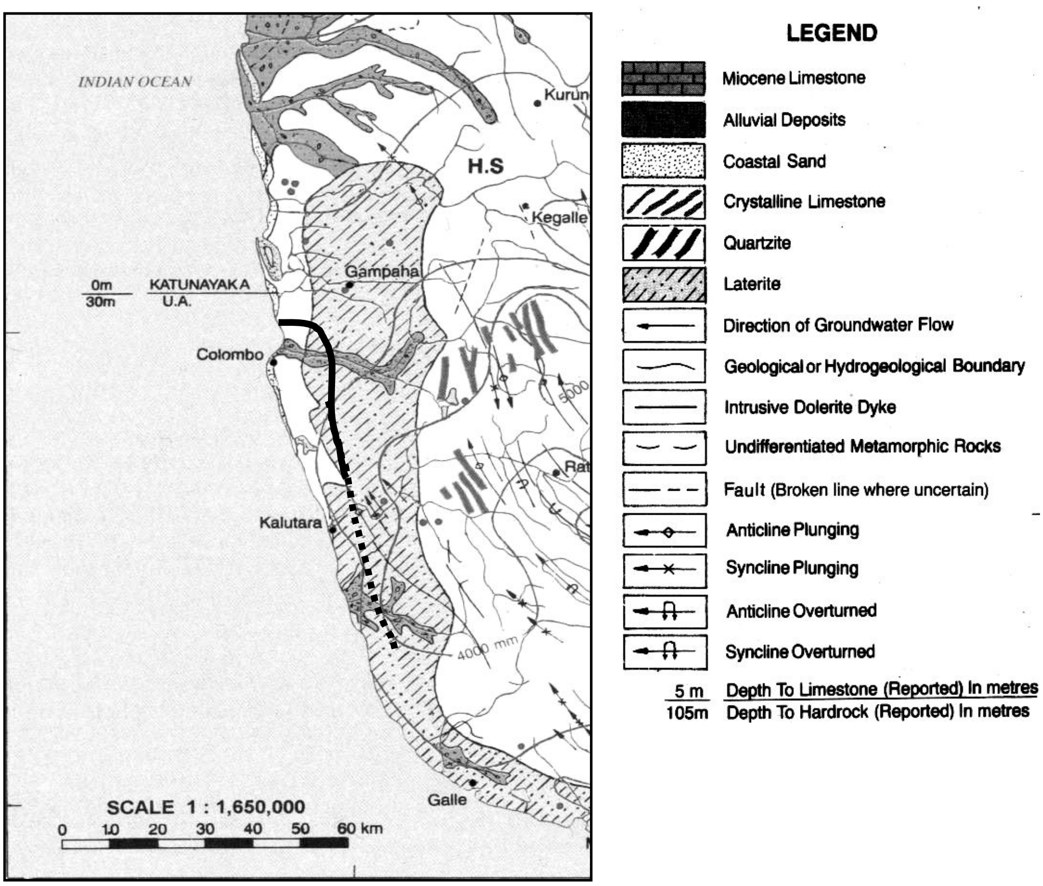


Fig. 4.9.1 Geological Distribution Map

The design standard of cutting slope based on the soil conditions has been decided as shown in **Table 4.8. 1**.

Table 4.9. 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 consists mostly of residual soil and also because there is a need to reduce the volume of borrow embankments.

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. 3.2.11** is the typical treatment.

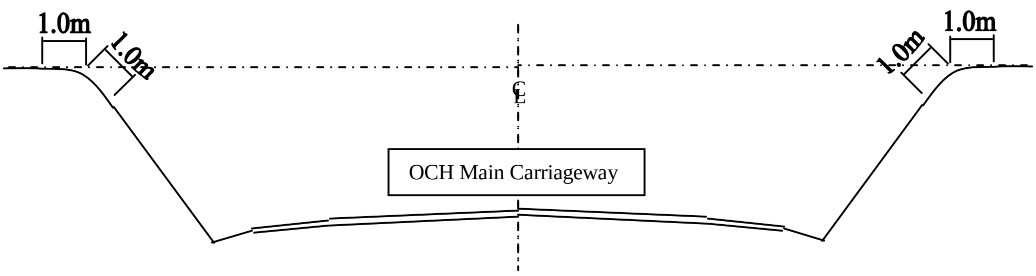


Fig. 4.9.2 Rounding of Top of Cut Slope

3) Berm

For cuts exceeding 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.

(3) Embankments (Filling)

The provision for the slope and the berm at the embankment section based on Japanese standards are given in **Table 4.9.2**.

Table 4.9.2 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.

Once RDA has requested to alter the ratio of filling slope from 1:1.8 to 1:1.5 to adjust to STDP, but as a result of discussion we agreed to keep the ratio 1:1.8 taking into account of the stability of embankment structure and the volume of borrow materials.

(4) Standard for Earthworks

The standard earthworks cross - section is shown in **Fig. 4.9.3.**

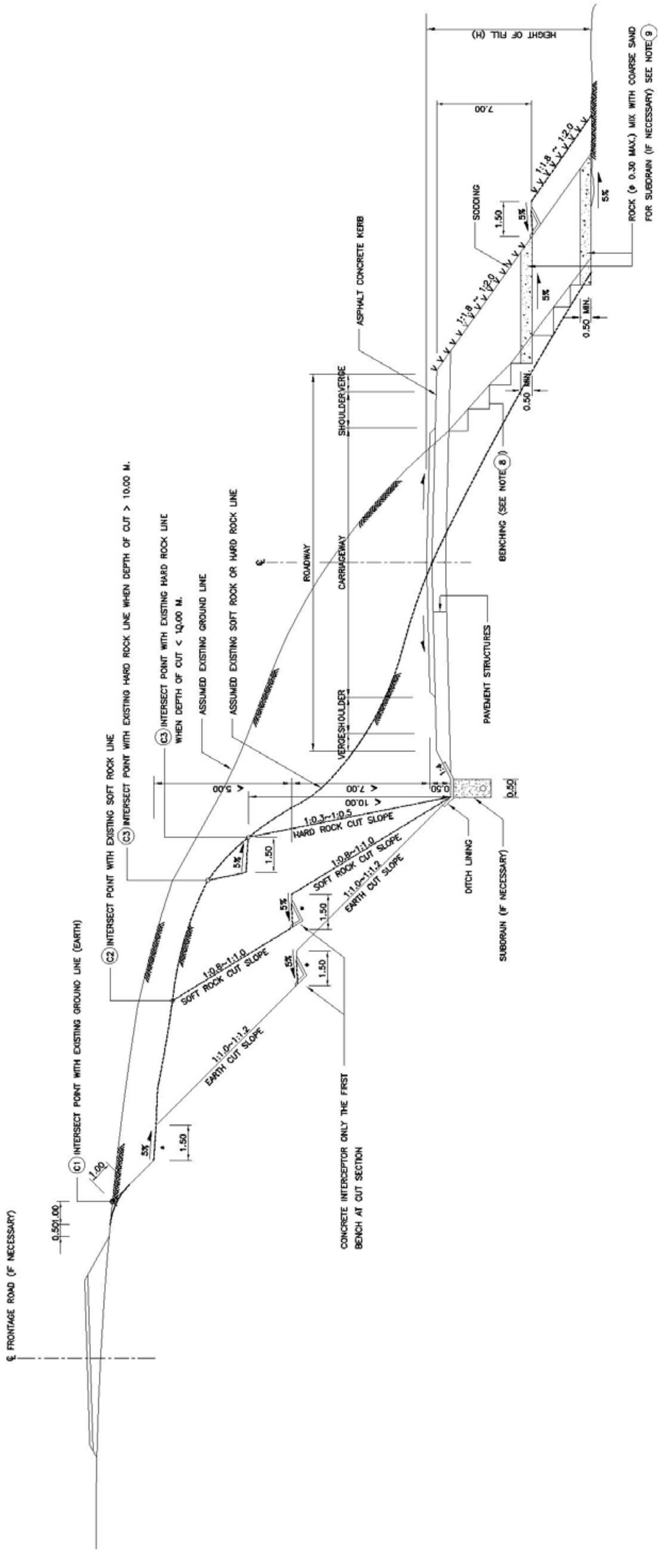


Fig. 4.9.3 Standard Earthworks Cross Section

4.10 Landscaping Plan

4.10.1. Existing Situation along the Alignment from the Viewpoint of Landscaping

(1) Land Forms and Land Use

Approximately 40% of the road traverses over the wetlands and paddy fields and the rest along the flat or little undulating lands. Areas which are adjoining the Kelani River act as flood detention areas. Other wetland areas are highly contributed to the free flow of the storm water drainage. Highland areas consist with urban/ semi urban housing and small scale rubber, coconut and home gardens.

(2) Existing Landscape Types

1 Marshes – natural wild character
2 Paddy fields and grasslands – Rural character
3 Coconut / Rubber & Home Gardens - village / rural character
4 Residential / Commercial / Mixed Development - Semi Urban / Urban Character

4.10.2. Basic Objectives of Landscaping

- To harmonize the massive earth shaping with the surrounding wetlands and adjoining areas and assist to mitigate the probable adverse impacts on Bio diversity. It will also ensure sound environment for the residents with the new development.
- To introduce green covers to balance and camouflage the appearances of hard concrete finishes, open brown earth mounds and excavated slopes.
- To ensure a healthy attractive highway environment and mitigate adverse impacts of the road development on fauna ,flora & their habitats
- To provide visual/aesthetic benefits through harmonizing with the desirable existing landscape character.

4.10.3. Greenery Plan

(1) General

In order to meet the requirements of road safety and the engineering designing aspects, the conditions mentioned below were critically evaluated and avoided as much as possible in greenery plan.

- Tree planting on the newly established slopes
- Shrub planting along the marshes, paddy lands and plantations
- Tree planting on slope shoulder, which pavement are planned
- Tree planting on center median strip, which drainage system are planned
- Disturbances to the Carriage way even in future
- Obstacles to the visibility of drivers at the interchanges with considering greenery function such as eye-inducing, introduction of amenity etc.

(2) Cut Slope Protection

According to the request from PMU for OCH, the JICA Study Team conducted the site inspection on June 16, 2007 for the cut slope protection in the STDP. The team visited three sites where the methods are adopted. According to the explanation by the Contractor the method is as following.

1) After surface preparation, coir fiber pith with including application of mulch, fertilizer and soil conditioners are fixed on the slope with bamboo anchors.
2) Grass blocks will be planted with bamboo anchors. The planting pattern of the grass block will be different depending on the slope conditions.

Success of the slope protection method will depend on the slope condition (slope stability) and weather (supply of rain fall after planting). Generally the grass blocks grow successfully. It seemed to be essential that the slope protection work should be conducted before rainy season so that sufficient water for growth of the grass will be expected.

Comparing the unit cost for turfing for embankment, the cost is considered reasonable. Therefore the team concluded the cut slope protection method will be adapted to the cut slope protection of Northern Section 1.

(3) Planting Methods

<Embankment>
10m x 2m strips of shrubs (5pots/m^2) along residential areas at 20 m intervals and turfs on both side of the whole trace

< Cut slopes>
Grass block planting and ground covers at the top edge in row at 300mm spacing (3 pots/m)

<Interchanges>
10m x 2m strips of shrubs (5pots/m^2) at 20 m intervals and turfs on both side of the whole trace

< Along the fences>
Informal hedges along residential areas with trees 1.0m in height in rows at 1.0m spacing for 50m and shrubs with wild effect for 20 m at 300mm spacing (3 pots/m)

< Approach roads>
Medium size trees at 10 m intervals for the toe and shrubs for the embankment at 10m x 2m strips in 20m intervals

(4) Soil Improvement Methods

<Turfing>
Loose the 150mm top soil layer and mixed excavated soil with sand, coir dust and cow dung/compost in .3:.3:.3 ratio mixture

<Shrubs on embankment, along the fences and foliage plant in interchanges>
300mm x 300mm x 300mm pit and refill the pits with a mixture of excavated soil with sand, coir dust and cow dung/compost in .3:.3:.3 ratio mixture

＜Ground covers at cut slope>
300mm x 300mm x 150mm pit and refill the pits with a mixture of excavated soil with sand, coir dust and cow dung/compost in .3:.3:.3 ratio mixture

< Trees along the fences and palms in interchange>
500mm x 500mm x 500mm pits for trees and refill the pits with a mixture of excavated soil with sand, coir dust and cow dung/compost in.3:.3:.3 ratio mixture

(5) Species of Trees and Shrubs

Recommended species of shrubs and trees are as follows.

Local Name	Scientific Name
<Shrubs: evergreen, maximum 5m height>	
Bata Cheena	*Bambusa multiplex*
Kaha Una	*Bambusa vulgaris*
Wetakeyiya	*Pandanus kaida*
Baludan	*Ardisa willisii*
<Ground Cover Plants: evergreen>	
Maha dupiyaliya	*Desmodium*
Heen udupiyaliya	*Desmodium heterophyllum*
Hulantala	*Ageratum conyzoides*
Monara kukudumbiya	*Vernonia cinerea*
Habarala	*Alocasia macrsorhiza*
<Trees-small size :evergreen, 5-10m height >	
Agladhara	*Justica adhatoda*
Ankenda	*Acronychia pedenculata*
Bowitiya	*Osbeckia aspera*
Etteriya	*Murraya paniculata*
Magul Karanda	*Pongamia pinnata*
Kopi	*Coffia Arabica*
<Trees-medium size :evergreen, 10-20m height >	
Godapara	*Dillenia retusa*
Diya Midella	*Barringtonoa asiatica*
Kaduru	*Cinnamomum verum*
Belipatta	*Hisbicus tiliaceus*
Muruta	*Lagrstroemia speciosa*
Diyapara	*Dillenia triquetra*
<Trees-large size :evergreen, height >20m >	
Bakmie	*Nauclea orientalis*
Dawata	*Carallia brachiata*
Diya Na	*Mesua ferra*
Wal Ehala	*Pterocarpus indicus*
Gas Kela	*Butea monosperma*
Havari Nuga	*Alstonia macrophylla*

(6) Procurement Plan for Planting and the Soil Improvement Materials

Planting materials are available at the nurseries of the Department of Forest (Ministry of Environment and Natural Resources) in advance request. If the recommended species can't be secured through the route, procurement by tenders from the private sector nurseries will be required. As for the grass planting or turfing, Axonopus compressus is recommended. It is available in abundance in coconut estates in Sri Lanka. Private sector coconut estates are the potential source of commercial turf.

The soil improvement materials, such as soil and sand are available in the vicinity of the project. The coir dust, a by product of coconut is abundantly available in the coconut estate in Sri Lanka. The compost and the cow dung are not presently available in large quantities at the market in Sri Lanka and need advance tendering from the private sector.

(7) Institutional Arrangement for the Greenery Management.

According to the Road Development Act No. 73 of 1981, the responsibility for greenery management along the roads lies with the Road Development Authority. There are provisions for the Staffs for greenery management under the Maintenance Division of RDA. However, there are proposals for the establishment of Expressway Management Authority with in the RDA for the maintenance of expressway system.

Necessary budget for greenery management should be allocated from the RDA Annual Maintenance budget.

(8) Annual Maintenance Schedule

After the planting of trees & shrubs adequate maintenance will be required for sound greenery. Annual maintenance schedule is recommended as below.

Watering :	Trees & grasses - Only 6 month's period until they established and Approx. 150 days per year days of insufficient rainfall for grass areas.
Grass Cutting & Weeding:	Once in 3 weeks where mown 'carpet' effect is required, once in 6 weeks in other areas.
Pruning	All the shrubs planted on the embankment will be pruned in every 3 months period.
Fertilizer application:	No need of fertilizer application if there is any growth defect found.

4.10.4. Drainage and Services

Efficient drainage system should be provided and where slopes adjoin the road, erosion control measures also should be included. The planting plays an important role of mitigating soil erosion. For the highland areas, the storm water drains which are coming into the excavated area should be diverted or allowed to come in properly designed cement masonry drains and direct them to the existing canals of the wetlands. In the wetland areas, the water collected on the road should be drained out properly with underground drains connected to a main drain, which opens to the wetlands in equal distance. If there are any infrastructure services they should be laid in a services duct constructed on the roadside.

4.11 Pavement

4.11.1. Introduction

Pavement design which is practiced in Sri Lanka now has been mainly carried out by using “A Guide to The Structural Design of Roads under Sri Lankan Condition published by RDA on April 1999”. The design method had been originally made by extracting some parts from the “Transport & Road Research Laboratory, United Kingdom Road Note 31 of 1977 (RN 31-1977)”. The method of design in the guideline has presently been modified based on the latest Transport Research Laboratory Method (RN31-1993) to suit local conditions.

As per request from RDA, the pavement structure for OCH, National highways and rural roads will be determined in accordance with the above RDA guideline.

The pavement structure of a road is defined as the combination of sub-base, base and surfacing placed on subgrade to support the traffic. The pavement structure should withstand and transmit traffic loads without dissatisfaction of intended usage and levels of services of the road. The selection of suitable pavement structure depends on the traffic load and subgrade strength.

Pavement Structure	Application
Surfacing	The binder application, dressing or surface course constructed on the base as a protective measure to the pavement structure, against traffic action and the effects of the weather and climate.
Base	The layer or layers of specified or selected materials of designated thickness placed on a sub base or subgrade on which surfacing will be constructed.
Subbase	The layer or layers of specified or selected materials of designated thickness placed on a subgrade on which road base will be constructed.
Subgrade	The layer immediately beneath the road formation. It may be compacted existing ground or fill material.

4.11.2. Type of Pavement Structure

The type of pavement structures for OCH and other roads will be adopted as shown in **Table 4.11.1**.

Table 4.11.1 Type of Pavement

Road Classification	Type of Pavement Structure
Main Carriageway + Ramp	Asphalt Concrete Pavement
Improved Arterial Roads at Interchange (A, B class)	Asphalt Concrete Pavement
Approach Roads (B, C, D, E class)	Asphalt Concrete Pavement, Surface Treatment
Frontage Roads	Surface Treatment
Footpath	Surface Treatment or Gravel Surface

4.11.3. Design Life

The design life specified in RDA guideline is recommended that a period of 10 to 20 years is advisable. It is essential to carry out economic analysis to determine the most cost effective design life for this particular highway. In addition, the step construction is also considerable option to determine the pavement structure, taking into account of the maintenance frequency.

For OCH and relative road, 10 years is considered as the design life for the pavement structure.

4.11.4. Subgrade Strength

(1) Laboratory Test Result

For the determination of subgrade strength, sampling and laboratory test for prospective borrow pits were implemented. The results of above tests are summarized in **Table 2.4.2**.

Table 4.11.2 Laboratory CBR Test for Prospective Borrow Pit

Pit No.	Location	Depth	Remarks	Soil Description	Unified Soil Class	NMC (%)	SG	Compaction Natural		CBR Natural Soil %	CBRm
								OMC (%)	MDD g/cm3		
B7	Kalukodayawa	0.00-0.50		Silty Sand with Gravel	SM	18.09	2.63	11.2	1.930	27.60	24.56
		0.50-1.00		Silty Sand with Gravel	SM	16.83	2.61	11.8	1.930	21.75	
B8	Kapapu Thotupola	0.00-0.50		Sility Sand with Gravel	SM	18.66	2.67	13.0	1.795	16.55	17.56
	Udamapitigama	0.50-1.00		Well Graded Gravel with Slit	GW-GM	14.04	2.57	13.8	1.870	18.60	
B9	Hanwella RD	0.00-0.50		Poorly Graded Gravel with Slit and Sand	GP	22.77	2.56	17.6	1.675	15.60	15.65
	Udamapitigama	0.50-1.00		Clayey Sand with Gravel	SC	32.56	2.53	17.6	1.620	15.70	
B10	Kalukodayawa	0.00-0.50	Upper	Well Graded Gravel with Sand	GW-GM	21.14	2.69	16.5	1.740	14.70	13.99
		0.50-1.00		Poorly Graded Gravel with Slit and Sand	GP-GM	19.84	2.75	16.6	1.730	13.30	
		0.00-0.50	Lower	Silty Sand with Gravel	SM	24.42	2.6	15.0	1.820	15.60	13.33
		0.50-1.00		Silty Gravel with Sand	GM	20.29	2.56	14.6	1.770	11.30	
B12	Darshanie Estate	0.00-0.50		Silty Sand with Gravel	SM	14.53	2.59	10.5	1.830	19.70	20.05
	Malwana	0.50-1.00		Silty Sand with Gravel	SM	15.38	2.63	15.2	1.920	20.40	
B26	Delgoda	0.00-1.00		Sandy Silt	ML			17.6	1.710	12.25	12.25

Representative CBR value of each borrow pit was calculated using following equation.

$$CBRm = \left(\frac{h1CBR1^{1/3} + h2CBR2^{1/3} + \cdots\cdot + hnCBR^{1/3}}{100} \right)^3$$

CBRm : Representative CBR value at each borrow pit
CBRn : CBR value of each layer
hn (cm) : Thickness of each layer

(2) Determination of CBR Value for Pavement Design

CBR value of subgrade applying for pavement design of northern section 1 was calculated using following equation.

$$\begin{aligned} CBRa &= CBRm_{(Ave.)} - \sigma_{n-1} \\ &= 16.77 - 4.35 \\ &= 12.42 = \boxed{\mathbf{12}} \end{aligned}$$

CBRa : CBR value of subgrade applying for pavement design
$CBRm_{(Ave.)}$: Averaged CBR value of prospective borrow pits
σ_{n-1} : Standard deviation of CBRm

According to guideline of RDA, this CBR value corresponds to **S4 class (CBR 8 – 14)**.

4.11.5. Traffic Volume

Normal passenger cars are not critical to the structural damage of road pavement by vehicular traffic. Therefore, for the pavement design, only heavier vehicles (unloaded weight exceeding 1.5 tones) are considered. For estimation of design traffic for OCH and A1 road, 10 years design life is assumed and the traffic demand forecast from 2012 to 2022 is given in the **Table 2.4.3** and **Table 2.4.4** for OCH and A1 Road.

Table 4.11.3 Traffic Demand Forecast of Northern Section 1 of OCH (2012 – 2022)

Year	Cars/Pass	Buses	Trucks	Total
2012	22,201	8,788	6,721	37,710
2022	37,740	8,277	7,524	53,540

Table 4.11.4 Traffic Demand Forecast of A1 Road (2012 – 2022)

Year	Cars/Pass	Buses	Trucks	Total
2012	11,822	7,757*	3,711*	29,501
2022	24,388	7,042	5,142	46,579

*- For the A1 Bypass road CNSA value, 80% of A1 Road Traffic demand is considered.

This forecast was estimated on the basis of the assumption that OCH construction reaches at Rt. A1 in year 2012. According to the forecast, the growth rate of bus and truck are -0.60% and 1.14% respectively for OCH. Similarly, growth rate of bus and truck are -0.96% and 3.32% respectively for A1 Bypass road.

4.11.6. Axle Load

Axle load survey data of the western province of Sri Lanka were obtained from the Traffic and Planning Division in the Road Development Authority. Axle load survey data of seven stations where was expected considerable contributions to OCH were used to calculate average equivalent standard axles (ESA) in millions for each section of the OCH.

Table 4.11.5 Classification of Vehicle for Axle Load Survey

No.	Type of Vehicle	No.	Type of Vehicles
1	S.L.T.B Long Bus	6	2A/6W <8.5 Tons
2	S.L.T.B Short Bus	7	2A/6W >8.5 Tons
3	Private Large Bus	8	3A Commercial
4	Medium Bus 2A/4W	9	4A Commercial
5	Light Lorry	10	5A Commercial

Table 4.11.6 Summary of Average ESA

Sta.	Vehicle Type	1	2	3	4	5	6	7	8	9	10
1	Volume	612	29	1,190	3,345	5,441	783	437	27	39	2
	ESA	0.288	0.288	0.288	0.056	0.005	0.048	0.75	1.811	0.504	0
2	Volume	355	24	304	645	1,616	1,268	1,154	57	17	4
	ESA	0.4373	0.4373	0.4373	0.103	0.0243	0.3013	3.2753	7.106	3.7469	0.4365
3	Volume	1,013	117	1,684	3,382	6,159	1,494	1,538	201	286	13
	ESA	0.5465	0.5465	0.5465	0.0734	0.0178	0.2043	5.5313	7.4559	10.6075	0
4	Volume	68	18	167	2,202	2,448	1,099	1,516	126	22	9
	ESA	0.3927	0.3927	0.3927	0.0235	0.0022	0.2394	2.9795	4.3502	3.0669	2.0968
5	Volume	32	2	26	967	3,188	757	234	35	4	0
	ESA	0.2119	0.2119	0.2119	0.0072	0.0073	0.2712	2.3452	6.2992	3.162	0
6	Volume	51	2	2,012	4,766	9,571	2,324	3,545	289	685	13
	ESA	0.185	0.185	0.185	0.05	1.678	0.176	3.164	5.195	5.094	12.25
7	Volume	439	13	299	706	958	478	860	33	4	23
	ESA	0.1371	0.1371	0.1371	0.056	0.0327	0.3443	0.7485	0.79	1.71	0
Sub Total of Vehicle		**2,570**	**205**	**5,682**	**16,013**	**29,381**	**8,203**	**9,284**	**768**	**1,057**	**64**

Total No of Buses (Type 1,2,3,4)	= 24,470
Total No of Trucks (Type 5,6,7,8,9,10)	= 48,757
Grand Total	= 73,227

Table 4.11.7 Average ESA of Bus & Truck

Vehicle Type	1	2	3	4	5	6	7	8	9	10
Total of each type	2,570	205	5,682	16,013	29,381	8,203	9,284	768	1,057	64
% Composition	10.503	0.8378	23.22	65.439	60.26	16.824	19.041	1.5752	2.1679	0.1313
Ave. ESA	0.3845	0.4509	0.3309	0.0524	0.5547	0.2154	3.1819	5.532	6.3325	2.8104
Average ESA considering all Buses together							**0.1553**			
Average ESA considering all Trucks together							**1.2044**			

4.11.7. Estimating the Cumulative Number of Standard Axles

The cumulative number of standard axles for the design life can be determined by use of the formula given as follow:

$$A=365^* \sum_{i=1}^{m} P_i\,[(1+r_i)^n - 1]/r_i$$

Where
A= Cumulative Number of Standard Axles (CNSA) for design life
P_i=Number of Standard Axles per Day as an average for the 1st Year after construction for vehicle type-i
r_i= Rate of growth of traffic for vehicle type-i
m=Number of type of vehicles
n= Design life in years

CNSA by the end of year 2022	= A
No of lanes	= 4
Maximum lane using ratio	= 1/2 x 2/3 = 0.33
So, CNSA per lane	= 0.33 x A

Based on above formula, the CNSA value per lane of OCH and A1 Bypass road shall be calculated as follows.

Table 4.11.8 CNSA Value per Lane of OCH

Vehicle Type	CNSAx10^6
Bus	4.85
Truck	31.11
Total	35.96
per Lane	**11.87**

According to guideline of RDA, this value corresponds to **T7 class (CNSA 10 – 17)**.

Table 4.11.9 CNSA Value per Lane of A1 Bypass Road

Vehicle Type	CNSAx10^6
Bus	3.37
Truck	15.18
Total	18.55
per Lane	**6.12**

According to guideline of RDA, this value corresponds to **T6 class (CNSA 6 – 10)**.

4.11.8. Determination of Pavement Structure for OCH and A1 Bypass Road

The pavement structure is determined by the combination with the subgrade strength and the cumulative axle load in accordance with the RDA guideline (see **Fig. 4.11.1**).

	Granular/Double Bituminous Surface Treatment and Sand Seal		Granular Bese/ Asphalt ConcreteSurfacing		
	T3	T4	T6	T7	T8
S1	SD 225 275 300	SD 225 325 300	100 200 225 350	125 225 225 350	150 250 250 350
S2	SD 225 225 200	SD 225 275 200	100 200 225 200	125 225 225 200	150 250 250 200
S3	SD 225 225	SD 225 275	100 200 250	125 225 250	150 250 275
S4	SD 225 150	SD 225 200	100 200 175	125 225 175	150 250 175
S5(a)	SD 200 100	SD 225 125	100 200 100	125 225 100	150 250 100
S5(b)	SD 200	SD 225	100 200 100	125 225 100	150 250 100
S6	SD 175	SD 220	100 200	125 225	150 250

- Asphalt Concrete Surfacing: WC and BC or WC only
- Granular Base: DGAB / WBM / DBM
- Subbase Cource
- Selected Material Subgrade Fill: CBR 15+

Fig. 4.11.1 Pavement Structure on RDA Guideline

(1) Pavement Structure of OCH

As per the above figure, the pavement structure of OCH main carriageway is given in **Fig. 4.11.2**. The pavement structure and its cross section, which was applied for main carriageway of southern section, can be applied for this northern section 1 as well.

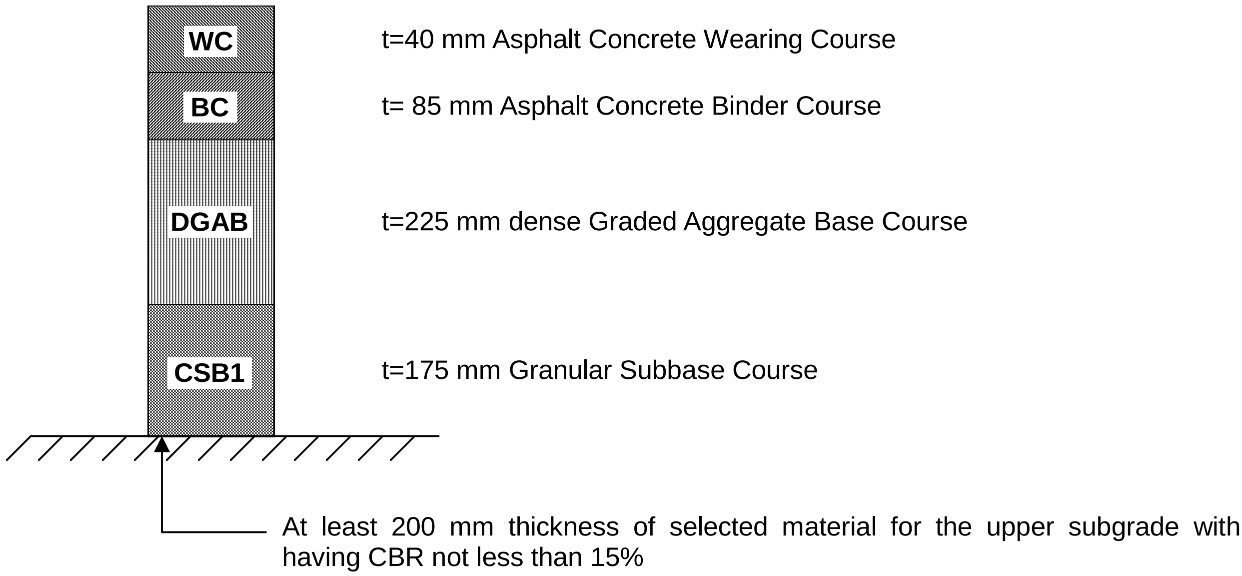


Fig 4.11.2 Pavement Structure of OCH

(2) Pavement Structure of A1 Bypass

As well as the OCH main carriageway, the pavement structure for A1 Bypass road was determined in accordance with the RDA guideline and given in **Fig. 4.11.3**.

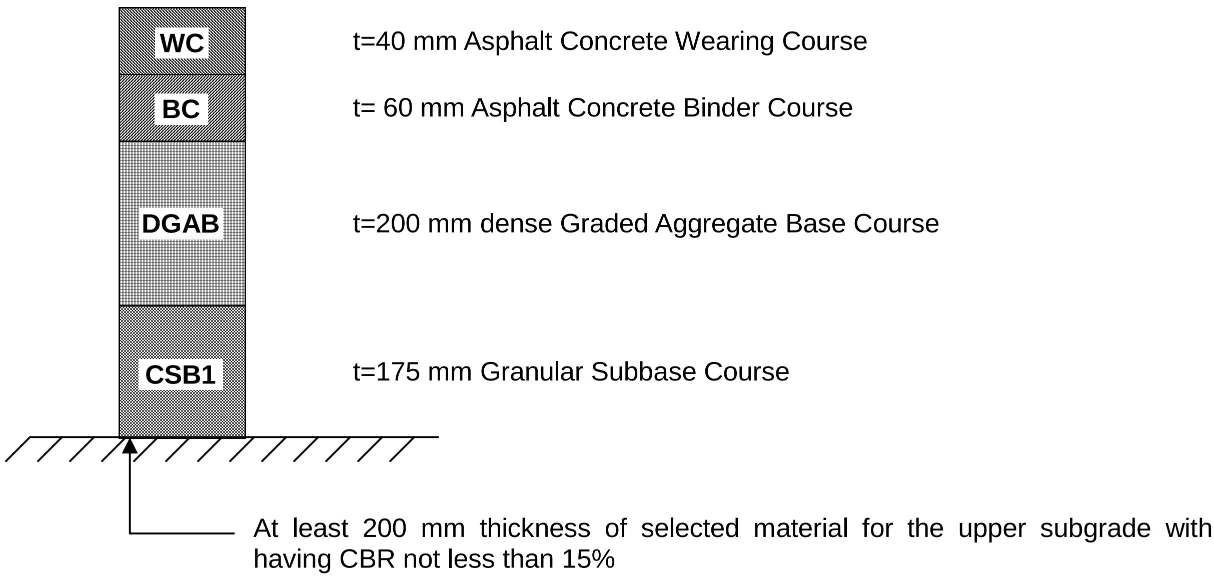


Fig 4.11.3 Pavement Structure of A1 Bypass

4.11.9. Pavement Structure for National Highways & Rural Roads

For determination of the pavement structure of national highway, the proposed future pavement for A1 road and B214 road are provided from RDA and the pavements of Sapugaskanda-Biyagama Road are referred to B214. The pavement structure of C class road is adopted for B169 and B401 as the grade of pavement of the future plan of these roads are lower than the pavement of C class road.

For the Class C, D and E roads, since they will be embankment structures, the same borrow bit material used to construct the OCH will be applied, meaning that subgrade strength will be Class S4 (CBR of 8 to 15%). As for traffic, since Class T4 is normally used for Class C roads, this will be adopted for this class of road, while Class T3 will be adopted for lesser traveled Class D and E roads.

The summary of design Pavement Structures for A1 Bypass, other National Highways and Class C, D and E roads are given in below **Table 4.11.10**.

Table 4.11.10 Pavement Structure for National Roads

Class	Road	Pavement Structure	Remarks
A	A1 Road Colombo-Kandy	AC: 125 DGAB: 225 SSB: 275 GCL: 200	Improvement Structure by RDA
A	A1 Bypass Road	AC: 100 DGAB: 200 SSB: 175 GCL: 200	Design by RDA Standard (T6,S4)
B	B169 Road Kadawatha-Mawaramandiya	DBST DGAB: 225 SSB: 200	Adopted the Pavement for C class
B	B401 Road Siyabalape-Galwalakada	DBST DGAB: 225 SSB: 200	Adopted the Pavement for C class
B	B214 Road Kelaniya-Mudungoda	AC: 125 DGAB: 250 SSB: 175	Improvement Structure by RDA
B	Sapugaskanda - Biyagama Road	AC: 125 DGAB: 250 SSB: 175	Adopted the Pavement for B214
C to E	C Class Road	DBST DGAB: 225 SSB: 200	Design by RDA Standard (T4,S4)
C to E	D&E Class Road	SBST DGAB: 225 SSB: 150	Design by RDA Standard (T3,S4)

4.12 Highway Facility

4.12.1.Highway Lighting Facilities

(1) Luminary Specification

The highway lighting system will be installed in accordance with the design manual for Japan Highway Public Corporation. The luminary specification for lighting facility, which is adopted in STDP, is conformable to use for the OCH for the sake of the uniformity in the sustainable traffic flow through directional link. The detailed calculation is made based on this policy.
The summary of luminary specification is comprised in the table as follows.

Table 4.12.1 Standard Specification of Highway Luminary

	OCH	STDP
Average Road Luminosity (Standard Luminosity)	1.0 cd/m2	
Standard Height of Lighting Post	12.0m	
Type of Pole	Round type steel pole	
Luminary Capacity	250 watt (High Pressure Sodium Lamp)	
Calculation Formula of Luminosty	F/S=W*K*L/(N*U*M) where: F: Lamp Lumens (lum) S: Interval of Light (m) W: Road width (m) K: Conversion ratio to Average illuminance (Lx/(cd/m2)) L: Standard Luminosity (cd/m2) N: Layout coefficient N=1: Stagger, Single N=2: Parallel U: Utilization Factor M: Maintenance Factor	Fd=I*e*Ltb*R/UV where: Fd: Lamp Lumens (Lum) I: Wide of Road way (m) e: Interval of Light (m) Ltb: cd/m2 U: Coefficeint V: Coefficient

(2) Location of Lighting Facility

Based on the discussion with RDA, the locations where lighting facilities shall be provided for OCH has been proposed as follow.

- Both sides of main carriageway of interchange and at approach section for the interchanges
- An either side of ramp at Interchange

(3) Standards layout of Highway Luminary

The standard layout specified in the design manual for Japan Highway Public Corporation is recommended as follows. In case of Single layout for one side carriageway (no lighting at center median), 36 to 42 m depending on the applicable type of lighting will be standardized at ultimate six-lane width of OCH.

Table 4.12.2 Standard Layout of Highway Luminary (Lighting Post Height: 12m)

Lane Width	3.0 – 3.5 (One Lane)		6.0-7.0 (Two Lane)		9.0-10.5 (Three Lane)	
Type of Light	Cut-off	Semi Cut-off or Back Cut-off	Cut-off	Semi Cut-off or Back Cut-off	Cut-off	Semi Cut-off or Back Cut-off
Single	42	48	42	48	36	42
Staggered	-	-	-	-	36	42
Parallel	-	-	-	-	42	48

(4) Standards Layout of Highway Luminary at Loop

The layout of lighting facilities shall be modified at curve section particularly at loop ramp. The Japanese standards recommends that lay out should be modified if the radius of horizontal curve is less than 1,000m to induce the driver's sight at the curve section. According to the table given below, the interval of street luminary depending on the radius of curve shall be reduced. If the interval from **Table 4.12.13** is larger than the value from **Table 4.12.12**, the value from **Table 4.12.12** shall be selected.

Table 4.12.3 Standard Interval of Highway Luminary at Curve Section

Radius of Curve	>300m	>250m	>200m	<200m
Interval of Lighting Facility	<40m	<35m	<30m	<25m

(5) Luminary facilities at Interchange

The average standard illumination at interchange shall be the same (1.0 cd/m2) with the street luminary. The criteria for layout of lighting at interchange will be planned in accordance with the table as follows.

Table 4.12.4 Scale for Highway Luminary at Interchange

Main Carriageway at Interchange	Main Carriageway AADT	>50,000	<50,000
	Lighting Scale Category	A	B
	Installation Scale	100%	-

Merging / Diverging Section or Ramp Section	Entry and Exit Traffic Volume of Interchange		>25,000	>15,000 <20,000	>5,000 <15,000	<5,000
	Lighting Scale Category		*1	*2	*3	*4
	Installation Scale	Mer. / Div. with Main Carriageway	100%	75%	50%	25%
		Ramp	100%	50%	50%	-
		Mer. / Div. of Ramps	100%	100%	100%	50%

Note: Installation Scale for Merging and Diverging Section with Main Carriageway shall be 100%, regardless of the values as above Table, when Main Carriageway AADT is more than 50,000 veh/day.

Examples of lighting distributions by Scale Category in above table are shown in **Fig. 4.12.1 - 4.12.5**.

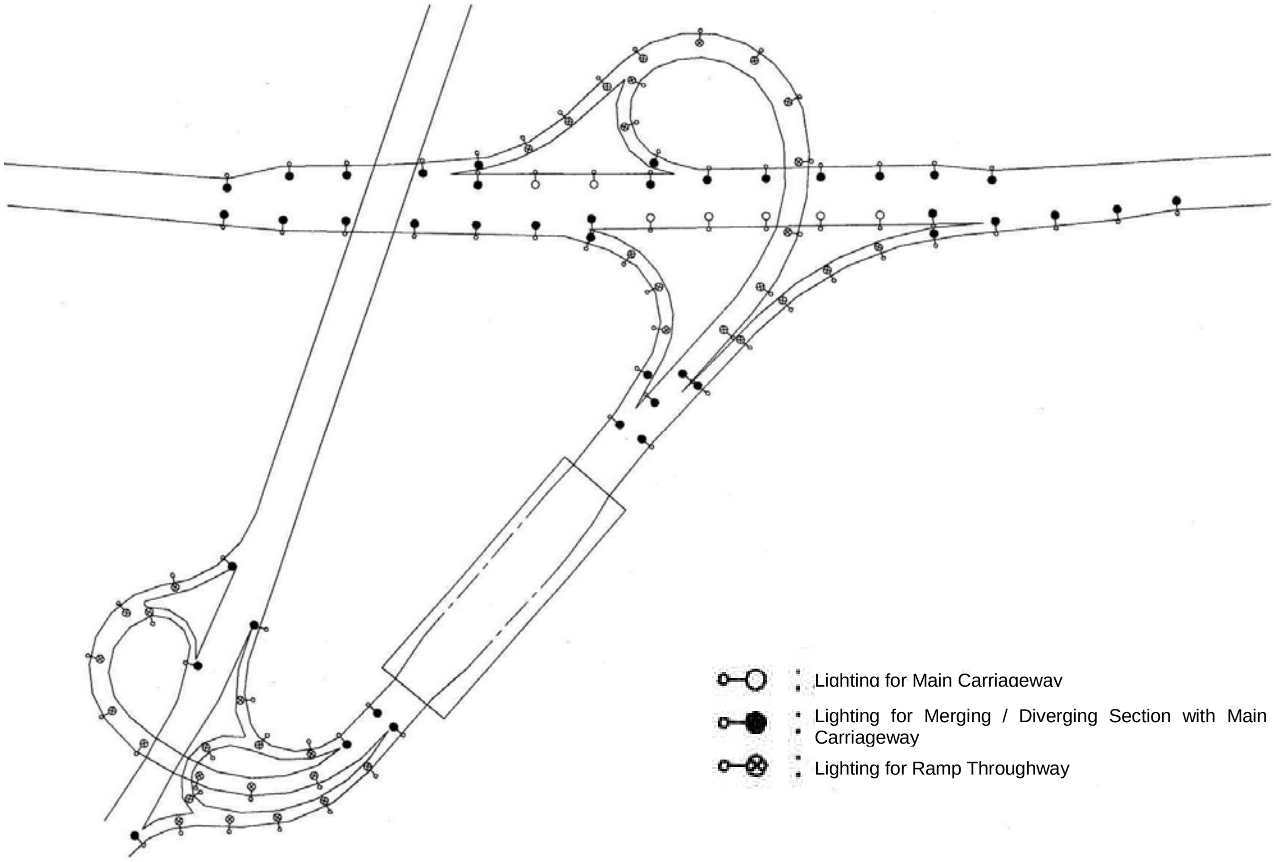


Fig. 4.12.1 Example for Lighting Distribution at Interchange Category A-*1

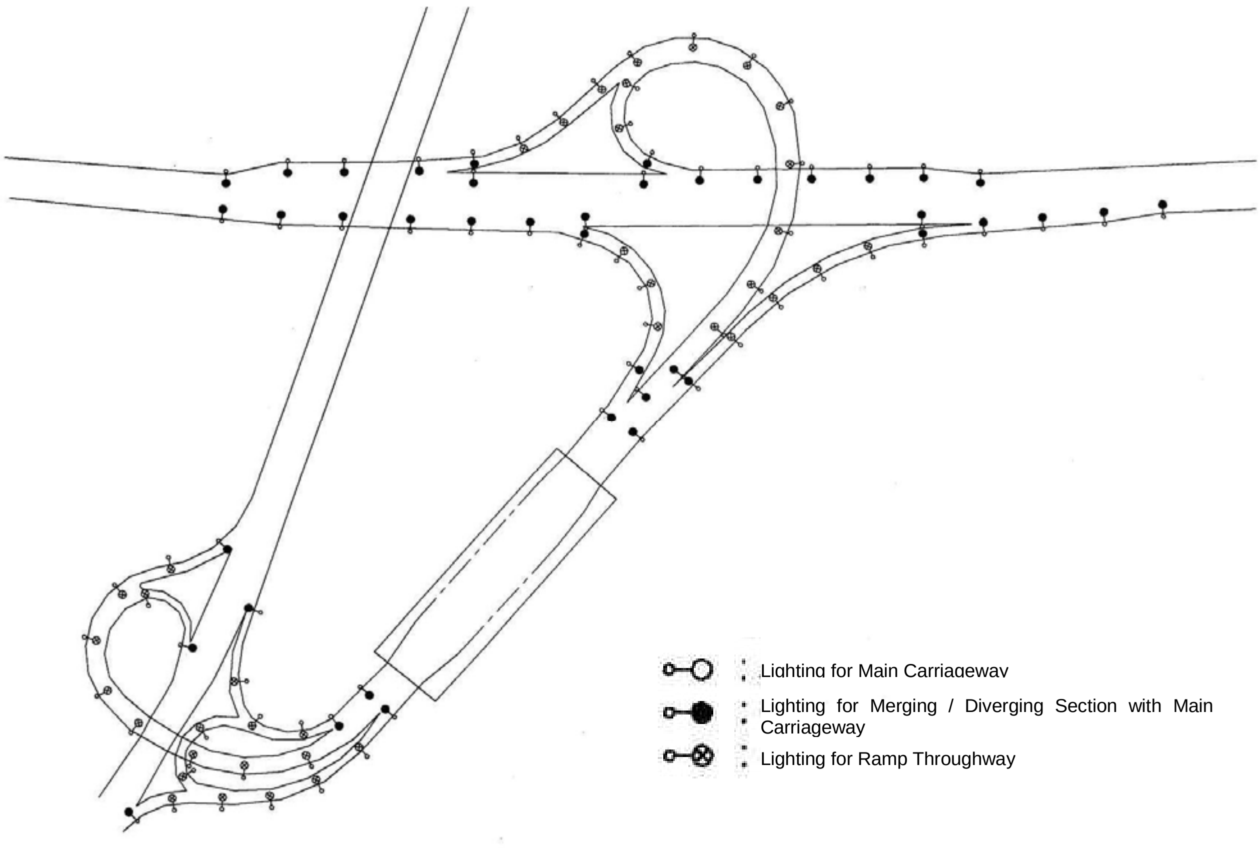


Fig. 4.12.2 Example for Lighting Distribution at Interchange Category B-*1

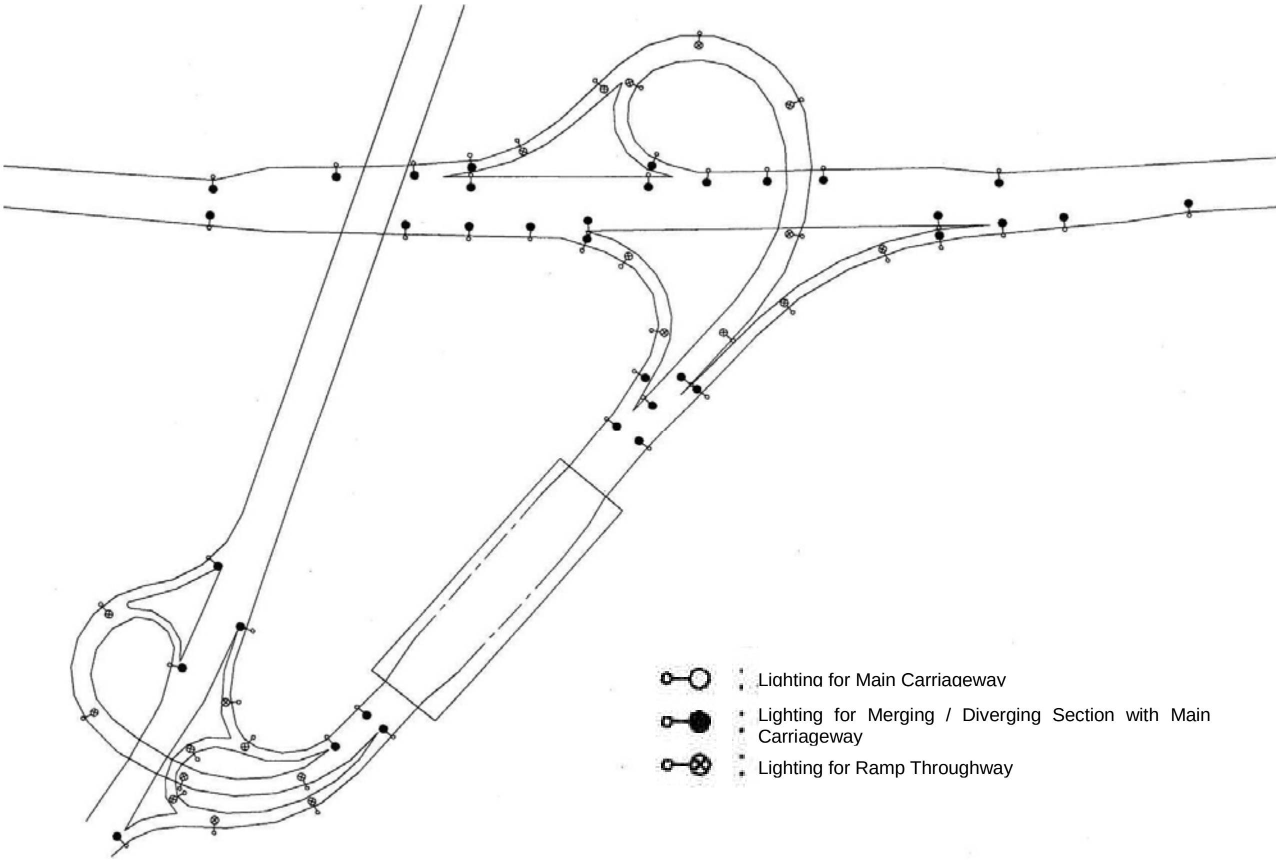


Fig. 4.12.3 Example for Lighting Distribution at Interchange Category B-*2

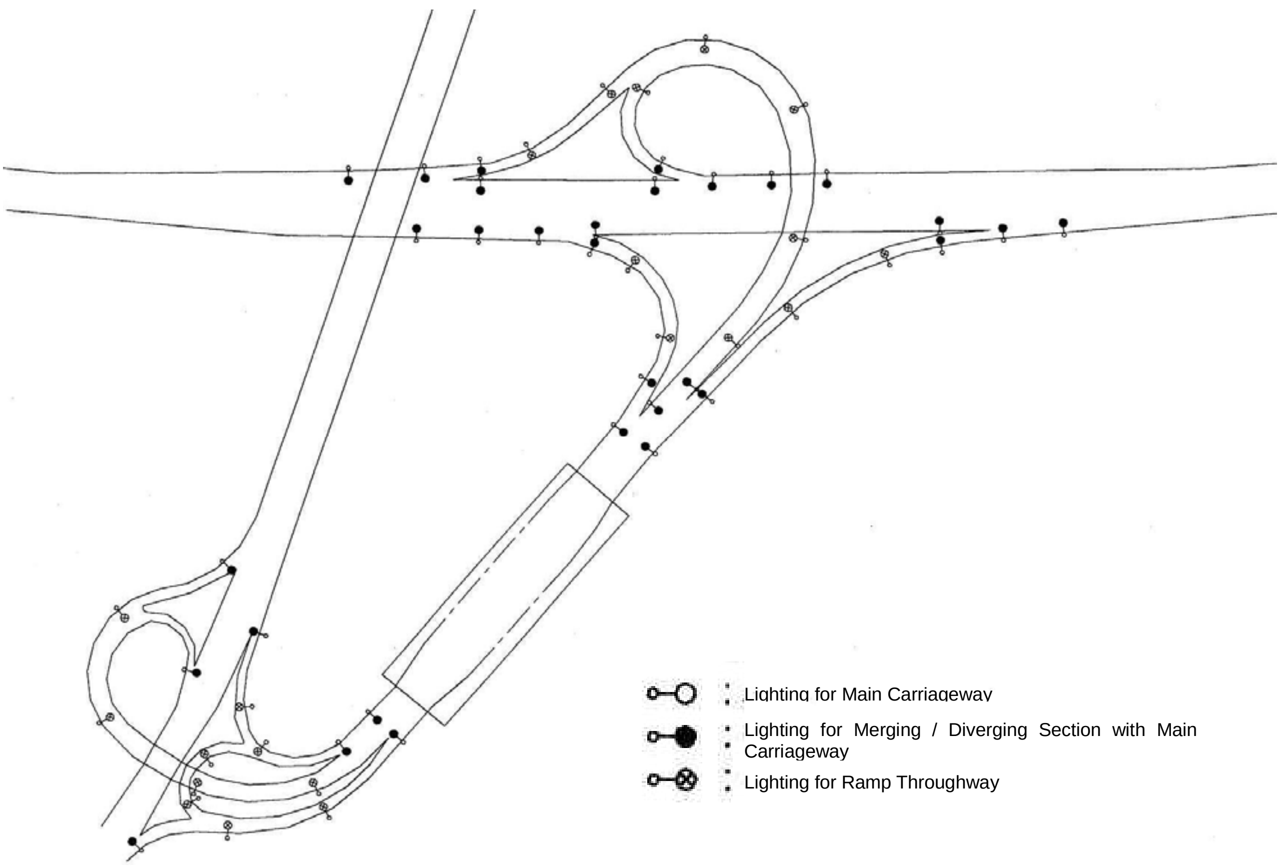


Fig. 4.12.4 Example for Lighting Distribution at Interchange Category B-*3

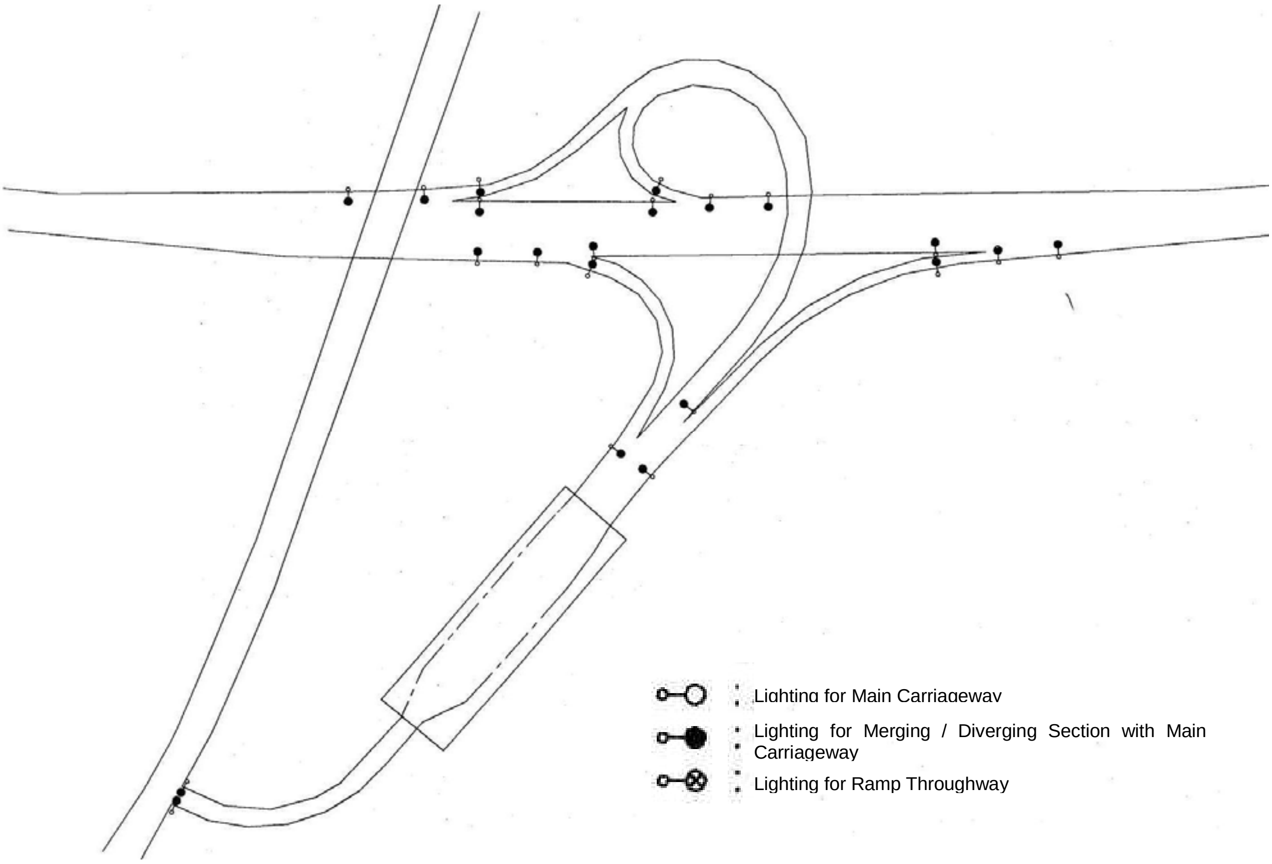


Fig. 4.12.5 Example for Lighting Distribution at Interchange Category B-*4

(6) Classification of Luminary Light Distributions

Luminaries – A luminary is composed of a light source, a reflector, and usually a glass or plastic lens or reflector. It is the function of the reflector and refractor to gather the light from the source, direct it toward the roadway, and shape it into a desired pattern on the roadway. Proper distribution of the light flux from the luminary is one of the essential factors in good roadway lighting. All luminary light distributions are classified according to their vertical and lateral distribution patterns and the light control in the upper portion of the beam. Glare shields may be added to reduce objectionable light emissions toward adjacent buildings or areas.
Selection of the light source will be made from high intensity discharge (HID) sources because of their luminous efficacy.

a) Lighting Type

HPS (High Pressure Sodium) lighting is the most energy efficient source which has an acceptable color rendition. This was the criteria to select HPS lighting for Outer Circular Highway.
Metal halide luminaries have a good color rendition, but luminous efficacy, lumen maintenance (lumen output diminishes more rapidly throughout life), length of life, and restrike time make them a less than desirable source for many applications.
Low pressure sodium lighting is not included, as the colour is monochromatic and therefore is not considered suitable for general use. The amount of sodium in low-pressure sodium luminaries requires special disposal methods if such luminaries

are not to pose a fire hazard. Incandescent lighting has not been included because of the extremely low luminous efficacy and short luminary life. Mercury vapor lighting is also not included because of lower luminous efficiencies, poor lumen maintenance, higher life cycle costs, and environmental considerations (mercury propagation and disposal).

b) Lighting Control and Wiring System

On Off Control: Luminaires for dusk to dawn operation will normally be controlled by a photoelectric cell installed on each Main Distribution board. Also an automatic system using a time switch with an astronomical dial or a manual on-off control is proposed. Multiple wiring systems will be installed, except for extensions to existing series systems for long access roads where voltage drops exceeding that permitted in the IEE wiring regulations (16 Edition) for multiple lighting systems would occur. Circuits for multiple lighting is designed to utilize the highest low-voltage level appropriate (see Appendix) for the installation in order to keep wire sizes and voltage drops to a minimum. Luminaries will be connected phase-to-neutral rather than phase-to-phase. Protection and disconnection of lighting circuits are provided. All lighting circuits will include an equipment grounding conductor. The equipment grounding conductor will be bonded to the noncurrent-carrying metal parts of each lighting standard and luminaries.

c) Control of Distribution above Maximum Candlepower

Disability and discomfort glare are largely a result of light emission into the driver's eye. For design purposes, it is necessary that luminaries be classified according to their relative glare effects. Thus, luminaries are classified as cutoff, semi-cutoff and non-cutoff. Our selection of luminaries for this design was based on semi-cutoff and cutoff category. Descriptions of these classification categories are as follows:

- Cutoff: A luminary light distribution is classified as cutoff when the candle power per 1000 bare lamp lumens does not exceed 25 at an angle of 90 degrees above nadir (a vertical axis through the light source); and 100 at an angle of 80 degrees above nadir.
- Semi-cutoff: A luminary light distribution is classified as semi-cutoff when the candlepower per 1000 bare lamp lumens does not exceed 50 at an angle of 90 degrees above nadir and 200 at a vertical angle of 80 degrees above nadir.
- Non-cutoff: The classification when there is no candlepower limitation in the zone above maximum candlepower.

4.12.2.Emergency Telephone System

The expressway, motorway and other toll-way are like lockout space where is allowably limited to access at the operational control. A traffic accident might incur critical situation if it is happen. As per the design manual of Japan Highway Public Corporation, the emergency telephone system shall be provided for corresponding to the emergency situation along the expressway. The emergency telephone system comprises the roadside telephone facility and telephone exchange unit. The emergency call from the roadside telephone shall be transferred to the control center as the information administrative through the exclusive circuits linked by exchange units.

(1) Standards and Regulations

The Telephone System shall be manufactured to international standards recognized by the Sri Lanka Telecommunication Regulatory Authority. Generally all work associated with the Telephone System shall comply with the requirements of the Sri Lanka Telecommunication Regulatory Authority.

The intelligent system as used in Europe, United States and Japan will be optionally considered. It is considered that the emergency telephone system for OCH provides the fundamental requirements of the emergency terminal as follows:

- Hearing clearly in both directions without being immediately adjacent to the phone
- Hands-free operation allows flexibility to use both hands for writing notes, providing first aid or continuing the conversation some distance from the phone
- Industrially designed to meet the needs of users including those with a disability
- Hands-free operation, essential in emergency situations, is made practical in high noise environments by using advanced signal processing techniques.
- Adaptive volume control measures the ambient noise level and automatically adjusts the volume.
- High audio output and sound quality from tamper resistant speaker.
- Tilt/vandal feature where a sensitivity level can be set, causing the phone to dial the maintenance number if it is pushed over by a vehicle or struck by an object.
- Easily installed, no exposed cables, on-sire configuration available but not required.
- Lighting and radio frequency interference protection.

It would be also required to establish multilink system with relevant expressway networks.

(2) Location of Emergency Telephone

The roadside telephones are located as following standards.

Location	Standards
Along the expressway and interchange	At the interval of 1km on both side of road except specific location

According to the above standards, there are totally 16 roadside telephones provided for the section of Northern Section 1 for OCH.

4.13. Traffic Control Devices

Traffic control devices are defined as devices to direct and assist road users so that they may be able to travel safely and efficiently on a road network. Traffic control devices control all modes of traffic that use a road, including motorized and non-motorized modes. Note that these devices should not be hidden or obscured by other attachments such as advertisement signs and normally consist of traffic safety devices, traffic signs, road markings, and traffic signals.

4.13.1 Traffic Safety Devices

(1) Guard Fence

A guard fence refers to a facility that has the purpose of preventing a vehicle from running the wrong way or from straying out of its lane into an opposing lane, as well as restoring a wayward vehicle back to its correct path with minimum damage to the vehicle and its passengers. A secondary function of a guard fence is to guide a driver's eyes. Note that the design of the guard fence for the OCH is based on Japanese standards and the types of guard fences used are as follows:

- Guard rail
- Concrete wall

A guard fence, as a safety device, shall provide the following functions:

- Prevent vehicles from drifting out of their lane.
- Allow a vehicle that collides with a guard fence to its normal path.
- Ensure the safety of passengers in vehicle that collides with a guard fence.
- Prevent a vehicle that collides with a guard fence or the guard fence itself from becoming a traffic hindrance after a severe collision.
- Prevent a collision against pedestrians or other vehicles
- Minimizing loss in properties
- Guiding the driver's eyes

The physical characteristics of a guard fence are usually as follows:

- A guard fence usually consists of connected rails supported by piles and has a modest rigidity so that it shows a large plastic distortion upon the collision of a vehicle. As to maintenance, damaged parts can be easily replaced locally.
- A concrete wall consists of reinforced concrete and is rigid and good at keeping automobiles from deviating from a road. However, it is poor for ensuring passenger safety and is claustrophobic for drivers. On the other hand, it is free from corrosion and relatively maintenance free.

(2) Installation Method and Treatment of Terminal Posts

1) Outer Side of Throughway

(a) Guardrail

(i) The starting point of a guardrail shall be located on a cut section 10 to 12 m from the boundary between it and an adjacent fill section. The beginning of the guardrail shall be at the side of the slope of the cut at least 75 cm from the clearance limit. The run-off should be a tapering curve and extend for three spans from the starting point of the guardrail (see **Fig. 2.6.1**). The end point of the guardrail shall be located on the next cut section 4 to 8 m from the boundary between it and the preceding fill section, with the beam at the end point not set back from the clearance limit.

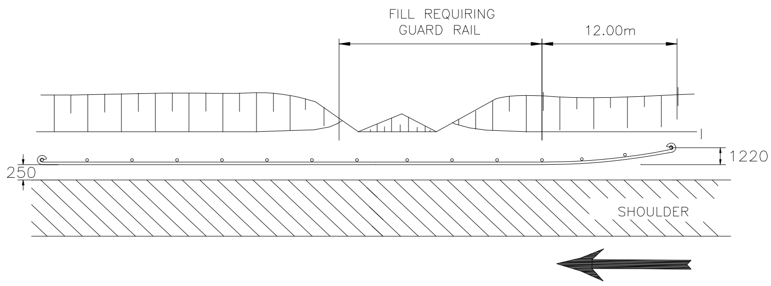


Fig. 4.13.1. Cut to Fill Section

(ii) The reference point of a guard fence shall be as follows:

- For a structure-protecting guard fence, the center of the structure is the reference point. However, the point may be shifted in consideration of the situation on the ground.
- For continuous sections having no structure-protecting guard fence, the end post is the reference point, and is to be located on a cut section 6 m from the boundary between it and a fill section.
- For continuous sections having a structure-protecting guard fence of one type, the end post of the guard fence is the reference point.
- For continuous sections having different types of guard fences, the end post of a fence of one type at which a fence of another type follows is the reference point.

(b) Concrete Wall or Handrail

The starting point of a concrete wall or handrail shall be located at the approaches to a bridge section (see **Fig. 2.6.2**).

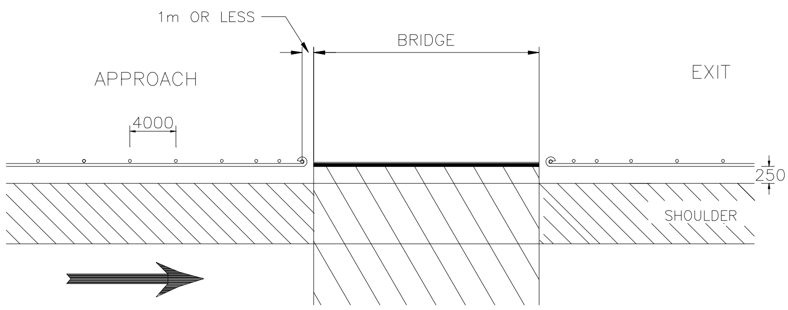


Fig. 4.13.2. Bridge Approach

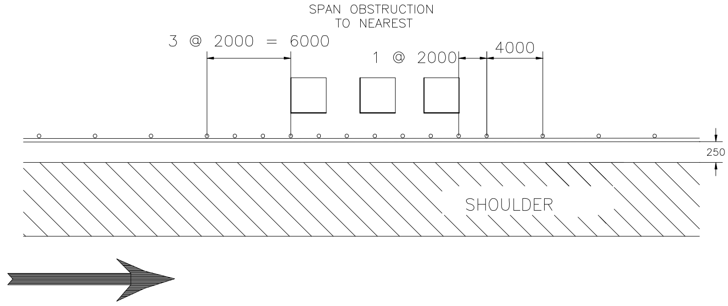


Fig. 4.13.3. Near Obstructions

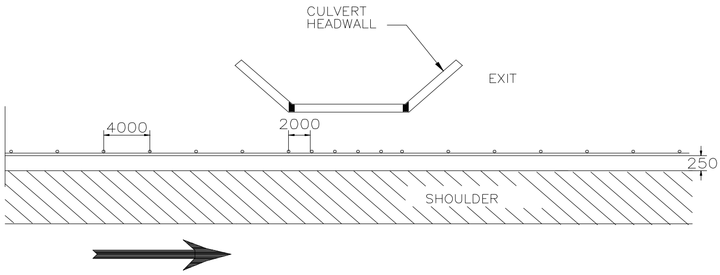


Fig. 4.13.4. Culvert Approach

(c) Approach to Overpass

In principle, the length of guardrail for overpasses shall be 60 m from the end of an overpass. However, the length may be varied taking into consideration topographic features and the presence of structures near the end of the overpass.

(d) Other Roads

The specifications for throughways and ramps shall apply correspondingly. At the initial stage of OCH the effective median width is 10 m. According to AASHTO and the Australian Standard, for a median width of 10 m a median barrier is optional. However, median barriers are still required for the following locations:

- Road medians containing bridge piers
- Bridge approaches
- Median edges with large elevation differences (Median barriers should be provided on the higher side.)
- Median openings
- Medians containing rigid objects

Roadside barriers shall be provided for shielding rigid objects such as bridge piers, overhead sign supports, abutments and retaining wall ends. Other locations where roadside barriers are required are high embankments and sharp horizontal curves. Barriers shall normally be constructed of steel beams (i.e., a guardrail).

Alternatively, a modified New Jersey concrete barrier could be used.

(e) Median and Roadside Barriers

According to the typical roadway section at the initial stage of the OCH, the median width is to be 0.75+7.50+0.75 = 9.00 meters. However since the inner shoulder width, which is planned to be 1.25m, needs only to be 0.75m for marginal strip, the remaining 0.5m can be added to the median width. Therefore, the effective median width will be 0.50+9.00+0.50 = 10.0 meters. As mentioned previously, the requirement for a median barrier then becomes optional (see figures below) and can reduce construction costs. However, at some locations median barriers are still required (see preceding item).

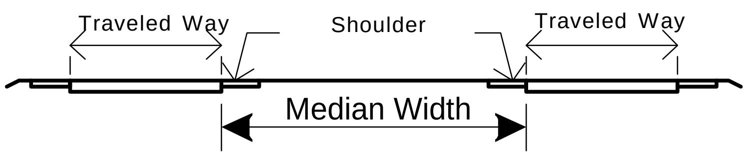


Fig. 4.13.5. Cross Section of Median

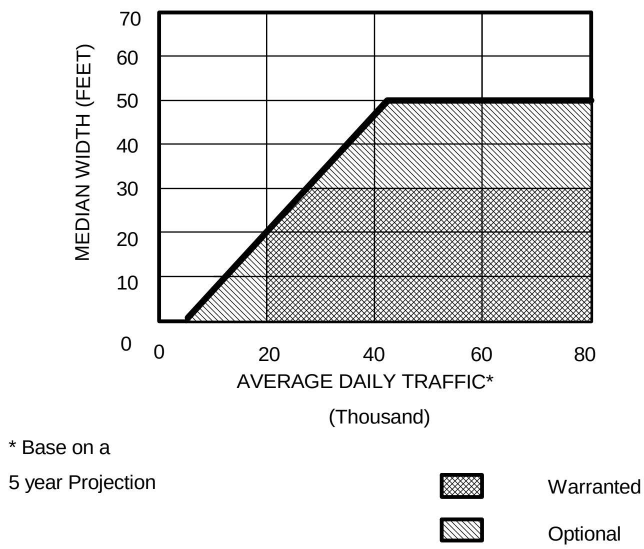
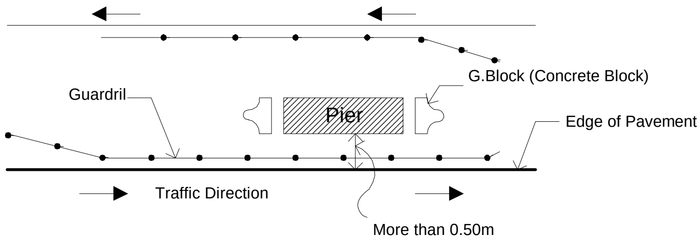


Fig. 4.13.6. Median Barrier Warrants for Freeways and Expressways

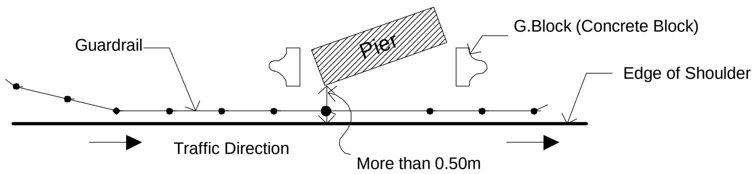
Source: Road Side Design Guide 1989 published by AASHTO

Barriers shall normally be constructed of steel beams. Alternatively the modified New Jersey concrete barrier could be used. The installation of guardrail for a bridge pier is shown in the figures below.

I. Where a pier is parallel to the roadway



II. Where a pier is skew to the roadway



For one-way or divided roads, a G-Block in the downstream direction of traffic is not required. Also, a guardrail in the downstream direction of traffic would be limited to the end of the pier.

(3) Fencing

Security fencing is installed to prevent unauthorized persons or animals from entering a national expressway or highway and to ensure the safety of traffic as well as to prevent the right-of-way from being occupied illegally. A security fence shall be installed at the following locations:

- Sections adjacent to another road
- Sections with houses or buildings nearby
- Sections where there is a difference of 3 m or more as compared to the adjacent ground
- Sections with an interchange, service area, or parking area in the vicinity
- Unused right-of-way
- Space beneath an elevated section
- Sections adjacent to future development area with the high risk of being occupied illegally
- Other sections where it is considered necessary from a security standpoint.

Notwithstanding the above, a security fence shall not be installed at the following sections:
- Sections that contain a water channel or pond and do not have any risk of being trespassed or occupied illegally

- Sections that have a retaining wall or stone masonry 1.5 m or more in height and that do not have any risk of being trespassed or occupied illegally
- Sections connected to a national highway and that have no risk of being occupied illegally because of topography or structure

(4) Curb

Curbs are installed on the throughway, the auxiliary lane of various facilities and ramps. Note that curb design for the OCH conforms to Japanese Standards.

(a) Asphalt Curb

Curbs on the throughway and ramps shall be 12 cm high and have an oblique front face with a 45° inclination (see **Fig 2.6.7**).

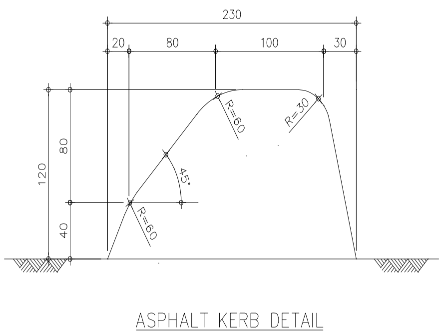


Fig. 4.13.7. Asphalt Curb

(b) Concrete Curb

Curbs at ramp medians shall be 22.5 cm in height and have a collision angle of 15° or less. It was decided that half-batter type curbs would be installed on the median of ramps at interchanges (see **Fig. 2.6.8** for details)

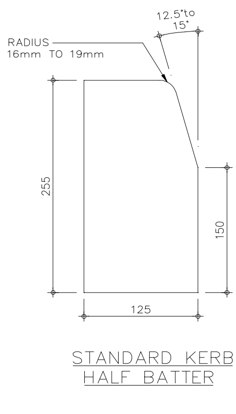


Fig. 4.13.8. Concrete Curb

4.13.2 Traffic Signs and Road Markings

(1) Traffic Signs

Road signs give drivers the necessary guidance, warnings, and instructions to ensure a safe and smooth traffic flow. Traffic signs are similar to ordinary commercial signs and billboards in the sense that they transmit information to the person who sees them. However, they are of much greater public importance.

The design of the traffic signs and mounting details for the OCH is in conformance with the Manual on Traffic Control Devices Part I of the National Road Safety Secretariat of Ministry of Transport, Highways and Civil Aviation of Sri Lanka. Traffic signs are provided alongside the road at the following locations:

- Beside a shoulder or foot walk
- If lateral clearance is limited on a shoulder or foot walk
- On a center median
- As overhead signs placed across the road.

The basic functions and types of traffic signs are as listed below.

Informative Signs for:

- Expressway entrance
- Advance direction map type sign at expressway entrance
- Exit direction sign at diverge nose
- Advance direction and notice of exit 500 m and 1 km from exit
- Notice of exit 2 km from exit
- Confirmatory sign 1 km from exit
- Advance directions at nose between Ramps
- Beginning of expressway at on-ramp
- End of expressway at off-ramp
- Route number sign for expressway (indicated at all major interchanges and approx. every 10 km)

Danger Warning Signs for:

- Left or right bend ahead
- Traffic from left merges ahead
- Reduction in number of lanes ahead

Prohibitory Signs for:

- Speed limit
- Minimum operation speed
- Road closed for cyclists, mopeds, three-wheelers, animal drawn vehicles
- No entry
- No right turn
- No left turn

(a) Advance Direction Signs

Place names on these signs are the names of the more important towns that are easily recognized by motorists. These names are to be written in Sinhala, Tamil and English. The generally recommended letter height is 75 mm to 150 mm. On overhead mounted signs, a letter height of 200 mm or 300 mm may be used.

There are two distinct types of advance direction sign; namely, a "Map Type" and "Stock Type". The map type signs indicate the layout of an intersection with arrowheads pointing towards each destination.

(b) Direction Signs

These signs are posted at intersections to direct motorists to destinations of special importance. The arrowhead indicates the direction of the destination and may be a flag type sign or with a direction symbol.

(c) Overhead Mounted Direction Signs

These signs may be mounted on gantries across the roadway or on cantilevers from the sides. Overhead signs may be either of Map Type or Lane Assignment Type. The recommended letter height is 200 mm or 300 mm.

(d) Lane Assignment Signs

Lane assignment signs are used to give information of destinations pertaining to the different lanes in a lane or at an interchange. Lane assignment signs shall always be mounted above the carriageway. The number of arrows on the sign shall be consistent with number of lanes. The arrows on the sign board shall, as for as possible, be located over the center of the lane that each arrow refers to or shall at least be within the lane limits marked by road markings. Also, the arrows shall be pointing downwards, except for existing lanes where the arrows shall be pointing upwards and be leaning towards the direction for the exit.

The lane assignment sign for expressways shall have a blue background with white symbol and text. If used on other road categories than expressways, the colour of the sign shall be white background with black symbols and text or green background with white symbols and text.

(e) Direction Signs for Expressways

Advance direction signs

Advance direction signs shall be placed ahead of an intersection and the distance of the sign from the intersection shall normally be equal to or more than the distance of the warning signs from the hazard given in the table citing details of warning signs.

Direction Signs

Direction signs shall be placed close to an intersection and the arrowhead of the sign shall clearly indicate the direction of the place being referred to.

Mounting Heights

The recommend height of the lowest edge of the sign above the level of the road shoulders or foot walk is 2100 mm. Where supplementary plots are sued along with the sign, the above recommended height shall be to the lowest edge of the plot. When overhead signs are provided the lowest edge of the sign shall be 5700 mm above the level of the road pavement.

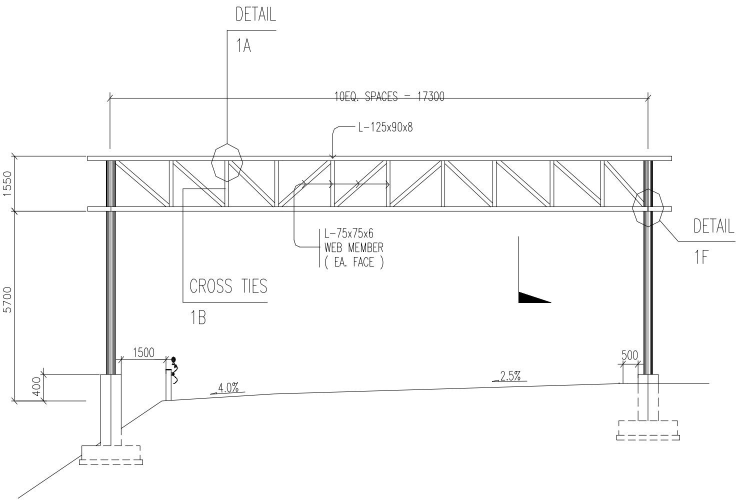


Fig. 4.13.9. Overhead Sign Boards

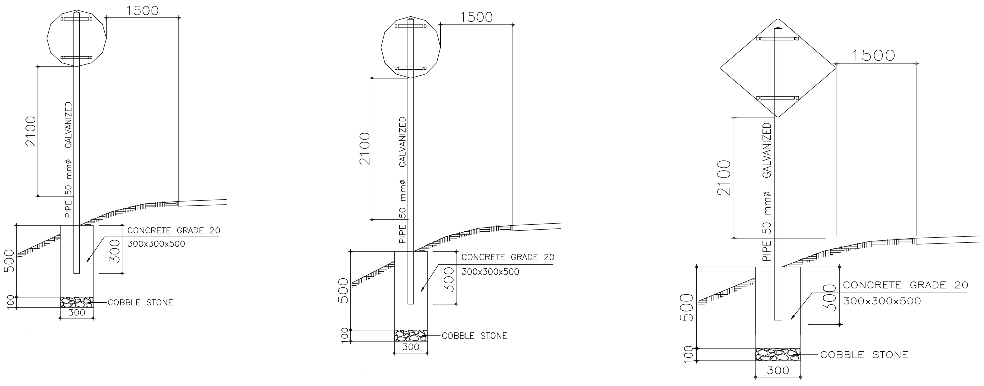


Fig. 4.13.10. Regular Sign Boards

(2) Reflectorisation of Signs

Reflectorisation of traffic signs could be achieved by using either reflective symbols or background or both depending on the sign design and local condition. Initial minimum reflectorisation R1 (cd/l/m2) efficiency for signs is measured in accordance with the procedure specified in CIE 54 using the CIE* standard illuminate A.

* International Commission on Illumination, CIE Central Bureau, Kegelgasse 27, A-1-3- Wien, Austria)

Reflectorisation (R1) efficiency of all printed colors, except white, shall be not less than 70% of the values for Class 1 (C1) and Class 2 (C2) signs, respectively.

(3) Road Markings

Road markings essentially consist of continuous lines, broken lines, letters and symbols. These may be applied on the road surface in different arrangements to convey distinct messages to the road user. These markings may be either of road paint or of thermoplastic material. Reflective studs are also used along with road markings, to better effect, both during the day and during the night.

High contrast is necessary for road markings to command the maximum attention of motorists. As the road surface is generally black in color, white and yellow are chosen as the colors for road markings in order to provide a high contrast. To ensure attention is drawn to road markings, a minimum lateral dimension of 100 mm for any longitudinal line is specified. Transverse lines need to be wider due to a perspective effect.

Markings normally convey a simple clear-cut message to a driver so that he is able to understand it readily and respond quickly. The design of the road markings for the OCH conforms to the Manual on Traffic Control Devices Part I of the National Road Safety Secretariat of Ministry of Transport, Highways and Civil Aviation of Sri Lanka and Japanese Standards.

Road Markings consist of:

- Transverse markings, which are at right angles to the centerline of a road surface
- Longitudinal markings, which are parallel to the centerline of a road
- Worded markings, which are arrows and box junctions

Note that the character of longitudinal lines is such that broken lines are permissive in nature and continuous lines restrictive and the color prescribed for all markings is white.

(a) Road Marking Materials

The material for road markings should be reflective thermoplastic material. It is important that all road markings be skid resistant during all types of weather. To enhance the reflective surface of markings (small spherical glass beads) should be added. These may be added to the road marking paint surface or to the thermoplastic material prior to the application or after application as the case may be. Quantities of balloting that should be added are given in the "Standard Specifications" and also in BS 3262 (1976).

(b) Marginal Strip

The portion of the shoulder with the same pavement structure of the traveled way is usually 0.25 m - 0.75 m in width. This is also the space for road marking at both ends of a carriageway.

(4) Road Side Delineators

These are light retro-reflecting devices mounted at the side of a roadway to indicate the roadway alignment and are considered a guidance device rather than a warning device. The design of delineators for OCH conforms to the Manual on Traffic Control Devices Part I of the National Road Safety Secretariat of Ministry of Transport, Highways and Civil Aviation of Sri Lanka and Japanese Standards.

Delineators shall consist of reflector units capable of clearly reflecting light under normal atmospheric conditions from a distance of at least 200 m when illuminated by the upper beam of standard automobile lights. The delineators may be designed to be mounted either on guardrails or on special delineator posts.

Delineator posts must be white and may have a black section on the upper part. The posts may be of flexible material. The reflective elements shall be white and consist of either height intensity reflective sheeting or prismatic reflectors. Reflective elements for delineators mounted on posts shall have an area of minimum 70 cm^2. Delineator posts, if used, shall have the top of the reflector unit about 1.2m above the roadway edge.

Delineators shall be placed not less than 1.0 m and not more than 2.0 m outside the outer edge of a shoulder or, if appropriate, in the line of a guardrail. Delineators mounted on guardrails may be placed at a height less than 1.2 m. Note that delineators should be placed at a constant distance from the edge of a roadway and are normally placed every 60 m to 100 m. On expressways and similar roads, normal delineator spacing is 100 m. Spacing should be adjusted at approaches and throughout horizontal bends so that several delineators are always visible to the driver.

4.13.3 Traffic Signals

Signal control is provided to reduce traffic conflicts and delays, allocate time between conflicting movements, and reduce accidents. Note that it is also possible to facilitate the movements of pedestrians at intersections using special pedestrian phases. Since the road layouts permit for larger center medians, pedestrians could negotiate the road in stages. Note that signal controlled intersections, even when pedestrian facilities are not provided, offer considerable assistance to pedestrian movements.

The traffic signaling design for the AB10 Interchange is based on a method adopted from the Australian Road Research Board, Research Report No. 123 (APR No. 123). The whole process involves the estimation of base saturation flows for each and every approach, effects of geometry, environment and other difficulties in the analysis of capacity and timing requirements of traffic at signalised intersections.

(1) Signal Timing Design

The method for determining such variables as cycle length, saturation ratio and signal indication intervals for traffic signals, is called ‘signal timing design.’ Its general procedure is shown in **Fig. 4.13.11**

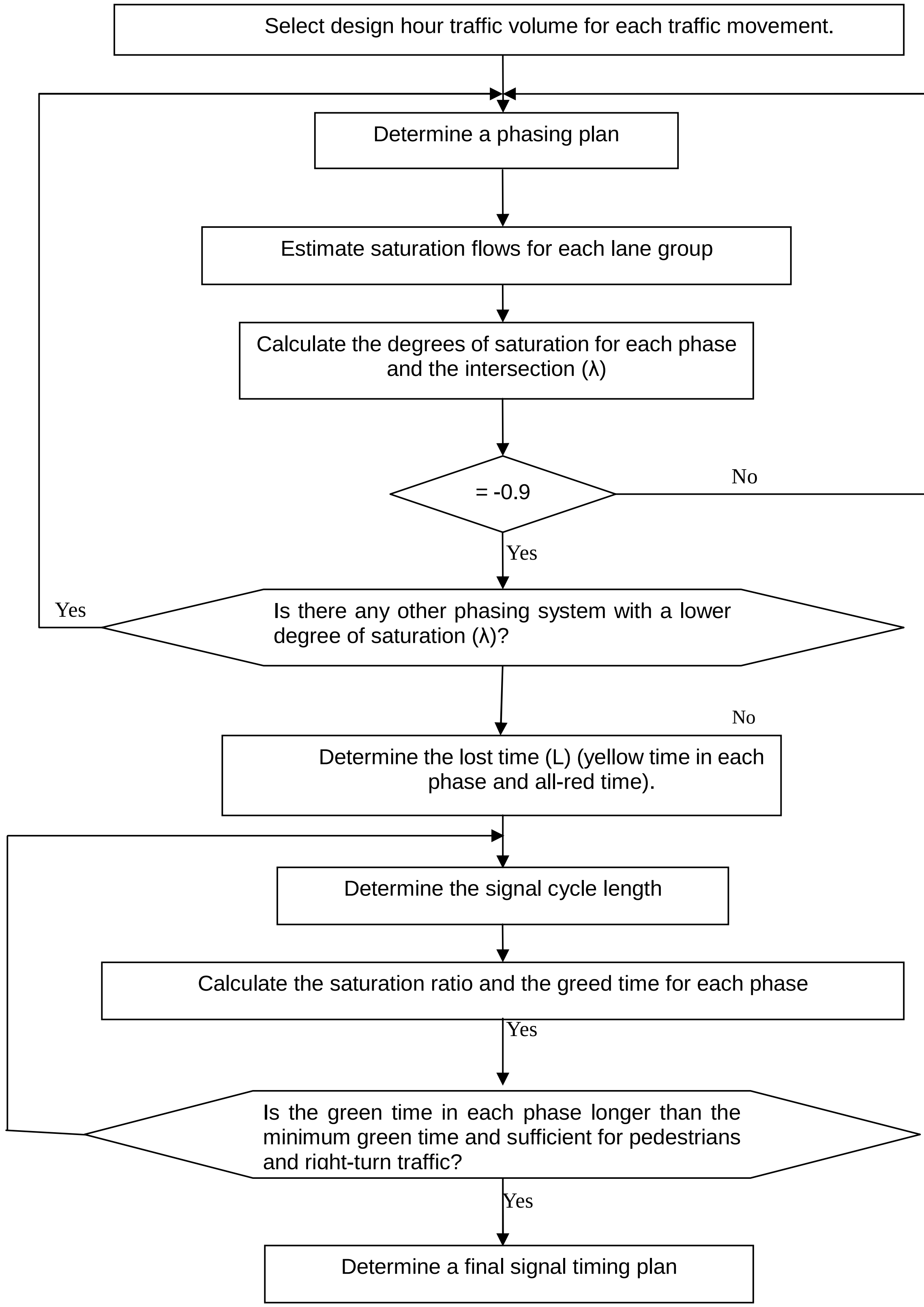


Fig. 4.13.11 Process for Signal Timing Design

(2) Determination of Signal Phasing & Calculation of Degree of Saturation for an Intersection

(a) Determination of Signal Phasing Method

Signal phasing is determined by considering the road geometry and traffic conditions (directional flows, vehicle type compositions, etc.) of an intersection. Typical steps in the process of phasing design are as follows:

1. Identify traffic streams for different directions for each approach.
2. Classify traffic movements into non-conflicting groups and designate a phase for each group.
3. Determine indication sequence of the phases.

The number of the phases should be kept minimum to reduce the lost time inherent in the changing of phases.

(b) Calculation of Degree of Saturation for Phase and Intersection

Calculate the degree of saturation for Phase I (rho zero i) and the degree of saturation for the intersection by using the following equation, where q_i denotes the design volume of an approach in Phase I and s_i the saturation flow rate of that approach in Phase i.

Flow ratio: $\rho_i = q_i / S_i$
Degree of saturation in phase i: $\rho°\iota = Max\{ \rho_i, \ \rho^1{}_c, \ \rho^n{}_i \ldots\ldots\ldots\}$
Degree of saturation at intersection: $\lambda = \sum_i \rho°\iota$

(3) Determination of Lost Time

Lost time (L) in signal control occurs during the change of phases and cannot be used to let vehicles pass through an intersection. Such lost time is the sum of the clearance time and the starting delay. The former is the time required for clearing vehicles by avoiding collisions with crossing vehicles and the latter is the lost time due to the fact that the saturation flow rate cannot be reached immediately after right-of-way is provided. In practice, a part of the yellow time provided for clearance purposes is used as the effective green time, but the beginning of the green time of the succeeding phase is not used for that purpose due to starting delay. For this reason, the sum of the lost time (L) in the signal cycle can practically be obtained by adding the yellow time and the all-red time to the cycle.

(a) Determination of Yellow Time and All-Red Time for a Phase

The sum of the yellow time (Y) and the all-red time (AR) of a phase equals the clearance time needed to discharge vehicles from the intersection, and its duration varies according to the speed of the traffic on the approaches, and the physical dimensions of the intersection.

(4) Determination of Cycle Length Time

(a) Determination of Cycle Length

The cycle length is determined from the degree of saturation of the intersection (lambda) and the lost time (L) within the cycle. At an intersection with relatively low traffic volume and a highly random arrival pattern, the value determined by the following equation can be used as a guide for an optimum cycle length.

$$C_o = \frac{1.5\,L + 5}{1 - \lambda} \qquad \text{.... (1)}$$

The above equation sometimes gives values somewhat higher than required, particularly in cases where traffic volume is relatively high and the traffic arriving is not random. If the C_p determined by Equation (1) is bigger than the C_{min} in Equation (2), then the C_p should be considered in practicality the longest cycle length (in the range of 120 to 180 seconds).

$$C_{min} = \frac{1.5\,L + 5}{1 - \lambda_0} \qquad \text{.... (2)}$$

Where, λ_0 is the degree of saturation of the intersection by setting the design saturation flow rate at 0.9 times the (possible) saturation flow rate, which is as follows:

$$C0 = \frac{1}{0.9}\ \lambda.$$

The C_{min} of Equation (2) is usually smaller than the C_0 of Equation (1). This is not, however, always true and $C_{min} > C_0$ when $\lambda > (0.45L + 4.5)\ /\ (0.6\ L + 5)$. Hence, by making C_0 the cycle length, it has to be admitted that delay will be too large as a practical cycle length (i.e. $C_0 >= 180$) and alternative phasing patterns or intersection geometrics should be examined.

(b) Calculation of Saturation Rations and Green Time

The saturation ratio of a phase is obtained as the ratio between the degree of saturation of an intersection (lambda) and the saturation rate of the phase rho zero i. The green time of a phase (G_i) is calculated by the following equation by using the cycle length (C), the lost time and the saturation ratio.

$$G_i = (C - L) \ \text{X}\ g_i = (C - L)\ \text{X}\ \rho^{1}{}_{c},\lambda \text{.......................................} (3)$$

(c) Minimum Green time (for vehicular traffic)

For safety purposes the necessary green time of a phase is calculated with Equation (3) and should be bigger than the minimum value shown below. It is necessary that the green time for vehicular traffic be 15 seconds longer for major traffic (usually through traffic) and 5 seconds for minor traffic (such as a non-exclusive right-turn phase).

As for the phase for pedestrians, the minimum green time is determined by the method explained below.

When cycle length is determined for proposed phasing, vehicles turning right should be considered. In the case where there is no exclusive phase for right-turns, it should be carefully examined whether or not right turning vehicles are assured of completing the turning movements. This can be judged form the opposing through-traffic volume, cycle length and split length.

(d) Preparation of Final Signal Timing Plan

Cycle length is extended from the viewpoint of minimum green time and crossing pedestrians and is shortened from the viewpoint of right-turning vehicles. As changes in signal cycle lengths and phasing plans occur, the above process should be repeated to prepare an optimum signal timing plan.

(5) Signal Displays and Their Locations

(a) Types of Signal Displays

Based on the position of the lantern, it may be categorized as a primary, secondary, tertiary or dual primary lantern.

- *PRIMARY LANTERNS*

A primary lantern for any approach is that display mounted on a post on the approach side of an intersection at or near the left of the stop line.

- *SECONDARY LANTERN*

The secondary lantern for an approach is the lantern mounted on a post on he far right on departure.

- *TERTIARY LANTERN*

The tertiary lantern for an approach is that lantern display mounted on a post to the left of an approach on the departure side of an intersection.

- *DUAL PRIMARY LANTERN*

The dual primary lantern for any approach is that lantern display mounted on a post at or near the right of the stop line on the approach side of an intersection.

(6) Locations

Lateral Positions

A curb/shoulder side post and mast arm are located 1.0m from a curb/shoulder and face towards the curb/shoulder edge and are not closer than 0.6m. Center median posts are located centrally in a median.

Longitudinal Positions

- Primary Lantern: Primary and dual primary posts are placed at the projection of the adjacent stop line and are not more than 3.0 m beyond. They are placed in advance of the adjacent stop line. Posts are placed not less than 1.2m from an island nose.
- Secondary Lanterns: These lanterns are placed on the primary post of the opposite approach.

CHAPTER 5

SOFT SOIL COUNTERMEASURES

CHAPTER 5 SOFT SOIL COUNTERMEASURES

5.1. General

This section of the report summarizes the results of the study for slope stability and embankment settlements with the necessary countermeasures on soft soil layer along the OCH Northern Section 1 (Sta.08+200 – 16+560), which consists of peat, organic & inorganic-clay and loose sand.
During the analysis and study, further discussions on the selection of soft soil countermeasures were made between RDA and the JICA Study Team. It is therefore a consequent which the countermeasures proposed were selected taking into account the technical requirements and the clarifications based on the practical experiences of on-going highway projects in Sri Lanka.

5.2. Subsurface Investigation

5.2.1. Geotechnical Investigation Survey

The geotechnical investigation for OCH was initially planned in 2001 for basic design of OCH, however it couldn't be carried out due to negative social impact.
After the series of discussions upon the soft soil ground treatments with the RDA, the Geotechnical Investigation for OCH Northern Section1 has been carried out by JICA Study Team in the period between November 2006 and July 2007.
According to topography and soil investigation result shown in Chapter 2, along 8km of the total alignment length of OCH Northern Section 1, the soft soils are distributed at marsh, small valley and paddy field area in about 4km long with the depth of 1 to 8 meters.

5.2.2. Materials Survey

Materials survey for OCH was finalized at the end of August 2004 (see details in Material Survey Report (2001) and Material Survey Report (2004) Vol. 2A and 2B), however the report of design review carried out by RDA (January, 2007) has pointed out decrease of numbers of borrow pits due to abandonment and suspended of borrow pits operation.
Because of these reasons, materials survey for OCH Northern Section 1 was carried out again in this study as shown in Chapter 2, the design parameters for soft soil treatment were selected taking into account these material and geological investigation results and also the practical experiences of on-going highway projects in Sri Lanka.

5.3. Analysis Method and Procedure

5.3.1. Slope Stability Analysis

(1) Local Stability

The local stability of the embankment side slope, is considered as follows:

$$\frac{H}{L_s} \leq \frac{\tan \varphi'_{cv}}{f_{ms}}$$

where, H = is the height of fill in the embankment;

L_s = is the horizontal length of the sideslope of the embankment;

φ'_{cv} = is the large strain angle of the friction of the embankment fill under effective stress conditions;

f_{ms} = is the partial material factor applied to tan φ'_{cv}, BS8006 recommends $f_{ms} = 1$

In this project, fill material is sandy soil ($\varphi'_{cv} = 30°$) then the side slope of 1:1.8 for embankment is satisfied.

(2) Overall Stability

The modified simplified method for slope stability analysis, which uses circular slip surfaces and slices, has been applied to confirm embankment stability and was carried out with a commercial computer program. The factor of safety is a constant proportion of applied strength at every point on a surface to potential sliding resistance and is determined as follows:

$$F.S. = \frac{MR}{MO}$$

Where, MR = Resisting moment
MO = Overturning moment

It can be expressed as follows;

$$Fs = \frac{\sum(C_u \cdot \ell + W \cdot \cos\alpha \cdot \tan\phi_u)}{\sum W \cdot \sin\alpha}$$

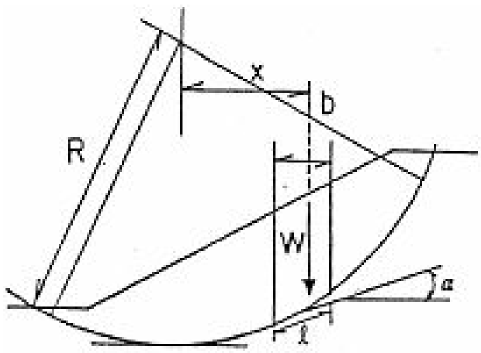


Fig. 5.3.1 Circular Slip of Embankment

Where
l : length of slip line of a block
W : Total weight of a block
α : angle of inclination of slip line

The factor of safety for stabilizing slopes with geotextile is calculated by adding geotextile tensile resistance, which acts as an additional resisting moment to slope failure and is calculated as follows:

$$F.S. = \frac{MR + (T \times R_1)}{MO}$$

Where, T = Geotextile tensile resistance
R_1 = Distance from center of rotation to geotextile position

(3) Improvement of Stability by Gravel Compaction Pile (GCP)

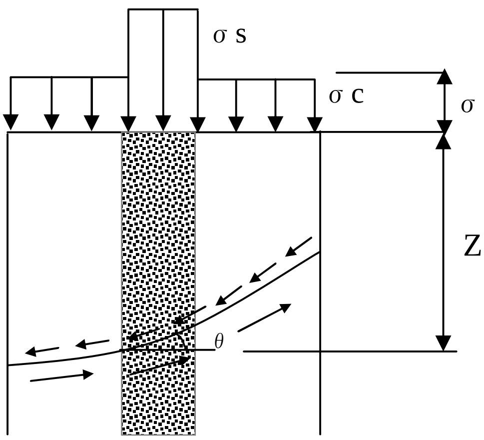
Soft ground treatment by Gravel Compaction Pile (GCP) is planned in OCH Northern Section 1 to secure the stability of high embankment. Due to usage of GCP, composite foundation composed of soft soil and gravel pile is made and increased strength of the foundation also increases factors of safety against failure of embankment stability and reduces settlements.

The shear strength of composite soil (τ) by GCP is calculated using the following equation:

$$\bar{\tau} = (1-a_s)(c_u+\Delta c) + (\gamma'_s \cdot z + \mu_s \cdot \sigma)\cdot a_s \cdot \tan\phi_s \cdot \cos^2\theta$$

$$\Delta c = c/p \cdot (\gamma'_c \ z + \mu_c \cdot \sigma - P_c)\cdot U \qquad \sigma_s = \sigma\left[\frac{m}{1+(m-1)a_s}\right] = \mu_s\sigma$$

$$\sigma_c = \sigma\left[\frac{1}{1+(m-1)a_s}\right] = \mu_c\sigma \qquad m = \sigma_s/\sigma_c$$



U: Consolidation degree of Soft Soil
Cu: Cohesion of original soil
γ s: Unit volume weight of pile sand
γ c: Unit volume weight of Soft Soil
ϕ s: Friction angle of pile edge
θ :Horizontal angle of sheared edge
μ s: Stress concentration factor of sand
μ c:Stress concentration factor of Soft Soil

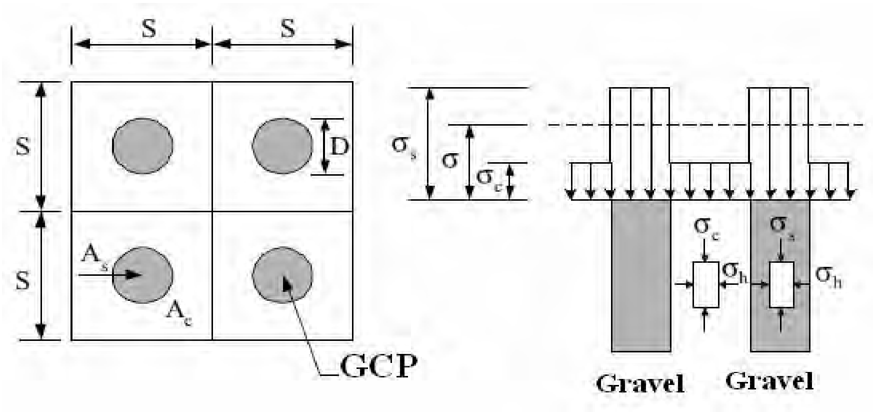


Fig. 5.3.2 Concept of Composite Ground by Gravel Compaction Pile (GCP)

Settlement working on soft soil ground is reduced by stress concentration phenomena of composite ground. Reduced settlement is calculated by following equation;

Reduced Settlement S = β x Sf

Where, Sf : Final settlement of untreated ground,
β = μc : Settlement Reduction Factor β = μc
m : stress concentration ration (= 4 in case of GCP).

5.3.2. Settlement Analysis

(1) Immediate Settlement Calculation

In coarse and fine soils, any volume change resulting from a change in loading occurs immediately. This is so called immediate settlement.
The immediate settlement can be evaluated by used of a semi-empirical strain influence factor proposed by Schmertmann and Hartman (1978). Acccording to this method, the immediate settlement is calculated as follows;

$$S_e = C_1 C_2 (\bar{q} - q) \sum_0^{z_2} \frac{I_z}{E_s} \Delta Z$$

Where, I_z = strain influence factor

C_1 = a correction factor for the depth of foundation embedment $= 1 - 0.5\left[\frac{q}{(\bar{q} - q)}\right]$ = 1.0 (q = 0)

C_2 = a correction factor to account for creep in soil

= $1 + 0.2\log\left(\frac{\text{time in year}}{0.1}\right)$ = 1.1 (time in year = 0.25)

$\bar{q}$ = stress at the level of the foundation

q = $\gamma D_f = 0$ ($D_f = 0$)

Es = Young's Modulous (Es=100 Cu for cohesive soil,
Es=390+45N by Schultze & Menzenbach
Where Cu: Cohesion, N: N value of SPT)

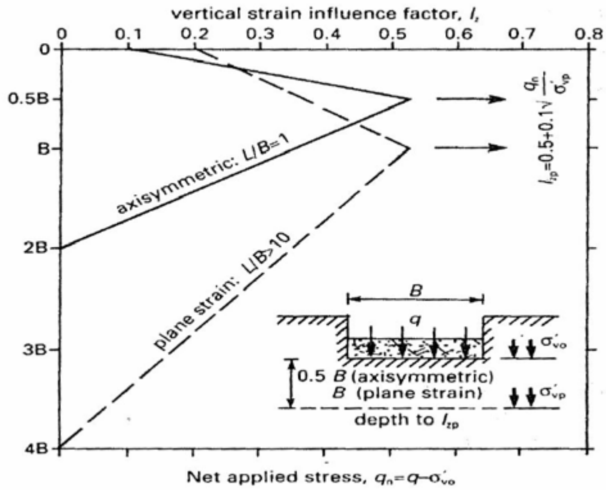


Fig. 5.3.3 Vertical Strain Influence Factor (after Schmertmann et al, 1978)

(2) Primary Consolidation Settlement

When soil is loaded undrained, the pore pressures should increases. Then, under site conditions, the excess pore pressures dissipate and water leaves the soil, resulting in consolidation settlement. This process takes time, and the rate of settlement decreases

over time. The amount of settlement which occurs in a given time depends on the permeability of the soil, length of the drainage path and compressibility of the soil.
It should consider the site on clay soil with initial steady-state groundwater conditions. An embankment is built, the loading is undrained: the pore pressure in the soil increases, seepage flow and therefore volume changes commences. As consolidation takes place, settlement occurs, and continues at a decreasing rate until steady-state conditions are regained.

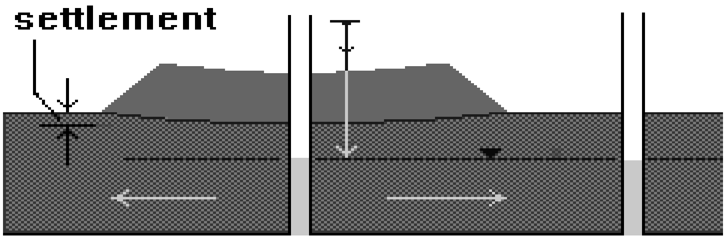


Fig. 5.3.4 Process of Consolidation

In herewith, the Terzaghi theory of one-dimensional consolidation is used to calculate primary consolidation settlement by applying the following function:

$$S_c = H\left[RR \times \log\frac{P'_c}{P'_o} + CR \times \log\frac{P'_o + \Delta P}{P'_c}\right]$$

Where,	Sc	=	consolidation settlement
	H	=	compressible layer thickness
	RR	=	recompression ratio
	CR	=	compression ratio
	P′c	=	maximum past pressure
	P′o	=	effective overburden pressure
	ΔP	=	Increased vertical stress by applied load

Increased vertical stress ΔP by applied load is calculated by the following formula and Osterburg's Chart shown in **Fig.5.3.5** for vertical stress by trapezoidal load.

$$\Delta p = I \cdot q_E = I \cdot H_E \cdot \gamma_E$$

Δp : Increased vertical stress
I : Influence factor
q_E : Embankment Load
H_E : Embankment Height
γ_E : Unit weight of embankment

Time and settlement relation is calculated base on the following theory;

- Untreated ground : Terzaghi's Theory
- Treated by Band Drains : Barron's Theory

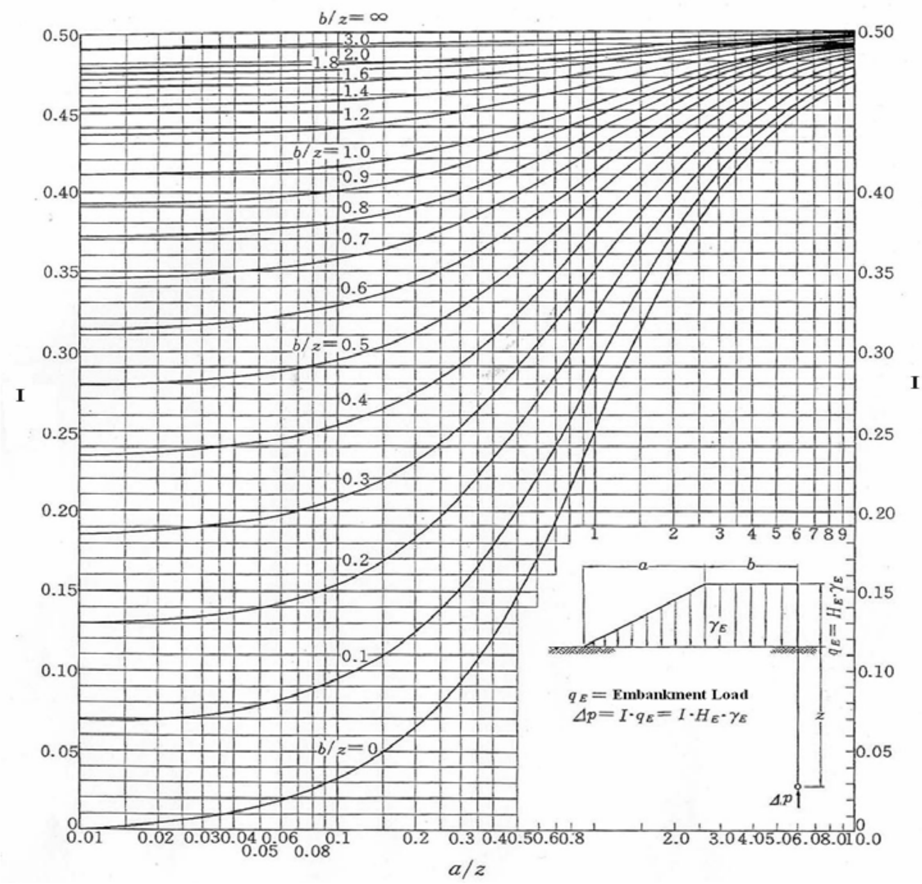


Fig. 5.3.5 Influence Factor (after Osterberg)

1) Untreated Ground

In case of untreated ground, time and settlement relation is calculated based on Terzaghi's Theory. Pore water of soft soil is discharged in vertical direction as shown in **Fig 5.3.6**.

$$S_t = U \cdot S_f$$

$$U = 1 - \frac{8}{\pi^2} \cdot \sum_{n=0}^{\infty} \frac{1}{(2n+1)^2} \cdot \exp\left\{-\left(\frac{2n+1}{2} \cdot \pi\right)^2 \cdot T_v\right\}$$

$$T_v = \frac{C_v \cdot t}{D^2}$$

Where St : Settlement at time t
Sf : Final settlement
U : Consolidation degree
Tv : Time factor for vertical drainage
Cv : Coefficient of consolidation for vertical drainage
t : Elapsed time after loading
D : Length of drainage path

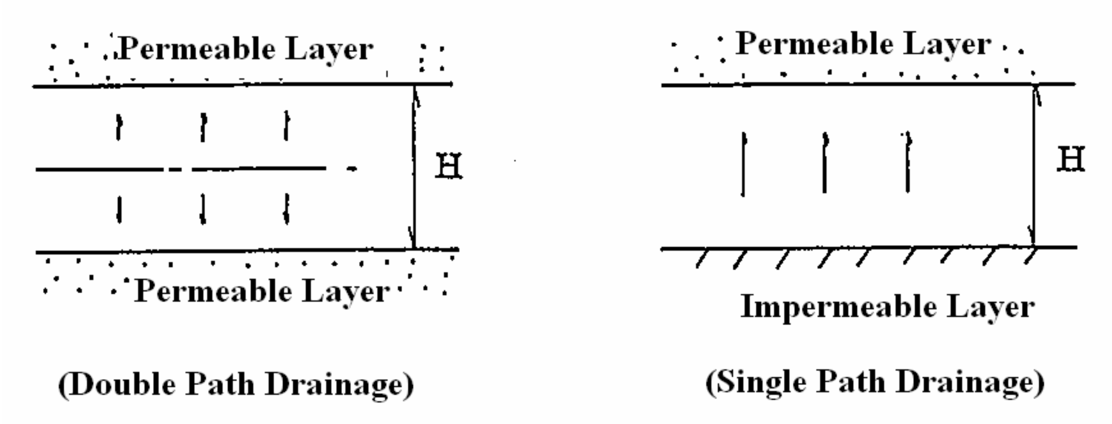


Fig. 5.3.6 Drainage Path of Consolidation

2) Treated Ground by Band Drains

In case of treated ground by band drains, time and settlement relation is calculated based on Barron's Theory because pore water of soft soil is discharged in horizontal direction through band drains as shown in **Fig 3.2.5**.

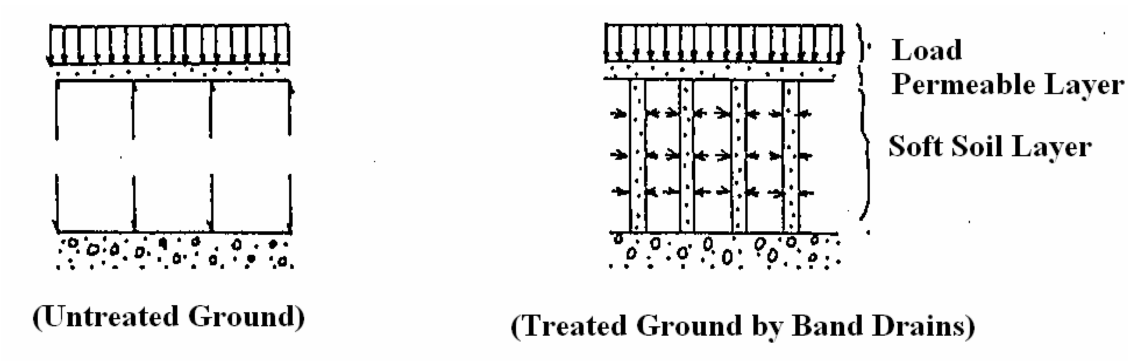


Fig. 5.3.7 Drainage Direction for Treated Ground by Band Drains

$$S_t = U \cdot S_f$$

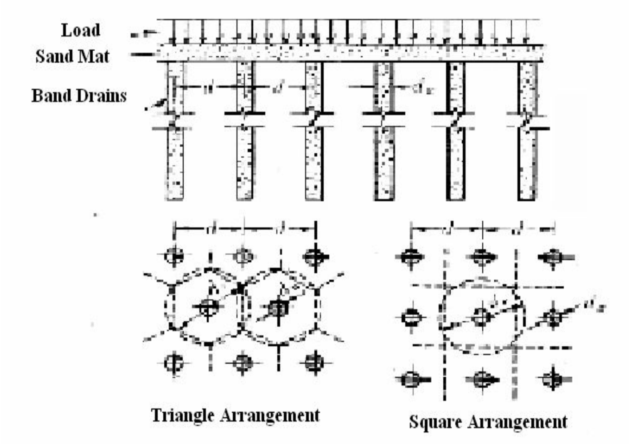
$$U = 1 - \exp\left(\frac{-8T_h}{F(n)}\right)$$

$$T_h = \frac{C_h \cdot t}{d_e^2}$$

$$F(n) = \frac{n^2}{n^2 - 1} \cdot \log_e n - \frac{3n^2 - 1}{4n^2}$$

$$n = \frac{d_e}{d_w}$$

Where St : Settlement at time t
Sf : Final settlement
U : Consolidation degree
Th : Time factor for horizontal drainage
Ch : Coefficient of consolidation for horizontal drainage
t : Elapsed time after loading
D : Length of drainage path
de : Diameter of effective circle
dw : ameter of band drain



de=1.05d (Square arrangement)
de=1.13d (Regular triangular arrangement)
where d : Interval of band drains

Fig. 5.3.8 Effective Diameters by Arrangement Type

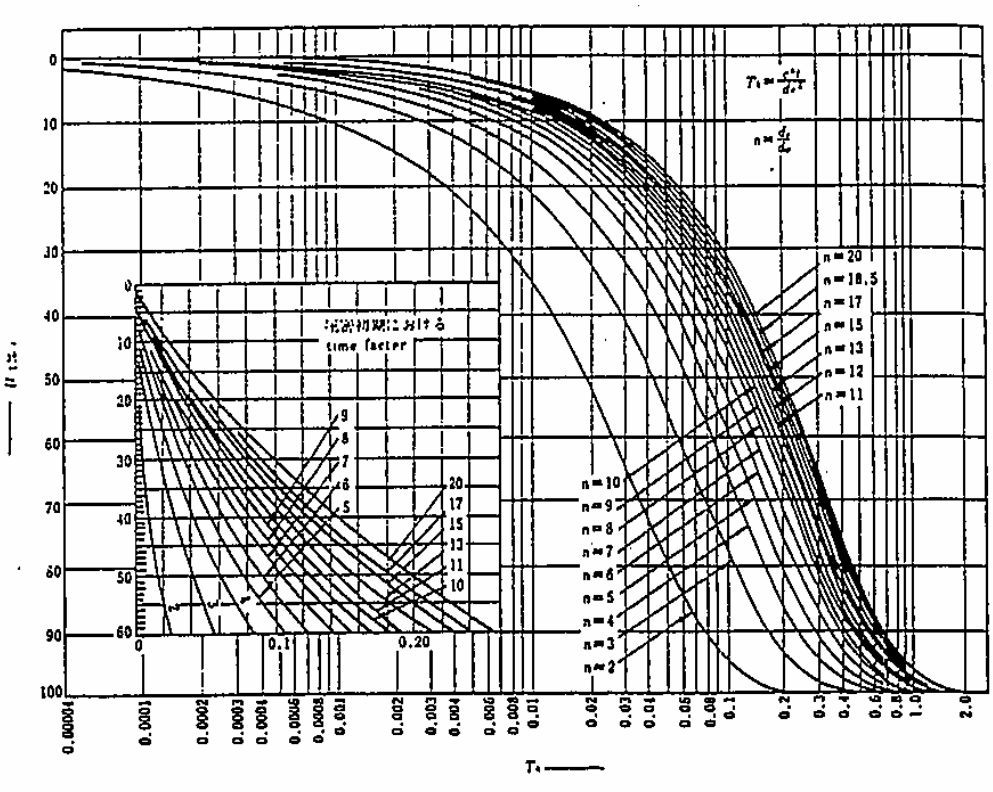


Fig. 5.3.9 Relation between Time Factor and Consolidation Degree
(Treated by Band Drains)

3) Multi-Layered Soil

a) Untreated Ground

The equivalent thickness method is applied to calculate the time and settlement relation of multi-layered soil for untreated ground.

In the equivalent thickness method, equivalent thickness of each layer is converted by the following formula using the any representative coefficient of consolidation (Cv). Then time and settlement relation of multi-layer soil is calculated as one layer with total converted thickness of layers by Terzaghi's formula.

$$H_0 = \sqrt{\frac{Cv_0}{Cvi}} \cdot Hi$$

where H_0 : Thickness of layer converted
Cv_0 : Representative Cv
H_i : Thickness of the i-th layer
Cv_i : Coefficient of consolidation of the i-th layer

b) Treated Ground by band drains

In case of treated ground by band drains, Barron's theory is applied to each layer and calculated settlement for each layer. The time and settlement relation of treated ground by band drains is come out by summing up the settlement of each layer.

(3) Secondary Consolidation Settlement

In some soils (especially organic and peat) one-dimensional compression continues under constant loading after all of the excess pore pressure has dissipated, for example after primary consolidation has ceased - this is so called secondary compression or creep.

The secondary consolidation settlements were hereunder calculated based on the following equation:

$$\rho_S = \sum_{l=1}^{i=n} C_{\alpha i} \log\left(\frac{t_f}{t_{100}}\right) H_i$$

Where,
$C_{\alpha i}$ = secondary consolidation coefficient
t_f = time, $t_f > t_{100}$
t_{100} = 100% primary consolidation time (Usually secondary compression curve appears at a consolidation degree between 90 and 100 %. Therefore the time of 90 % of consolidation degree was taken as t_{100}.)
H_i = thickness of compressible layer

After the series of discussions between the RDA and the JICA Study Team, the criterion for consolidation settlement was agreed as Residual Settlement (Rs) shall be less than less than 15cm (primary + secondary) after 3 years from traffic opening.

The secondary consolidation settlement defines to incur after almost 100% of primary consolidation settlement or excess pore pressure becomes zero. In our project, all the primary consolidation are assumed to complete within 1- 2 years after constructing embankment. Therefore, Rs (15 cm) after 3 years will be almost only secondary consolidation settlement.

(4) Increase of Shear Strength for Soft Cohesive Soil

The increase of shear strength after loading due to consolidation can be considered by the following formula. If embankment can be maintain the stability considering increase of shear strength, staged construction method is applied. In case band drains is installed, they are designed to perform 90% of consolidation degree in six months after loading.

For $P_0+\Delta P \leqq Pc'$, $Cu = Cu_0,$

for $P_0+\Delta P > P_c'$, $Cu = Cu_0 + Cu/P\times(P_0-P_c'+\Delta P)\times U$

Where
- Cu_0 : Undrained strength of soil before loading
- Cu/P : Increase ratio of shear strength
- P_0 : Overburden pressure before loading
- P_c' : Cu_0 / (Cu/P)
- ΔP : Increased stress by loading
- U : Consolidation degree

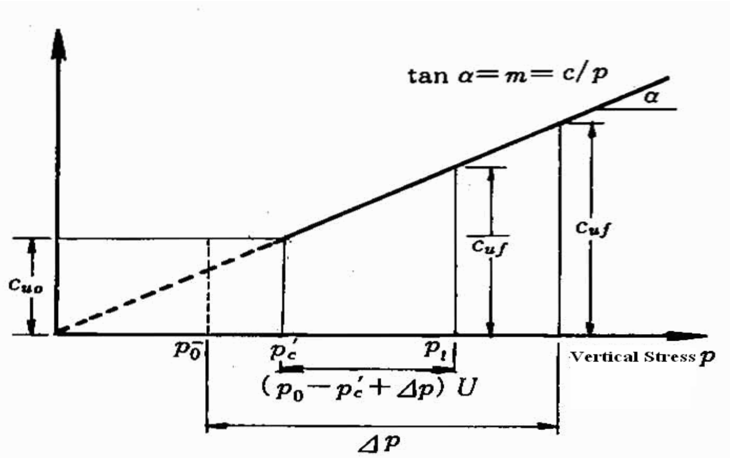


Fig. 5.3.10 Concept for Increase of shear strength

5.4. Design Parameters

Design parameters for soft soil treatment have been evaluated and selected for the sake of further analysis and design of embankment after several times of discussions between JICA study team and RDA as below.

5.4.1. Design parameters for Stability Analysis

The angle of internal friction of sand is evaluated from correlations between SPT-N and ϕ as shown in **Table 5.4.1**. It was resulted that the average angle of internal friction of sand varies from 27°-33° for loose to medium dense sand.

Table 5.4.1 Correlations between SPT-N and ϕ for Sand

SPT-N Blows (Blows/300 mm)	Relative Density	Angle of Internal Friction (ϕ), (deg.)
0 – 4	Very Loose	< 28°
4 – 10	Loose	28° - 30°
10 – 30	Medium Dense	30° - 36°
30 - 50	Dense	36° - 41°
> 50	Very Dense	> 41°

Source: Peck, R.B., Hansen, W.E., and Thornburn, T.H. (1974), Foundation Engineering, 2nd ed., John Wiley & Sons, New York, USA.

The shear strength, which implicates the characteristic of cohesive soil, is determined based on SPT-N and shear strength of clay as shown in **Table 5.4.2**.

Table 5.4.2 Correlations between SPT-N and S_u for Clay

SPT-N Blows (Blows/300 mm)	Consistency	Undrained Shear Strength S_{uc} (kN/m^2)
< 2	Very Soft	< 15
2 – 4	Soft	15 - 25
4 – 8	Medium	25 - 50
8 – 15	Stiff	50 - 100
15 – 30	Very Stiff	100 - 200
> 30	Hard	> 200

Source: Peck, R.B., Hansen, W.E., and Thornburn, T.H. (1974), Foundation Engineering, 2nd ed., John Wiley & Sons, New York, USA.

Considering the above empirical correlation between SPT N and internal friction angle for sandy soil, undrained shear strength for cohesive soil and also field and laboratory tests result, soil parameters for embankment stability analysis has determined.
After the series of discussions between RDA and JICA DD study team for OCH, the following design parameters for stability analysis of embankment are agreed and applied as shown in **Table 5.4.3.**

Table 5.4.3 Soil parameters for embankment stability analysis

Soil Layer	**Symbol**	**Cohesion Cu (kpa)**	**Friction Angle ϕ**	**Unit Weight γt (kN/m3)**	**Submerged Unit Weight γ' (kN/m3)**	**m (= Cu/P)**	**Remarks**
Peat	Pt	5	0	13	3	0.40	I)
Organic Clay	Oc	5	0	14	4	0.30	I)
Alluvial Clay	Ac	10	0	16	6	0.30	I)
Alluvial Sandy Clay	Acs	10	0	17	7	0.30	I)
Alluvial Sand	As	0	30	18	8	-	II)
Residual Soil	Rs	50 - 200	0	18	8	-	II)
Completely Weathered Rock	Wr	50 - 200	0	18	8	-	II)
Embankment Material (Sandy Soil)	E	0	30	20	10	-	

I) Cu is selected incase of less than or equal to 1 of SPT N-value. In case of more than 1 of SPT N-value, Cu is estimated by the formula; Cu = 20N/3 (kPa).
II) Cu and ϕ are estimated by Cu = 20N/3 for cohesive soil and from ϕ=0.3N+27 (by Peck) for cohesionless soil.

5.4.2. Design Parameters for Consolidation Settlement

1) Design Parameters for Primary consolidation settlement
Consolidation settlement is calculated based on the design parameters; P'c, CR, RR, $C_{\alpha\varepsilon}$ and Cv.

The coefficient of consolidation (C_v) is the representative parameter for determining the rate of consolidation. The average Cv is calculated from the normal compression line (NCL) in which the effective stress is linearly increased.
Note that the current state of soil is on the normal compression line it is said to be normally consolidated. If the soil is unloaded it becomes overconsolidated.

As an empirical method for determination of coefficient of secondary consolidation ($C_{\alpha\varepsilon}$), there is one relation between natural water content (Wn) and $C_{\alpha\varepsilon}$ given by Mesri et al (1997) as shown in **Fig. 5.4.1.** According to this figure, The value of $C_{\alpha\varepsilon}$:0.015, for example, is given by cohesive soil with having the natural water content of 150%.

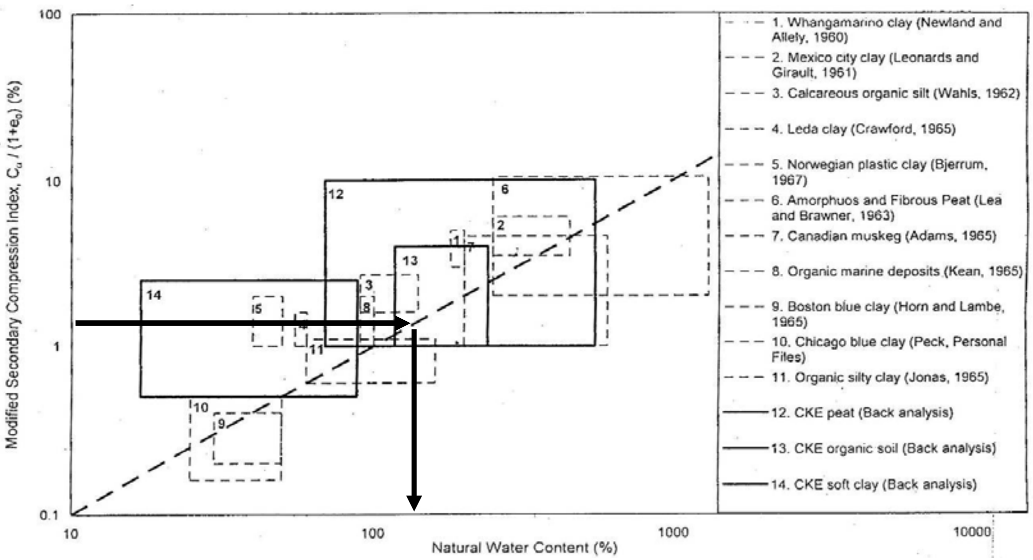


Fig 5.4.1 Modified Secondary Compression Index versus Natural Water Content (Mesri et. al (1997) and after Mesri

Considering the above empirical correlation between natural water content and CR, RR, $C_{\alpha\varepsilon}$ and Cv, and field and laboratory test result and also based on the practical experiences of on-going highway projects in Sri Lanka, soil parameters for settlement analysis has been determined.

After the series of discussions between JICA DD study team and RDA, the following design parameters for settlement analysis of embankment are agreed and applied as shown in **Table 5.4.4**.

Table 5.4.4 Soil parameters for settlement analysis

Soil Layer	Symbol	Compression Ratio CR Cc/(1+e0) 111)	Recompression Ratio RR Cs/(1+e0) IV)	Secondary Consolidation Factor Cαε Cα/(1+e0) V)	Coefficient of Vertical Consolidation Cv (m2/year) VI)	Coefficient of horizontal Consolidation Ch (m2/year) VII)	Remarks
Peat	Pt	0.45	0.045	0.055	5	10	
Organic Clay	Oc	0.35	0.035	0.035	1	2	
Alluvial Clay	Ac	0.30	0.030	0.030	1	2	N<8
Alluvial Sandy Clay	Acs	0.23	0.023	0.0075	2	4	
Firm Clay	Ac	0.23	0.023	0.015	1	2	N>8
Firm Organic Clay	Oc	0.25	0.025	0.025	1	2	N>8

III), V), VI) VII) : Values instructed by RDA
III) : RR=CR x 0.1 (empirically RR=CR x (0.1-0.2))

5.5. Conditions for Analysis

5.5.1. Sections and Models for Analysis

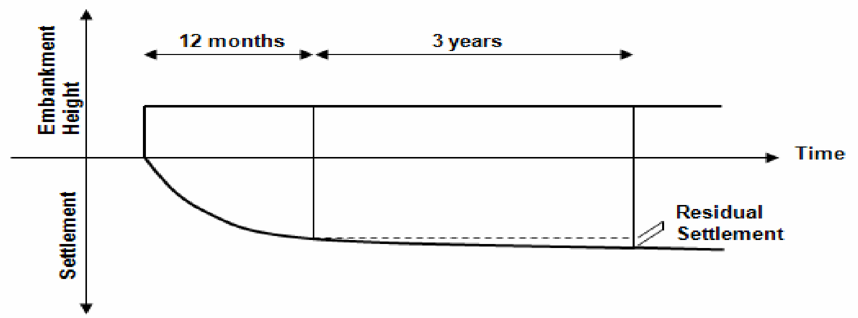
Analysis has been carried out for 36 sections between Sta.16+500 - 28+500. Analysis model for each sections are tabulated in **Table 5.5.1**.

5.5.2. Allowable Residual Settlement

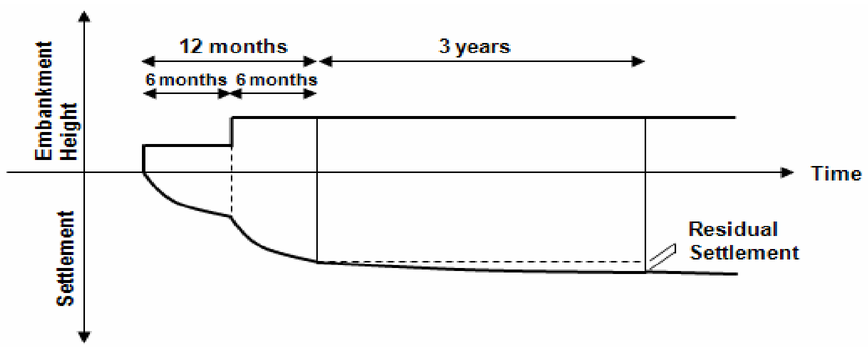
Allowable Residual Settlement (Rs) shall be less than less than 15cm (primary + secondary) after 3 years from traffic opening.
In this analysis, traffic opening is assumed to be one year (12 months) later after the completion of embankment from safety point of view for construction period. Therefore residual settlement is calculated as follows ;

$$Rs = (St4 - St1) + Ssr$$

Where St4 : Settlement of primary consolidation at the time of 4 years later from completion of embankment. (t4=1460 days))
St1 : Settlement of primary consolidation at the time of 1 year later from completion of embankment (Assumed that it is equal to the time of traffic opening.) (t1=365days)
Ssr : Residual secondary consolidation settlement for three years between the time t1 and t4. (Ssr = Cαε x log (t4/t1) x H, where H: thickness of soft soil, Cαε: Coefficient of Secondary Consolidation.)



(a) One step Loading



(b) Two Steps Loading

Fig. 5.5.1 Residual Settlement of soft ground by consolidation

Table 5.5.1-1 Sections and Model for Analysis

Section No.	Road	Section and Embankment Height						Ground Conditions					Soil Parameter										
		Sec.	Analysis Sec.	Analysis	Zone	Bor.	Emb. H	Total T. of Soft to Medium Soil	Layer	Thick.	SPT N	Ave. N	Cu/Pc	Pc	P0	CR	RR	Cαe	γ	γ'	Cu	φ	Young's E
		No.	No.	Sta. No.	Length	No.	(m)	(m)	Name	(m)	(Blows)	(Blows)		(KN/m2)	(KN/m2)				(KN/m3)	(KN/m3)	(KN/m2)	(degree)	(KN/m2)
1	A1-bypass	21	1	0+225 to 0+370	145	A1-BH-05	8.0	3.7	Ac	1.00			0.30	33.3	3.0	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Oc	2.00	1, 2	1.5	0.30	33.3	10.0	0.35	0.035	0.0350	14	4.0	10.0	0	1,000
									Ac	0.65	1	1.0	0.30	33.3	16.0	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Br	1.00	50<	50.0							18	8.0	200.0	0	20,000
		22	2	0+370 to 0+480	110	A1-AB-02	5.0	1.9	Ac	0.50			0.30	33.3	1.5	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Acs	1.40			0.30	33.3	7.9	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Wr	2.60									18	8.0	135.0	0	13,500
		23	3	0+480 to 0+780	300	A1-BH-06	5.0	2.4	Acs	0.80			0.30	33.3	2.8	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Ac	1.60	4, 5	4.5	0.30	100.0	10.4	0.30	0.030	0.0300	16	6.0	30.0	0	3,000
									Wr	8.35	31, 50<	31.0							18	8.0	200.0	0	20,000
		25	4	0+880 to 0+980	100	A1-BH-03	4.0	3.5	Ac	2.00			0.30	33.3	6.0	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Acs	1.50			0.30	33.3	17.3	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Wr	5.00									18	8.0	135.0	0	13,500
			5	0+980 to 1+080	100	A1-AB-03	4.0	4.0	Ac	2.60			0.30	33.3	7.8	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Oc	0.80			0.30	16.7	17.2	0.35	0.035	0.0350	14	4.0	5.0	0	500
									Acs	0.60			0.30	33.3	20.9	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Wr	5.00									18	8.0	135.0	0	13,500
			6	1+080 to 1+160	80	A1-BH-04	4.5	4.0	Ac	2.50			0.30	33.3	7.5	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Acs	1.50	7	7.0	0.30	153.3	20.3	0.23	0.023	0.0075	17	7.0	46.0	0	4,600
									Wr	5.00	10, 14, 23	15.7							18	8.0	104.0	0	10,400
			7	1+160 to 1+300	140	A1-BH-04,7A,7B	8.0	2.0	Ac	0.90			0.30	33.3	2.7	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Acs	1.10	7	7.0	0.30	153.3	9.3	0.23	0.023	0.0075	17	7.0	46.0	0	4,600
									Wr	6.65	10, 14, 23	15.7							18	8.0	104.0	0	10,400
		26	8	1+300 to 1+680	380	A1-BH-04,7A,7B	8.0	1.5	Ac	1.50	3	3.0	0.30	66.7	4.5	0.30	0.030	0.0300	16	6.0	20.0	0	2,000
									As	1.50									18	8.0	0.0	30	3,900
									Wr		50<	50.0							18	8.0	200.0	0	20,000
	Ramp-1	27	9	0+025 to 0+260	235	A1-BH-06	9.0	2.4	Ac	2.40	4, 5	4.5	0.30	100.0	7.2	0.30	0.030	0.0300	16	6.0	30.0	0	3,000
									Wr	8.35	31, 50<	31.0							18	8.0	200.0	0	17,850
	Ramp-4	28	10	0+020 to 0+280	260	A1-AB-04	3.0	4.0	Acs	0.70			0.30	33.3	2.5	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Oc	1.20			0.30	16.7	7.3	0.35	0.035	0.0350	14	4.0	5.0	0	500
									Acs	2.10			0.30	33.3	17.1	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Wr	5.00									18	8.0	135.0	0	13,500
		29	11	0+280 to 0+365	85	A1-AB-04 AD-BH-	6.5	3.5	Ac	2.00			0.30	33.3	6.0	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Acs	1.50			0.30	33.3	17.3	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Wr	5.00									18	8.0	135.0	0	13,500

Table 5.5.1-2 Sections and Model for Analysis

Sectuion No.	Road	Section and Embankment Height						Total T.of Soft to Medium Soil	Ground Conditions				Soil Parameter										
		Sec.	Analysis Sec.	Analysis	Zone	Bor.	Emb. H		Layer	Thick.	SPT N	Ave. N	Cu/Pc	Pc	P0	CR	RR	Cce	γ	γ'	Cu	φ	Young's E
		No.	No.	Sta. No.	Length	No.	(m)	(m)	Name	(m)	(Blows)	(Blows)		(KN/m2)	(KN/m2)				(KN/m3)	(KN/m3)	(KN/m2)	(degree)	(KN/m2)
1	OCH	1	12	8+200 to 8+270	70	NS1-BH-	12.5	2.4	Ac	2.40	1, 2	1.5	0.30	33.3	7.2	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Wr	5.40	13, 14, 49	13.5							18	8.0	90.0	0	9,000
			13	8+270 to 8+330	60	NS1-AB-	12.0	4.0	Acs	4.00			0.30	33.3	14.0	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Wr	5.00									18	8.0	135.0	0	13,500
		2	14	8+330 to 8+460	130	NS1-BH-	12.0	1.7	Acs	1.70	1	1.0	0.30	33.3	6.0	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									As	1.00	7	7.0							18	8.0	0.0	29	7,050
									Wr	7.00	12, 13, 19, 23	16.8							18	8.0	111.0	0	11,100
		3	15	8+460 to 8+668	208	NS1-BH-	12.0	2.8	Oc	1.25			0.30	16.7	2.3	0.35	0.035	0.0350	14	4.0	5.0	0	500
									Acs	0.55	4	4.0	0.30	86.7	6.9	0.23	0.023	0.0075	17	7.0	26.0	0	2,600
									Ac	1.00	1	1.0	0.30	33.3	11.9	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Wr	6.80	11, 22, 23	18.7							18	8.0	124.0	0	12,400
2	OCH	4	16	9+330 to 9+365	35	NS1-AB-	6.0	5.0	Acs	2.40			0.30	33.3	8.4	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Ac	2.60			0.30	33.3	24.6	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Rs	1.00									18	8.0	125.0	0	3,900
									Wr	5.00									18	8.0	135.0	0	13,500
		5	17	9+700 to 9+900	200	NS1-BH-	5.5	2.2	Oc	2.20	0	0.0	0.30	16.7	4.4	0.35	0.035	0.0350	14	4.0	5.0	0	500
									Wr	6.40	4, 4, 5, 7,	5.0							18	8.0	33.0	0	6,150
		6	18	10+460 to 10+800	340	NS1-BH- NS1-BH-25	10.0	2.0	Acs	2.00	0	0.0	0.30	33.3	7.0	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									As	4.00	14, 12, 19, 25	17.5							18	8.0	0.0	32	11,775
									Wr		50<	50.0							18	8.0	200.0	0	20,000
		8	19	11+660 to 11+800	140	NS1-BH-	6.5	1.2	Oc	1.20			0.30	16.7	2.4	0.35	0.035	0.0350	14	4.0	5.0	0	500
									As	1.60	3, 28	3.0							18	8.0	0.0	28	5,250
									Ac	4.20	19, 18, 24, 28	22.3	0.30	493.3	30.2	0.23	0.023	0.0150	18	8.0	148.0	0	14,800
									Wr	5.00	41, 22, 27	24.5							18	8.0	163.0	0	14,925
		9	20	11+800 to 11+930	130	NS1-BH- NS1-BH-	7.0	2.0	Oc	1.20			0.30	16.7	2.4	0.35	0.035	0.0350	14	4.0	5.0	0	500
									Acs	0.80	7	7.0	0.30	153.3	7.6	0.23	0.023	0.0075	17	7.0	46.0	0	4,600
									As	5.00	4, 2, 5, 8	4.8							18	8.0	0.0	28	6,038
									Rs	5.00	10, 6, 5,	10.4							18	8.0	69.0	0	6,900
									Wr	3.20	23, 50<	23.0							18	8.0	153.0	0	15,300
		10	21	12+000 to 12+080	80	NS1-BH-40.41	5.0	1.8	Pt	0.70			0.40	12.5	1.1	0.45	0.045	0.0550	13	3.0	5.0	0	500
									Ac	1.10	6	6.0	0.30	133.3	5.4	0.30	0.030	0.0300	16	6.0	40.0	0	4,000
									Acs	3.10	13, 17, 23	17.7	0.30	390.0	19.6	0.23	0.023	0.0075	17	7.0	117.0	0	11,700
									Wr	4.10	30, 41,	30.0							18	8.0	200.0	0	20,000
		11	22	12+235 to 12+395	160	AD-BH-	4.0	4.0	Ac	3.00	1, 4, 6	3.0	0.30	66.7	8.0	0.30	0.030	0.0300	16	6.0	10.0	0	2,000
									As	2.00	2, 1	1.0							18	8.0	0.0	27	4,350
									Pt	1.00	1	1.0	0.40	12.5	33.3	0.45	0.045	0.0550	13	3.0	5.0	0	500
									As	1.00	13	13.0							18	8.0	0.0	31	9,750
									Wr	2.00	33, 50	40.0							18	8.0	200.0	0	20,000

Table 5.5.1-3 Sections and Model for Analysis

Sectuion No.	Road	Section and Embankment Height: Sec. No.	Analysis Sec No.	Analysis Sta. No	Zone Length	Bor. No.	Emb. H (m)	Total T. of Soft to Medium Soil (m)	Ground Conditions: Layer Name	Thick (m)	SPT N (Blows)	Ave. N (Blows)	Soil Parameter: Cu/Pc	Pc (KN/m2)	P0 (KN/m2)	CR	RR	Cαe	γ (KN/m3)	γ' (KN/m3)	Cu (KN/m2)	φ (degree)	Young's E (KN/m2)
3	OCH	12	23	12+700 to 12+825	125	NS1-AB-	6.0	4.0	Oc	0.90			0.30	16.7	1.8	0.35	0.035	0.0350	14	4.0	5.0	0	500
									Acs	1.50			0.30	33.3	6.9	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Oc	1.60			0.30	16.7	17.3	0.35	0.035	0.0350	14	4.0	5.0	0	500
									Rs	2.00									18	8.0	125.0	0	12,500
									Wr	3.00									18	8.0	135.0	0	13,500
		13	24	12+890 to 13+010	120	NS1-BH-	8.5	2.0	Oc	1.00			0.30	16.7	2.0	0.35	0.035	0.0350	14	4.0	5.0	0	500
									Acs	1.00			0.30	33.3	7.5	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Rs	5.00	4,	10.5							18	8.0	70.0	0	7,000
									Wr	10.00	21,25,32,1 9,15, 23,28,44,5	23.3							18	8.0	155.0	0	15,500
			25	13+010 to 13+150	140	NS1-BH-	8.5	3.9	Oc	1.90			0.30	16.7	3.8	0.35	0.035	0.0350	14	4.0	5.0	0	500
									Acs	2.00	5, 6	5.5	0.30	130.0	14.6	0.23	0.023	0.0075	17	7.0	36.0	0	3,600
									Wr	2.55	17, 50<	17.0							18	8.0	113.0	0	11,300
			26	13+150 to 13+250	100	Ns1-BH-	9.5	4.7	Pt	2.10	0	0.0	0.40	12.5	1.3	0.45	0.045	0.0300	13	3.0	5.0	0	500
									Ac	2.60	6, 5, 10	7.0	0.30	133.3	14.1	0.30	0.030	0.0300	16	6.0	46.0	0	4,600
									As	3.30	3, 7, 30	5.0							18	8.0	0.0	29	6,150
									Wr	2.90	39, 19	19.0							18	8.0	126.0	0	12,600
		14	27	13+250 to 13+350	100	NS1-BH-	10.0	6.3	Pt	2.00	0	0.0	0.40	12.5	3.0	0.45	0.045	0.0550	13	3.0	5.0	0	500
									Acs	0.90	0	0.0	0.30	33.3	9.2	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									Ac	2.10	2, 7	3.0	0.30	66.7	18.6	0.30	0.030	0.0300	16	6.0	20.0	0	2,000
									Oc	1.30	4	4.0	0.30	86.7	27.5	0.35	0.035	0.0350	14	4.0	26.0	0	2,600
									As	3.70	12, 46, 50, 25	18.5							18	8.0	0.0	33	13,225
									Wr	1.20	50<	50.0							18	8.0	200.0	0	20,000
		15	28	13+350 to 13+450	100	NS1-BH-	10.0	4.3	Pt	2.70	0, 0	0.0	0.40	12.5	4.1	0.45	0.045	0.0550	13	3.0	5.0	0	500
									Ac	1.60	6, 8	7.0	0.30	133.3	12.9	0.30	0.030	0.0300	16	6.0	46.0	0	4,600
									As	1.70	9	9.0							18	8.0	0.0	30	7,950
									Ac	0.90	12	12.0	0.30	266.7	34.0	0.23	0.023	0.0150	16	6.0	80.0	0	8,000
									As	4.10	19, 29,	24.0							18	8.0	0.0	34	14,700
									Wr	1.70	50<	50.0							18	8.0	200.0	0	20,000
			29	13+450 to 13+690	240	NS1-BH-	10.0	5.0	Pt	1.30	0	0.0	0.40	12.5	2.0	0.45	0.045	0.0550	13	3.0	5.0	0	500
									Ac	3.70	0, 3, 6	3.0	0.30	46.7	13.8	0.30	0.030	0.0300	16	6.0	20.0	0	2,000
									Ac	0.90	19	19.0	0.30	420.0	28.8	0.23	0.023	0.0150	16	6.0	126.0	0	12,600
									As	1.80	16, 47	16.0							18	8.0	0.0	32	11,100
									Wr	3.00	47, 50<	47.0							18	8.0	200.0	0	20,000
			30	13+690 to 13+775	85	NS1-BH-	10.0	2.5	Pt	2.50	0, 0	0.0	0.40	12.5	1.8	0.45	0.045	0.0550	13	3.0	5.0	0	500
									Rs	6.40	20, 38, 16, 28	18.0							18	8.0	120.0	0	12,000
									Wr	2.20	29, 50<	29.0							18	8.0	193.0	0	19,300

Table 5.5.1-4 Sections and Model for Analysis

Sectuion No.	Road	Section and Embankment Height						Total T.of Soft to Medium Soil	Ground Conditions				Soil Parameter										
		Sec.	Analysis Sec.	Analysis	Zone	Bor.	Emb. H		Layer	Thick	SPT N	Ave. N	Cu/Pc	Pc	P0	CR	RR	Cαe	γ	γ'	Cu	φ	Young's E
		No.	No.	Sta. No	Length	No.	(m)	(m)	Name	(m)	(Blows)	(Blows)		(KN/m2)	(KN/m2)				(KN/m3)	(KN/m3)	(KN/m2)	(degree)	(KN/m2)
3	OCH	16	31	14+050 to 14+250	200	NS1-BH-	9.0	2.9	Ac	1.00			0.30	33.3	3.0	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Pt	1.90	0, 0	0.0	0.40	12.5	8.9	0.45	0.045	0.0550	13	3.0	5.0	0	500
									As	1.25	7	7.0							18	8.0	0.0	29	7,050
									Acs	0.85	13	13.0	0.30	286.7	24.7	0.23	0.023	0.0075	17	7.0	86.0	0	8,600
									Wr	5.00	10, 50<	10.0							18	8.0	66.0	0	6,600
		17	32	14+280 to 14+410	130	NS1-BH-	8.5	2.5	Oc	2.45	0, 0	0.0	0.30	16.7	4.9	0.35	0.035	0.0350	14	4.0	5.0	0	500
									As	1.45	6	6.0							18	8.0	0.0	29	6,600
									Wr	6.10	21, 34, 29, 50<	28.0							18	8.0	186.0	0	18,600
		18	33	14+600 to 14+650	50	NS1-BH-	8.0	2.6	Oc	2.60	0, 0	0.0	0.30	16.7	5.2	0.35	0.035	0.0350	14	4.0	5.0	0	500
									As	1.60	22	22.0							18	8.0	0.0	34	13,800
									Wr	6.30	29, 50<	29.0							18	8.0	193.0	0	19,300
		19	34	15+060 to 15+115	55	NS1-BH-	8.0	4.0	Oc	4.00	0, 0, 9	0.0	0.30	16.7	8.0	0.35	0.033	0.0350	14	4.0	5.0	0	500
									As	2.20	20, 7	7.0							18	8.0	0.0	29	7,050
									Wr	10.50	11, 32, 46	11.0							18	8.0	73.0	0	7,300
		20	35	15+495 to 15+610 16+520 to 16+560	155	NS1-BH- 72, 73	9.0	4.9	Ac	2.50	1, 1	1.0	0.30	33.3	7.5	0.30	0.030	0.0300	16	6.0	10.0	0	1,000
									Pt	2.40	2, 14	2.0	0.40	32.5	18.6	0.45	0.045	0.0550	13	3.0	13.0	0	1,300
									As	1.50	12	12.0							18	8.0	0.0	31	9,300
									Wr		50<	50.0							18	8.0	200.0	0	20,000
4	Approach	30	36	0+50 to 0+130	80	NS1-BH-	7.0	4.0	Acs	1.90	1	1.0	0.30	33.3	6.7	0.23	0.023	0.0075	17	7.0	10.0	0	1,000
									As	2.10	5, 3	3.0							18	8.0	0.0	28	5,250
									Wr	3.00	5, 8, 10	7.6							18	8.0	0.0	29	7,320

5.5.3. Minimum Safety Factor for Circular Slip Failure of Embankment

- During and after construction (Short Term) : Fs = 1.20
- At the service stage (Long Term) : Fs = 1.25

5.5.4. Load for Embankment Stability Analysis

Loads for embankment stability analysis are as follows;

Table 5.5.2 Load for Embank Stability Analysis

Time	Dead Load	Live Load
During and after construction (Short Term)	Embankment Load	Traffic Load (P=5KN/m2)
At the service stage (Long Term)	Embankment Load	Traffic Load (P=10KN/m2)

*Embankment Load = γE x HE = 20H (KN/m2)

5.5.5. Water Table

Water table is assumed to be at the ground surface level (embankment toe level).

5.5.6. Installation Spacing of Band Drains

In case band drains is installed, they are designed to perform 90% of consolidation degree in six months after loading. As shown in **Fig. 5.5.2**, spacing of 1.0 m in square arrangement is necessary to obtain required consolidation degree (90 %) in 6 months.

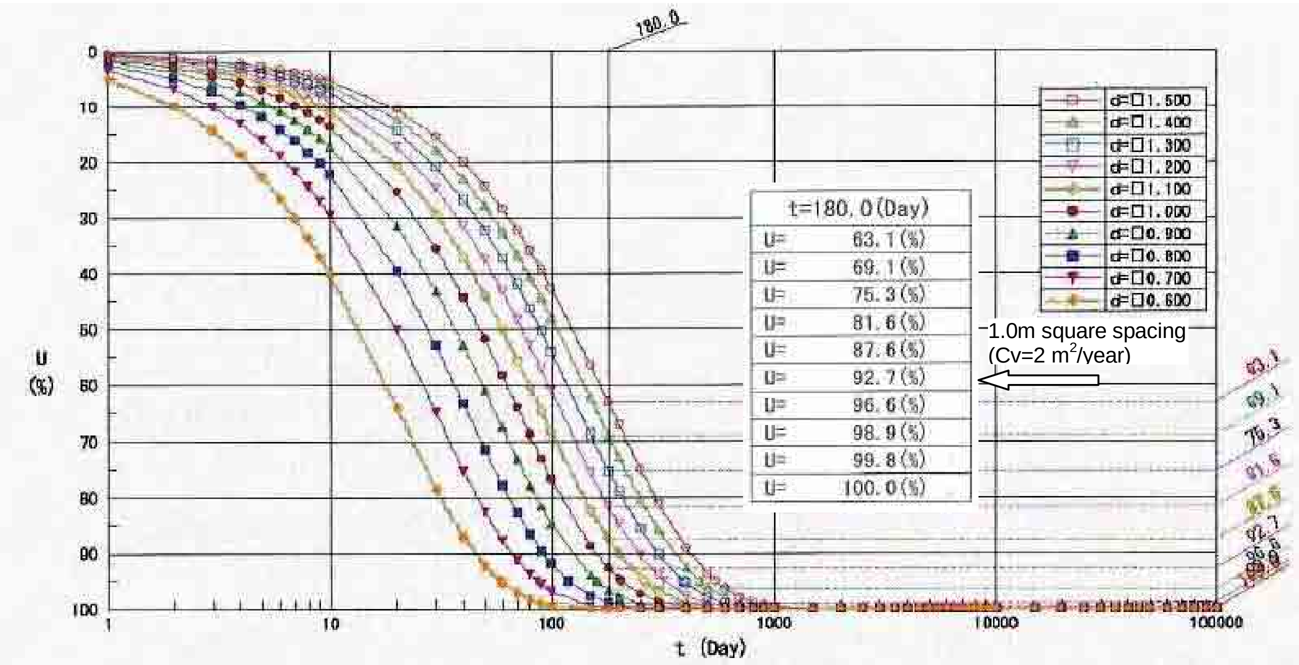


Fig. 5.5.2 Band drains spacing selection

5.5.7. U – T Relation of Gravel Compaction Pile

Based on Barron's theory, U (consolidation degree) and t (time) relations for gravel compaction piles in 1.2 meter and 2.0 meter square spacing are calculated as shown in **Fig. 5.5.3**.

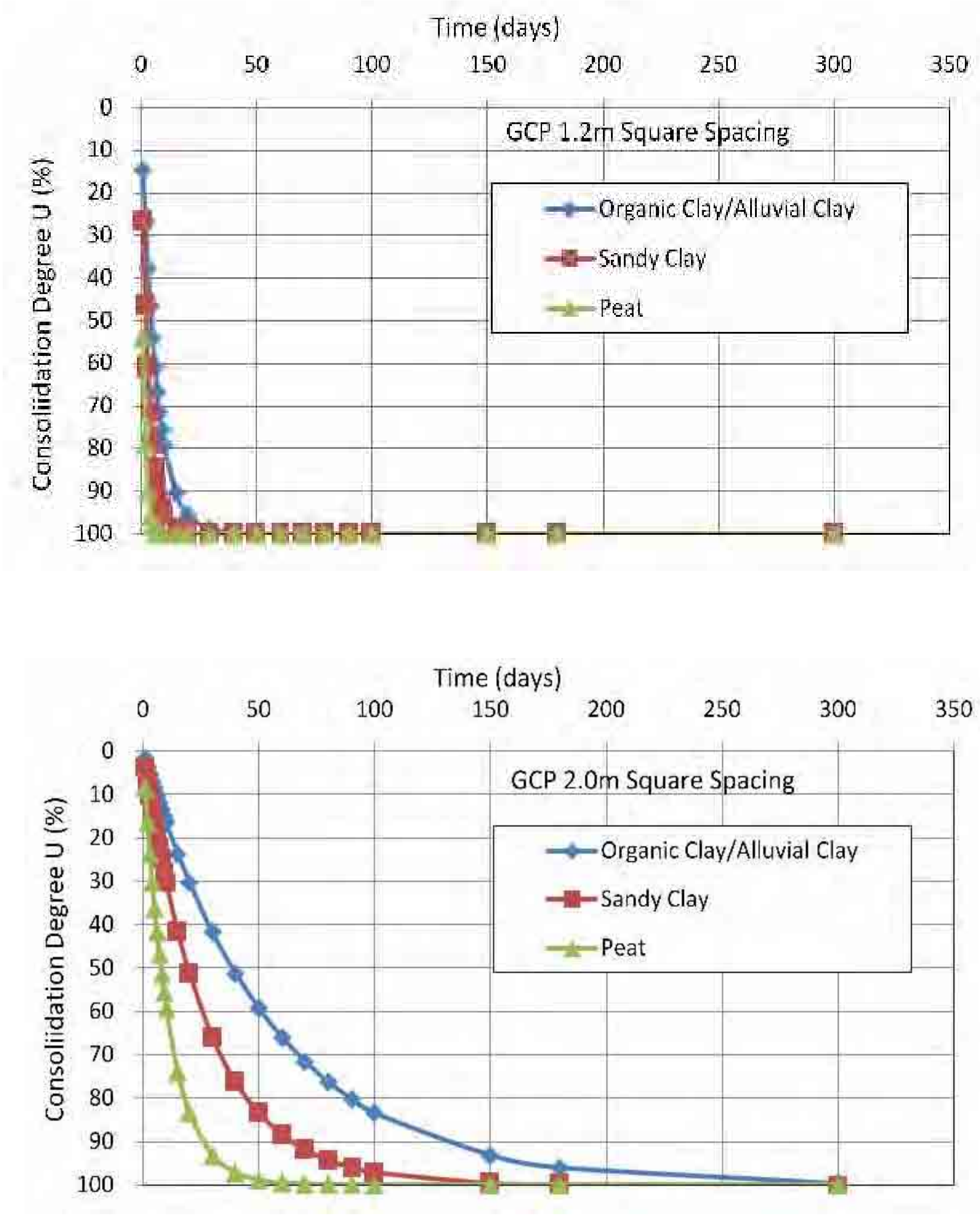


Fig. 5.5.3 U - Time Relation of Gravel Compaction Piles

5.5.8. Staged Construction

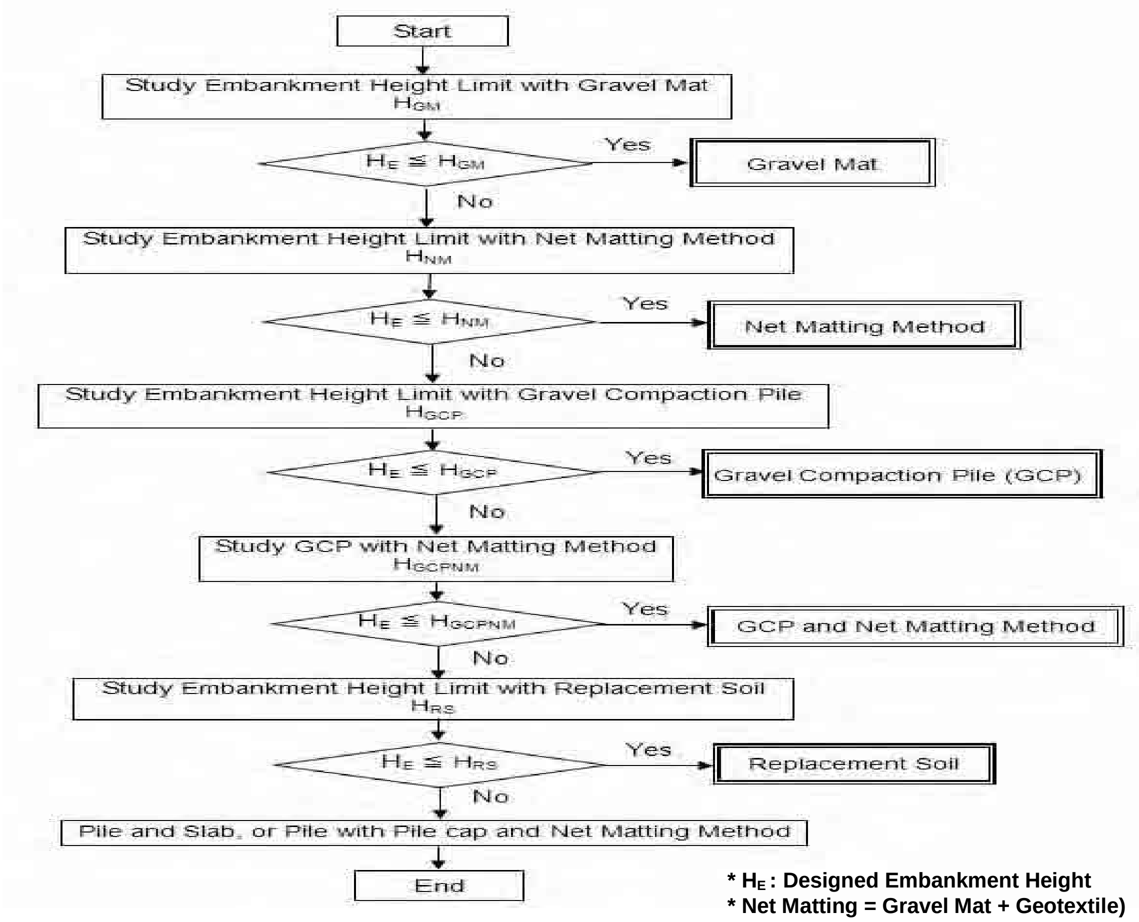
If embankment can be maintain the stability considering increase of shear strength, staged construction method is applied. In this case, the maximum numbers of stage is limited up to two stages considering the construction period. (refer to **Fig. 5.2.1**)

5.6. Selection Procedure for Soft Soil Treatment Method

Soft soil treatment methods are studied among the several possible treatment methods shown in **Table 5.6.1** to meet the technical requirements and the clarifications based on discussion result between RDA and JICA Study Team and also the practical experiences of on-going highway projects in Sri Lanka considering to reduce the volume of excavated soil for replacement to be dumped as much as possible.

5.6.1. Stability Analysis

During construction and after completion of the embankment the FS is greater than 1.20. When the service stage of OCH the FS should secure more than 1.25. The analysis steps have been conducted according to the flow chart below.



*** H_E : Designed Embankment Height**
*** Net Matting = Gravel Mat + Geotextile)**

Fig. 5.6.1Flow Chart of Embankment Stability Analysis

From Table 5.6.1, Gravel Compaction Pile Method (GCP) has been finally selected as a main countermeasure for the stability of embankment due to the following reasons;

1) To stabilize the high embankment on the soft ground, mainly 3 countermeasures can be considered, Replacement Soil, Sand Compaction Pile (SCP) and Cement Mixing (CM) from technical and economical point of views. And Dynamic Compaction (DC) and Vacuum Consolidation (VC) Method also can be considered.

2) However CM Method is not accepted by RDA due to the reason of no experience in Sri Lanka. And with consideration to reduce, as much as possible, the volume of excavated unsuitable soil for replacement to be dumped, the priority of Replacement Soil Method becomes low. On the other hand, SCP method has been used in many projects for long years and also can be

applied for not only soft clay but also very weak peat ground. As a result of those reasons, SCP can be selected as a main countermeasure to maintain the stability of embankment. Furthermore DC takes a lot of time to complete the required treatment area according to the actual performance in STDP.

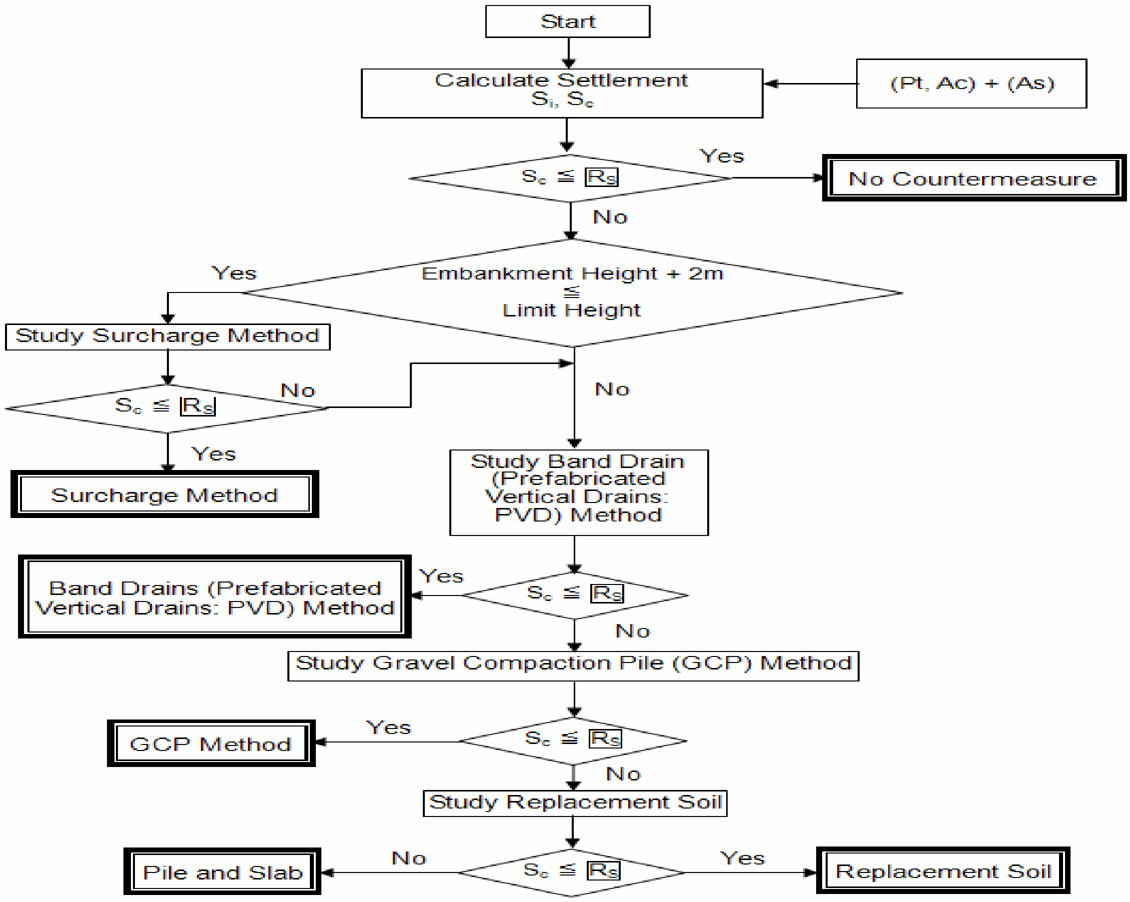
3) VC is very effective only for peat and organic clay with high water content. However, in OCH NS1, most of very weak peat distributed area is planned to construct viaducts so that only small peat distributed area remains for embankment construction, which is about 300 m in longitudinal distance. Therefore VC is not so much suitable in this case from geotechnical and economical point of view.

4) Finally, due to difficulties of procurement of sand material in the region, GCP has been selected as a main countermeasure to maintain the stability of embankment and facilitate the consolidation settlement instead of SCP.

5) Actually GCP is one of SCP methods and only the point that gravel is used as pile material is different form SCP, However the design concept and procedure is quite same. And also GCP has higher pile strength than SCP.

5.6.2. Settlement Analysis

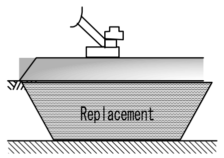
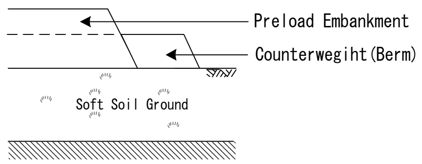
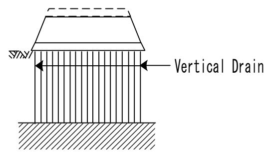
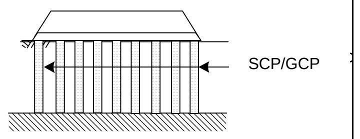
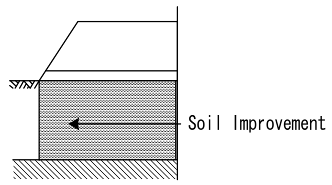
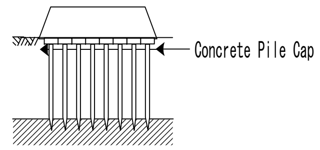
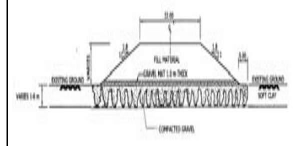
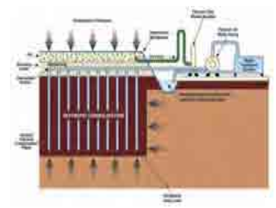
After a slope stability analysis of all analysis sections, consolidation settlement is calculated and compared with allowable residual settlement using the flow chart below as a guideline.



Rs: Allowable residual settlement (=15cm)
Sc: Residual settlement calculated under each countermeasure

Fig. 5.6.2 Flow Chart of Settlement Analysis with Countermeasure

Table 5.6.1 Soft Ground Treatment Methods

Improvement Method	Excavation Replacement with Sand Mat	Pre-loading with Counterweight	Vertical Drain with Pre-loading	Sand/ Gravel Compaction Pile	Deep Mixing Method (Cement, Lime Improvement)	Concrete Pile	Dynamic Compaction	Vacuum Consolidation
Sketch								
Introduction	This method involves the removal of all or part of the soft deposits and replaces it with material, which increases the strength and decreases the settlement.	This method, which places counterweight fill to prevent sliding accelerate the settlement by pre-loading.	This method places material like sand and cardboard vertically to drain the soft ground and to accelerate the consolidation by shortening the drainage path.	With this method, sand/ gravel columns of large diameter are placed in the ground and compacted by vibration or impact to increase the strength of ground.	This method, use of chemical improvement is more attractive than conventional methods such as vertical drains, sand compaction pile and counterweight fill methods to prevent affecting neighboring area.	This method, use of concrete piles conveys the fill loads to the foundation via piles to increase bearing capacity and prevent settlement	With this method, sand or gravel column are made by free fall of heavy weight using special improved crane	With this method, water in soft soil is pumped up by vacuum pressure through PVD.
Effectiveness	• To increase the bearing capacity by replacement • Necessary sand mat to ensure the traffic –ability prior to excavation	• Pre-loading to accelerate the consolidation settlement and increase the shear strength • To ensure the stability of embankment by counterweight	• Pre-loading with dewatering through vertical drains to accelerate the consolidation settlement and increase the shear strength	• To increase the shear strength by the sand columns and ensure the stability of embankment • To reduce the consolidation settlement • To accelerate the settlement	• To improve the soil properties by the chemical improvement (Cement, Lime etc.) and ensure the stability of embankment	• To ensure the bearing capacity by the rigid piles	• To make the sand or gravel column by forced replacement of soft soil. • To increase the shear strength by sand or gravel column. • To reduce the consolidation settlement	• Consolidation settlement will occur by this suction without any embankment. • To accelerate consolidation settlement by vacuum load as surcharge.
Criteria for Application	Shallow deposit less than 6m	Shallow deposit is effective	More advantageous for deep deposit	More advantageous for deep deposit	Deep deposit is preferable	More advantageous for deep deposit	Shallow Deposit less than 6m	expected maximum load by suction is about 65/m^2
	Clay, Peat or organic soil	Clay, Peat or Organic Soil	Clay, Peat or Organic Clay	Clay, Peat or Organic Soil	Peat or Organic clay, loose cohesionless soil	Peat or Organic clay, loose cohesionless soil	Clay Peat and Organic soil, loose cohesionless soil	• Clay, Peat, Organic Soil
	Widely used in Sri Lanka	Widely used in Sri Lanka	Necessary technique and equipment from abroad	Necessary technique and equipment from abroad	Necessary technique and equipment from abroad	Locally possible construction method in Sri Lanka	Used in Southern High Way only	Necessary technique and equipment from abroad
Clarifications or other attentions to pay for at construction or design stage	• Difficult complete excavation may leave the small pockets of peat that may cause severe settlement after the completion. • Very low shear strength peat may cause failure of the side slope excavation that results in increasing the quantities • Rarely used in recent time because difficult estimating of material volume • The settlement often continuously occurred after completion	• This may result in very long construction times depend on the depth of peat. • The height of preloading is generally 2m or about 1m, where residual settlement is low due to shallow peat deposit or where differential residual settlement may be difficult to achieve on flat land.	• Generally, very thick sand mats are necessary prior to the placement of the sand columns because the equipment used is very heavily. • The sand drain method does not effect appreciable acceleration of settlement, but it is incorporated into design as a composite foundation with the strength of the sand columns.	• Necessary monitoring must be carried out to pay attention of uplift or deformation of surroundings and adjacent structure during placing of sand columns. • In small and medium rivers with narrow beds there may be blockage of low-lying waterways.	• This method is not necessary to consider greatly influence adjacent ground during construction however, depending on ground condition and work condition, uplift or lateral deformation of the adjacent ground may occur so that necessary treatment shall be considered. • Consideration shall be given to great variation in the strength of improved columns after mixing and stirring.	• This method has abundant experiences in Japan to prevent excessive settlement approaching to the abutment for bridge and to control the longitudinal settlement of the road surface. • The sand mat or other surface treatment to ensure the traffic ability shall be required before the pile driving. • By use of rigid concrete piles, it can be easily to ensure the bearing capacity but very costly method.	• As number of tamping is very much for one location, it takes time very much. • Confirmation of improvement effectiveness to required depth shall be checked area by area. • Material for sand column shall be sand or gravel. • Trial execution is necessary to confirm the design criteria • No excavation and No disposal site for removed soil are necessary. • Uncommon	• Sand ream in the soft soil layer reduce the effectiveness of vacuum because of leakage of air and water through it. • Monitoring of vacuum pressure and water volume pumped up must be carried out to confirm the effectiveness of vacuum pressure. • Effectiveness of soil improvement by this method shall be checked by monitoring the settlement and strength of soil. • Air tightness is essential for this method. Holes or tear of sheet and Other cause of leakage shall be checked strictly • Uncommon
Cost Effectiveness	Cost effective but applicable depth is less than estimate 6m	Cost effective but takes time	Cost effective	Cost effective	Costly	Very costly	Cost effective but takes time	Costly

note : ( ) is minimum

5.7. Analysis Results and Possible Countermeasures

5.7.1. Analysis Result

After the series of discussions between the RDA and the JICA DD Study Team for OCH, the criteria below for the selection of countermeasures were agreed.

a) In case soft soil layer are more than 2 m in thick, band drains are basically installed in a spacing of 1.0 m in square arrangement to accelerate the consolidation settlement.

b) Soil properties should be clarified if it is required to replace with by the geotechnical investigation including trial excavation and embankment prior to the construction.

In this study, analysis for the slope stability and settlement for the embankment was carried out taking into account the above agreement and the combinations with the following eight soft soil treatments (refer to **Table 5.7.3**). These analysis results are summarized in **Table 5.7.1**, **Table 5.7.2** and **Fig. 5.7.1**.

Type A: Gravel mat
Type B: Gravel mat and 1 layer geotextile
Type C: Gravel mat, 1 layer geotextile and band drains
Type D: Gravel mat, 1 layer geotextile and Gravel Compaction Pile
Type E: Gravel mat, Band Drains and Gravel Compaction Pile
Type F: Gravel mat, 1 layer geotextile, Band Drains and Gravel Compaction Pile
Type G: Gravel mat, 1 layer geotextile and Gravel Compaction Pile (Whole Area)
Type H: Replacement by Excavation

For the locations where box culverts and abutments of viaducts are constructed on the soft ground, gravel compaction pile (GCP; ϕ=0.7m, □1.2m) and preload (up to designed road level) is applied in order to remove the negative influence to the structures due to consolidation settlement by embankment.
All locations of box culverts and abutments of viaducts on the soft ground treated by “GCP and Preload” method are tabulated in **Table 5.7.5**.

Table 5.7.1-1 Summary of Settlement Calculation Result

Sectuion No.	Road	Section and Embankm,ent Height						Ground Conditions					Soil Parameter										Load on ground	Immediate Settlement									Actual Embank. Load	Primary Consolidation		Secondary Consolidation				Sub Total	Grand Total
		Sec. No.	Analysis Sec. No.	Analysis Sta. No.	Zone Length	Bor. No.	Emb. H (m)	Total T.of Soft to Medium Soil (m)	Layer Name	Thick. (m)	SPT N (Blows)	Ave. N (Blows)	Cu/Pc	Pc (KN/m2)	P0 (KN/m2)	CR	RR	Cαε	γ (KN/m3)	γ' (KN/m3)	Cu (KN/m2)	Young's E (KN/m2)	Dp (KN/m2)	B (m)	Z (m)	Z/B	Iz	C1	C2	Siz (m)	Si (m)	Dp' (KN/m2)	Spz (m)	S_p (m)	t1 (days)	t2 (days)	Ssz (m)	Ss (m)	Sp+Ss (m)	Si+Sp+Ss (m)
1	A1-bypass	21	1	0+225 to 0+370	145	A1-BH-	8.0	3.7	Ac	1.00			0.30	33.3	3.0	0.30	0.030	0.0300	16	6.0	10.0	1,000	160	61.8	0.50	0.008	0.202	1.0	1.1	0.036	0.145	160	0.238	0.917	365	1460	0.018	0.072	0.989	1.134
									Oc	2.00	1, 2	1.5	0.30	33.3	10.0	0.35	0.035	0.0350	14	4.0	10.0	1,000	160	61.8	2.00	0.032	0.210	1.0	1.1	0.074		160	0.532		365	1460	0.042			
									Ac	0.65	1	1.0	0.30	33.3	16.0	0.30	0.030	0.0300	16	6.0	10.0	1,000	160	61.8	3.33	0.054	0.216	1.0	1.1	0.025		160	0.147		365	1460	0.012			
									Br	1.00	50<	50.0							18	8.0	200.0	20,000																		
		22	2	0+370 to 0+480	110	A1-AB-	5.0	1.9	Ac	0.50			0.30	33.3	1.5	0.30	0.030	0.0300	16	6.0	10.0	1,000	100	51.0	0.25	0.005	0.201	1.0	1.1	0.011	0.048	103	0.095	0.283	365	1460	0.009	0.015	0.298	0.346
									Acs	1.40			0.30	33.3	7.9	0.23	0.023	0.0075	17	7.0	10.0	1,000	100	51.0	1.20	0.024	0.207	1.0	1.1	0.032		103	0.188		365	1460	0.006			
									Wr	2.60									18	8.0	135.0	13,500	100	51.0	3.20	0.063	0.219	1.0	1.1	0.005										
		23	3	0+480 to 0+780	300	A1-BH-	5.0	2.4	Acs	0.80			0.30	33.3	2.8	0.23	0.023	0.0075	17	7.0	10.0	1,000	100	51.0	0.40	0.008	0.202	1.0	1.1	0.018	0.041	100	0.110	0.178	365	1460	0.004	0.033	0.210	0.251
									Ac	1.60	4, 5	4.5	0.30	100.0	10.4	0.30	0.030	0.0300	16	6.0	30.0	3,000	100	51.0	1.60	0.031	0.209	1.0	1.1	0.012		100	0.068		365	1460	0.029			
									Wr	8.35	31, 50<	31.0							18	8.0	200.0	20,000	100	51.0	6.58	0.129	0.239	1.0	1.1	0.011										
		25	4	0+880 to 0+980	100	A1-BH-	4.0	3.5	Ac	2.00			0.30	33.3	6.0	0.30	0.030	0.0300	16	6.0	10.0	1,000	80	47.4	1.00	0.021	0.206	1.0	1.1	0.036	0.073	84	0.302	0.478	365	1460	0.036	0.043	0.521	0.593
									Acs	1.50			0.30	33.3	17.3	0.23	0.023	0.0075	17	7.0	10.0	1,000	80	47.4	2.75	0.058	0.217	1.0	1.1	0.029		84	0.176		365	1460	0.007			
									Wr	5.00									18	8.0	135.0	13,500	80	47.4	6.00	0.127	0.238	1.0	1.1	0.008										
			5	0+980 to 1+080	100	A1-AB-	4.0	4.0	Ac	2.60			0.30	33.3	7.8	0.30	0.030	0.0300	16	6.0	10.0	1,000	80	47.4	1.30	0.027	0.208	1.0	1.1	0.048	0.098	85	0.398	0.690	365	1460	0.047	0.067	0.757	0.855
									Oc	0.80			0.30	16.7	17.2	0.35	0.035	0.0350	14	4.0	5.0	500	80	47.4	3.00	0.063	0.219	1.0	1.1	0.031		85	0.221		365	1460	0.017			
									Acs	0.60			0.30	33.3	20.9	0.23	0.023	0.0075	17	7.0	10.0	1,000	80	47.4	3.70	0.078	0.223	1.0	1.1	0.012		85	0.072		365	1460	0.003			
									Wr	5.00									18	8.0	135.0	13,500	80	47.4	6.50	0.137	0.241	1.0	1.1	0.008										
			6	1+080 to 1+160	80	A1-BH-	4.5	4.0	Ac	2.50			0.30	33.3	7.5	0.30	0.030	0.0300	16	6.0	10.0	1,000	90	49.2	1.25	0.025	0.208	1.0	1.1	0.051	0.070	94	0.411	0.437	365	1460	0.045	0.052	0.489	0.559
									Acs	1.50	7	7.0	0.30	153.3	20.3	0.23	0.023	0.0075	17	7.0	46.0	4,600	90	49.2	3.25	0.066	0.220	1.0	1.1	0.007		94	0.026		365	1460	0.007			
									Wr	5.00	10, 14, 23	15.7							18	8.0	104.0	10,400	90	49.2	6.50	0.132	0.240	1.0	1.1	0.011										
			7	1+160 to 1+300	140	A1-BH-	8.0	2.0	Ac	0.90			0.30	33.3	2.7	0.30	0.030	0.0300	16	6.0	10.0	1,000	160	61.8	0.45	0.007	0.202	1.0	1.1	0.032	0.066	164	0.218	0.262	365	1460	0.016	0.021	0.284	0.350
									Acs	1.10	7	7.0	0.30	153.3	9.3	0.23	0.023	0.0075	17	7.0	46.0	4,600	160	61.8	1.45	0.023	0.207	1.0	1.1	0.009		164	0.044		365	1460	0.005			
									Wr	6.65	10, 14, 23	15.7							18	8.0	104.0	10,400	160	61.8	5.33	0.086	0.226	1.0	1.1	0.025										
		26	8	1+300 to 1+680	380	A1-BH-	8.0	1.5	Ac	1.50	3	3.0	0.30	66.7	4.5	0.30	0.030	0.0300	16	6.0	20.0	2,000	160	61.8	0.75	0.012	0.204	1.0	1.1	0.027	0.041	160	0.229	0.229	365	1460	0.027	0.027	0.256	0.297
									As	1.50									18	8.0		3,900	160	61.8	2.25	0.036	0.211	1.0	1.1	0.014										
									Wr		50<	50.0							18	8.0	200.0	20,000																		
	Ramp-1	27	9	0+025 to 0+240	215	A1-BH-	9.0	2.4	Ac	2.40	4, 5	4.5	0.30	100.0	7.2	0.30	0.030	0.0300	16	6.0	30.0	3,000	180	65.4	1.20	0.018	0.206	1.0	1.1	0.033	0.054	180	0.278	0.278	365	1460	0.043	0.043	0.322	0.376
									Wr	8.35	31, 50<	31.0							18	8.0	200.0	17,850	180	65.4	6.58	0.101	0.230	1.0	1.1	0.021										
	Ramp-4	28	10	0+020 to 0+240	220	A1-AB-	3.0	4.0	Acs	0.70			0.30	33.3	2.5	0.23	0.023	0.0075	17	7.0	10.0	1,000	60	43.8	0.35	0.008	0.202	1.0	1.1	0.009	0.073	60	0.062	0.521	365	1460	0.003	0.038	0.559	0.632
									Oc	1.20			0.30	16.7	7.3	0.35	0.035	0.0350	14	4.0	5.0	500	60	43.8	1.30	0.030	0.209	1.0	1.1	0.033		60	0.269		365	1460	0.025			
									Acs	2.10			0.30	33.3	17.1	0.23	0.023	0.0075	17	7.0	10.0	1,000	60	43.8	2.95	0.067	0.220	1.0	1.1	0.031		60	0.190		365	1460	0.009			
									Wr	5.00									18	8.0	135.0	13,500	60	43.8	6.50	0.148	0.245	1.0	1.1	0.006										
		29	11	0+240 to 0+365	125	A1-BH-	6.5	3.5	Ac	2.00			0.30	33.3	6.0	0.30	0.030	0.0300	16	6.0	10.0	1,000	130	56.4	1.00	0.018	0.205	1.0	1.1	0.059	0.117	139	0.428	0.670	365	1460	0.036	0.043	0.713	0.830
									Acs	1.50			0.30	33.3	17.3	0.23	0.023	0.0075	17	7.0	10.0	1,000	130	56.4	2.75	0.049	0.215	1.0	1.1	0.046		139	0.242		365	1460	0.007			
									Wr	5.00									18	8.0	135.0	13,500	130	56.4	6.00	0.106	0.232	1.0	1.1	0.012										
	OCH	1	12	8+200 to 8+270	70	NS1-BH-	12.5	2.4	Ac	2.40	1, 2	1.5	0.30	33.3	7.2	0.30	0.030	0.0300	16	6.0	10.0	1,000	250	81.0	1.20	0.015	0.204	1.0	1.1	0.135	0.171	250	0.687	0.687	365	1460	0.043	0.043	0.730	0.901
									Wr	5.40	13, 14, 49	13.5							18	8.0	90.0	9,000	250	81.0	5.10	0.063	0.219	1.0	1.1	0.036										
			13	8+270 to 8+330	60	NS1-AB-	12.0	4.0	Acs	4.00			0.30	33.3	14.0	0.23	0.023	0.0075	17	7.0	10.0	1,000	240	79.2	2.00	0.025	0.208	1.0	1.1	0.219	0.241	255	0.870	0.870	365	1460	0.018	0.018	0.888	1.129
									Wr	5.00									18	8.0	135.0	13,500	240	79.2	6.50	0.082	0.225	1.0	1.1	0.022										
		2	14	8+330 to 8+460	130	NS1-BH-	12.0	1.7	Acs	1.70	1	1.0	0.30	33.3	6.0	0.23	0.023	0.0075	17	7.0	10.0	1,000	240	79.2	0.85	0.011	0.203	1.0	1.1	0.091	0.136	240	0.369	0.369	365	1460	0.008	0.008	0.376	0.513
									As	1.00	7	7.0							18	8.0		7,050	240	79.2	2.20	0.028	0.208	1.0	1.1	0.008										
									Wr	7.00	12, 13, 19,	16.8							18	8.0	111.0	11,100	240	79.2	6.20	0.078	0.223	1.0	1.1	0.037										
		3	15	8+460 to 8+650	190	NS1-BH-	12.0	2.8	Oc	1.25			0.30	16.7	2.5	0.35	0.035	0.0350	14	4.0	5.0	500	240	79.2	0.63	0.008	0.202	1.0	1.1	0.134	0.233	240	0.545	0.893	365	1460	0.026	0.047	0.940	1.173
									Acs	0.55	4	4.0	0.30	86.7	6.9	0.23	0.023	0.0075	17	7.0	26.0	2,600	240	79.2	1.53	0.019	0.206	1.0	1.1	0.011		240	0.071		365	1460	0.002			
									Ac	1.00	1	1.0	0.30	33.3	11.9	0.30	0.030	0.0300	16	6.0	10.0	1,000	240	79.2	2.30	0.029	0.209	1.0	1.1	0.055		240	0.277		365	1460	0.018			
									Wr	6.80	11, 22, 23	18.7							18	8.0	124.0	12,400	240	79.2	6.20	0.078	0.223	1.0	1.1	0.032										

Table 5.7.1-2 Summary of Settlement Calculation Result

Sectuion No.	Road	Section and Embankm,ent Height								Ground Conditions					Soil Parameter										Load on ground	Immediate Settlement								Actual Embank. Load	Primary Consolidation		Secondary Consolidation				Sub Total	Grand Total
		Sec.	Analysis Sec.	Analysis			Zone	Bor.	Emb. H	Total T.of Soft to Medium Soil	Layer	Thick.	SPT N	Ave. N	Cu/Pc	Pc	P0	CR	RR	Cαe	γ	γ'	Cu	Young's E	Dp	B	Z	Z/B	Iz	C1	C2	Siz	Si	Dp'	Spz	S_p	t1	t2	Ssz	Ss	Sp+Ss	Si+Sp+Ss
		No.	No.	Sta. No.			Length	No.	(m)	(m)	Name	(m)	(Blows)	(Blows)		(KN/m2)	(KN/m2)				(KN/m3)	(KN/m3)	(KN/m2)	(KN/m2)	(KN/m2)	(m)	(m)					(m)	(m)	(KN/m2)	(m)	(m)	(days)	(days)	(m)	(m)	(m)	(m)
2	OCH	4	16	9+330	to	9+365	35	NS1-AB-	6.0	5.0	Acs	2.40			0.30	33.3	8.4	0.23	0.023	0.0075	17	7.0	10.0	1,000	120	54.6	1.20	0.022	0.207	1.0	1.1	0.065	0.161	133	0.379	0.914	365	1460	0.011	0.058	0.972	1.133
											Ac	2.60			0.30	33.3	24.6	0.30	0.030	0.0300	16	6.0	10.0	1,000	120	54.6	3.70	0.068	0.220	1.0	1.1	0.076		133	0.535	(0.000)	365	1460	0.047	(0.000)		
											Rs	1.00									18	8.0	125.0	3,900	120	54.6	5.50	0.101	0.230	1.0	1.1	0.008			(By replacement)				(By replacement)			
											Wr	5.00									18	8.0	135.0	13,500	120	54.6	8.50	0.156	0.247	1.0	1.1	0.012										
		5	17	9+700	to	9+900	200	NS1-BH-	5.5	2.2	Oc	2.20	0	0.0	0.30	16.7	4.4	0.35	0.035	0.0350	14	4.0	5.0	500	110	52.8	1.10	0.021	0.206	1.0	1.1	0.110	0.139	119	0.715	0.715	365	1460	0.046	0.046	0.761	0.900
											Wr	6.40	4, 4, 5, 7,	5.0							18	8.0	33.0	6,150	110	52.8	5.40	0.102	0.231	1.0	1.1	0.029										
		6	18	10+460	to	10+800	340	NS1-BH- NS1-BH-	12.5	2.0	Acs	2.00	0	0.0	0.30	33.3	7.0	0.23	0.023	0.0075	17	7.0	10.0	1,000	250	81.0	1.00	0.012	0.204	1.0	1.1	0.112	0.132	250	0.439	0.439	365	1460	0.009	0.009	0.448	0.580
											As	4.00	14, 12, 19,	17.5							18	8.0		11,775	250	81.0	4.00	0.049	0.215	1.0	1.1	0.020										
											Wr		50<	50.0							18	8.0	200.0	20,000																		
		8	19	11+660	to	11+800	140	NS1-BH-	6.5	1.2	Oc	1.20			0.30	16.7	2.4	0.35	0.035	0.0350	14	4.0	5.0	500	130	56.4	0.60	0.011	0.203	1.0	1.1	0.070	0.100	137	0.422	0.494	365	1460	0.025	0.063	0.557	0.657
											As	1.60	3, 28	3.0							18	8.0		5,250	130	56.4	2.00	0.035	0.211	1.0	1.1	0.009										
											Ac	4.20	19, 18, 24,	22.3	0.30	493.3	30.2	0.23	0.023	0.0150	16	6.0	148.0	14,800	130	56.4	4.90	0.087	0.226	1.0	1.1	0.009		137	0.072		365	1460	0.038			
											Wr	5.00	41, 22, 27	24.5							18	8.0	163.0	14,925	130	56.4	9.50	0.168	0.251	1.0	1.1	0.012					365	1460				
		9	20	11+800	to	11+930	130	NS1-BH- NS1-BH-	7.0	2.0	Oc	1.20			0.30	16.7	2.4	0.35	0.035	0.0350	14	4.0	5.0	500	140	58.2	0.60	0.010	0.203	1.0	1.1	0.075	0.146	147	0.435	0.459	365	1460	0.025	0.029	0.488	0.634
											Acs	0.80	7	7.0	0.30	153.3	7.6	0.23	0.023	0.0075	17	7.0	46.0	4,600	140	58.2	1.60	0.027	0.208	1.0	1.1	0.006		147	0.024		365	1460	0.004			
											As	5.00	4, 2, 5, 8	4.8							18	8.0		6,038	140	58.2	4.50	0.077	0.223	1.0	1.1	0.028										
											Rs	5.00	10, 6, 5,	10.4							18	8.0	69.0	6,900	140	58.2	9.50	0.163	0.249	1.0	1.1	0.028										
											Wr	3.20	23, 50<	23.0							18	8.0	153.0	15,300	140	58.2	13.60	0.234	0.270	1.0	1.1	0.009										
		10	21	12+000	to	12+080	80	NS1-BH-	5.0	1.8	Pt	0.70			0.40	12.5	1.1	0.45	0.045	0.0550	13	3.0	5.0	500	100	51.0	0.35	0.007	0.202	1.0	1.1	0.031	0.098	107	0.329	0.430	365	1460	0.023	0.057	0.487	0.585
											Ac	1.10	6	6.0	0.30	133.3	5.4	0.30	0.030	0.0300	16	6.0	40.0	4,000	100	51.0	1.25	0.025	0.207	1.0	1.1	0.006		107	0.044		365	1460	0.020			
											Acs	3.10	13, 17, 23	17.7	0.30	390.0	19.6	0.23	0.023	0.0075	17	7.0	117.0	11,700	100	51.0	3.35	0.066	0.220	1.0	1.1	0.006		107	0.058		365	1460	0.014			
											Wr	4.10	30, 41,	30.0							18	8.0	200.0	20,000	100	51.0	6.95	0.136	0.241	1.0	1.1	0.005										
		11	22	12+235	to	12+395	160	NS1-BH-	4.0	3.5	Pt	1.35			0.40	12.5	2.0	0.45	0.045	0.0550	13	3.0	5.0	500	80	47.4	0.68	0.014	0.204	1.0	1.1	0.049	0.093	90	0.574	0.756	365	1460	0.045	0.054	0.811	0.903
											Acs	2.15	2, 2, 3	2.3	0.30	50.0	11.6	0.23	0.023	0.0075	17	7.0	15.0	1,500	80	47.4	2.43	0.051	0.215	1.0	1.1	0.027		90	0.183		365	1460	0.010			
											Wr	9.20	14, 22	18.0							18	8.0	120.0	12,000	80	47.4	8.10	0.171	0.251	1.0	1.1	0.017										
3	OCH	12	23	12+700	to	12+825	125	NS1-AB-	6.0	4.0	Oc	0.90			0.30	16.7	1.8	0.35	0.035	0.0350	14	4.0	5.0	500	120	54.6	0.45	0.008	0.202	1.0	1.1	0.048	0.198	133	0.316	1.087	365	1460	0.019	0.059	1.147	1.345
											Acs	1.50			0.30	33.3	8.9	0.23	0.023	0.0075	17	7.0	10.0	1,000	120	54.6	1.65	0.030	0.209	1.0	1.1	0.041		133	0.237		365	1460	0.007			
											Oc	1.60			0.30	16.7	17.3	0.35	0.035	0.0350	14	4.0	5.0	500	120	54.6	3.20	0.059	0.218	1.0	1.1	0.092		133	0.534		365	1460	0.034			
											Rs	2.00									18	8.0	125.0	12,500	120	54.6	5.00	0.092	0.227	1.0	1.1	0.005										
											Wr	5.00									18	8.0	135.0	13,500	120	54.6	8.50	0.156	0.247	1.0	1.1	0.012										
		13	24	12+890	to	13+010	120	NS1-BH-	8.5	2.0	Oc	1.00			0.30	16.7	2.0	0.35	0.035	0.0350	14	4.0	5.0	500	170	63.6	0.50	0.008	0.202	1.0	1.1	0.076	0.175	170	0.387	0.569	365	1460	0.021	0.026	0.595	0.769
											Acs	1.00			0.30	33.3	7.5	0.23	0.023	0.0075	17	7.0	10.0	1,000	170	63.6	1.50	0.024	0.207	1.0	1.1	0.039		170	0.182		365	1460	0.005			
											Rs	5.00	4,	10.5							18	8.0	70.0	7,000	170	63.6	4.50	0.071	0.221	1.0	1.1	0.030										
											Wr	10.00	21,25,32,1	23.3							18	8.0	155.0	15,500	170	63.6	12.00	0.189	0.257	1.0	1.1	0.031										
			25	13+010	to	13+150	140	NS1-BH-	8.5	3.9	Oc	1.90			0.30	16.7	3.8	0.35	0.035	0.0350	14	4.0	5.0	500	170	63.6	0.95	0.015	0.204	1.0	1.1	0.145	0.177	170	0.719	0.847	365	1460	0.040	0.049	0.896	1.073
											Acs	2.00	5, 6	5.5	0.30	120.0	14.6	0.23	0.023	0.0075	17	7.0	36.0	3,600	170	63.6	2.90	0.046	0.214	1.0	1.1	0.022		170	0.128		365	1460	0.009			
											Wr	2.55	17, 50<	17.0							18	8.0	113.0	11,300	170	63.6	5.18	0.081	0.224	1.0	1.1	0.009										
			26	13+150	to	13+250	100	Ns1-BH-	9.5	4.7	Pt	2.10	0	0.0	0.40	12.5	3.2	0.45	0.045	0.0550	13	3.0	5.0	500	190	67.2	1.05	0.016	0.205	1.0	1.1	0.180	0.243	214	1.229	1.445	365	1460	0.070	0.116	1.561	1.804
											Ac	2.60	6, 5, 10	7.0	0.30	153.3	14.1	0.30	0.030	0.0300	16	6.0	46.0	4,600	190	63.6	3.40	0.053	0.216	1.0	1.1	0.026		214	0.216		365	1460	0.047			
											As	3.30	3, 7, 30	5.0							18	8.0		6,150	190	63.6	6.35	0.100	0.230	1.0	1.1	0.026										
											Wr	2.90	39, 19	19.0							18	8.0	126.0	12,600	190	63.6	9.45	0.149	0.245	1.0	1.1	0.012										

Table 5.7.1-3 Summary of Settlement Calculation Result

Sectuion No.	Road	Section and Embankm,ent Height								Ground Conditions					Soil Parameter										Load on ground	Immediate Settlement								Actual Embank. Load	Primary Consolidation		Secondary Consolidation				Sub Total	Grand Total
		Sec.	Analysis Sec.	Analysis			Zone	Bor.	Emb. H	Total T.of Soft to Medium Soil	Layer	Thick.	SPT N	Ave. N	Cu/Pc	Pc	P0	CR	RR	Cαε	γ	γ'	Cu	Young's E	Dp	B	Z	Z/B	Iz	C1	C2	Siz	Si	Dp'	Spz	S_p	t1	t2	Ssz	Ss	Sp+Ss	Si+Sp+Ss
		No.	No.	Sta. No.			Length	No.	(m)	(m)	Name	(m)	(Blows)	(Blows)		(KN/m2)	(KN/m2)				(KN/m3)	(KN/m3)	(KN/m2)	(KN/m2)	(KN/m2)	(m)	(m)					(m)	(m)	(KN/m2)	(m)	(m)	(days)	(days)	(m)	(m)	(m)	(m)
3	OCH	14	27	13+250	to	13+350	100	NS1-BH-	10.0	6.3	Pt	2.00	0	0.0	0.40	12.5	3.0	0.45	0.045	0.0550	13	3.0	5.0	500	200	72.0	1.00	0.014	0.204	1.0	1.1	0.180	0.312	225	1.190	1.999	365	1460	0.066	0.136	2.135	2.447
											Acs	0.90	0	0.0	0.30	33.3	9.2	0.23	0.023	0.0075	17	7.0	10.0	1,000	200	67.2	2.45	0.036	0.211	1.0	1.1	0.042		225	0.187	(1.538)	365	1460	0.004			
											Ac	2.10	2, 7	3.0	0.30	66.7	18.6	0.30	0.030	0.0300	16	6.0	20.0	2,000	200	67.2	3.95	0.059	0.218	1.0	1.1	0.050		225	0.389		365	1460	0.038			
											Oc	1.30	4	4.0	0.30	86.7	27.5	0.35	0.035	0.0350	14	4.0	26.0	2,600	200	67.2	5.65	0.084	0.225	1.0	1.1	0.025		225	0.234		365	1460	0.027			
											As	3.70	12, 46, 50,	18.5							18	8.0		12,225	200	67.2	8.15	0.121	0.236	1.0	1.1	0.016										
											Wr	1.20	50<	50.0							18	8.0	200.0	20,000																		
		15	28	13+350	to	13+450	100	NS1-BH-	10.0	4.3	Pt	2.70	0, 0	0.0	0.40	12.5	4.1	0.45	0.045	0.0550	13	3.0	5.0	500	200	72.0	1.35	0.019	0.206	1.0	1.1	0.244	0.292	230	1.606	1.772	365	1460	0.089	0.126	1.899	2.191
											Ac	1.60	6, 8	7.0	0.30	153.3	12.9	0.30	0.030	0.0300	16	6.0	46.0	4,600	200	65.4	3.50	0.054	0.216	1.0	1.1	0.017		230	0.148		365	1460	0.029			
											As	1.70	9	9.0							18	8.0		7,950	200	65.4	5.15	0.079	0.224	1.0	1.1	0.011										
											Ac	0.90	12	12.0	0.30	266.7	34.0	0.23	0.023	0.0150	16	6.0	80.0	8,000	200	65.4	6.45	0.099	0.230	1.0	1.1	0.006		230	0.018		365	1460	0.008			
											As	4.10	19, 29,	24.0							18	8.0		14,700	200	65.4	8.95	0.137	0.241	1.0	1.1	0.015										
											Wr	1.70	50<	50.0							18	8.0	200.0	20,000																		
			29	13+450	to	13+690	240	NS1-BH-	10.0	5.0	Pt	1.30	0	0.0	0.40	12.5	2.0	0.45	0.045	0.0550	13	3.0	5.0	500	200	72.0	0.65	0.009	0.203	1.0	1.1	0.116	0.229	223	0.781	1.486	365	1460	0.043	0.118	1.604	1.833
											Ac	3.70	0, 3, 6	3.0	0.30	66.7	15.0	0.30	0.030	0.0300	16	6.0	20.0	2,000	200	65.4	3.15	0.048	0.214	1.0	1.1	0.087		223	0.685		365	1460	0.067			
											Ac	0.90	19	19.0	0.30	420.0	28.8	0.23	0.023	0.0150	16	6.0	126.0	12,600	200	65.4	5.45	0.083	0.225	1.0	1.1	0.004		223	0.019		365	1460	0.008			
											As	1.80	16, 47	16.0							18	8.0		11,100	200	65.4	6.80	0.104	0.231	1.0	1.1	0.008										
											Wr	5.00	47, 50<	47.0							18	8.0	200.0	20,000	200	65.4	10.20	0.156	0.247	1.0	1.1	0.014										
			30	13+690	to	13+750	60	NS1-BH-	10.0	2.5	Pt	2.50	0, 0	0.0	0.40	12.5	3.8	0.45	0.045	0.0550	13	3.0	5.0	500	200	72.0	1.25	0.017	0.205	1.0	1.1	0.226	0.258	226	1.482	1.482	365	1460	0.083	0.083	1.564	1.823
											Rs	6.40	20, 38, 16,	18.0							18	8.0	120.0	12,000	200	67.2	5.70	0.085	0.225	1.0	1.1	0.026										
											Wr	2.20	29, 50<	29.0							18	8.0	193.0	19,300	200	67.2	10.00	0.149	0.245	1.0	1.1	0.006										
		16	31	14+070	to	14+250	180	NS1-BH-	9.0	2.9	Ac	1.00			0.30	33.3	3.0	0.30	0.030	0.0300	16	6.0	10.0	1,000	180	65.4	0.50	0.008	0.202	1.0	1.1	0.040	0.245	202	0.268	1.350	365	1460	0.018	0.085	1.435	1.679
											Pt	1.90	0, 0	0.0	0.40	12.5	8.9	0.45	0.045	0.0550	13	3.0	5.0	500	180	63.6	1.95	0.031	0.209	1.0	1.1	0.157		202	1.063		365	1460	0.063			
											As	1.25	7	7.0							18	8.0		7,050	180	63.6	3.53	0.055	0.217	1.0	1.1	0.008										
											Acs	0.85	13	13.0	0.30	286.7	24.7	0.23	0.023	0.0075	17	7.0	86.0	8,600	180	63.6	4.58	0.072	0.222	1.0	1.1	0.004		202	0.019		365	1460	0.004			
											Wr	5.00	10, 50<	10.0							18	8.0	66.0	6,600	180	63.6	7.50	0.118	0.235	1.0	1.1	0.035										
		17	32	14+280	to	14+410	130	NS1-BH-	8.5	2.5	Oc	2.45	0, 0	0.0	0.30	16.7	4.9	0.35	0.035	0.0350	14	4.0	5.0	500	170	63.6	1.23	0.019	0.206	1.0	1.1	0.189	0.212	170	0.920	0.920	365	1460	0.052	0.052	0.972	1.183
											As	1.45	6	6.0							18	8.0		6,600	170	63.6	3.18	0.050	0.215	1.0	1.1	0.009										
											Wr	6.10	21, 34, 29,	28.0							18	8.0	186.0	18,600	170	63.6	6.95	0.109	0.233	1.0	1.1	0.014										
		18	33	14+600	to	14+650	50	NS1-BH-	8.0	2.6	Oc	2.60	0, 0	0.0	0.30	16.7	5.2	0.35	0.035	0.0350	14	4.0	5.0	500	160	61.8	1.30	0.021	0.206	1.0	1.1	0.189	0.207	175	0.986	0.986	365	1460	0.055	0.055	1.041	1.247
											As	1.60	22	22.0							18	8.0		13,800	160	61.8	3.40	0.055	0.217	1.0	1.1	0.004										
											Wr	6.30	29, 50<	29.0							18	8.0	193.0	19,300	160	61.8	7.35	0.119	0.236	1.0	1.1	0.014										
		19	34	15+050	to	15+130	80	NS1-BH-	8.0	4.0	Oc	4.00	0, 0, 9	0.0	0.30	16.7	8.0	0.35	0.035	0.0350	14	4.0	5.0	500	160	61.8	2.00	0.032	0.210	1.0	1.1	0.295	0.372	183	1.527	1.527	365	1460	0.084	0.084	1.612	1.984
											As	2.20	20, 7	7.0							18	8.0		7,050	160	61.8	5.10	0.083	0.225	1.0	1.1	0.012										
											Wr	10.50	11, 32,46	11.0							18	8.0	73.0	7,300	160	61.8	11.45	0.185	0.256	1.0	1.1	0.065										
		20	35	15+445	to	15+640	195	NS1-BH-	9.0	4.9	Ac	2.50	1, 1	1.0	0.30	33.3	7.5	0.30	0.030	0.0300	16	6.0	10.0	1,000	180	65.4	1.25	0.019	0.206	1.0	1.1	0.102	0.188	204	0.651	1.581	365	1460	0.045	0.125	1.705	1.894
											Pt	2.40	2, 14	2.0	0.40	32.5	18.6	0.45	0.045	0.0550	13	3.0	13.0	1,300	180	65.4	3.70	0.057	0.217	1.0	1.1	0.079		204	0.930		365	1460	0.079			
											As	1.50	12	12.0							18	8.0		9,300	180	65.4	5.65	0.086	0.226	1.0	1.1	0.007										
											Wr		50<	50.0							18	8.0	200.0	20,000																		
4	Approach	30	36	0+50	to	0+130	80	NS1-BH-	7.0	4.0	Acs	1.90	1	1.0	0.30	33.3	6.7	0.23	0.023	0.0075	17	7.0	10.0	1,000	140	58.2	0.95	0.016	0.205	1.0	1.1	0.060	0.088	140	0.314	0.314	365	1460	0.009	0.009	0.323	0.410
											As	2.10	5, 3	3.0							18	8.0		5,250	140	58.2	2.95	0.051	0.215	1.0	1.1	0.013										
											Wr	3.00	5, 8, 10	7.6							18	8.0		7,320	140	58.2	5.50	0.095	0.228	1.0	1.1	0.014										

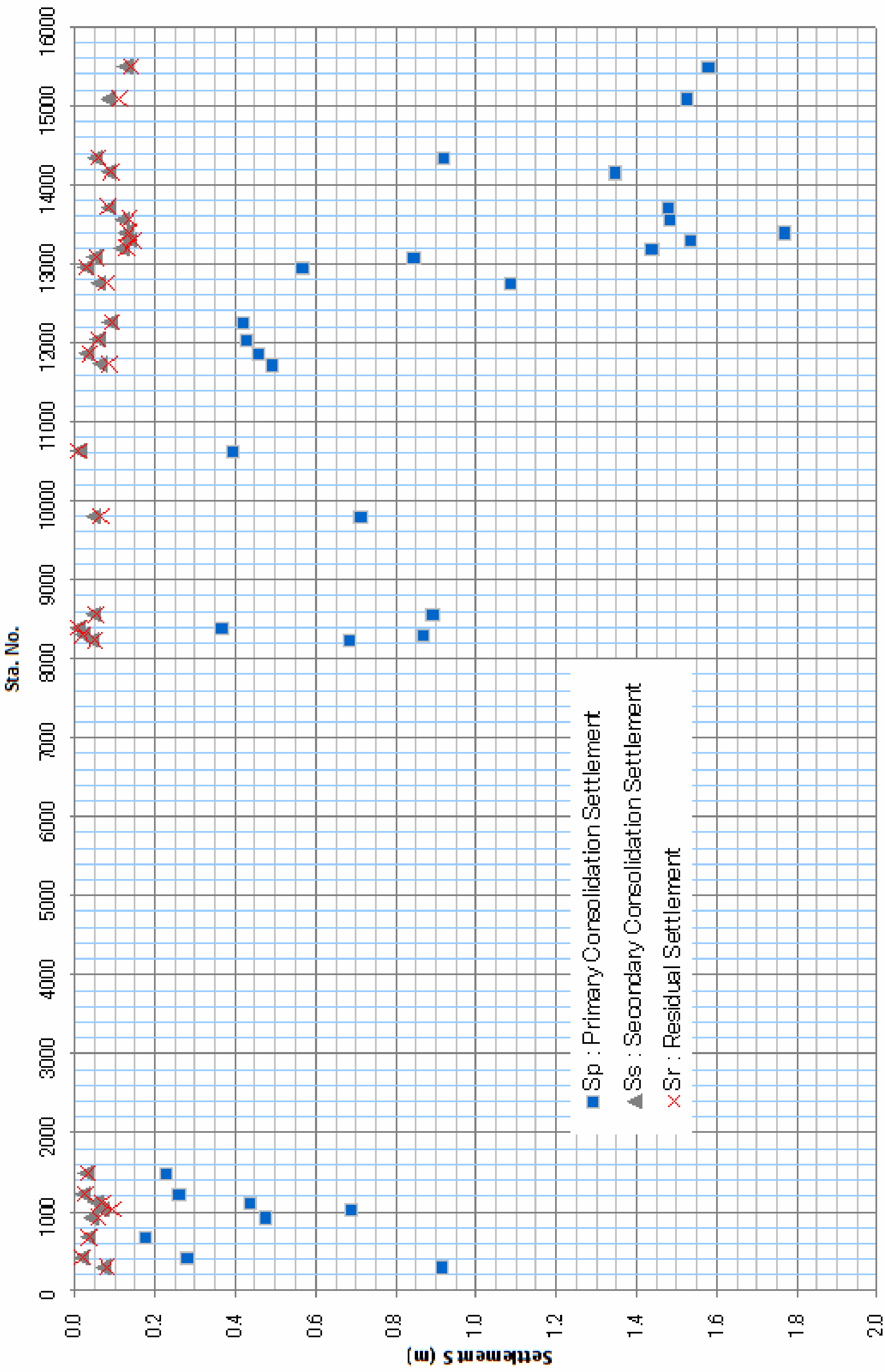


Fig. 5.7.1 Settlement Calculation Result

Table 5.7.2 Summary of Analysis Results

Analysis Section No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Beginning Point of Section	0 + 225	0 + 370	0 + 480	0 + 880	0 + 980	1 + 80	1 + 160	1 + 300	0 + 25	0 + 20	0 + 280	8 + 200	8 + 270	8 + 330	8 + 460	9 + 330	9 + 700	10 + 460
Embankment Height (m)	8.0	5.0	5.0	4.0	4.0	4.5	8.0	8.0	9.0	3.0	6.5	12.5	12.0	12.0	12.0	6.0	5.5	10.0
Thickness of Soft to Medium Soil (m)	4.65	4.50	10.75	3.50	4.00	4.00	8.65	3.00	10.75	4.00	3.50	7.80	4.00	9.70	9.60	6.00	8.60	6.00
Thickness of Cohesive Soil (m)	3.65	1.90	2.40	3.50	4.00	4.00	2.00	1.50	2.40	4.00	3.50	2.40	4.00	1.70	2.80	5.00	2.20	2.00
Selected Treatment Type	F	C	C	C	C	C	C	B	E	C	F	F	F	D	F	H	C	F
Untreated: Embank. Height (m)	8.0	5.0	5.0	4.0	4.0	4.5	8.0	8.0	9.0	3.0	6.5	12.5	12.0	12.0	12.0	6.0	5.5	10.0
Untreated: Allowable Safety Factor	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25
Untreated: Minimum S.F. (Fsmin)	0.421	0.599	0.870	0.606	0.480	0.595	0.798	0.949	0.971	0.513	0.454	0.575	0.427	0.713	0.565	0.435	0.372	0.569
Untreated: Judgement	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
Treated: Nos. Loading Steps	1	2	1	2	2	2	2	1	1	1	2	1	2	1	1	2	2	1
Treated: Treatment Method: Geotextile (kN/m)	110	120	70	130	150	130	150	160	—	120	60	110	170	80	100	—	190	100
Treated: Treatment Method: Inside Portion	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	—	Band Drain	Band Drains	Band Drains	Band Drains	Band Drains	—	Band Drains	Replacement	Band Drains	Band Drain
Treated: Treatment Method: Slope Portion	GCP	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	—	GCP	Band Drains	GCP	GCP	GCP	GCP	GCP	Replacement	Band Drains	GCP
Treated: 1st Step Loading (Short): Embank. Height (m)	8.0	3.0	5.0	2.0	2.0	2.5	6.0	8.0	9.0	3.0	4.5	12.5	10.0	12.0	12.0		3.5	10.0
Treated: 1st Step Loading (Short): Allowable Safety Factor	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20		1.20	1.20
Treated: 1st Step Loading (Short): Minimum S.F. (Fsmin)	1.208	1.602	1.269	2.031	1.905	1.842	1.346	1.252	1.425	1.205	1.407	1.216	1.244	1.233	1.209		1.356	1.240
Treated: 2nd Step Loading (Short): Embank. Height (m)	—	5.0	—	4.0	4.0	4.5	8.0	—	—	—	6.5	—	12.0	—	—		5.5	—
Treated: 2nd Step Loading (Short): Allowable Safety Factor	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20		1.20	1.20
Treated: 2nd Step Loading (Short): Minimum S.F. (Fsmin)	—	1.237	—	1.215	1.214	1.226	1.277	—	—	—	1.265	—	1.263	—	—		1.253	—
Treated: After Completion (Long Term): Embank. Height (m)	8.0	5.0	5.0	4.0	4.0	4.5	8.0	8.0	9.0	3.0	6.5	12.5	12.0	12.0	12.0	6.0	5.5	10.0
Treated: After Completion (Long Term): Allowable Safety Factor	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25
Treated: After Completion (Long Term): Minimum S.F. (Fsmin)	1.253	1.259	1.252	1.371	1.382	1.320	1.253	1.251	1.404	1.519	1.251	1.254	1.253	1.257	1.252	1.400	1.272	1.261
Treated: Judgement	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Settlement at the Center of Embankment: Primary Consolidation: Final Settlement Sf (m)	0.917	0.283	0.178	0.478	0.690	0.437	0.262	0.229	0.278	0.521	0.670	0.687	0.870	0.369	0.893	-	0.715	0.396
Center: Primary Consolidation: Settlement at 4years later after raising up St4 (m)	0.917	0.283	0.178	0.478	0.690	0.437	0.262	0.228	0.278	0.521	0.670	0.687	0.870	0.368	0.893	-	0.715	0.396
Center: Primary Consolidation: Settlement at 1year later after raising up St1 (m)	0.906	0.279	0.175	0.462	0.663	0.420	0.257	0.222	0.273	0.519	0.657	0.681	0.869	0.366	0.886	-	0.697	0.396
Center: Primary Consolidation: Residual Settlement St4−St1=Spr (m)	0.009	0.004	0.003	0.016	0.027	0.017	0.005	0.006	0.005	0.002	0.013	0.006	0.001	0.002	0.007	-	0.018	0.000
Center: Secondary Consolidation: 1 year later after raising up t1 (days)	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	-	365	365
Center: Secondary Consolidation: 4 years later after raising up t4 (days)	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	-	1460	1460
Center: Secondary Consolidation: Secondary Consolidation for 3 years Ssr (m)	0.072	0.015	0.033	0.043	0.067	0.052	0.021	0.027	0.043	0.038	0.043	0.043	0.018	0.008	0.047	-	0.046	0.009
Center: Total Residual Settlement Σ Sr=Spr+Ssr (m)	0.081	0.019	0.036	0.059	0.094	0.069	0.026	0.033	0.048	0.040	0.056	0.049	0.019	0.010	0.054	-	0.064	0.009
Center: Judgement (Σ Sr≦0.15m)	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	-	OK	OK
Settlement at the Shoulder of Embankment: Primary Consolidation: Final Settlement Sf (m)	0.496	0.277	0.166	0.451	0.649	0.420	0.257	0.226	0.150	0.480	0.354	0.376	0.467	0.202	0.489	-	0.702	0.217
Shoulder: Primary Consolidation: Settlement at 4years later after raising up St4 (m)	0.496	0.277	0.166	0.451	0.649	0.42	0.257	0.225	0.150	0.480	0.354	0.376	0.467	0.202	0.489	-	0.702	0.217
Shoulder: Primary Consolidation: Settlement at 1year later after raising up St1 (m)	0.496	0.273	0.163	0.436	0.624	0.403	0.252	0.219	0.150	0.478	0.354	0.376	0.467	0.202	0.489	-	0.685	0.217
Shoulder: Primary Consolidation: Residual Settlement St4−St1=Spr (m)	0.000	0.004	0.003	0.015	0.025	0.017	0.005	0.006	0.000	0.002	0.000	0.000	0.000	0.000	0.000	-	0.017	0.000
Shoulder: Secondary Consolidation: 1 year later after raising up t1 (days)	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	-	365	365
Shoulder: Secondary Consolidation: 4 years later after raising up t4 (days)	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	-	1460	1460
Shoulder: Secondary Consolidation: Secondary Consolidation for 3 years Ssr (m)	0.072	0.015	0.033	0.043	0.067	0.052	0.021	0.027	0.043	0.038	0.043	0.043	0.018	0.008	0.047	-	0.046	0.009
Shoulder: Total Residual Settlement Σ Sr=Spr+Ssr (m)	0.072	0.019	0.036	0.058	0.092	0.069	0.026	0.033	0.043	0.040	0.043	0.043	0.018	0.008	0.047	-	0.063	0.009
Shoulder: Judgement (Σ Sr≦0.15m)	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	-	OK	OK
Settlement at the Toe of Embankment: Primary Consolidation: Final Settlement Sf (m)	0.015	0.009	0.012	0.019	0.074	0.022	0.010	0.009	0.008	0.021	0.010	0.008	0.009	0.004	0.012	-	0.021	0.002
Toe: Primary Consolidation: Settlement at 4years later after raising up St4 (m)	0.015	0.009	0.012	0.019	0.074	0.022	0.010	0.009	0.008	0.021	0.010	0.008	0.009	0.004	0.012	-	0.021	0.002
Toe: Primary Consolidation: Settlement at 1year later after raising up St1 (m)	0.015	0.009	0.012	0.018	0.074	0.021	0.010	0.009	0.008	0.021	0.010	0.008	0.009	0.004	0.012	-	0.021	0.002
Toe: Primary Consolidation: Residual Settlement St4−St1=Spr (m)	0.000	0.000	0.000	0.001	0.000	0.001	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	0.000	0.000
Toe: Secondary Consolidation: 1 year later after raising up t1 (days)	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	-	365	365
Toe: Secondary Consolidation: 4 years later after raising up t4 (days)	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	-	1460	1460
Toe: Secondary Consolidation: Secondary Consolidation for 3 years Ssr (m)	0.072	0.015	0.033	0.043	0.067	0.052	0.021	0.027	0.043	0.038	0.043	0.043	0.018	0.008	0.047	-	0.046	0.009
Toe: Total Residual Settlement Σ Sr=Spr+Ssr (m)	0.072	0.015	0.033	0.044	0.067	0.053	0.021	0.027	0.043	0.038	0.043	0.043	0.018	0.008	0.047	-	0.046	0.009
Toe: Judgement (Σ Sr≦0.15m)	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	-	OK	OK

Analysis Section No.	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36
Beginning Point of Section	11 + 660	11 + 800	12 + 0	12 + 235	12 + 700	12 + 890	13 + 10	13 + 150	13 + 250	13 + 350	13 + 450	13 + 690	14 + 50	14 + 280	14 + 600	15 + 60	15 + 495	0 + 50
Embankment Height (m)	6.5	7.0	5.0	4.0	6.0	8.5	8.5	9.5	10.0	10.0	10.0	10.0	9.0	8.5	8.0	8.0	9.0	7.0
Thickness of Soft to Medium Soil (m)	12.00	15.20	9.00	7.00	6.00	17.00	6.45	10.90	11.20	12.70	12.70	11.10	10.00	10.00	10.50	16.70	6.40	7.00
Thickness of Cohesive Soil (m)	1.20	2.00	4.90	4.00	4.00	2.00	3.90	4.70	6.30	4.30	5.90	2.50	2.90	2.45	2.60	4.00	4.90	1.90
Selected Treatment Type	B	C	C	C	F	F	F	F	G	F	F	F	F	F	F	F	F	D
Untreated: Embank. Height (m)	6.5	7.0	5.0	4.0	6.0	8.5	8.5	9.5	10.0	10.0	10.0	10.0	9.0	8.5	8.0	8.0	9.0	7.0
Untreated: Allowable Safety Factor	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25
Untreated: Minimum S.F. (Fsmin)	0.560	0.621	0.810	0.897	0.315	0.530	0.481	0.468	0.438	0.387	0.531	0.406	0.381	0.402	0.384	0.264	0.410	0.597
Untreated: Judgement	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
Treated: Nos. Loading Steps	2	2	2	1	2	1	1	2	2	2	2	2	2	1	2	2	2	1
Treated: Treatment Method: Geotextile (kN/m)	160	170	50	160	100	90	110	120	150	150	90	150	150	170	130	190	110	70
Treated: Treatment Method: Inside Portion	—	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	GCP (as=10%)	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	Band Drains	—
Treated: Treatment Method: Slope Portion	—	Band Drains	Band Drains	Band Drains	GCP	GCP	GCP	GCP	GCP	GCP	GCP	GCP	GCP	GCP	GCP	GCP	GCP	GCP
Treated: 1st Step Loading (Short): Embank. Height (m)	4.5	5.0	3.0	4.0	4.0	8.5	8.5	7.5	8.0	8.0	8.0	8.0	7.0	8.5	6.0	6.0	7.0	7.0
Treated: 1st Step Loading (Short): Allowable Safety Factor	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
Treated: 1st Step Loading (Short): Minimum S.F. (Fsmin)	1.295	1.269	1.380	1.322	1.384	1.229	1.212	1.246	1.261	1.216	1.250	1.231	1.272	1.209	1.321	1.312	1.259	1.237
Treated: 2nd Step Loading (Short): Embank. Height (m)	6.5	7.0	5.0	—	6.0	—	—	9.5	10.0	10.0	10.0	10.0	9.0	—	8.0	8.0	9.0	—
Treated: 2nd Step Loading (Short): Allowable Safety Factor	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
Treated: 2nd Step Loading (Short): Minimum S.F. (Fsmin)	1.311	1.299	1.307	—	1.268	—	—	1.273	1.272	1.264	1.269	1.275	1.268	—	1.275	1.278	1.263	—
Treated: After Completion (Long Term): Embank. Height (m)	6.5	7.0	5.0	4.0	6.0	8.5	8.5	9.5	10.0	10.0	10.0	10.0	9.0	8.5	8.0	8.0	9.0	7.0
Treated: After Completion (Long Term): Allowable Safety Factor	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25
Treated: After Completion (Long Term): Minimum S.F. (Fsmin)	1.274	1.265	1.258	1.259	1.255	1.254	1.253	1.257	1.257	1.251	1.257	1.261	1.253	1.253	1.256	1.261	1.250	1.251
Treated: Judgement	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Settlement at the Center of Embankment: Primary Consolidation: Final Settlement Sf (m)	0.494	0.459	0.430	0.421	1.087	0.569	0.847	1.441	1.538	1.770	1.486	1.482	1.350	0.920	0.986	1.527	1.581	0.314
Center: Primary Consolidation: Settlement at 4years later after raising up St4 (m)	0.459	0.459	0.430	0.421	1.087	0.569	0.847	1.441	1.538	1.770	1.486	1.482	1.350	0.920	0.986	1.527	1.581	0.313
Center: Primary Consolidation: Settlement at 1year later after raising up St1 (m)	0.437	0.450	0.429	0.415	1.068	0.565	0.841	1.428	1.527	1.762	1.468	1.482	1.344	0.913	0.969	1.499	1.567	0.311
Center: Primary Consolidation: Residual Settlement St4−St1=Spr (m)	0.022	0.009	0.001	0.006	0.019	0.004	0.006	0.013	0.011	0.008	0.018	0.000	0.006	0.007	0.017	0.028	0.014	0.002
Center: Secondary Consolidation: 1 year later after raising up t1 (days)	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365
Center: Secondary Consolidation: 4 years later after raising up t4 (days)	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460
Center: Secondary Consolidation: Secondary Consolidation for 3 years Ssr (m)	0.063	0.029	0.057	0.087	0.059	0.026	0.049	0.116	0.136	0.126	0.118	0.083	0.085	0.052	0.055	0.084	0.125	0.009
Center: Total Residual Settlement Σ Sr=Spr+Ssr (m)	0.085	0.038	0.058	0.093	0.078	0.030	0.055	0.129	0.147	0.134	0.136	0.083	0.091	0.059	0.072	0.112	0.139	0.011
Center: Judgement (Σ Sr≦0.15m)	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Settlement at the Shoulder of Embankment: Primary Consolidation: Final Settlement Sf (m)	0.486	0.455	0.424	0.368	0.570	0.311	0.458	0.757	1.058	0.944	0.784	0.796	0.730	0.503	0.530	0.809	0.826	0.171
Shoulder: Primary Consolidation: Settlement at 4years later after raising up St4 (m)	0.454	0.455	0.424	0.368	0.570	0.311	0.458	0.757	1.058	0.944	0.784	0.796	0.730	0.503	0.530	0.809	0.826	0.171
Shoulder: Primary Consolidation: Settlement at 1year later after raising up St1 (m)	0.433	0.447	0.423	0.362	0.570	0.311	0.458	0.757	1.058	0.944	0.784	0.796	0.730	0.503	0.530	0.809	0.826	0.171
Shoulder: Primary Consolidation: Residual Settlement St4−St1=Spr (m)	0.021	0.008	0.001	0.006	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Shoulder: Secondary Consolidation: 1 year later after raising up t1 (days)	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365
Shoulder: Secondary Consolidation: 4 years later after raising up t4 (days)	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460
Shoulder: Secondary Consolidation: Secondary Consolidation for 3 years Ssr (m)	0.063	0.029	0.057	0.087	0.059	0.026	0.049	0.116	0.136	0.126	0.118	0.083	0.085	0.052	0.055	0.084	0.125	0.009
Shoulder: Total Residual Settlement Σ Sr=Spr+Ssr (m)	0.084	0.037	0.058	0.093	0.059	0.026	0.049	0.116	0.136	0.126	0.118	0.083	0.085	0.052	0.055	0.084	0.125	0.009
Shoulder: Judgement (Σ Sr≦0.15m)	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Settlement at the Toe of Embankment: Primary Consolidation: Final Settlement Sf (m)	0.030	0.016	0.034	0.093	0.075	0.008	0.016	0.029	0.034	0.035	0.028	0.021	0.062	0.013	0.014	0.021	0.022	0.005
Toe: Primary Consolidation: Settlement at 4years later after raising up St4 (m)	0.021	0.016	0.034	0.093	0.075	0.008	0.016	0.029	0.034	0.035	0.028	0.021	0.062	0.013	0.014	0.021	0.022	0.005
Toe: Primary Consolidation: Settlement at 1year later after raising up St1 (m)	0.016	0.016	0.033	0.093	0.075	0.008	0.016	0.029	0.034	0.035	0.028	0.021	0.062	0.013	0.014	0.021	0.022	0.005
Toe: Primary Consolidation: Residual Settlement St4−St1=Spr (m)	0.005	0.000	0.001	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Toe: Secondary Consolidation: 1 year later after raising up t1 (days)	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365	365
Toe: Secondary Consolidation: 4 years later after raising up t4 (days)	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460	1460
Toe: Secondary Consolidation: Secondary Consolidation for 3 years Ssr (m)	0.063	0.029	0.057	0.087	0.059	0.026	0.049	0.116	0.136	0.126	0.118	0.083	0.085	0.052	0.055	0.084	0.125	0.009
Toe: Total Residual Settlement Σ Sr=Spr+Ssr (m)	0.068	0.029	0.058	0.087	0.059	0.026	0.049	0.116	0.136	0.126	0.118	0.083	0.085	0.052	0.055	0.084	0.125	0.009
Toe: Judgement (Σ Sr≦0.15m)	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK

- : Minimum Safety Factor which calculates tensile force.
- GCP: Gravel Compaction Pile in 1.2 m square spacing (as=27%).
- Band Darains: Band Drains in 1.0 m square spacing.

5.7.2. Selection of Applicable Countermeasure

Table 5.7.3 provides the possibly minimal countermeasures to be selected in this study based on the available geotechnical information and the subsoil profile illustrated from the results of investigation carried out at during this study. For the selection of countermeasure, it was assumed where the subsurface stratum uniformly distributes and provides physically homogeneous.

The geological feature generally observed in Western region of Sri Lanka shows it's most probably inhomogeneous. Therefore, some parts of subsoil conditions along the OCH would be probably different from the geotechnical profile illustrated based on such spot samples given by the geotechnical investigation. In this regards, the review and rectification of countermeasures should be considered to meet with the actual subsoil conditions prior to and during the construction.

Especially design of countermeasure against settlement has been carried out based on the soil data obtained at detailed design stage to achieve that the residual settlement for 3 years after traffic opening will not exceed 15 cm. Actually this 15cm of the allowable residual settlement for 3 years is a target value for design of soft soil treatment and also execution of construction. Because it can be anticipated that actual settlements for 3 years sometimes exceed due to inhomogeneous soil conditions and limited information of ground conditions even though construction work was executed properly according to the obtained data and design.

However, it predominates that the countermeasures studied this time should be reasonably applicable for the actual subsoil conditions on site to avoid the risk to increase the construction costs due to unknown subsurface conditions.

Taking into consideration on above subjects, the appreciable countermeasures against soft soil ground along the OCH Northern Section1 are described in **Table 5.7.4** and **Fig. 5.7.2.** All locations of box culverts and abutments of viaducts on the soft ground treated by GCP and Preload are shown in **Table 5.7.5**.

Detailed appraisal for the countermeasure will be extended in the Contract that the Contractor should review the soil profile and propose the eligible countermeasures suit to the site.

The supervision consultant shall properly check the Contractor's proposal whether the engineering justification makes minimal countermeasures to pursue the economical efficiency with satisfying the geotechnical requirements for soft soil ground.

Table 5.7.3 Type of Countermeasures

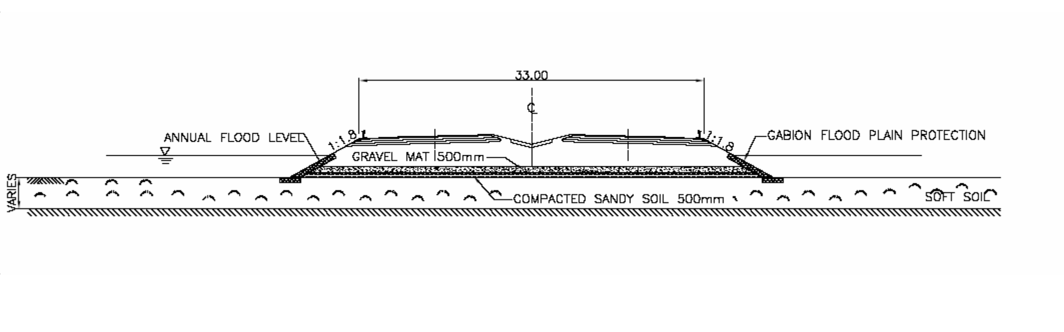
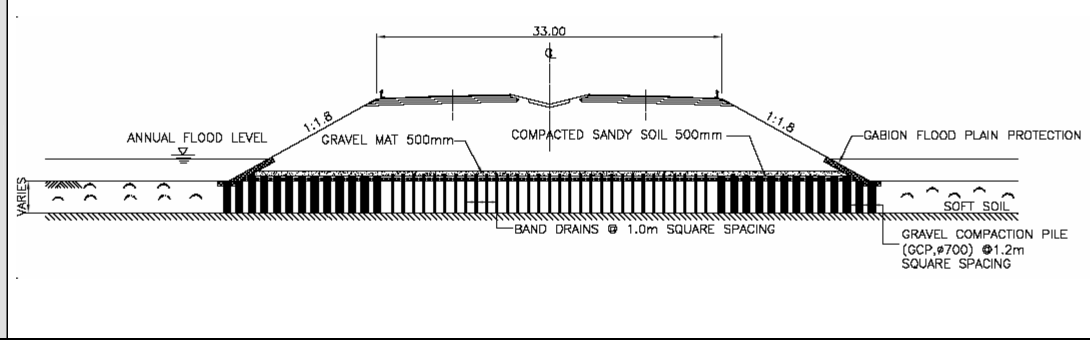
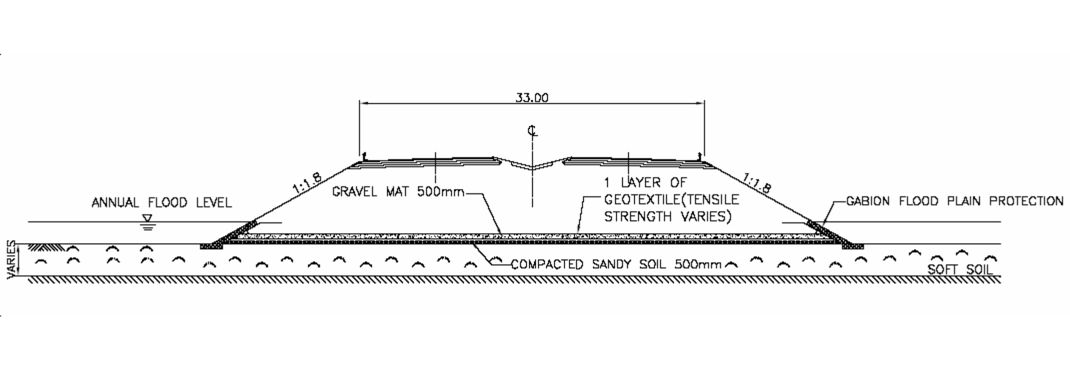
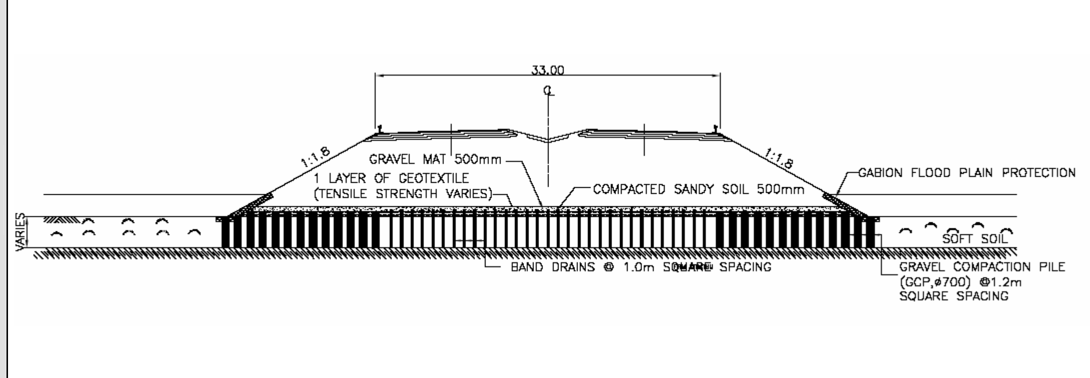
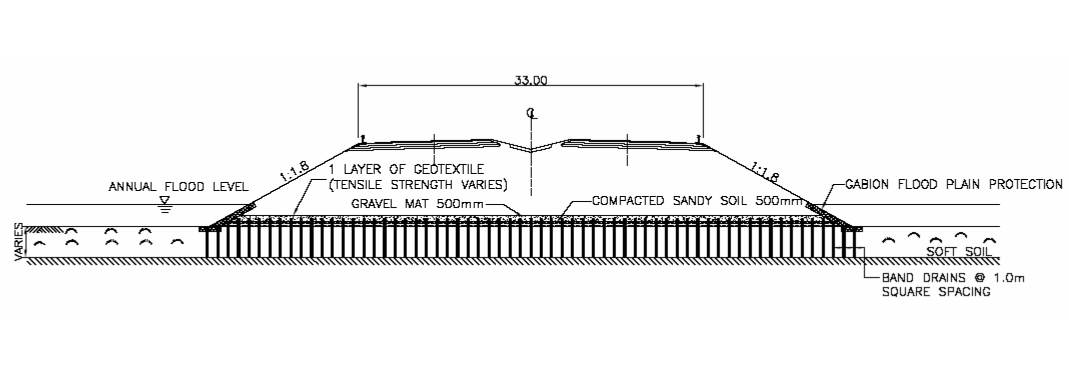
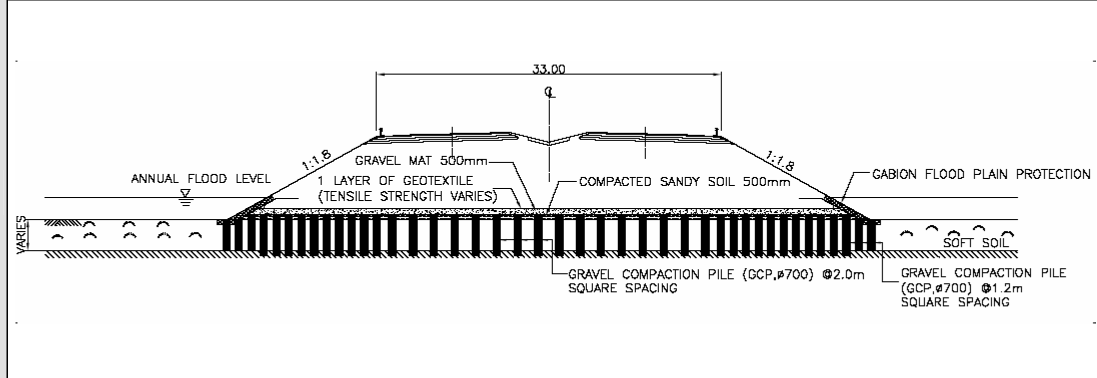
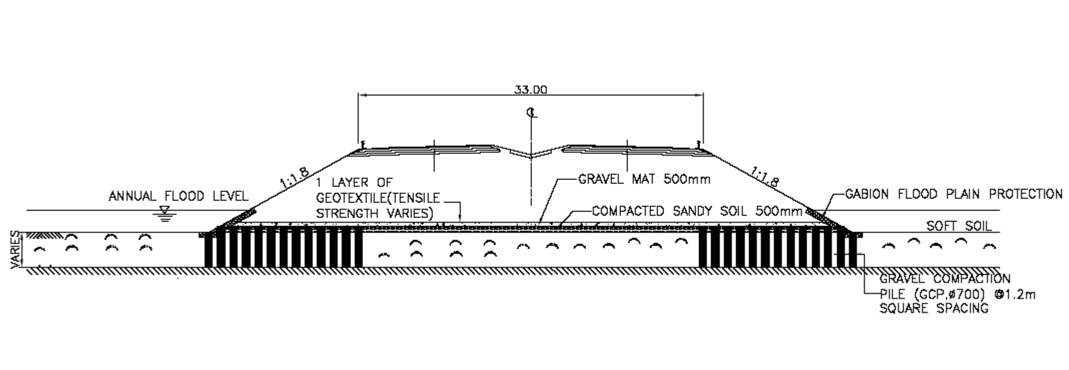
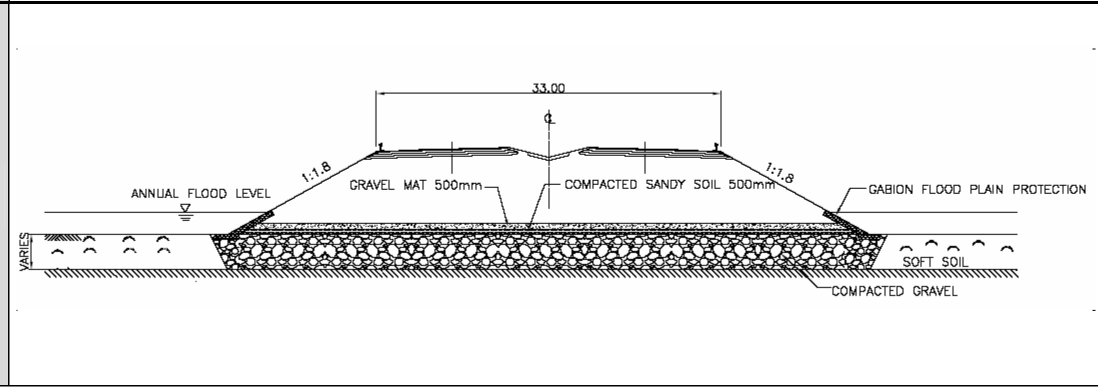
Type	Cross Section	Descriptions	Type	Cross Section	Descriptions
A		■Gravel Mat	**E**		■Gravel Mat ■Band Drains ■Gravel Compaction Pile
B		■Gravel Mat ■1 layer of Geotextile	**F**		■Gravel Mat ■1 Layer of Geotextile ■Band Drains ■Gravel Compaction Pile
C		■Gravel Mat ■1 layer of Geotextile ■Band Drains	**G**		■Gravel Mat ■1 Layer of Geotextile ■Gravel Compaction Pile (Whole Area)
D		■Gravel Mat ■1 layers of Geotextile ■Gravel Compaction Pile	**H**		■Replacement By Excavation.

Table 5.7.4 Soft Soil Countermeasures along the OCH Northern Section 1

Road	Sec.No.	Sta. No. Start	to	End	Distance (m)	Type of Treatment	Type of Geotextile (kN/m)	Depth of Soft Soil (m)	Nos. of Steps for Embankment	Analysis Sec. No.
OCH	1	8+200	to	8+330	130	F	400.0	3 - 4	2	12, 13
	2	8+330	to	8+460	130	D	200.0	1 - 2	1	14
	3	8+460	to	8+608	148	F	200.0	2 - 3	1	15
		8+608	to	8+668	60					
	4	9+330	to	9+365	35	H	-	4 - 5	1	16'
	5	9+700	to	9+900	200	C	400.0	2 - 3	2	17
	6	10+460	to	10+800	340	F	200.0	2 - 3	1	18
	7	10+800	to	10+930	130	A	-	0 - 1	1	-
	8	11+660	to	11+800	140	B	400.0	1 - 2	2	19
	9	11+800	to	11+930	130	C	400.0	2 - 3	2	20
	10	12+000	to	12+080	80	C	100.0	2 - 3	2	21
	11	12+235	to	12+395	160	C	400.0	2 - 4	1	22
	12	12+700	to	12+825	125	F	200.0	2 - 4	2	23
	13	12+890	to	13+250	360	F	300.0	2 - 5	2	24, 25, 26
	14	13+250	to	13+350	100	G	300.0	5 - 6	2	27
	15	13+350	to	13+775	425	F	300.0	2 - 5	2	28, 29, 30
	16	14+050	to	14+250	200	F	300.0	2 - 4	2	31
	17	14+280	to	14+410	130	F	400.0	2 - 3	1	32
	18	14+600	to	14+650	50	F	300.0	2 - 3	2	33
	19	15+060	to	15+115	55	F	400.0	2 - 4	2	34
	20	15+495	to	15+610	115	F	300.0	2 - 6	2	35
		16+520	to	16+560	40					
A1 Bypass	21	0+225	to	0+370	145	F	300.0	3 - 4	1	1
	22	0+370	to	0+480	110	C	300.0	2 - 3	2	2
	23	0+480	to	0+780	300	C	200.0	2 - 3	1	3
	24	0+780	to	0+880	100	A	-	0 - 1	1	-
	25	0+880	to	1+300	420	C	300.0	2 - 4	2	4, 5, 6, 7
	26	1+300	to	1+680	380	B	400.0	1 - 2	1	8
Ramp-1	27	0+025	to	0+260	235	E	-	2 - 3	1	9
Ramp-4	28	0+020	to	0+280	260	C	300.0	3 - 4	1	10
	29	0+280	to	0+365	85	F	200.0	3 - 4	2	11
Approach	30	0+50	to	0+130	80	D	200.0	1 - 2	1	36

* "Approach" means the approach road to overpass bridge at the Sta. No. 9+415 of OCH.

Table 5.7.5 Locations of box culverts and abutments of viaduct on the soft ground treated by GCP (Gravel Compaction Pile) and Preload

(Box Culvert)

Location		STA.	Skew Angle
Road	Main Line	13+327	82
		14+619	90
	A1 Ramp-1	0+136	79
	A1 Bypass	0+285	90
		1+293	82
Drainage	Main Line	9+760	70
		10+530	90
		10+690	75
		11+740	90
		12+350	90
		12+735	90
		13+210	90
		13+550	90
		14+340	90
		15+585	90
	A1 Ramp-4	0+025	90
	A1 Bypass	0+575	90
		0+675	90
		0+990	90
		1+312	90
Irrigation	Main Line	9+726	53
		9+876	69
		10+497.5	60
	A1 Bypass	0+500.5	78

(Abutment of Viaduct)

Location	STA.	ID No.
Main Line	8+648.0	V1
	13+755.0	V2
	14+070.0	
	15+095.0	V3
	15+515.0	
	16+524.5	H9
A1 I.C. Ramp2,3	0+241.3 0+239.8	V5, V6

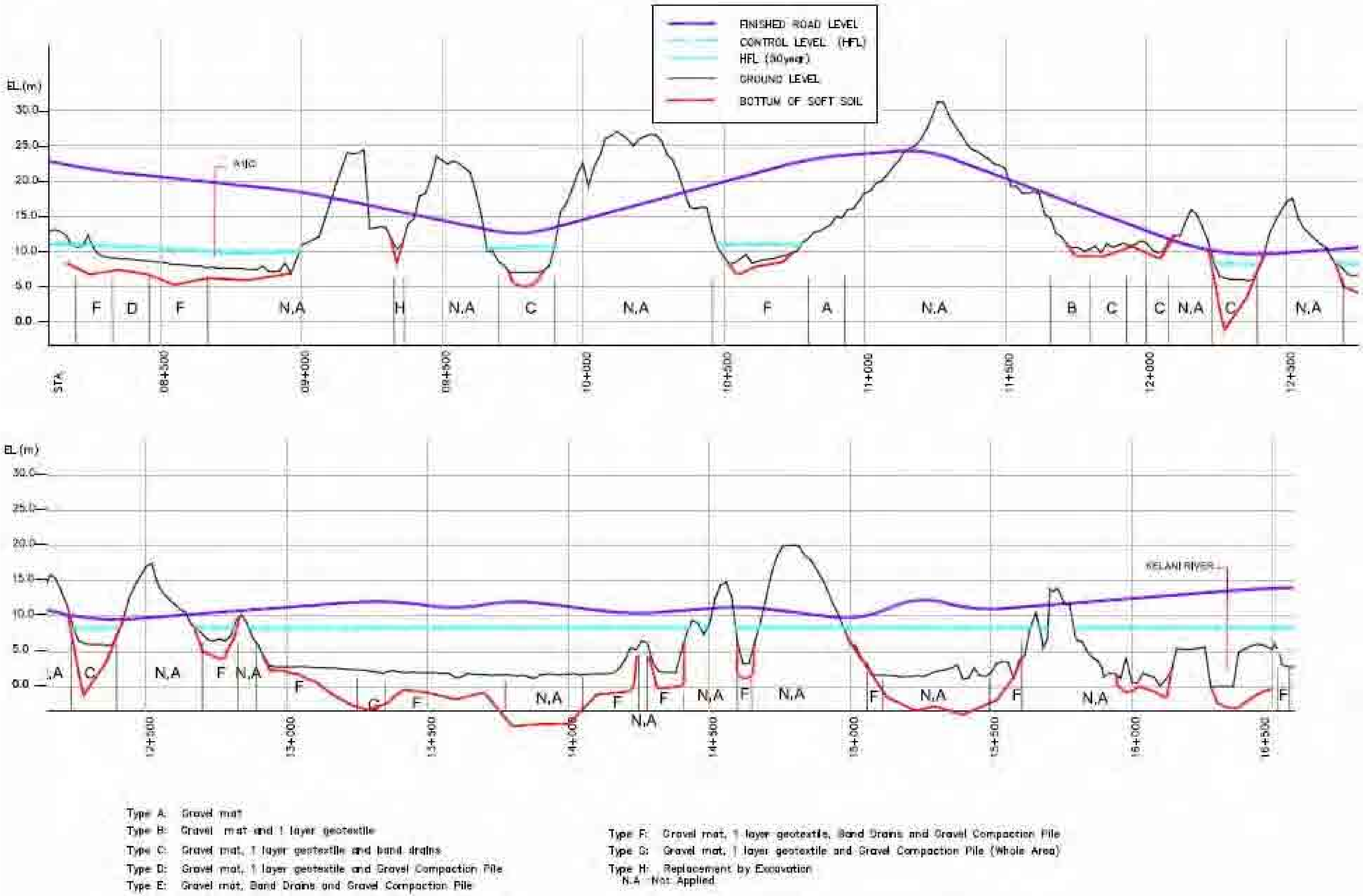


Fig. 5.7.2-1 Section of Countermeasures

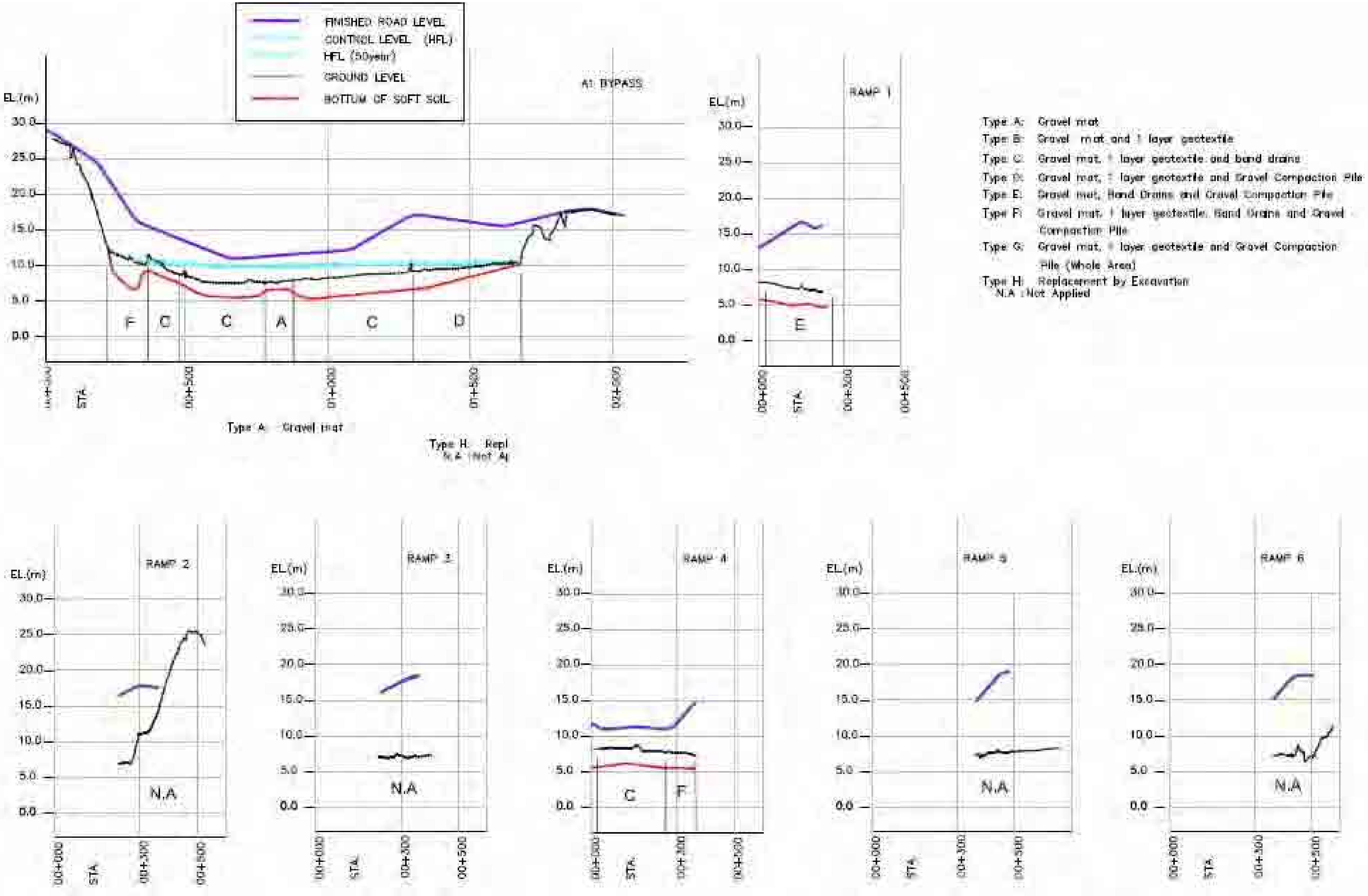


Fig. 5.7.2-2 Section of Countermeasures

5.7.3. Points to pay attention on for soft soil treatment

(1) Construction of Embankment on Soft Ground

a) It is important to fill the soil slowly with monitoring of ground deformation and pore water pressure of soil in order to maintain the stability of embankment.
b) It should be carried out carefully to spread the soil in thin layer with keeping the level of fill to uneven loading on soft soil.
c) It is essential to keep the leaving period for embankment as it is for more than half year after raising up the embankment to the designed level to perform the required consolidation degree (U=90%).
d) Even if countermeasures are taken for embankment stability against soft ground, those countermeasures are mainly considered for only toward the cross section. Therefore embankment slope angle for longitudinal direction should be taken very gentle during construction period. Because geotextile usually dose not cover the required stability in longitudinal direction in case of the geotextile with different strength in warp and weft directions.

(2) Countermeasures

a) Replacement Method

The required excavated depth should be where the firm clay or medium dense sand layer is confirmed. If the ground is very soft and has high water content, excavation itself usually becomes very difficult. In such area, excavation should be carried out only in mid of dry season to avoid the troubles due to water in the soil and under ground.

For hauling the replaced high water content soil such as peat or organic clay, it is very difficult to load and unload them without any drying. Therefore stock pile yard is necessary for drying them.

b) Geotextile

Geotextile has the different tensile strengths in two rectangular directions. The direction having bigger strength should meet with a cross direction of road alignment.

c) Band Drains

Required installation depth should be at the bottom of soft to medium consistency clay layer which needs to accelerate the consolidation settlement.

d) Gravel Compaction Pile (GCP)

Gravel compaction method usually causes the following negative influence to the surrounding area from its nature of the method.

*Noise and vibration
*Heaving of surrounding ground

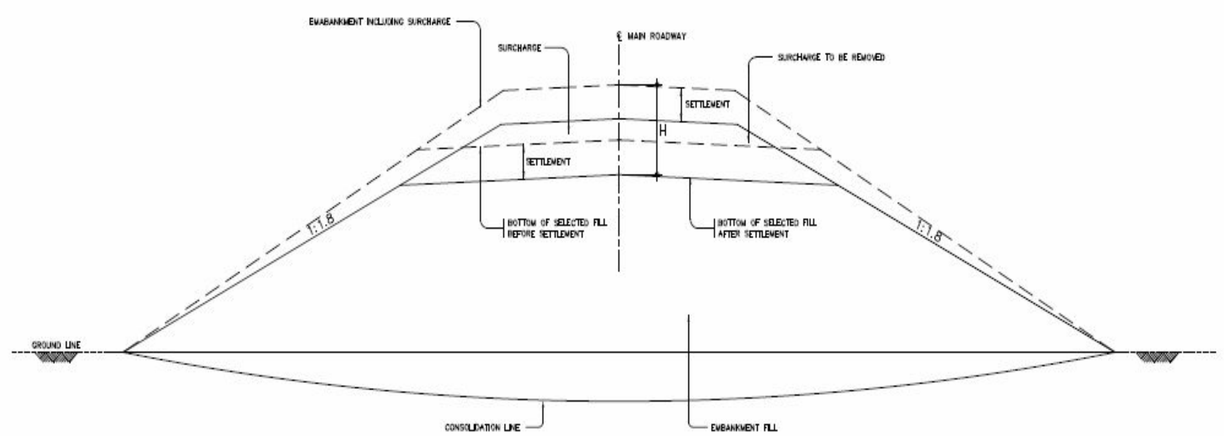
Therefore some measures such as introducing low noise and vibration machines and ditch excavation along the R.O.W. are should be considered depending on circumstances at site.

Required driving depth of GCP should be at the top of firm or medium dense sand layer. Furthermore as the strength of cohesive soil usually decreases temporarily by driving gravel compaction piles, it should be left for one to three months as it is to recover the strength again. Then embankment work should be started.

For the locations where box culverts and abutments of viaducts are constructed on the soft ground, gravel compaction pile (GCP; ϕ=0.7m, □1.2m) and preload (up to designed road level) is applied in order to remove the negative influence to the structures due to consolidation settlement by embankment. After removal of preload with leaving period of more than one to three months, those structures can be constructed (Refer to Fig. 5.7.3).

e) Surcharge

Surcharge shall be place on the embankment portion on the soft ground to cover the settlement volume of embankment and also to achieve the consolidation settlement as required. Surcharge height at each section of soft soil treatment area are shown in Fig. 5.7.4.



OCH	1	8+200	to	8+330	130	0.8
	2	8+330	to	8+460	130	0.8
	3	8+460	to	8+608	148	0.8
		8+608	to	8+668	60	
	4	9+330	to	9+365	35	0.8
	5	9+700	to	9+900	200	0.8
	6	10+460	to	10+800	340	0.8
	7	10+800	to	10+930	130	0.8
	8	11+660	to	11+800	140	0.8
	9	11+800	to	11+930	130	0.8
	10	12+000	to	12+080	80	0.8
	11	12+235	to	12+395	160	0.8
	12	12+700	to	12+825	125	1.5
	13	12+890	to	13+250	360	1.5
	14	13+250	to	13+350	100	1.5
	15	13+350	to	13+775	425	1.5
	16	14+050	to	14+250	200	1.5
	17	14+280	to	14+410	130	1.5
	18	14+600	to	14+650	50	1.5
	19	15+060	to	15+115	55	1.5
	20	15+495	to	15+610	115	1.5
		16+520	to	16+560	40	
A1 Bypass	21	0+225	to	0+370	145	0.8
	22	0+370	to	0+480	110	0.8
	23	0+480	to	0+780	300	0.8
	24	0+780	to	0+880	100	0.8
	25	0+880	to	1+300	420	0.8
	26	1+300	to	1+680	380	0.8
Ramp-1	27	0+025	to	0+260	235	0.8
Ramp-4	28	0+020	to	0+280	260	0.8
	29	0+280	to	0+365	85	0.8
Approach	30	0+50	to	0+130	80	0.8

* "Approach" means the approach road to overpass bridge at the Sta. No. 9+415 of OCH.

Fig.5.7.4 Surcharge for embankment on soft soil

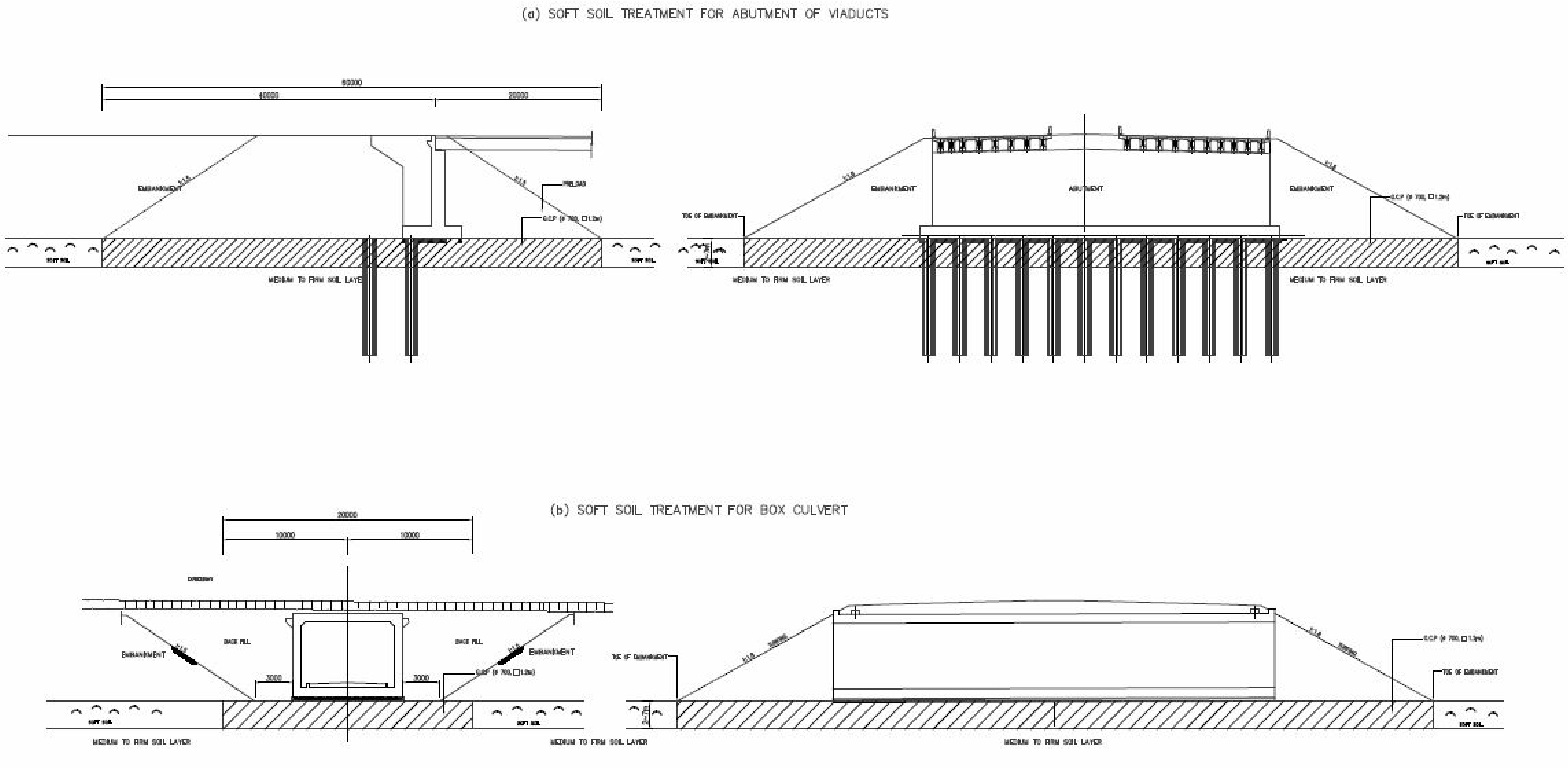


Fig. 5.7.3 Soft Ground Treatment for Box Culvert and Abutment of Viaduct

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CHAPTER 6

HYDROLOGY & DRAINAGE DESIGN

CHAPTER 6 HYDROLOGY AND DRAINAGE DESIGN

6.1. Background

This chapter presents the hydrological and hydraulic analysis along with details of drainage provisions for OCH (Northern Section-1) section, from Sta. 8+200 to Sta. 16+560. The basic design of entire OCH from Sta. 0+000 to Sta. 28+500 was conducted in 2001 and detailed design of OCH (Southern Section) from Sta. 16+560 to Sta. 28+500 was completed in year 2005. In addition, Supplementary Environmental Impact Assessment for Biyagama deviation from Sta. 13+000 to 16+000 was carried out by Road Development Authority in 2005-2006.

During the detail design study, all related institutions such as Irrigation Department, Sri Lanka Land Reclamation and Development Corporation and Department of Agrarian Services were consulted through Road Development Authority and their suggestions / recommendations were also incorporated. In addition, detail design of culverts, bridges and canals/side ditches for irrigation/drainage, design of Rakkgahawatha minor flood protection scheme, clear opening span for Kelani River bridge along with hydrological analysis were submitted and discussed with these institutions.

6.2. Existing Condition

The proposed Outer Circular Highway (OCH) Northern-Section 1 begins at station 8+300 close to Colombo-Kandy Road (A1). The beginning section from station 8+300 to 8+800 traverses through low line areas consisting of paddy fields in Ihala Biyanwila and Gonahena in Mahara Division. The interchange (A1) for Kandy Road, its access roads and ramps are located between station 8+650 and 9+000. The reach between station 8+800 to 15+850 consists of intermittent hilly and low ground areas with small streams draining water from minor sub catchments. The OCH passes through the flood plain of Kelani River in Biyagama area between station 15+850 and 16+100 and crosses a tributary of Kelani River called Rakgahawatta Ela at station 15+850 and Kelani River around station 16+350. The access roads to OCH from Kelaniya-Mudungoda Road (B214) and Colombo-Avissawella Road (A110) have been provided. The OCH Northern Section-1 connects to its Southern Section after crossing the Colombo-Avissawella Road, at station 16+700.

The OCH has been designed with elevated bridge structures mainly at four locations other than structures provided for road crossings, in consultation with Road Development Authority analyzing several issues especially soft soil treatment, related with OCH construction. These bridges are located from station 8+648 to 8+970, 13+755 to 14+070, 15+130 to 15+445 and 15+895 to 16+525. Though provision of these bridges was not a main issue in hydrological consideration, it remarkably mitigates the adverse impacts on hydrological environment providing free flow (uninterrupted drainage) especially at A1 interchange and sensitive flood plain areas of Kelani River at Biyagama.

6.3. Design Standard

Recommended design storm return periods are given below. In OCH (Southern Section), same design standards were applied with the consent of Road Development Authority.

Table 6.3.1 Design Standards

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

6.4. Data Collection

(1) Topographical Data

The topographic maps produced by Survey Department at 1:10,000 scale covering sub catchments of the study area including the Kelani tributary, Mudun Ela were collected. In addition, survey plan in 1:2000 produced by Survey Department conducting the areal survey for the Outer Circular Highway and the ground survey data provided by Survey Department and local consultants were also utilized. These maps were used to identify the existing waterways (drainage, irrigation), low line areas, flood plains, high water elevations and also to demarcate the sub catchment boundaries of waterways crossing the OCH trace. The consultants made many site visits to gather information from public as well as to check the accuracy of data collected from various sources.

(2) River / Waterways / Canal Cross Sections

Following surveys were done for hydrological analysis in addition to the alignment cross section survey conducted at every 20m intervals for entire road trace.

- Kelani River
  - Altogether nine cross sections of Kelani River was surveyed. Three cross sections were surveyed at the centre line of the proposed bridge, 150m upstream and 150m downstream. In addition, four cross sections were taken two each at upstream and downstream of the bridge at an interval of around 15m. Further, two cross sections were obtained at the confluence of Kelani River – Rakgahawatta Ela (Mudun Ela) and around 100m downstream of the confluence.

  - Additional 65 cross sections covering a reach of 90.6 km from Kitulgala to Colombo were also collected (Source: Department of Irrigation and Lanka Hydraulic Institute) for mathematical modeling of Kelani River by MIKE 11 software.

- Rakgahawatta Ela (Mudun Ela)
  - Thirty eight (38) cross sections of Rakgahawatta Ela were surveyed from Biyagama area to Kelani river confluence at close intervals around 50-100m. It included the existing minor flood protection structure of 6 cell openings each 1.8 m x 1.3 m and existing bridge at Kelaniya-Mudungoda Road (B214).

- Irrigation / Drainage Canals
  - All waterways that intersect OCH trace were surveyed.

(3) Rainfall Data

During the Basic Design Study, Intensity-Duration-Frequency Curve/Table for Colombo was developed by Gumbel extreme value probability distribution for short term rainfalls For this purpose, rainfall data at relevant gauging stations including Colombo Met Station were collected from Meteorological Department for the period of 1951-2000. During the hydrological analysis, both short duration and long duration rainfall events for standard return periods were applied. For flash floods, 3 hr storm duration was adopted as the time of concentration in the sub catchment is about 3 hours. However for long duration flood storms, standard 24 hr storm duration was taken into account.

The hyetographs for 3 hour were derived using Intensity Duration – Frequency curves (IDF Curves) at Colombo. The design 24 hour storm hyetographs were obtained from the "Study on Storm Water Drainage Plan for Colombo Metropolitan Region 2003" by JICA Study.

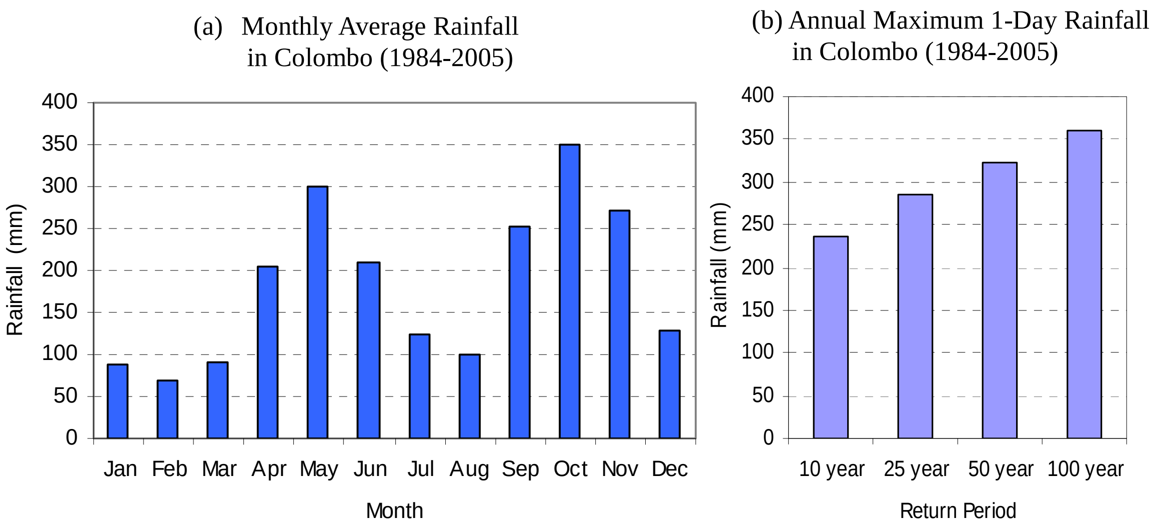

Fig 6.4.1 Rainfall Pattern in Colombo

Table 6.4.1 Intensity of Rainfall in Colombo (mm)

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

(4) Discharge / Water Level Data

The daily discharge and water level data for gauging stations along the Kelani River at Kitulgala, Deraniyagala, Holombuwa, Glencourse, Hanwella were obtained from Irrigation Department and encoded, basically targeting the past 22 years. It was collected to set up a MIKE 11 model for Kelani River Basin. In addition, hourly water levels at Kitulgala, Deraniyagala, Holombuwa, Glencourse, Hanwella, Nagalagum street and flood gauging posts located at downstream of Hanwella were also recorded for past flood events in May/June 1989, May 1990, Nov 2004.

6.5. High Water Level Survey

High water level survey along the OCH trace was conducted by flood hearing from long time residents in the area. The flood marks/levels were noted and subsequently flood elevations were determined by topograpghic survey tying up these points to the nearest bench marks. In addition, high water levels at streams crossing the OCH were also calculated analyzing the corresponding sub catchment discharge. In case of Keleni river basin, high water level is dependent on the river water level as there are no flood control dykes in this area.

Table 6.5.1 High Water Level along OCH

No.	Station	Ground Level m.MSL	Annual HWL (m.MSL)	50 year HWL (m.MSL)	Remarks
1	8+150	13.0 (A 1); 10.3 (GL)		11.0	Kalu Ela Basin - upper reach
2	8+760	8.5 (Shramadana Mw); 7.3 (GL)	8.9	9.7	
3	10+630	9.5 (Paddy)	10.7	11.2	
4	12+300	5.8	6.8	8.2	Kelani Ganga Basin - High water level dependent on water level in Kelani River
5	13+200	2.7	4	8.2	
6	15+200	1.6	4	8.2	
7	16+100	5.3 (B 214); 5.0 (GL)		8.2	
8	16+500	6.0 (A110); 5.3 (GL)		8.2	

Note: HWL:High Water Level; GL:Ground Level

6.6. Hydrology / Hydraulic Analysis

The hydrological and hydraulic analysis was mainly conducted for the following designs.

- Kelani River flood analysis and selection of bridge span
- Scour analysis of proposed Kelani River bridge
- Selection of bridge span for Rakgahawatta Ela
- Design of minor flood protection structure at Kelani-Rakgahawatta Ela confluence and
- Design of drainage provisions such as culverts for drainage/irrigation, bridges, side canals/ditches etc. for waterways crossing the OCH trace and related minor roads.

6.6.1. Kelani River Flood Analysis and Selection of Bridge Span

(1) Introduction

The Kelani River is the third largest river in Sri Lanka with a catchment area of 2292 km2. The river basin is entirely located in the wet zone of the of country and the annual average rainfall varies from 2500 mm at lower elevations to 5000 mm at the upper reaches. The river drops from 2500 m MSL elevation from the central mountain to the sea level, traversing on the western slope. The river enters the sea along the northern boundary of the city of Colombo. The total length of the river had been estimated as 150 km and average annual runoff to the sea is 5500 million m3.

The areas below Yatiyantota, Ruwenwella up to the city of Colombo are vulnerable to floods of Kelani River and the Colombo suburbs had been protected by a system of dykes constructed in 1925 to 1930. These dykes provide protection only for a distance of 15 km from the mouth of the river. Fig. 6.6.1 shows the Sub Catchments in Kelani Basin with Rainfall and Hydrometric Gauging Stations.

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This section provides the brief description of data collection, model set up, calibration, and validation and study results.

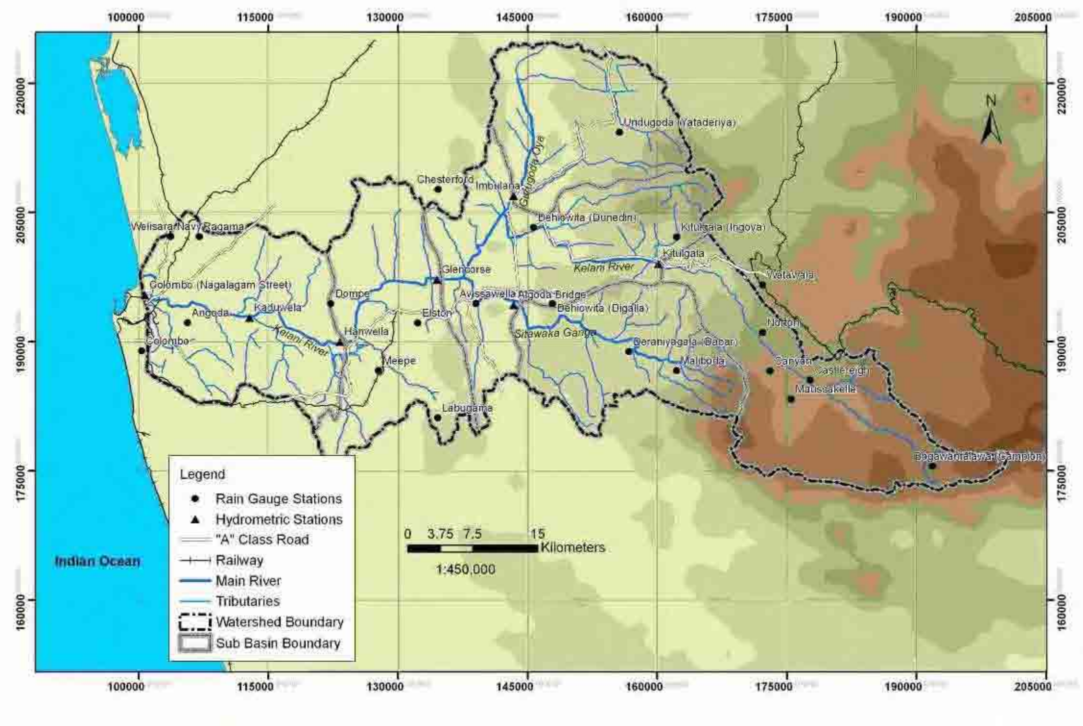


Fig. 6.6.1 Sub Catchments in Kelani Basin with Rainfall and Hydrometric Gauging Stations

(2) Objectives

Proposed Outer Circular Highway has to traverse through the flood plain of Kelani River passing the towns of Biyagama, and Kaduwela, which are frequently subjected to floods of Kelani River. When Kelani River overflows during heavy rains, water spreads on either side of the river to an approximate distance of 2 to 3 km between Pugoda and Ambatale. This area is located along the river between 15 km and 40 km from the mouth. The reach between 0 to 15 km is protected from the Kelani flood bunds as mentioned earlier. The proposed OCH crosses the river at a location, called Rakgahawatta,19 km upstream from the sea and has an average formation level of 12.5 m M.S.L. The average ground level of the flood plain area is approximately at 6.0 m M.S.L. Therefore considerable amount of obstruction to the natural passage of flood is inevitable. The flood analysis will focus on the impact of the proposed highway to the natural flooding in the Kelani valley closer to the proposed bridge site in between Kaduwela to Ambatale located in between 20 to 15 km along the river. Several options of bridge spans had been examined to minimize the environmental impact due to the proposed bridge. Fig. 6.6.3 shows the longitudinal section of the river between Colombo and Kitulgala selected for the model study.

(3) Study Methodology

The Kelani river reach between Kitulgala to Colombo was numerically modeled by using the Mike II one dimension hydrodynamic model developed by the Denish Hydraulic Institution (DHI). The Mike II hydrodynamic model has been developed on the basis of the solution to the well-known St. Venant equations for unsteady flow. Basically it solves the equation of continuity and equation of momentum by using the finite difference method. The computational grid consisting of alternative Q and H points at each time step and each distance step [Δ t & Δx].

The model has the flexibility to use different frictional factors along the river for different reaches. In addition it can use different frictional factors at each cross section for varying river depths. Generally higher resistance factor for flood plains and lower factors for the river channel are adopted. In this study the hydrological module NAM for simulating rainfall-runoff process in river catchments was linked MIKE 11 HD for modeling of river basins. Three different schematizations were used to represent the inundation areas as flood plains, flood cells and additional flooded areas.

The Kelani Estuary is subjected to morphological changes due to excessive sand mining and therefore cross sections and bed levels are changing with time in the reach below Hanwella at 34.6 km.

(4) Data Collection

In order to set up the Mike II model for a river reach, following basic data are required.

- River network data - coordinates of few points along the center line of the river with longitudes and latitudes to describe the river geometry.
- River cross sections at 2 to 3 km intervals to describe the conveyance capacity of the river canal, flood plain, elevations of dykes etc.
- Dimensions of all man made structures such as bridges, culverts, weirs across the river.
- Both upstream and downstream boundary data. Normally upstream boundary is a discharge hydrograph and the downstream is a water level graph.
- Historical observations of water levels at intermediate locations of the river reach for the selected boundary data in order to calibrate the model and

thereafter for verification.

- Rainfall and evaporation data of the intervening catchment during the selected events for calibration and verification. These data are required to simulate the runoff process for the river reach within the model length laterally between model boundaries.
- Synthetic design hydrograph or probable design discharge of different return periods such as 50 to 100 years for simulation.

<u>River Cross Sections</u>

The river cross sections and bridge sections available from Irrigation Department and Lanka Hydraulic Institute were used in setting up the model. In addition, nine cross sections were surveyed close to the proposed bridge centerline as mentioned above. The raw data is available in the form of (x, z) coordinates, x being the horizontal distance across the cross section and z the elevation above a selected datum (MSL). The bed resistance at river cross sections was specified in terms of Manning's roughness coefficients. The model processed these data and computed the hydraulic parameters such as cross sectional area, width, hydraulic radius, conveyance at different water levels as "processed data". The longitudinal profile of the river was also automatically generated. Figure below shows the longitudinal profile of the main Kelani River.

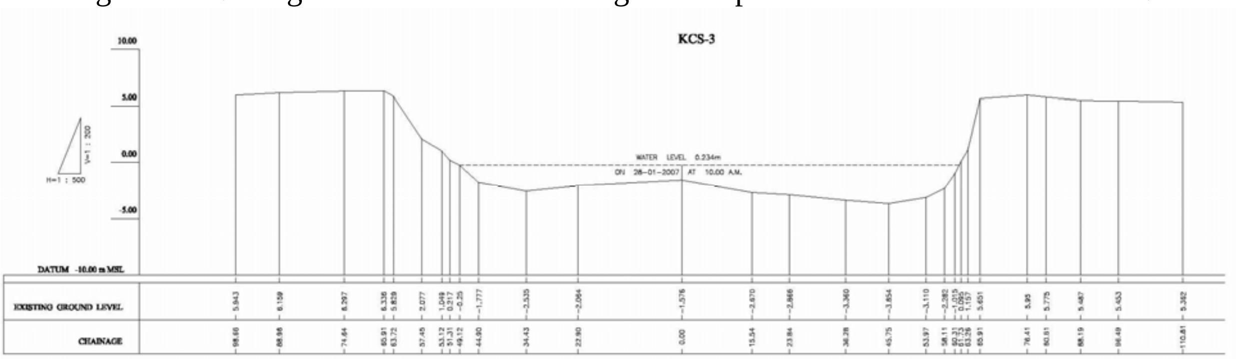
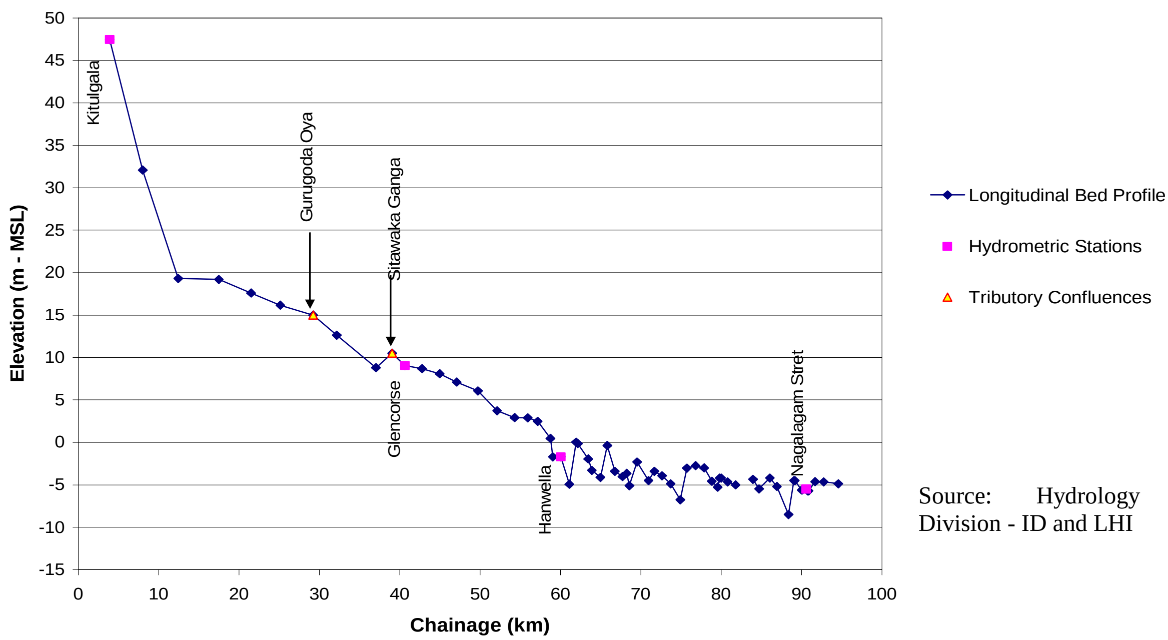


Fig. 6.6.2 River Cross Section of Proposed Kelani River Bridge



Source: Hydrology Division - ID and LHI

Fig. 6.6.3 Longitudinal Profile of Kelani River

Water Level and Discharge Data

The water level and discharge data was collected from Irrigation Department. The data at Glencourse is quite reliable as it is a stable river section according to Irrigation Department.

Table 6.6.1 Annual Maximum Flood Levels in Colombo, Hanwella and Glencourse

Water Year	Nagalagam	Hanwella	Glencourse
	Water Level (m MSL)	Discharge (m3/s)	Discharge (m3/s)
1960	1.42	N/A	971.3
1961	1.14	N/A	787.8
1962	1.68	N/A	894.8
1963	1.30	N/A	1765.8
1964	1.96	N/A	871.9
1965	1.55	N/A	1024.8
1966	2.64	N/A	1413.6
1967	2.74	N/A	3058.6
1968	2.79	N/A	1902.3
1969	1.42	N/A	963.6
1970	1.07	N/A	956.0
1971	2.23	N/A	1981.0
1972	1.63	N/A	1428.0
1973	0.91	N/A	883.3
1974	1.65	2378.6	2067.1
1975	2.01	2619.3	1948.8
1976	1.42	1704.4	887.2
1977	1.50	1776.9	1550.6
1978	1.60	1911.4	2665.2
1979	1.45	2217.2	1182.5
1980	0.74	579.9	608.5
1981	1.09	1670.7	1873.4
1982	1.42	N/A	1529.0
1983	1.07	N/A	1249.9
1984	1.57	2009.2	2647.0
1985	1.37	1401.4	1620.6
1986	0.76	1254.4	925.4
1987	0.94	522.8	1322.1
1988	1.24	979.6	1582.9
1989	2.79	2440.0	3428.0
1990	1.20	568.0	1032.4
1991	1.50	1007.0	963.6
1992	1.55	1076.0	1346.2

1993	1.60	1292.0	1519.5
1994	1.52	1172.4	452.4
1995	1.27	764.0	719.0
1996	1.52	1135.0	1360.6
1997	1.65	1216.0	1550.6
1998	1.45	1017.0	803.1
1999	1.96	1282.2	1413.6
2000	1.52	989.7	810.6

N/A: Not Available

Rainfall Data
All available daily rainfall data since 1950 at the selected rain gauging stations to represent the Kelani catchment in spatial distribution was collected from Meteorological Department. Sub basin daily mean rainfall was automatically calculated by NAM model according to the Thiessen Weights when rain gaining stations and rainfall time series files are given to the model.

(5) Boundary Conditions

The boundary conditions are required at all external boundaries of the model. The boundary conditions used for Kelani River and the two main tributaries are given. The NAM simulated runoff for the 3 intermediate sub catchments, Glencorse, Hanwella and Colombo were linked as uniformly distributed lateral inflows between upstream and downstream river stations.

(6) Calibration and Verification of Model

The model calibration was carried out by setting up the hydrodynamic model downstream of Glencourse. The inflow boundary input for hydrodynamic modeling was obtained from recorded hourly stream flow data at Glencourse gauging station for the flood events considered. Downstream boundary is the water level at the river mouth. Model was calibrated for 1989 flood event and validated for 1990 flood event.

(7) Analysis of Proposed Kelani River Bridge

The proposed Kelani River Bridge was analyzed with following conditions.

- Mathematical model was developed using MIKE 11 river modeling (explained above)
- Model was calibrated for 1989 flood and validated for 1990 flood
- Design Return Period : 100 year
- Design Discharge : 4400 m3/s at Glencourse
- Simulated scenarios
    - o Existing condition
    - o 350m long Kelani Bridge at OCH crossing point about 1km downstream of Kaduwela

Design discharge of 100 year flood at Glencourse (4400 m3/s) was estimated by frequency analysis, extreme value type I Gumble distribution, using annual maximum discharge data collected from Irrigation Department. Analyzing the distribution of flood hydrographs at Glencourse, 100 year return period flood hydrograph was derived and applied in the simulation.

The change of water profile along the river is a tool to identify the direct impact on flood behaviour due the construction of proposed highway. When proposed bridge section reaches peak of the flood, water level along the river is given in the Table 6.6.2. According to the results, back water effect is limited to a 10cm height for the bridge span of 350m.

Table 6.6.2 Water Profile along Kelani River for 100 Year Flood with / without 350m Span Kelani River Bridge

Location	Distance from Sea (km)	100 Year Flood		
		Water Elevation (m MSL)		Difference (m)
		Existing Condition	With Bridge (350m span)	
Nagalagam	4.2	4.004	4.010	0.006
Kotuwila	5.7	4.235	4.241	0.006
Wennawatta	8.8	4.797	4.803	0.006
Kelanimulla	12.0	6.002	6.007	0.005
Ambatale	14.9	7.305	7.310	0.005
Rakgahawatta	18.0	8.151	8.154	0.003
350m downstream of bridge		8.370	8.367	-0.003
Proposed Bridge	19.0	8.396	8.460	0.064
350m upstream of bridge		8.412	8.479	0.067
Kaduwela	19.9	8.421	8.521	0.100
Bomiriya	23.8	9.146	9.227	0.081
Nawagamuwa	26.6	9.953	10.004	0.051
Ranala	28.1	10.382	10.422	0.040
Atigala	30.9	11.066	11.091	0.025
Hanwella	34.0	11.551	11.569	0.018

(8) Proposed Bridge Opening Span

- During the detail design study, necessary clear span for proposed Kelani bridge is considered as 350m due to the following reasons.
  - Bridge span included the main roads (A110 & B214) either side of river eliminating a small embankment reach between river and A110 road, thereby to prevent possible high erosion at the toe of embankment due to high velocity during severe floods.
  - To minimize the possible backwater effect with urbanization of flood plain area in future.
  - To minimize the obstruction to free flow of water and thereby to mitigate the adverse impacts of floods to the surrounding area.

- Model results confirm that 350m clear span is sufficient for minimizing the backwater effect and hence, it is recommended.

- All related data, river cross sections, MIKE 11 model details and results obtained were submitted to Irrigation Department for their approval. Irrigation Department has given the approval mentioning that it has not exceeded the allowable backwater effect.

- However, bridge length has extended from station 15+895 to 16+524, covering the low line flood plain area due to several other issues related with OCH construction, especially soft ground treatment, in consultation with Road Development Authority.

6.6.2. Scour analysis of proposed Kelani River bridge

(1) General

The HEC-RAS Model; River Analysis System developed by US Army Corps of Engineers, Hydrologic Engineering Center was used for hydraulic modelling of the Kelani Bridge. HEC-RAS has the capability to compute one-dimensional water surface profiles for both steady and unsteady flow. Sub-critical, supercritical and mix flow regime profiles can be calculated.

Water surface profiles are computed from one cross section to the next by solving the energy equation using standard-step method. Energy losses are evaluated by friction (Manning’s equation) and contraction/expansion coefficients. HEC-RAS requires inputs for boundary conditions of upstream discharge and either downstream water level or known energy gradient.

(2) HEC-RAS Model for Kelani River Bridge

HEC-RAS model was set up for proposed Kelani Bridge. Then computation of scour at the bridge was carried out within HEC-RAS based upon the method outlined in Hydraulic Engineering Circular No. 18 (FHWA, 2001).

Contraction scour occurs when the flow area of a stream is reduced by a natural contraction or a bridge constricting the flow. There are two forms of contraction scour that can occur depending on how much bed material is already being transported upstream of the bridge contraction reach. The two types of contraction scour are called live-bed contraction scour and clear-water contraction scour. Live-bed contraction scour occurs when bed material is already being transported into the contracted bridge section from upstream of the approach section (before the contraction reach). Clear-water contraction scour occurs when the bed material sediment transport in the uncontracted approach section is negligible or less than the carrying capacity.

If the flow upstream is transporting bed material (i.e., live-bed contraction scour), the model calculates the critical velocity for beginning of motion Vc (for the D50 size of the bed material) and compares it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge at the approach section. If the critical velocity of the bed material is greater than the mean velocity at the approach section ($V_c > V$), then clear-water contraction is assumed. If the critical velocity of the bed material is less than the mean velocity at the approach section ($V_c < V$), then live-bed contraction scour is assumed in the HEC-RAS model.

(3) Model Set-up

The HEC-RAS geometry file was set up using main four cross sections at 150m upstream, 30m upstream, 30m downstream, 150m downstream and 250m downstream of proposed bridge section. Standard Manning's 'n' coefficient was used for main channel (n=0.03). According to the results of sieve analysis of samples taken from main channel, D50 and D90 are 0.1mm and 0.5mm. Contraction and expansion coefficients do not apply in the natural situation and default values of 0.1 and 0.3 respectively were adopted.

The boundary conditions needed to run the model is upstream discharge and either downstream water level or known energy gradient. Model simulation was carried out for the design discharge of 4,400 m3/s; 100 year return period flood which was estimated by the frequency analysis of Extreme Value Type I – Gumble Distribution for historical annual maximum discharge data recorded at Glencourse Gauging station.

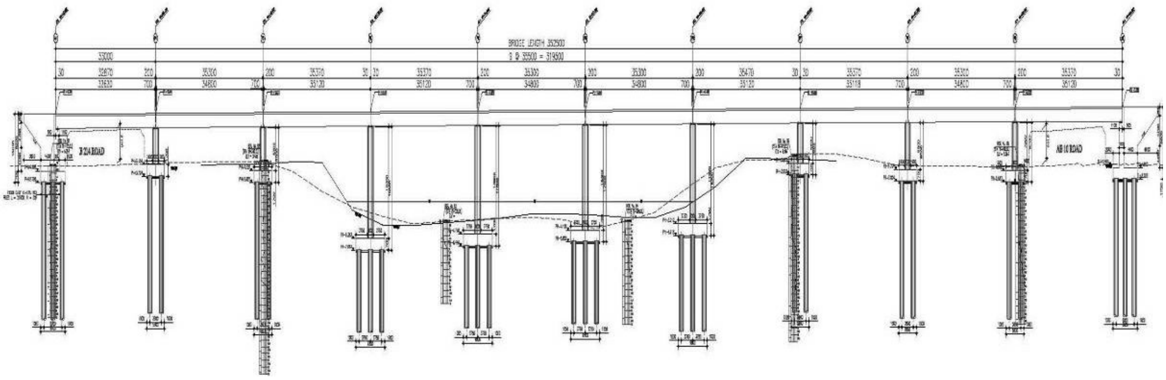
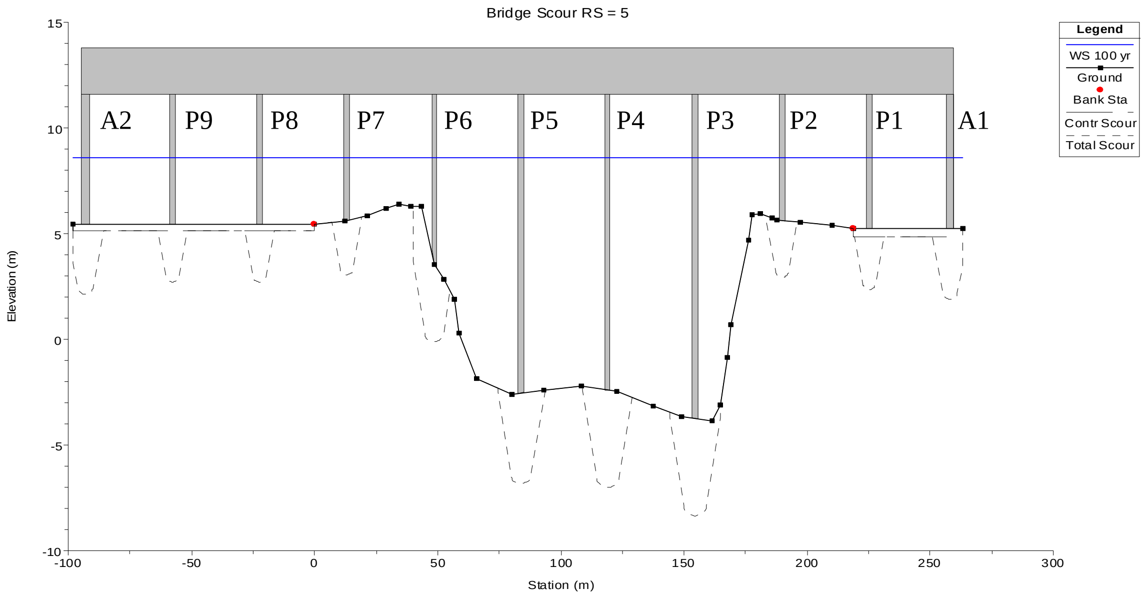


Fig. 6.6.4 Proposed Kelani River Bridge

(4) Scour Depth

Scour depth at each pier calculated in HEC-RAS model is given in Table 6.6.3. All the details of scour calculation and results are attached in Appendix 6.2.



Note: In HEC-RAS modeling, river cross sections are given, looking from upstream to downstream. Accordingly, left and right banks are shown in the figure. Piers are numbered as of structural design drawings.

Fig. 6.6.5 Scour at Bridge Piers

Table 6.6.3 Scour Depth at Proposed Kelani Bridge

No	Contraction Scour	Local Scour	Combined Scour Depth
A1	0.57	2.93	3.50
P1	0.57	2.47	3.04
P2	0.46	2.68	3.14
P3	0.46	4.60	5.07
P4	0.46	4.60	5.06
P5	0.46	4.27	4.73
P6	0.46	3.83	4.29
P7	0.46	2.58	3.04
P8	0.48	2.43	2.91
P9	0.48	2.43	2.91
A2	0.48	3.00	3.48

Unit: m

6.6.3. Hydrological Modeling of Rakgahawatta Ela for Design of Bridge Span

(1) Introduction

The OCH trace between station 14+000 to 16+000 passes through lowland area crossing the marshes and paddy fields and runs parallel to the right bank tributary of Kelani River (Rakgahawatta Ela) which confluences with the river about 250m downstream of OCH crossing point of Kelani river. The ground level along the trace in this section is quite low and varies between 1.0-3.0m MSL except for intermittent highlands. Therefore, when water level in the Kelani River is high, entire surrounding area where ground level is below the Kelani water level, is inundated. It is further aggravated with the local precipitation in the area as water does not flow into the Kelani River. However, this situation can not be easily overcome without implementing a flood control plan for Kelani River. The flood level for a 50 year return period is dependent on Kelani River flood level and is about 8.2m.m.s.l.

As OCH crosses Rakgahawatta Ela at sta. 15+865, necessary drainage provisions are to be designed. The catchement area of Rakgahawatta Ela is around 60km^2 and consists of several branches of small waterways. The HEC-HMS model is applied to analyze runoff of the catchement and clear span of the bridge was determined through HEC-RAS model simulations. The same model was updated further to design the minor flood protection structure at Kelani- Rakgahawatta Ela confluence as given in Chapter 6.6.4.

(2) Hydrological Modelling

The drainage area of Kelani tributary was divided into several sub basins as shown below and it was modeled by hydrological model (HEC-HMS model). The transform method used to compute direct runoff from excess precipitation is the SCS unit hydrograph technique.

1) Topographical Data

The topographic maps produced by Survey Department at 1:10,000 scale covering sub catchments of the study area including the Kelani tributary, Mudun Ela were collected. In addition, survey plan in 1:2000 produced by Survey Department conducting the areal survey for the Outer Circular Highway and the ground survey data conducted by Survey Department and local consultants were also utilized. These maps were used to demarcate the sub catchment boundaries of waterways crossing the OCH trace. Further, for hydrological modelling of Rakgahawatta Ela at OCH crossing point (Sta.15+865), catchment area was divided into several sub basins following the stream network.

2) River Cross sections

Rakgahawatta Ela were surveyed with thirty eight (38) cross sections from Biyagama area to Kelani river confluence at close intervals around 50-100m. It included the existing minor flood protection structure of 6 cell openings each 1.8 m x 1.3 m and existing bridge at Kelaniya-Mudungoda Road (B214).

3) Design Rainfall Events

Both short duration and long duration rainfall events for various return periods are considered. For flash floods, 3 hr storm duration was adopted as the time of concentration in the sub catchment is about 3 hours. However for long durations flood storm, standard 24 hr storm duration was taken into account.

The hyetographs for 3 hour and 24 hour storms for different return periods were derived using Intensity Duration – Frequency curves (IDF Curves) at Colombo and design storm hyetographs used in the "Study on Storm Water Drainage Plan for Colombo Metropolitan Region 2003" by JICA Study are given in Fig. 6.6.8.

4) Runoff Discharge

The estimated peak runoff discharge by HEC-HMS model for 50 year return period is given below.

Table 6.6.4 Peak Discharge at Each Sub Basin

Sub Basin	Catchment Area (km^2)	Flash Storm (3 hr) m^3/s	Long Duration Storm (24 hr) m^3/s
1	22.5	83.7	95.5
2	24.2	80.0	96.6
3	4.2	45.9	37.1
4	7.6	39.3	39.1
5	2.0	32.9	23.1
6	1.2	16.0	12.3

According to HEC-HMS model results, peak discharge of the Rakgahawatta Ela at the OCH crossing point, station 15+900, is 191.7 m3/s for 3hr flash storm and 232.4 m3/s for 24hr long duration storm, 50 year return period.

(3) Hydraulic Modelling

The Rakgahawatta Ela was modeled by HEC-RAS model developed by Hydrological Engineering Center. The model is used to assess the river flow and high water level along the reach together with backwater levels caused by constrictions in the river/floodplains. The input into the model is geometric sketch of the river, cross sectional data, details of river reach and flow data.

Hydrological and hydraulic analysis through model studies has been carried out by local consultants and Sri Lanka Land Reclamation and Development Authority (SLLRDC) for Supplementary Environmental Impact Assessment in 2005-2006 and the recommended clear span was 40m.

JICA study team also analyzed the necessary clear span together with improvement of flood protection structure at Kelani River – Rakgahawatta Ela Confluence through model studies. Model simulations conducted for several case studies are presented in Appendix 6-1.

According to the simulation results, 40m clear span is recommended for bridge opening in case of earthen embankment structure for OCH. The reach where Rakgahawatta Ela crosses the OCH lies within the open area of newly designed elevated road structure; 630m long bridge from station 15+895 to 16+525. Therefore, free flow of water is not obstructed by OCH. While entire area is open for existing water flow, a clear span of 35m is provided for river channel by widening the present channel width of 10-15m. In addition, it is designed to straighten the section of 100m both upstream and downstream meandering section of Rakgahawatta Ela. In order to maintain the necessary high flood level with freeboard without being obstructed by the Ramp, it is therefore decided to construct the bridge at sta. 15+915 (center) shifting the waterway by few meters from the existing location at station 15+950. Also, it straightens the river course, eliminating the high degree of meandering pattern. .

Table 6.6.5 Clear Span of Bridge at Rakgahawatta Ela

Station	Direction	Design Discharge	Clear Span of Bridge
15+915	L - R	232.4	35m

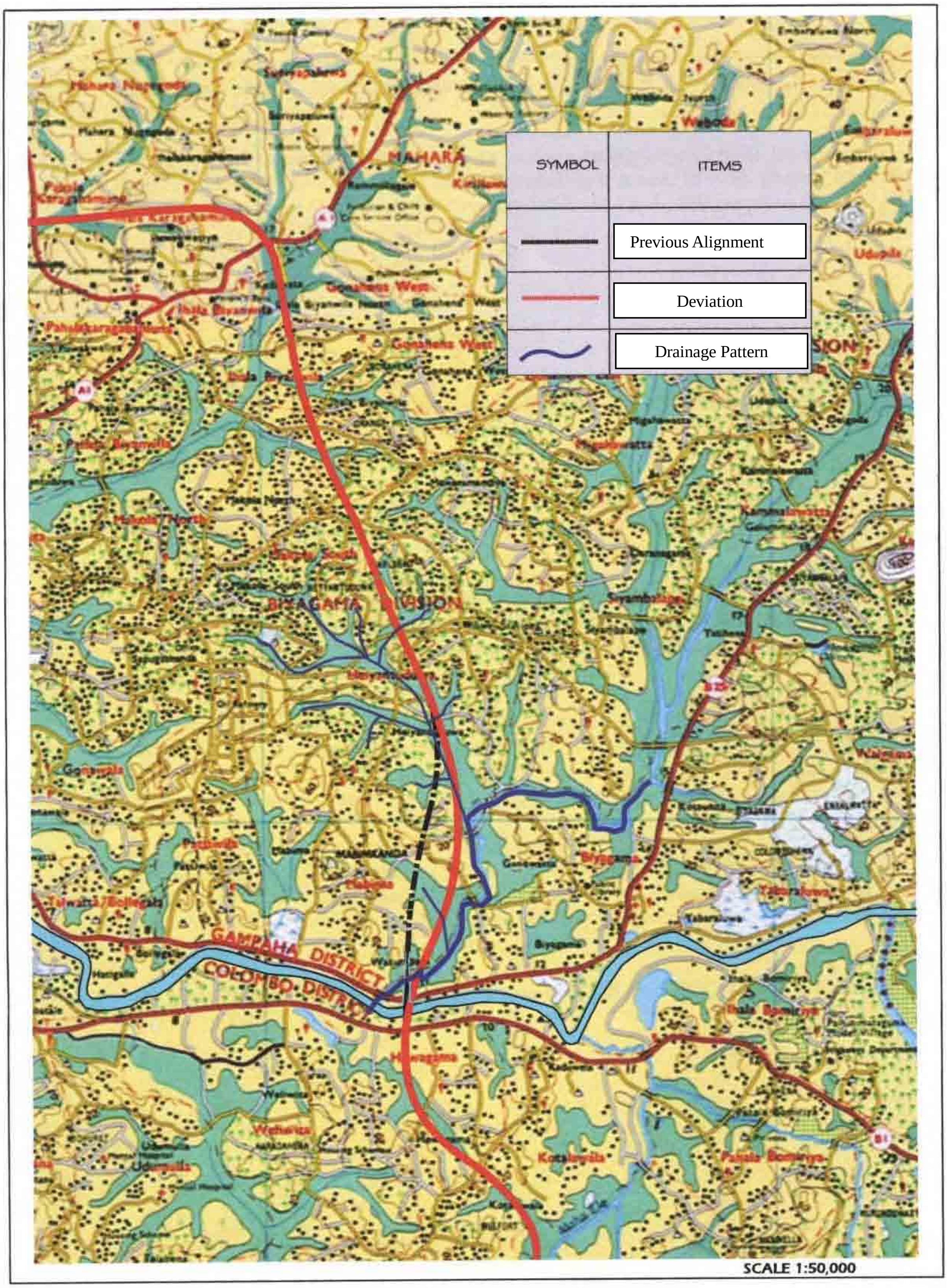
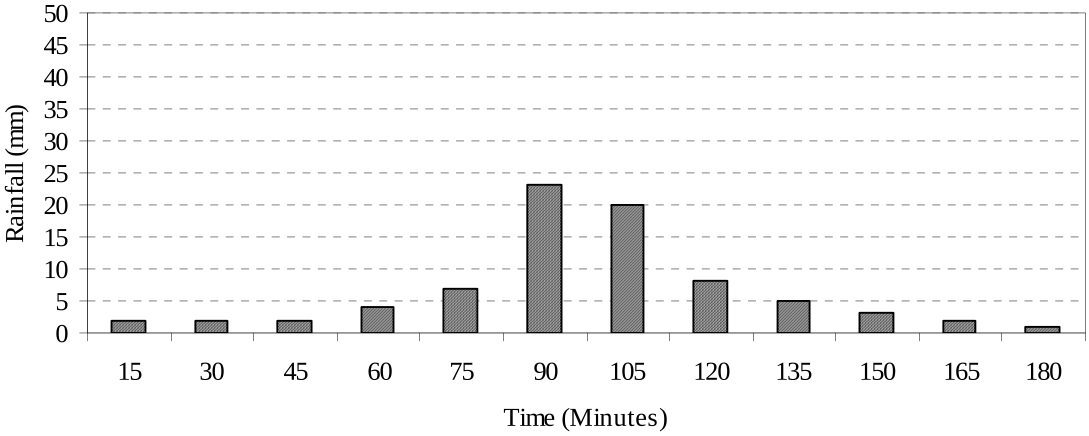
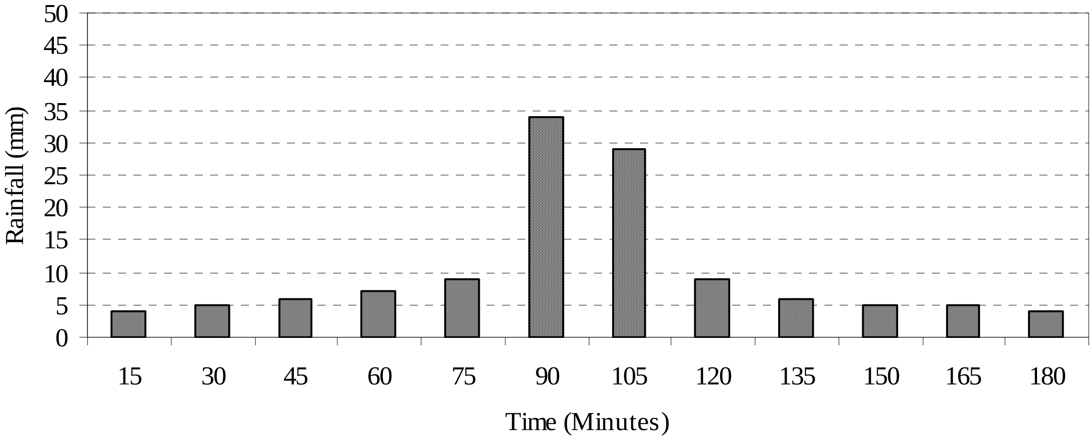


Fig. 6.6.6 Existing Drainage System

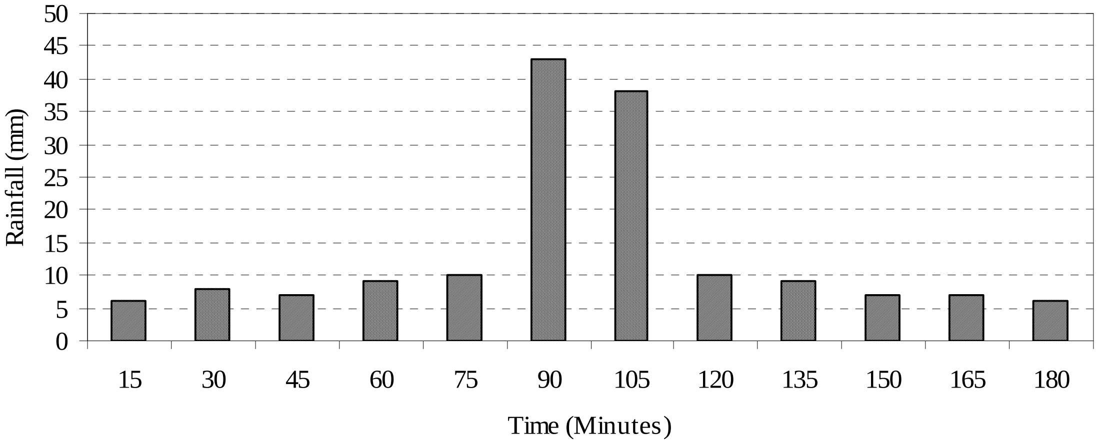
Fig 6.6.7 Sub Catchments in Rakgahawatta Ela Basin



Hyetograph of 3hr rainfall for 2 year return period in Colombo

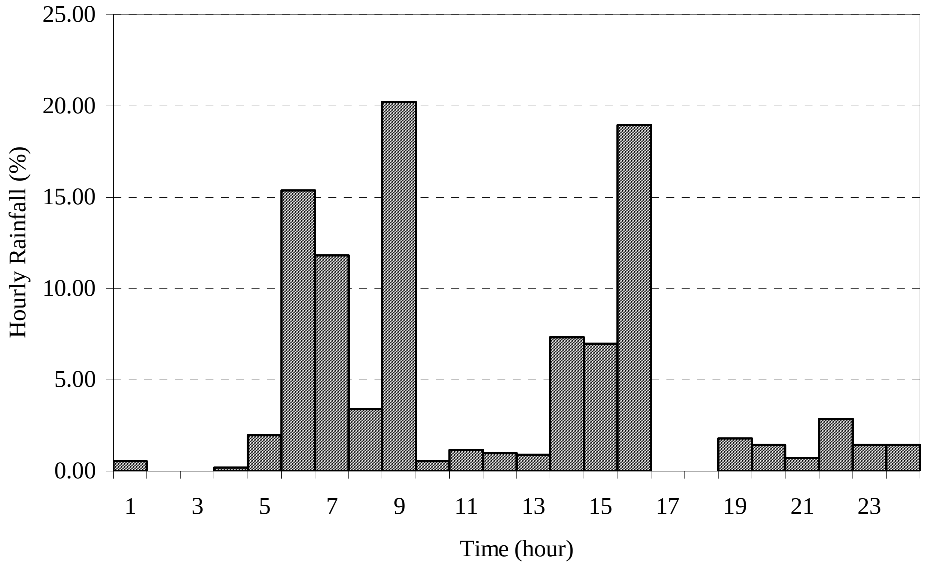


Hyetograph of 3hr rainfall for 10 year return period in Colombo



Hyetograph of 3hr rainfall for 50 year return period in Colombo

Fig. 6.6.8 (1) Design Hyetographs for 3 Hour Storm in Colombo



Design hyetograph for 24 hour rainfall in Colombo

Duration (hr)	Rainfall (%)	Duration (hr)	Rainfall (%)
1	0.54	13	0.89
2	0.00	14	7.33
3	0.00	15	6.98
4	0.18	16	18.96
5	1.97	17	0.00
6	15.38	18	0.00
7	11.81	19	1.79
8	3.40	20	1.43
9	20.21	21	0.72
10	0.54	22	2.86
11	1.16	23	1.43
12	0.98	24	1.43

Return Period (years)	Greater Colombo Rainfall (mm)
2	117
5	176
10	214
25	270
50	320

Source: Storm Water Drainage Plan for the Colombo Metropolitan Region; JICA Study (March 2003)

Fig. 6.6.8 (2) Design Hyetograph for 24 Hour Storm in Colombo

Fig. 6.6.9 Location Map of Rakgahawatta Ela Cross Sections

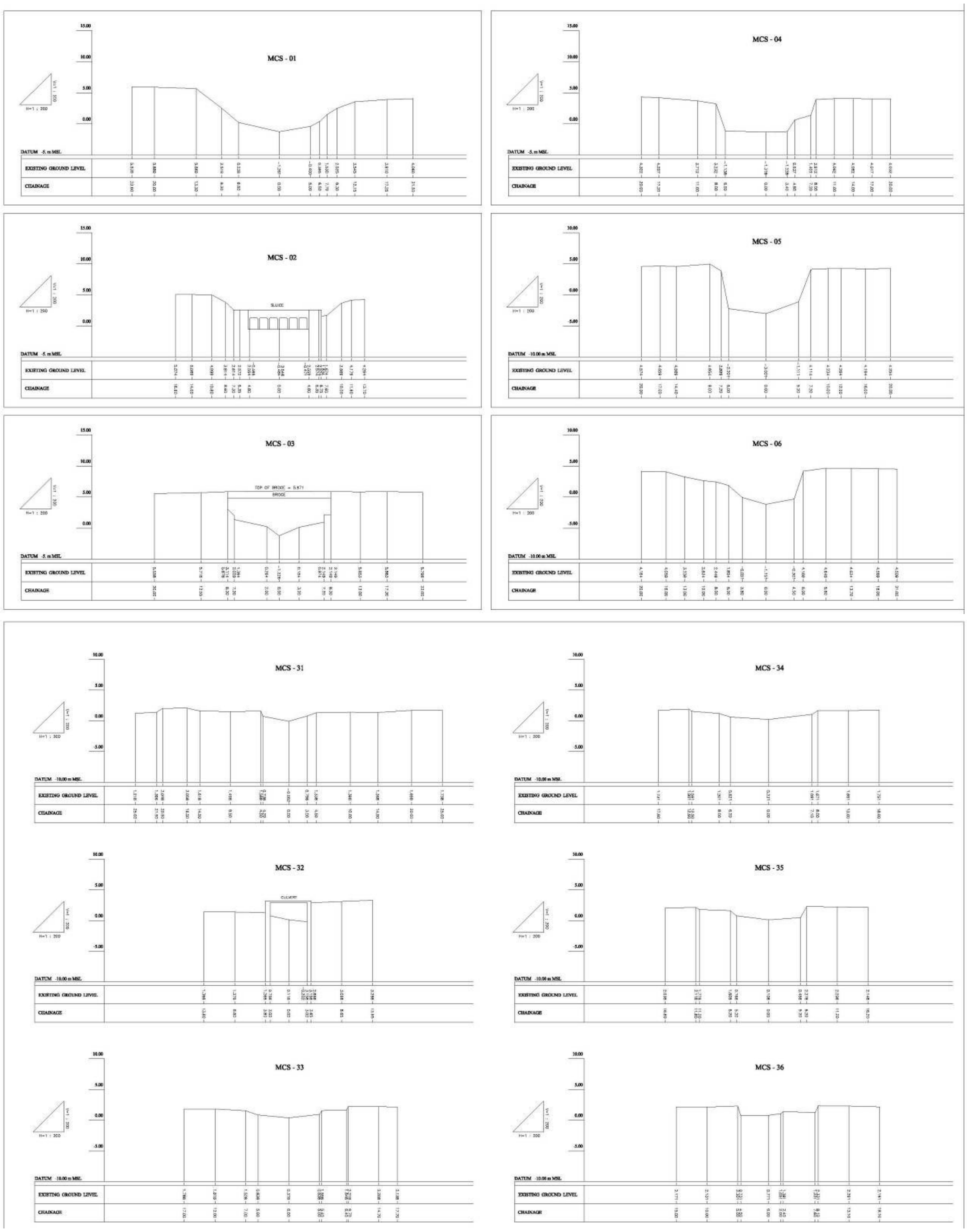


Fig. 6.6.10 Several Cross Sections of Rakgahawatta Ela

6.6.4. Minor Flood Protection Structure at Rakgahawatta Ela

The existing Rakgahawatta Ela minor flood protection structure located at Kelani River – Rakgahawatta Ela confluence is of inadequate capacity for a smooth flow even under present conditions. The downstream area of Rakgahawatta Ela is inundated every year with the prevailing low ground elevation around 3m msl or less, and due to influence of Kelani River water level and local rainfall in the catchment.

As a part of Outer Circular Highway Project, it was earlier decided to carry out a detail design of minor flood protection structure, to minimize any adverse impacts caused due to embankment construction of OCH obstructing the natural passage of water flow. However, structure type in this reach has been changed to elevated structure addressing various issues in consultation with Road Development Authority. The elevated structure covers the entire reach of flood plain at the crossing of Rakgahawatta Ela, with a bridge of 630m long from station 15+895 to 16+525. Therefore, construction of OCH will not basically cause any obstruction to free flow of water within this reach.

During the discussions held between Road Development Authority and Irrigation Department, it is agreed that construction of minor flood protection structure will not be included under Outer Circular Highway Project. However, design of the flood protection structure has been carried out by the consultants and it is presented in Appendix.

6.6.5. Drainage Design in Minor Sub Catchments

(1) Introduction

Most of sub catchments in OCH (Northern) trace are quite small in extent except Rakgahawatta Ela as shown in Fig. 6.6.11. Rational method widely used in Sri Lanka has been applied for drainage design in small catchments.

(2) Rational Method

Rational Method is given by the formula;

$$Q_d = \frac{1}{3.6 \times 10^6} C \cdot I \cdot A$$

where: Q - Discharge in m3/s.
C - Runoff coefficient
I - Average intensity of storm (mm/hr)
A - Catchment area (m^2)

1) Runoff Coefficient

The estimation of runoff coefficient ‘C’ is a major source of uncertainty in the application of Rational method. The coefficients are given by tables of suggested values, graphs or by means of the sum of the ‘scores’ given for each of several factors such as slope, permeability of soil and vegetation. In this design, the run-off coefficient is calculated using Table 6.6.6. The calculated run-off coefficient has been increased by 10%

considering future development. In addition, recommended value of 'C' given by Sri Lanka Land Reclamation & Development Corporation (SLLRDC) is also taken into consideration in determining the runoff coefficients.

Table 6.6.6 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

2) Rainfall Intensity

The rainfall intensity 'I' is the average rainfall rate for a particular drainage basin or sub basin. The intensity is selected on the basis of the design rainfall duration and return period. The design duration is equal to the time of concentration for the drainage area under consideration. The rainfall Intensity-Duration-Frequency for Colombo is given in Table 6.4.1.

Runoff is assumed to reach a peak at the time of concentration (Tc) when the entire watershed is contributing to flow at the outlet. The time of concentration is the time for a drop of water to flow from the remotest point in the watershed to the point of interest.

The time of concentration Tc is calculated as given below

$$T_c = \frac{L}{V \times 60} + 15\,\text{min}$$

where; L is the length of the longest water course in meters
V is the average velocity in meters/sec

As the inlet time which is the time taken for a drop of water to travel before it reaches a defined water course, 15 minutes is added to the flow time as given in the above equation.

The average flow velocity in the natural watercourse is estimated based on the gradient of the stream as given in Table 6.6.7 recommended by Design of Irrigation Headworks.

Table 6.6.7 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

3) Design of Culverts

For the minor catchments, Rational formula was used to estimate the design discharge (Table 6.6.8). The software “Hydro Culvert” which is widely used in culvert design taking into consideration downstream condition, head losses at inlet and outlet, was applied to determine the design dimensions. The dimensions of the culverts along OCH trace were given in Table 6.6.9.

4) Design of Ditch / Canals

The proposed road that passes through the marsh and paddy area, divides the flood plain into two parts in left and right sides. Under the existing condition, shallow flow can be seen in main water course and both sides get simultaneously inundated with heavy rainfalls. According to the proposed road alignment, main water course should be realigned at some locations. Then, at some points it is required to change flow direction from one side of road embankment to other side to fall in line with the existing drainage pattern. The drainage canals were designed for 10 year return period of rainfall event while allowing sufficient overflow condition in order to keep the existing condition with distributed flow that is required for agricultural needs and paddy cultivation. This design was carried out in consultation with Sri Lanka Land Reclamation & Development Corporation. The drainage canal dimensions are given in the Table 6.6.10.

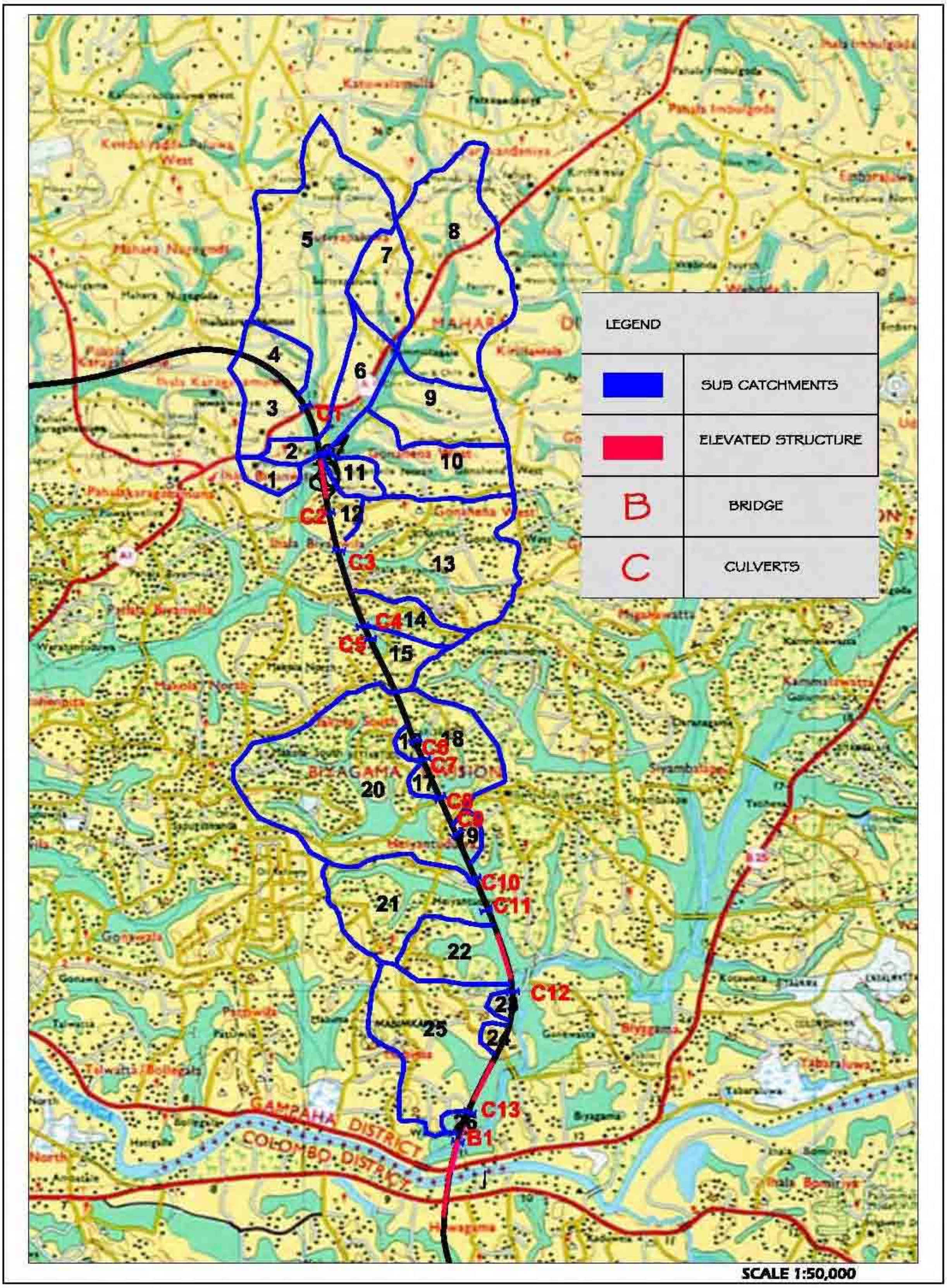


Fig. 6.6.11 Drainage Provisions at Minor Sub Catchments

Table 6.6.8 Discharge at Minor Sub Catchments

Sub Catchment No.	Area (km^2)	Runoff Coefficient (C)	Time of Concentration (Min)	Intensity (mm/hr)		Discharge (m^3/s)	
				10 yr	50yr	10 yr	50yr
1	0.080	0.4	26.36	128.14	164.43	1.14	1.46
2	0.100	0.4	24.16	129.50	165.89	1.44	1.84
3	0.380	0.4	25.99	128.41	164.67	5.42	6.95
4	0.179	0.4	16.83	133.80	170.70	2.66	3.40
5	2.195	0.4	80.57	68.40	85.60	16.68	20.88
6	0.440	0.4	20.46	131.72	168.36	6.44	8.23
7	0.448	0.4	31.48	123.75	159.00	6.16	7.91
8	1.697	0.4	75.11	71.93	89.92	13.56	16.95
9	0.551	0.4	45.05	103.43	131.40	6.33	8.04
10	1.078	0.4	55.98	87.03	109.17	10.42	13.08
11	0.216	0.4	25.99	128.41	164.67	3.08	3.94
12	0.060	0.45	15.00	135.00	172.00	1.01	1.29
13	1.817	0.4	64.18	78.49	97.93	15.85	19.77
14	0.289	0.4	27.82	127.31	163.45	4.09	5.25
15	0.200	0.4	25.99	128.41	164.67	2.85	3.66
16	0.060	0.45	15.00	135.00	172.00	1.01	1.29
17	0.100	0.45	15.00	135.00	172.00	1.69	2.15
18	0.621	0.4	23.20	130.08	166.53	8.98	11.49
19	0.070	0.45	15.00	135.00	172.00	1.18	1.51
20	2.620	0.4	77.84	70.30	93.45	20.47	27.20
21	1.035	0.4	69.64	75.22	89.84	8.65	10.33
22	0.545	0.4	23.20	130.08	166.53	7.88	10.08
23	0.100	0.45	15.00	135.00	172.00	1.69	2.15
24	0.060	0.45	15.00	135.00	172.00	1.01	1.29
25	1.301	0.4	40.64	110.04	140.37	15.91	20.29
26	0.050	0.45	15.00	135.00	172.00	0.84	1.08

Table 6.6.9 Storm Water Drainage Crossing Along Outer Circular Highway

Culvert /Bridge No.	Culvert I.D. (Station)	Main Catchment I.D.	Sub Catchments included	Direction	Catchment Area (km2)	Return Period (Years)	Design Discharge (m3/sec)	Culvert/ Bridge slope Design	Culvert/Bridge Dimension [H(m)xW(m)]	Minimum Freeboard (m)	Remarks
Highway crossings											
Culverts											
C1	8+150	Stream	4+5	L to R	2.374	50	24.27	0.001	2 (3.0 x 3.25)	0.500	
C2	9+340	Sheet flow	12	L to R	0.060	50	1.29	0.001	2.0 x 3.25	0.500	
C3	9+760	Stream	13	L to R	1.817	50	19.77	0.001	2 (3.0 x 3.25)	0.500	Skew angle 30^{0}
C4	10+530	Stream	14	L to R	0.289	50	5.25	0.001	2.0 x 3.25	0.500	
C5	10+680	Stream	15	L to R	0.200	50	3.66	0.001	2.0x 3.25	0.500	Skew angle 45^{0}
C6	11+740	Sheet flow	16	R to L	0.060	50	1.29	0.001	2.0 x 3.25	0.500	
C7	11+935	Sheet flow	17	R to L	0.100	50	2.15	0.001	2.0 x 3.25	0.500	
C8	12+350	Stream	18	L to R	0.621	50	11.49	0.001	2 (2.0 x 3.25)	0.500	
C9	12+735	Sheet flow	19	L to R	0.050	50	1.51	0.001	2.0 x 3.25	0.500	
C10	13+210	Secondary canal	16+17+18+19+20	R to L	3.241	50	43.64	0.001	2 (3.0 x 3.25)	0.500	
C11	13+550	Stream	21	R to L	1.035	50	10.33	0.001	3.0 x 3.25	0.500	
C12	14+340	Sheet flow	23	R to L	0.150	50	2.15	0.001	2.0 x 3.25	0.500	
C13	15+600	Sheet flow	26	R to L	0.150	50	1.08	0.001	2.0 x 3.25	0.500	
Bridges											
B 1	15+915	Mudun Ela		L to R	61.600	50	231.40	0.001	35m clear span		
A1 Bypass crossings											
A1-1	0+580	For Canal along OCH (R)		L to R	2.854	50	33.070	0.001	3 (3.0 x 3.25)	0.500	
A1-2	0+670	For Canal along OCH (L)		L to R	0.440	50	8.230	0.001	3.0 x 3.25	0.500	
A1-3	0+990			L to R					3.0 x 3.25	0.500	
A1-4	1+300			L to R	0.448	50	7.910	0.001	3.0 x 3.25	0.500	

Culvert /Bridge No.	Culvert I.D. (Station)	Main Catchment I.D.	Sub Catchments included	Direction	Catchment Area (km2)	Return Period (Years)	Design Discharge (m3/sec)	Culvert/ Bridge slope Design	Culvert/Bridge Dimension [H(m)xW(m)]	Minimum Freeboard (m)	Remarks
Ramp 4 crossings											
R4-1	0+130	For Canal along the A1 Bypass (R)		L to R	3.774	50	45.990	0.001	3 (3.0 x 3.25)	0.500	
Minor Roads											
M1	Sharamadana Road					10	72.19	0.001	5(2.0x3.25)		Across the canal extending from A1bypass (R)
M3 & M4	Sharamadana Road					10	1.44	0.001	2.0x3.25 each		For the crossings of canal along the A1 Bypass (R)
M5	Bandaranayake Road (crossing at A1 Bypass)					10	6.16	0.001	2.0x3.25		For canal along the A1 Bypass (L)
M6	Bandaranayake Road (crossing at A1 Bypass)					10	19.72	0.001	3(2.0x3.25)		For canal along the A1 Bypass (R)
M7	Parallel to A1 Bypass (1+650)(L) for connecting new road					10	6.16	0.001	2.0x3.25		Drainage for residents
M8	Ranmuthugala Road Parallel to A1 Bypass (1+880)(R)					10	6.16	1.000	2.0x3.25		
M9	Siriketha Road (Left side of OCH at ST. 13+320 along side canal)					10	33.33	0.001	2(2.0x3.25)		Secondary canal
M10	Siriketha Road (Right side of OCH at ST. 13+320 along side canal)					10	8.98	0.001	2.0x3.25		

Table 6.6.10 Ditch/Canal Provisions along Outer Circular Highway (Northern Section - 1)

No	Ditch / Canal Station	Side	Sub Catch-ment No.	Contributing Canals	Catch-ment Area (km2)	Return Period (Year)	Discharge (m3)	Flood plain width* (m)	Channel size (m) Width Top	Channel size (m) Width Bot	Channel size (m) Ht.	Channel size (m) Slope Critical	Channel size (m) Slope Design	Water Depth (m) Main Canal	Water Depth (m) Flood plain	Velocity (m/s) Main Canal	Velocity (m/s) Flood plain	Remarks
(1) Reinforced Concrete Canals (Manning's Roughness applied n = 0.015)																		
- **Along Highway**																		
CH-2	8+145 to 8+640	L	4	-	0.18	10	2.66	NA	2	2	1.5	0.0048	0.0060	0.52		2.54		R/F Conc.
- **Along Access and Ramps (A1 Interchange)**																		
A1-CH1	A1 Bypass (0+125-0+590)	L	2	-	0.25	10	3.60	NA	2	2	1.5	0.0050	0.0050	0.43		4.22		R/F Conc.
A1-CH2	A1 Bypass (0+590-1+620)	L	6	-	0.44	10	6.44	NA	2.5	2.5	1.5	0.0047	0.0005-0.0040	1.03		2.50		R/F Conc.
A1-CH3	A1 Bypass (0+510) and along relocated Shramadana Road Right Side till existing canal	R	1	-	0.27	10	4.03	NA	2.5	2.5	1.5	0.0044	0.0043	0.42		3.80		R/F Conc.
A1-CH7	RAMP 4 (0+000 to 0+370)	R	11	-	0.30	10	4.20	NA	2	2	1.5	0.0051	0.0005-0.0050	0.55		3.81		R/F Conc.

No	Ditch / Canal Station	Side	Sub Catchment No.	Contributing Canals	Catchment Area (km2)	Return Period (Year)	Discharge (m3)	Flood plain width* (m)	Channel size (m) Width Top	Width Bot	Ht.	Slope Critical	Slope Design	Water Depth (m) Main Canal	Water Depth (m) Flood plain	Velocity (m/s) Main Canal	Velocity (m/s) Flood plain	Remarks
(2) Canals with side walls of rubble masonry (Manning's Roughness applied for channel n = 0.020; for floodplain n=0.025																		
Along Highway																		
CH-1	8+145 to 8+640	R	3+4+5	-	2.93	10	24.76	NA	10.5	7.5	1.5	0.0046	0.0014	1.41		1.98		Design velocity of main channel is confined below 2.0m/s as of earth channel bed.
CH-3	11+740 to 12+350	L	16+17+ 18	-	0.78	10	11.18	NA	6	3	1.5	0.0052	0.0015	1.41		1.79		
CH-4	13+000 to 13+755	R	21	-	1.04	10	8.65	20	5	2	1.5	0.0055	0.0006	1.69	0.19	1.20	0.32	
CH-5	13+210 to 14+300 and connect to Kelani tributory	L	16+17+ 18+19+ 20		3.47	10	33.33	40	8	5	1.5	0.0045	0.0020	1.87	0.37	1.86	0.62	
Along Access,m and Ramps (A1 Interchange)																		
A1-CH4	A1 Bypass (0+670 to 0+990)	R	6+7+8 +9+ 10+11	A1-CH2, A1-CH5, A1-CH7	4.43	10	45.99	7	15	12	1.5	0.0042	0.0010	1.68	0.18	1.98	0.40	Design velocity of main channel is confined below 2.0m/s as of earth channel bed.
A1-CH5	A1 Bypass (0+990 to 1+880)	R	6+7+8 +9	A1-CH2	3.14	10	32.49	20	9	6	1.5	0.0044	0.0010-. 0.0044	1.85	0.35	1.94	0.63	
A1-CH6	OCH 8+740 to 0+270 of Ramp	R	2+3+4 +5+6+ 7+8+9 +10+11	A1-CH8 A1-CH10	7.26	10	72.19	10	20	16	2	0.0040	0.0019- 0.0040	2.01	0.01	1.99	0.06	
A1-CH8	A1 Bypass 0+670 to A1-CH6	OCH / L-R	4+6+7 +8+9+ 10+11	CH2, A1-CH4	4.61	10	48.65	20	16	13	1.5	0.0042	0.0010	1.65	0.15	1.97	0.36	

A1-CH9	OCH 8+680 to 8+740 (Extension of CH-1)	R	3+4+5	CH1, A1-CH9	2.93	10	26.2	N A	12	9	1.5	0.0045	0.0040	1.28		1.99		

Note:

(1) * Floodplain width considered for analysis though actual flood plain width is wider for several cases.

(2) NA: Not applicable as overflow condition is not applied.

(3) Manning's roughness coefficient applied; n=0.015 (concrete canals), 0.020 (excavated dredged canals) and n=0.025 (floodplain) (Reference: HEC Manual)

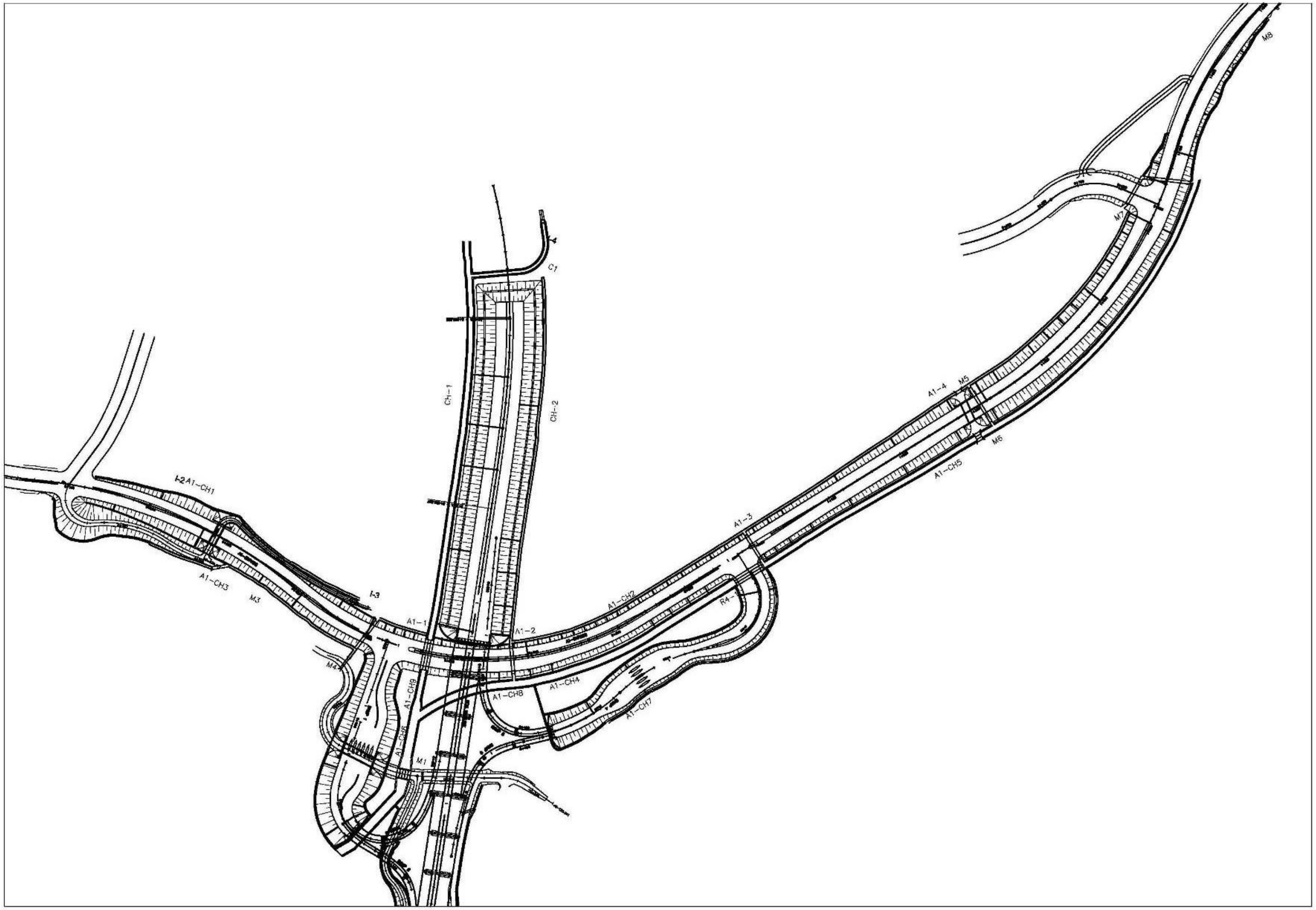


Fig. 6.6.12 Drainage Plan at A1- Interchange

6.6.6. Consultants Remarks

(1) Natural drainage paths shall not be disturbed except the stretches where highway and bypass roads are built up. Further, it is required to connect the natural drain paths to proposed built up drain or natural stream network in order to ensure a smooth flow pattern.

(2) During the construction, it is necessary to provide alternative canal routes (ditches) for irrigation canals obstructed so that farmers can continue their cultivation.

(3) Consultants propose RDA to record the annual high water level at low line areas especially at Kelani River flood basin and A1 interchange interviewing the public (in addition to data given in this report) and mark them permanently and keep the evidences to assess any adverse impacts on inundation water level during the construction stage. In addition, it will be a tool to check the validity if any complain arises from public regarding the local inundation due to construction.

(4) The hydrological and hydraulic analysis of Kelani River bridge opening (clear span) is presented to the Department of Irrigation and accordingly, it has been approved.

(5) The designing of Minor Flood Protection Scheme at Rakgahawatta has been done by the consultants. However, it will not be implemented under this project, as construction of elevated highway with via-duct option instead of earth fill embankment for low line area in Kelani river basin will not obstruct the existing hydrological environment. It has been accepted by the Department of Irrigation.

(6) Sri Lanka Land Reclamation Authority has carried out two hydrological review studies for Biyagama basin and A1 interchange. Its recommendations given on culvert sizes and canal/ditch sizes are also taken into consideration during the design.

(7) Sri Lanka Land Reclamation Authority has recommended to improve the downstream of drain A1-CH3 at A1 interchange, till it meets the existing Kalu Oya tributary, at least equivalent to dimension of A1-CH3, to minimize the impacts such as water logging and local inundation. In addition, it recommended to repair the existing damaged culverts at this downstream reach.

(8) Necessary drainage provisions for the affected irrigation canals were provided in consultation with Department of Agrarian Services.

6.6.7 Design of Channel Drop Structures

Channel slopes are very small, between Min. 0.0005 (A1 CH-3) and Max. 0.0050 (A1-CH1 and A1-CH7. CH-2 is out of limit of contract.). Therefore, total 40 channel drop structures were designed for Northern section-1.

Table 6.6.11 Channel Drop Structures

No.	**Channel**	**Nos. of Drop (H)**		**Max. drop**
		0.2m≤H ≤0.5 m	**0.5 m<H ≤1.0 m**	
1	A1-CH1	4	13	1.0 m
2	A1-CH3	5		0.5 m
3	A1-CH5	10	3	0.9 m
4	A1-CH6	4		0.5 m
5	A1-CH7		1	0.9 m
	Total	**23**	**17**	**Total 40 Nos.**

Channel: A1-CH5 (D_1=1.0 m)				
1	**Design conditions**			
	(1)	Channel		
		Width (B), average, 6+1=	7	m
		Slope, (I)	0.0044	m/m
		Discharge (Q)	32.49	cu.m./s
		Manning's n	0.02	
	(2)	Geotechnical conditions		
		Soft Clay		
		Unit weight	1.4	t/cu.m
		Unit weight of water	1.0	t/cu.m
		Angle of Internal friction	10	degree
	(3)	Design Load		
	(a)	Dead weight		
		Reinforced concrete, γ_c	2,400	kgf/cu.m
	(b)	Design seismic coefficent	0	
	(4)	Drop height		
		River bed level difference, D_1	**1.0**	**m**
		Projection of drop concrete, D_2	0.3	m
		Depth of drop pool, D_3	0.3	m
		Total drop, D	1.60	m
2	**Design of Main body and apron**			
	(1)	Apron length		
	(a)	Length of overflow drop		
	(i)	Discharge conditions		
		Discharge, Q=	32.49	cu.m./s
		Discharge per unit width: q=Q/B=	4.64	sq.m/s
		Critical depth: $h_c=q/v_c=q/(ghc)$^1/2		
		Then, $h_c=q^{2/3}xg^{1/3}$=	1.30	m
		Critical velocity: $v_c=(gh_c)$^1/2=	3.57	m/s
		Uniform flow depth of down stream; $h_2=q/v_2=q/1/n\ xh_2^{2/3}I^{1/2}$		
		Then, $h_2=(qxn/\ I^{1/2})$ ^3/5=	1.22	m
		Check of Overflow condition		
		$h_c+D_1+D_2$=	2.60	> h_2=1.22 m
		Complete overflow is confirmed even if maximum discharge.		
		Therefore, discharge is used for apron calculation.		
	(ii)	Calculation of overflow falling length		
		Using Rand's formula,		
		$L'/D=4.3x(h_c/D)^{0.81}$		
		Then, $L'=4.3x(h_c/D)^{0.81}xD$=	5.82	, say **6.0 m.**
		L' : Apron length, D: Total drop		
	(b)	Check of piping		
		Water levels at concrete drop and apron down stream end are required to check the piping.		
		Water levels at up and down streams to be determined for checking piping.		
	(i)	Dixcharge conditions		
		Discharge is used because of complete uniform flow.		
	(ii)	Depth at concrete drop		
		Control section will be at the concrete drop, critical depth is used.		

(ii)-1 Depth at falling point: h_{1a}

Formula:

$h_{1a}^3-(v_c^2/2g+D+h_c)xh_{1a}^2+q^2/2g=0$

h_{1a}^3-(3.57^2/2/2*9.8+1.6+1.30)xh_{1a}^2+4.64^2/2/2*9.8=0

h_{1a}^3-3.55xh_{1a}^2+1.098=0

When h_{1a}=		0.61	m
A=h_{1a}^3-3.55xh_{1a}^2+1.098=		0.0040	
Therefore, h_{1a}=		0.61	m

(ii)-2 Upstream max. water level difference

H = hc+D-h_{1a} = 1.30+1.6-0.61 = 2.29 m

(iii) Anchor length of seepage control work

Anchor length is calculated using Rein's formula.

$C \le (L/3+\Sigma l_y)/H$

where,

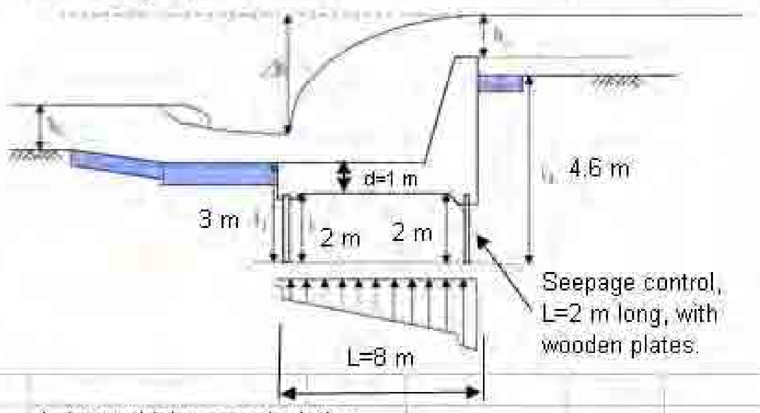
C: Creep ratio (Silt) 7

L: Horizontal seepage length

=Main body+Apron length=2+6= **8 m**

Σl_y: Vertical seepage length

=Main body+Apron thickness



d: Apron thickness calculation

$d=F_s x(u_{pm}-h_2 x W_0)/(\gamma_c-1)$

where,

d: Required thickness of apron (m),

u_{pm}: Maximum uplift force at the apron (t/sq.m),

γ_c: Unit weight of concrete (=2.4 t/cu.m)

F_s: Safety factor (usually 4/3 is used.),

On the other hand,

$u_{px}=\{h_2+Hx(\Sigma l - l_x)/\Sigma l\}xW_0$

where,

u_{px}: Uplift at x point (t/sq.m)

H: Water level difference of up and down streams (=1.0 m)

l_x: Seepage length at x point (m),

W_0: Unit weight of water (1 t/qu.m)

When x=0, $u_{px}=u_{pm}$ and h_2=1.22 m

		Therefore,							
		u_{pm}=(1.22+1.0)x1=		2.22	m				
		Then,							
		d=4/3x(2.22-1.22x1.0)/(2.4-1)=		0.95	m, say **d=1.0 m.**				
		On the other hand,							
		$\Sigma l_y \geq CxH-L/3$=7x1-8/3=		4.3	m				
		Required seepage control work length: l							
		l={Σ ly-(D_1+D_3+2xd)}/2		0.5	m				
		Seepage control work is min. 2 m long.							
		Therefore, Seepage control work at both up and down streams, **minimum 2 m long.**							
3	**Stability of Main Body**								
	(a)	External force							
		River bed level difference, D_1		1	m				
		Projection of drop concrete, D_2		0.3	m				
		Depth of drop pool, D_3		0.3	m				
		Total drop, D		1.6	m				
		Thickness of apron, d:		1.0	m				
		Thickness of vertical wall, W:		2.0	m				
		Apron length		6.0	m				
		Main body+Apron length=L		8.0	m				
		Gradient of vertical wall, l'		0.2					
		Critical depth, h_c		1.30	m				
		Overflow depth at falling point, h_{1a}		0.61	m				
		Seepage control work length							
		Upstream (lc_1)		2.0	m				
		Downstream (lc_2)		0 (2)	(Assumed 0 due to weepholes)				
		Width, B'		1.0	m, (Unit width)				
		Unit weight of concrete		2.4	t/cu.m				
		Unit weight of water		1.0	t/cu.m				
		Angle of internal friction							
		Normal case, K_{a1}		0.297					
		Seismic case, K_{ea}		0					
	(i)	Dead weight and inertia force calculation							
				Vertical force	Arm length (m)		Moment		Inertia force H_i
	Part	Volume (cu.m)		W_i (ton)	x_i	y_i	W_ix_i : Mx	H_iy_i : My	(earthquake)
	1	WxDxB'=	3.20	7.68	1.00	1.80	7.68	0	0
	2	l'xDxB'xDx1/2	0.26	0.61	2.11	1.53	1.30	0	0
	3	LxdxB'	8.00	19.20	4.00	0.50	76.80	0	0
		Total		27.49			85.78	0	0
				x 2.4					
		x=Σ(W_i x x_i)/ΣW=		85.78/27.49=	3.12	m			
		y=0							
	(ii)	Earth pressure calculation							
		Normal time							
		Earth pressure per sq.m	$p_1=K_{a1}x\gamma_{bw}x(D_1+D_3+d)$=0.297x1.0x(1+0.3+1.0)=					0.68	t/sq.m
		Earth pressure	P_1=1/2xp_1x(D1+D3+d)=1/2x0.68x(1+0.3+1)=					0.78	ton
		Horizontal force	$P_{1H}=P_1$xcos (2Φ/3)=0.78xcos(2x10/3°)=					0.77	ton
		Vertical force	$P_{1v}=P_1$xsin (2Φ/3)=0.78xsin(2x10/3°)=					0.09	ton
		Arm length, y	y=(D1+D3+d)/3=2.3/3=					0.77	m

(iii) Water pressure calculation

Normal time

Backside: $pw_{1b}=\gamma w \times h_c=1.0x1.44=$ 1.44 t/sq.m

$pw_{2b}=\gamma w \times (h_c+D+d)=1.0x(1.44+1.6+1)=$ 4.04 t/sq.m

$PW_{1b}=1/2x(pw_{1b}+pw_{2b})x(D+d)=1/2x(1.44+4.04)x(1.6+1.0)=$ 7.12 t/m

$y=(D+d)/3x(2xpw_{1b}+pw_{2b})/(pw_{1b}+pw_{2b})=(1.6+1.0)/3x(2x1.44+4.04)/(1.44+4.04)=$ 1.09 m

Frontside: $pw_{1f}=\gamma w \times h_{1a}=1.0x0.70=$ 0.70 t/sq.m

$pw_{2f}=\gamma w \times (h_{1a}+d)=1.0x(0.7+1.0)=$ 1.70 t/sq.m

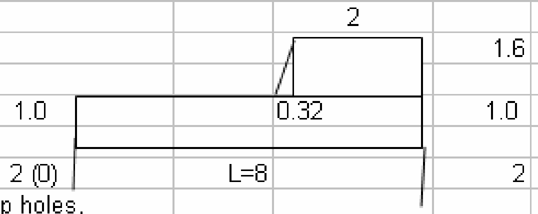
$PW_{2f}=1/2x(pw_{1f}+pw_{2f})xd=1/2x(0.70+1.7)x1.0=$ 1.20 t/m

$y=d/3x(2xpw_{1f}+pw_{2f})/(pw_{1f}+pw_{2f})=1.0/3x(2x0.7+1.7)/(0.7+1.7)=$ 0.43 m

(iv) Uplift force

(a) Normal time

$\Sigma l=1.6+1.0+2+2+8+1.0=$ 15.6 m



With weep holes,
assumed as 0

$U_a=(d+Hx(\Sigma l-l_a)/\Sigma l)xW_0=[1.0+2.34x\{15.6-(1.6+1.0+4+4)\}/15.6]x1.0$ 1.75 tf/sq.m

$U_b=(d+Hx(\Sigma l-l_b)/\Sigma l)xW_0=[1.0+2.34x\{15.6-(1.6+1.0+4+4+8)\}/15.6]x1.0$ 0.55 tf/sq.m

U=(1.75+0.55)x8.0x 1/2= 9.20 tf/sq.m

x=8/3x(2x0.55+1.75)/(0.55+1.75)= 3.30 m

(b) Stability calculation (normal time)

	N (tf)	H (tf)	x (m)	y (m)	N・x(tf・m)	H・y(tf・m)
Dead weight	27.49		3.12		85.78	
Earth pressure (P_1)	0.09	-0.77	0	0.77	0	0.59
Water pressure (PW_1)		-7.12		1.09		7.80
Water pressure (PW_2)		1.20		0.43		-0.52
Uplift force	-9.20		3.30		-30.40	
Σ	18.38	-6.70			55.38	7.87

(i) Overturn

x=(N・x + H・y) /N= 3.44 m

e=L/2-x= 0.56 m<L/6=1.33 m, Ok

(ii) Sliding

Safety factor, F_s=N・f/H=18.38x0.6/6.70= 1.65 >1.50, Ok.

where f: friction factor=0.6, Clay

(iii) Bearing capacity of stratum

P=N/Bx(1+(6xe)/B)=18.38/7x{1+(6x0.56)/7}= 3.89 tf/sq.m<5, Clay, Ok.

Uplift is zero.

4 Channel Bed Protection Design

(1) Channel protection length calculation

(a) Natural hydraulic jump after water dropping

(i) Water depth at dropping point

Water depth at dropping point: (from above Item 3, h_{1a}= 0.70 m

Channel: A1-CH6 (D_1=0.5 m)				
1	**Design conditions**			
	(1)	Channel		
		Width (B), average, 16+1=	17	m
		Slope, (I)	0.004	m/m
		Discharge (Q)	72.19	cu.m./s
		Manning's n	0.02	
	(2)	Geotechnical conditions		
		Soft Clay		
		Unit weight	1.4	t/cu.m
		Unit weight of water	1.0	t/cu.m
		Angle of Internal friction	10	degree
	(3)	Design Load		
	(a)	Dead weight		
		Reinforced concrete, γ_c	2,400	kgf/cu.m
	(b)	Design seismic coefficent	0	
	(4)	Drop height		
		River bed level difference, D_1	**0.5**	**m**
		Projection of drop concrete, D_2	0.3	m
		Depth of drop pool, D_3	0.3	m
		Total drop, D	1.10	m
2	**Design of Main body and apron**			
	(1)	Apron length		
	(a)	Length of overflow drop		
	(i)	Discharge conditions		
		Discharge, Q=	72.19	cu.m./s
		Discharge per unit width: q=Q/B=	4.25	sq.m/s
		Critical depth: $h_c=q/v_c=q/(ghc)$^1/2		
		Then, $h_c=q^{2/3}xg^{1/3}$=	1.23	m
		Critical velocity: $v_c=(gh_c)$^1/2=	3.47	m/s
		Uniform flow depth of down stream; $h_2=q/v_2=q/1/n\ xh_2^{2/3}I^{1/2}$		
		Then, $h_2=(qxn/\ I^{1/2})$ ^3/5=	1.19	m
		Check of Overflow condition		
		$h_c+D_1+D_2$=	2.03	> h_2=1.19 m
		Complete overflow is confirmed even if maximum discharge.		
		Therefore, discharge is used for apron calculation.		
	(ii)	Calculation of overflow falling length		
		Using Rand's formula,		
		$L'/D=4.3x(h_c/D)^{0.81}$		
		Then, $L'=4.3x(h_c/D)^{0.81}xD$=	5.16	, say **5.5 m.**
		L' : Apron length, D: Total drop		
	(b)	Check of piping		
		Water levels at concrete drop and apron down stream end are required to check the piping.		
		Water levels at up and down streams to be determined for checking piping.		
	(i)	Dixcharge conditions		
		Discharge is used because of complete uniform flow.		
	(ii)	Depth at concrete drop		
		Control section will be at the concrete drop, critical depth is used.		

(ii)-1 Depth at falling point: h_{1a}

Formula:

$h_{1a}^3-(v_c^2/2g+D+h_c)xh_{1a}^2+q^2/2g=0$

$h_{1a}^3-(3.47^{\wedge}2/2^*9.8+1.6+1.23)xh_{1a}^2+4.25^{\wedge}2/2^*9.8=0$

$h_{1a}^3-3.44xh_{1a}^2+0.922=0$

When h_{1a}=	0.57	m
$A=h_{1a}^3-3.44xh_{1a}^2+0.922$=	-0.0105	
Therefore, h_{1a}=	0.57	m

(ii)-2 Upstream max. water level difference

H = hc+D-h_{1a} = 1.23+1.1-0.57 =	1.76	m

(iii) Anchor length of seepage control work

Anchor length is calculated using Rein's formula.

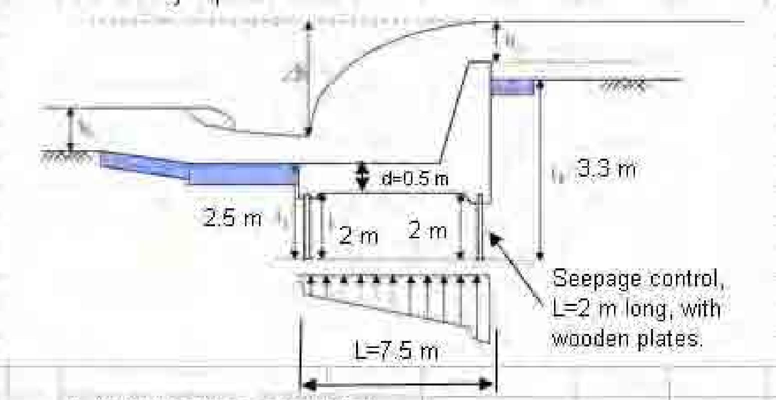
$C \leq (L/3+\Sigma l_y)/H$

where,

C: Creep ratio (Silt)	7	
L: Horizontal seepage length		
=Main body+Apron length=2+5.5=	**7.5**	**m**

Σl_y: Vertical seepage length

=Main body+Apron thickness



d: Apron thickness calculation

$d=F_s x(u_{pm}-h_2 x W_0)/(\gamma_c-1)$

where,

d: Required thickness of apron (m),

u_{pm}: Maximum uplift force at the apron (t/sq.m),

γ_c: Unit weight of concrete (=2.4 t/cu.m)

F_s: Safety factor (usually 4/3 is used.),

On the other hand,

$u_{px}=\{h_2+Hx(\Sigma l - l_x)/\Sigma l\}xW_0$

where,

u_{px}: Uplift at x point (t/sq.m)

H: Water level difference of up and down streams (=0.5 m)

l_x: Seepage length at x point (m),

W_0: Unit weight of water (1 t/qu.m)

When x=0, $u_{px}=u_{pm}$ and h_2=1.19 m

Therefore,

u_{pm}=(1.19+0.5)x1=	1.69	m		
Then,				
d=	0.48	m,	say **d=**	**0.5 m**
On the other hand,				
$\Sigma l_y \geq CxH-L/3=7x0.5-7.5/3$=	1.0	m		
Required seepage control work length: l				
$l=\{\Sigma ly-(D_1+D_3+2xd)\}/2$	-0.4	m		

Seepage control work is min. 2 m long.

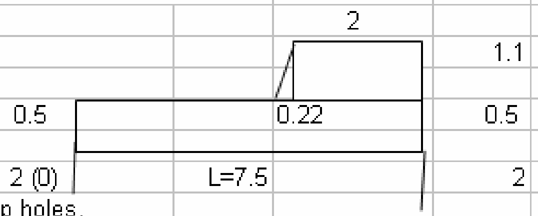
Therefore, Seepage control work at both up and down streams, **minimum 2 m long.**

3	**Stability of Main Body**								
	(a)	External force							
		River bed level difference, D_1		0.5	m				
		Projection of drop concrete, D_2		0.3	m				
		Depth of drop pool, D_3		0.3	m				
		Total drop, D		1.1	m				
		Thickness of apron, d:		0.5	m				
		Thickness of vertical wall, W:		2.0	m				
		Apron length		5.5	m				
		Main body+Apron length=L=		**7.5**	**m**				
		Gradient of vertical wall, l'		0.2					
		Critical depth, h_c		1.23	m				
		Overflow depth at falling point, h_{1a}		0.57	m				
		Seepage control work length							
		Upstream (lc_1)		2.0	m				
		Downstream (lc_2)		0 (2)	(Assumed 0 due to weepholes)				
		Width, B'		1.0	m, (Unit width)				
		Unit weight of concrete		2.4	t/cu.m				
		Unit weight of water		1.0	t/cu.m				
		Angle of internal friction							
		Normal case, K_{a1}		0.297					
		Seismic case, K_{ea}		0					
	(i)	Dead weight and inertia force calculation							
				Vertical force	Arm length (m)		Moment		Inertia force H_i
	Part	Volume (cu.m)		W_i (ton)	x_i	y_i	W_ix_i : Mx	H_iy_i : My	(earthquake)
	1	WxDxB'=	2.20	5.28	1.00	1.05	5.28	0	0
	2	l'xDxB'xDx1/2	0.12	0.29	2.07	0.86	0.60	0	0
	3	LxdxB'	3.75	9.00	3.75	0.25	33.75	0	0
		Total		14.57			39.63	0	0
				x 2.4					
		$x=\Sigma(W_i \times x_i)/\Sigma W=$			2.72	m			
		y=0							
	(ii)	Earth pressure calculation							
		Normal time							
		Earth pressure per sq.m	$p_1=K_{a1} x \gamma_{bw} x(D_1+D_3+d)=0.297x1.0x(0.5+0.3+0.5)=$					0.39	t/sq.m
		Earth pressure	$P_1=1/2xp_1x(D1+D3+d)=$					0.25	ton
		Horizontal force	$P_{1H}=P_1xcos\ (2\Phi/3)=0.25xcos(2x10/3°)=$					0.25	ton
		Vertical force	$P_{1v}=P_1xsin\ (2\Phi/3)=0.25xsin(2x10/3°)=$					0.03	ton
		Arm length, y	y=(D1+D3+d)/3=1.3/3=					0.43	m
	(iii)	Water pressure calculation							
		Normal time							
		Backside:	$pw_{1b}=\gamma w\ x\ h_c=1.0x1.23=$					1.23	t/sq.m
			$pw_{2b}=\gamma w\ x(h_c+D+d)=1.0x(1.23+1.1+0.5)=$					2.83	t/sq.m
		$PW_{1b}=1/2x(pw_{1b}+pw_{2b})x(D+d)=1/2x(1.23+2.83)x(1.1+0.5)=$						3.25	t/m
		$y=(D+d)/3x(2xpw_{1b}+pw_{2b})/(pw_{1b}+pw_{2b})=(1.1+0.5)/3x(2x1.23+2.83)/(1.23+2.83)=$						0.69	m
		Frontside	$pw_{1f}=\gamma w\ x\ h_{1a}=1.0x0.57=$					0.57	t/sq.m
			$pw_{2f}=\gamma w\ x(h_{1a}+d)=1.0x(0.57+0.5)=$					1.07	t/sq.m
		$PW_{2f}=1/2x(pw_{1f}+pw_{2f})xd=1/2x(0.57+1.07)x0.5=$						0.41	t/m
		$y=d/3x(2xpw_{1f}+pw_{2f})/(pw_{1f}+pw_{2f})=0.5/3x(2x0.57+1.07)/(0.57+1.07)=$						0.22	m

(iv) Uplift force

(a) Normal time

$\Sigma l=1.1+0.5+2+2+7.5+0.5=$ 13.6 m



With weep holes, assumed as 0

$U_a=(d+Hx(\Sigma l\text{-}l_a)/\Sigma l)xW_0=[0.5+1.76x\{13.6\text{-}(1.1+0.5+4+4)\}/13.6]x1.0$ 1.02 tf/sq.m

$U_b=(d+Hx(\Sigma l\text{-}l_b)/\Sigma l)xW_0=[0.5+1.76x\{13.6\text{-}(1.1+0.5+4+4+7.5)\}/15.6]x1.0$ 0.05 tf/sq.m

U=(1.02+0.05)x7.5x 1/2= 3.99 tf/sq.m

x=7.5/3x(2x0.05+1.02)/(0.05+1.02)= 2.61 m

(b) Stability calculation (normal time)

	N (tf)	H (tf)	x (m)	y (m)	N·x(tf·m)	H·y(tf·m)
Dead weight	14.57		2.72		39.63	
Earth pressure (P_1)	0.03	-0.25	0	0.43	0	0.11
Water pressure (PW_1)		-3.25		0.69		2.26
Water pressure (PW_2)		0.41		0.22		-0.09
Uplift force	-3.99		2.61		-10.42	
Σ	10.61	-3.09			29.21	2.27

(i) Overturn

x=(N·x + H·y) /N= 2.97 m

e=L/2-x= 0.78 m<L/6=1.33 m, Ok

(ii) Sliding

Safety factor, F_s=N·f/H= 2.06 >1.50, Ok.

where f: friction factor=0.6, Clay

(iii) Bearing capacity of stratum

P=N/Bx(1+(6xe)/B)= 0.80 tf/sq.m<5, Clay, Ok.

Uplift is zero.

4 Channel Bed Protection Design

(1) Channel protection length calculation

(a) Natural hydraulic jump after water dropping

(i) Water depth at dropping point

Water depth at dropping point: (from above Item 3, h_{1a}= 0.57 m

(ii) Hydraulic jump after water drop point:

$h_{1b}/h_2=1/2x((1+8\cdot F_2^2)^{1/2}-1)$

Then, $h_{1b}=1/2x((1+8\cdot F_2^2)^{1/2}-1)xh_2$

Froude Number at downstream depth of h_2, $=V_2/(g*h_2)^{1/2}$

where $V_2xh_2=v_cxh_c$,

Therefore, $V_2=v_cxh_c/h_2=(gxh_c)^{1/2}xh_c/h_2$

Then Froude Number becomes:

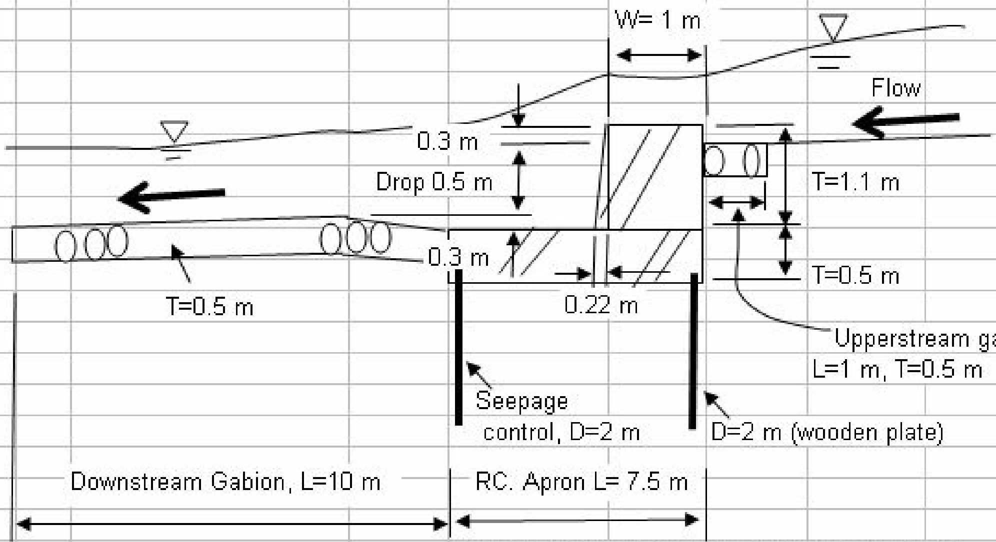
$F_2=(gxh_c)^{1/2}xh_c/h_2x(gxh_2)^{1/2}=(h_c/h_2)^{3/2}=$ 1.040

Then , h_{1b}= 1.26 m

(iii) Comparison between Overflow depth at dropping (h_{1a}) and Hydraulic jump depth after overflow (h_{1b})

h_{1b}=1.26>0.57=h_{1a},
therefore, channel bed protection should be provided for all hydraulic jump section. Referring to other design examples, gabion protection of L=10 m is adopted. L= 10 m (with gabion channel bed protection)

5 **Profile drawing of channel drop structure**
(in the case of 0.5 m drop)



Reinforcement Dia. 13 mm at 300 mm intervals,
Concrete coverage: 5 cm from top of concrete.

Profile

6.7. Surface Drainages

6.7.1. Basic Consideration

Several road drainage facilities would be installed to remove the water from the road surface (see **Fig. 6.7.1**).

(1) Shoulder Drainage

In the cut section, masonry ditch along the shoulder of the main highway plays a role to transport the water from the shoulder to the discharge point (see **Fig. 6.7.2**).
In the embankment section, to avoid the scouring of the filling, asphalt curb is installed to the shoulder. Road surface water, therefore, runs on the shoulder (see **Fig. 6.7.3**). To prevent the encroachment of water onto the carriageway, drainage openings that remove the water from the shoulder to the vertical drainage is required with some intervals that were calculated by the formula mentioned in the next (see **Fig. 6.7.4**).

(2) Median Drainage

Masonry ditch on the centre median was planned to collect the water and to transport it. This type of ditch will be removed when the 6 lane condition.
To bring the water form the ditch to toe end, catch basin connected with pipe was required.

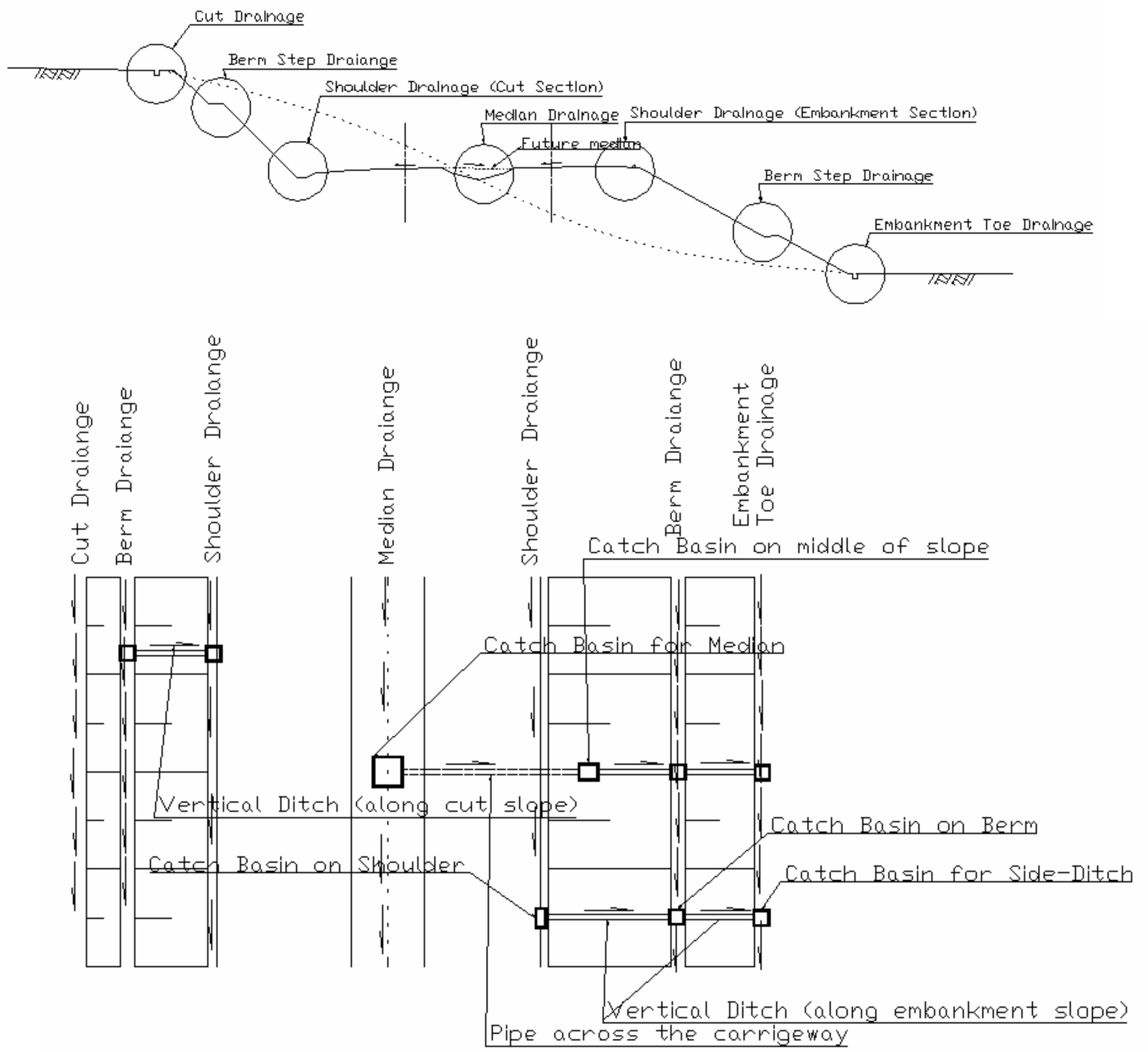
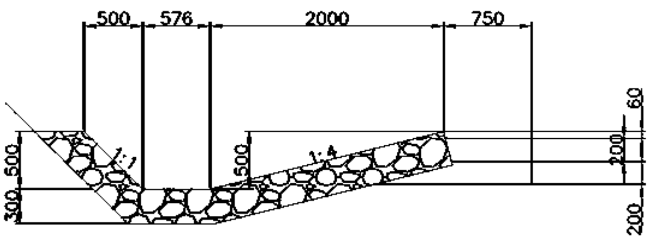


Fig. 6.7.1 Longitudinal/Cross-Sectional Drainage Facilities



(unit: mm)

Fig. 6.7.2 Shoulder Ditch on Cut Section

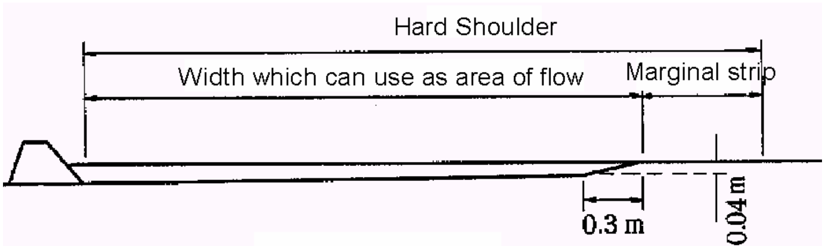


Fig. 6.7.3 Asphalt Concrete Curb and Cross-sectional Area of Flow

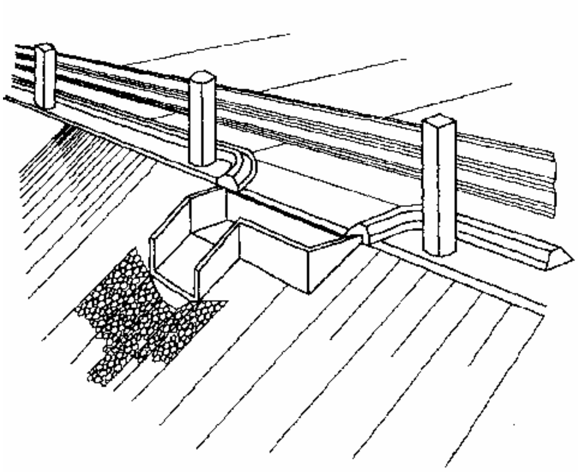
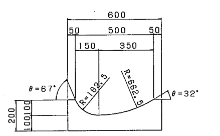
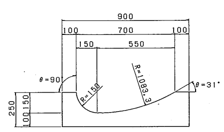
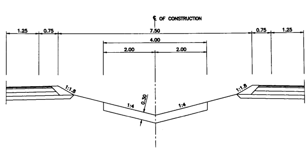


Fig. 6.7.4 Opening of the Asphalt Curb (CB-A)

(3) Layout of the Catch Basin on Median (CB-B)

To examine the locations of basins, rolled gutters were considered in the curve section (R>3,500m) and masonry ditches were considered in the tangent section (R<3,500m). Detail of these ditches are shown in the **Table 6.7.1**.
Some catch basins shown in the tables will not be installed at the 4-lane stage, such as the ones that were connected with longitudinal pipes at the 6-lane stage.

Table 6.7.1 Median Ditches and its Properties

Type	Cross-section	Hydraulic properties	
Rg-0.5		Hydraulic R	0.062
		Cross-sectional A	0.035
		Roughness C	0.015
Rg-0.7		Hydraulic R	0.090
		Cross-sectional A	0.073
		Roughness C	0.015
DS(M)-C		Hydraulic R	0.243
		Cross-sectional A	1.000
		Roughness C	0.025

6.7.2. Calculation Formula

To bring the water from road surface to the longitudinal canal/ditch, opening of the curb is required to install onto the embankment section's shoulder and catch basin which is connected to the longitudinal/cross-sectional pipe is required onto the median and cut section's shoulder.

Interval of the curb opening or catch basin was calculated using formula below.

$$L = 0.8Qa / Qd$$

Where Qa: design discharge capacity by using the Manning's formula
(full flow: m3/s)
Qd: discharge volume by using the Rational formula (m3/s)

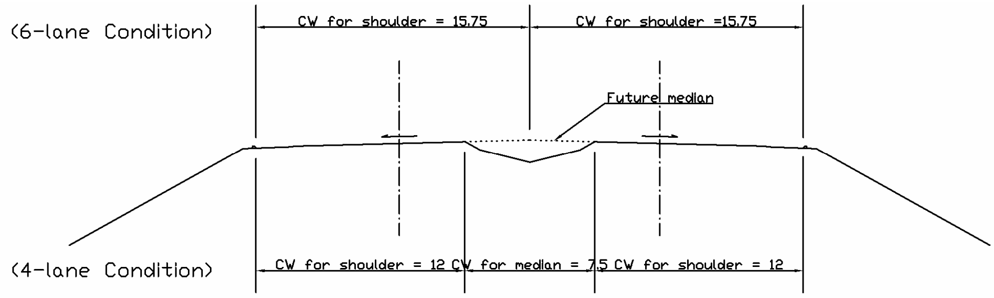
Full flow width of the embankment shoulder is considered the water flow not disturbing the traffic. In the design, the width was set as a hard shoulder width excluded by a marginal strip width (refer to **Fig. 6.7.3**). The gradient that required in the Manning's formula was estimated by the elevation difference between the adjacent two basins dividing by its interval length.

For the shoulder drainage in the normal condition, the calculated L was limited not more than 100m. But 200m of maximum interval was adopted in case the running water was collected from the shoulder width only.

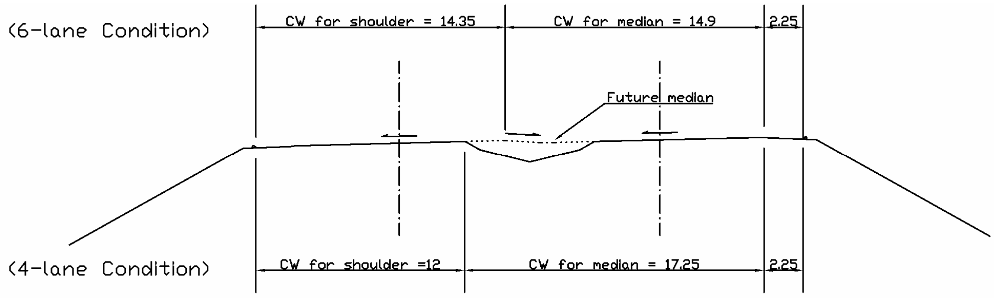
Regarding the median drainage, no limit was set where no median ditch will be required in the 6 lane condition (e.g. section whose radius more than 3,500m). On the other hand, 100m was taken as a limit for the rests of sections.

6.7.3. Stage-Construction

Since the stage construction method is adopted in OCH project, future 6-lane condition must be considered as well as 4-lane condition.
In the design, especially for the median drainage was affected by the stage construction, because both the type of ditch and the catchment width were different between the 4-lane condition and 6-lane condition (see **Fig. 6.7.5** and **Fig. 6.7.6**).
In some case, the drainage pipe under the carriageway were required at the 6-lane stage, but weren't required at the 4-lane stage, and if so, the pipe was planned in the design, since it will be difficult to install the pipe under the service condition.



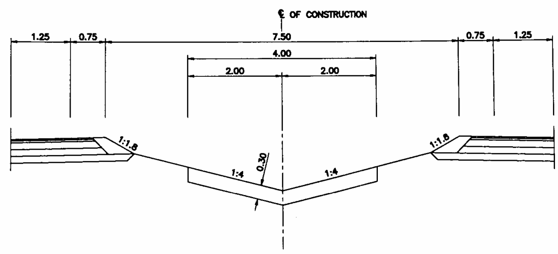
(Tangent Section)

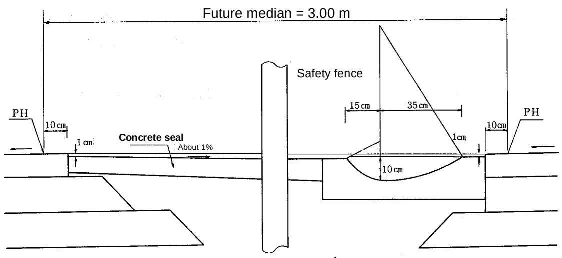


(Curve Section)

CW: Catchment Width

Fig. 6.7.5 Differences of Catchment Widths Between the 4/6-lane Stage





(4-lane Stage) **(6-lane Stage[1])**

1 No median ditch will be installed in the normal crown section.

Fig. 6.7.6 Differences of Median Drainage Facility

6.8. Flood Area Slope Protection (Gabions)

The gabions protections were provided where the highway passes through mainly marsh and flood plains. These protections were combination of two boxes or combination of boxes and cylinders. The top elevations of these protections were set to annual flood level (see **Fig. 6.8.1** and **Table 6.8.1**).

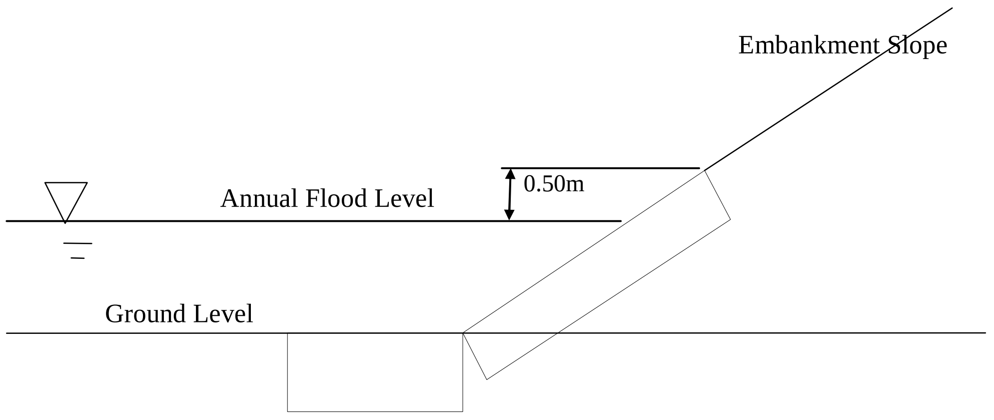


Fig. 6.8.1 Relation of Annual Flood Level and Gabion Protection

The drainage will be provided to prevent scour on cut slopes caused by water inflow from adjacent area. Most typical drainage facilities adopted were same as embankment toe drainage.

Table 6.8.1 Adopted Annual Flood Level

Station	Annual Flood Levels (MSL in m)
8+760	8.9
10+630	10.7
12+300	6.8
13+200	4.0
15+200	4.0

6.9. Gutter Capacity

6.9.1. Location of the Catch Basin on Shoulder CB (A)

(1) Flood

<u>Conditions</u>

Case : Road Surface Drainage (see **Fig. 6.9.1**)
Type of Drainage : Asphalt Concrete Curb (see **Fig. 6.9.2**)
Location : L=100m

Volume of discharge is calculated by Rational method as follows.

$$Qd = \frac{1}{3.6 \times 10^6} C \times I \times A = 0.0484 \text{ (m3/sec)}$$

Where
Qd : Discharge (m3/sec)
C : Runoff coefficient = 0.9
I : Average intensity of storm (mm/hr) = 135 (mm/hr)
A : Catchment area (m2) = 1,435.0 (m2)

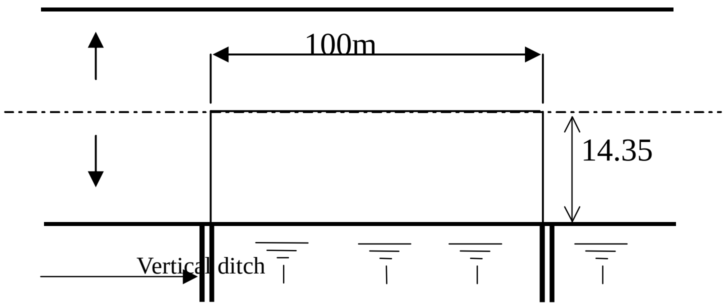


Fig. 6.9.1 Plan of Road Surface

(2) Discharge capacity CB (A)

$$Qa = \frac{1}{n} A \times R2/3 \times S1/2 = \text{ (m3/sec)}$$

Where
Qa : Discharge capacity (m3/sec)
n : Manning roughness coefficient = 0.013
A : Cross-sectional area of flow (m2) = 0.166
R : Hydraulic radius (A/P) (m) = 0.070
P : Wetted perimeter (m) = 2.370
S : Longitudinal gradient = 0.001~0.010

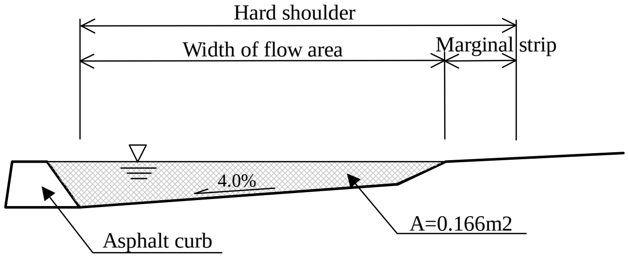


Fig. 6.9.2 Section of OCH Road Drainage Facility (Asphalt Concrete Curb)

$$Qa = \frac{1}{0.013}\ 0.167 \text{ x } 0.0702/3 \text{ x } S1/2 = 2.145 \text{ x } S1/2 \quad (m3/sec)$$

Table 6.9.1 Discharge capacity CB (A)

S	Qa	Remarks
0.001	0.068	
0.002	0.096	
0.003	0.117	
0.004	0.136	
0.005	0.152	
0.006	0.166	
0.007	0.179	
0.008	0.192	
0.009	0.203	
0.010	0.215	

Maximum Interval of Vertical ditch

The thing that rain water flows in the length section on the road has the following problem.
The accident such as slipping is caused when rain water stays, and it is undesirable on the running safety.
Rain water infiltrates easily under the pavement, and the weakening of the road body is caused.

Full drain	L=100m	Qd=0.0484 ＜Qa=0.068 S=0.001
Shoulder drain	L=200m	

Comment : It is possible to drain it by assuming the interval of to be 100m in 0.1% in the longitudinal slope.

6.9.2. Location of the Catch Basin on Shoulder CB (B)

Ramp of Interchange

(1) Flood

<u>Conditions</u>

Case	: Road Surface Drainage (see **Fig. 6.9.3**)
Type of Drainage	: Asphalt Concrete Curb (see **Fig. 6.9.4**)
Location	: L=50m

$$Qd = \frac{1}{3.6 \times 10^6} C \times I \times A = (m3/sec)$$

= Case 1 Stage 1 0.0118 (m3/sec. per50m)
Case 2 Stage 2 0.0232 (m3/sec.per50m)

Where

Qd	: Discharge (m3/sec)
C	: Runoff coefficient = 0.9
I	: Average intensity of storm (mm/hr) = 135 (mm/hr)
A	: Catchment area (m2) Case 1 = 350.0 (m2)
	Case 2 = 688.0 (m2)

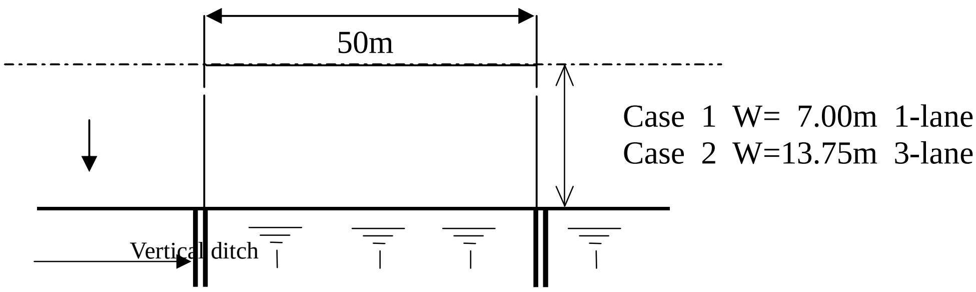


Fig. 6.9.3 Plan of Ramp Surface

(2) Discharge capacity CB (B)

$$Qa = \frac{1}{n} A \times R2/3 \times S1/2 = (m3/sec)$$

Where

Qa	: Discharge capacity (m3/sec)
n	: Manning roughness coefficient = 0.013
A	: Cross-sectional area of flow (m2) = 0.0781
R	: Hydraulic radius (A/P) (m) = 0.0308
P	: Wetted perimeter (m) = 2.563
S	: Longitudinal gradient = 0.001~0.010

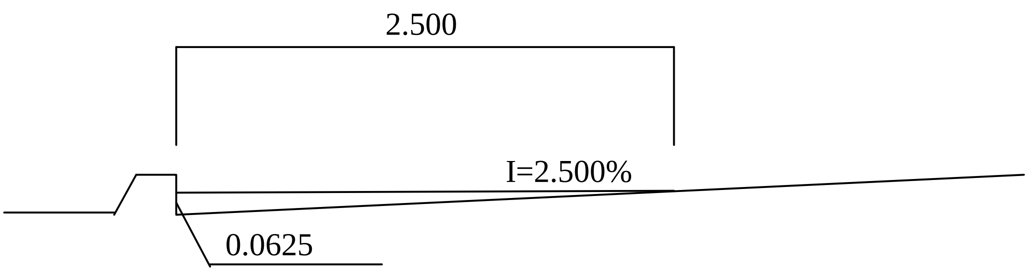


Fig. 6.9.4 Section of Ramp Drainage Facility (Asphalt Concrete Curb)

$$Qa = \frac{1}{0.013}\ 0.078 \times 0.03082/3 \times S1/2 = 0.589 \times S1/2 \quad (m3/sec)$$

Table 6.9.2 Discharge capacity CB (B)

S	Qa	Remarks
0.001	0.019	Case 2 L=50m
0.002	0.026	Case 1 L=50m
0.003	0.032	
0.004	0.037	
0.005	0.042	
0.006	0.046	
0.007	0.049	
0.008	0.053	
0.009	0.056	
0.010	0.059	

Case 1 : It is possible to drain it by assuming the interval of to be 50m in 0.2% in the longitudinal slope.
Case 2 : It is possible to drain it by assuming the interval of to be 50m in 0.1% in the longitudinal slope

Comment : It is possible to drain it by assuming the interval of to be 50m in the longitudinal slope.

6.9.3. Location of the Catch Basin on Shoulder CB (B)

A-3 Bypass

(1) Flood

<u>Conditions</u>

Case : Road Surface Drainage (see **Fig. 6.9.5**)
Type of Drainage : Asphalt Concrete Curb (see **Fig. 6.9.6**)
Location : L=50m

$$Qd = \frac{1}{3.6 \times 106}\ C \times I \times A = (m3/sec)$$

= 0.0169 (m3/sec. per50m)

Where

Qd : Discharge (m3/sec)

C : Runoff coefficient = 0.9

I : Average intensity of storm (mm/hr) = 135 (mm/hr)

A : Catchment area (m2) = 500.0 (m2)

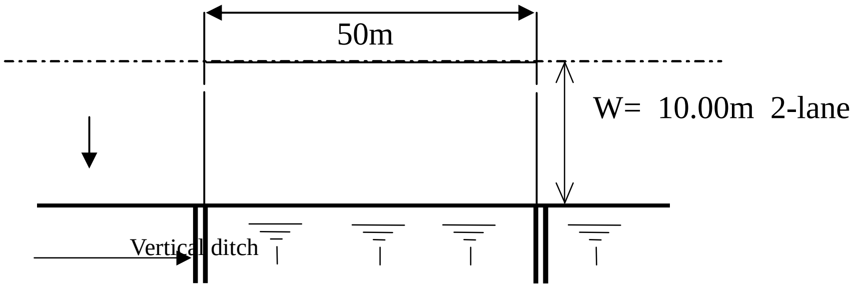


Fig. 6.9.5 Plan of A-3 Bypass Surface

(2) Discharge capacity CB (B)

$$Qa = \frac{1}{n} \text{ A x R2/3 x S1/2 = (m3/sec)}$$

Where

Qa : Discharge capacity (m3/sec)

n : Manning roughness coefficient = 0.013

A : Cross-sectional area of flow (m2) = 0.0781

R : Hydraulic radius (A/P) (m) = 0.0308

P : Wetted perimeter (m) = 2.563

S : Longitudinal gradient = 0.001~0.010

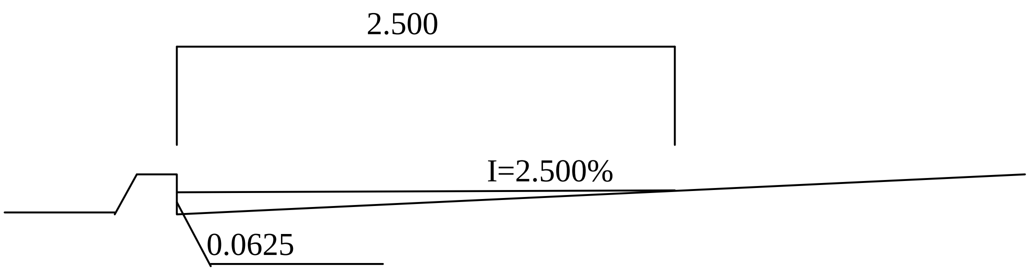


Fig. 6.9.6 Section of Ramp Drainage Facility (Asphalt Concrete Curb)

$$Qa = \frac{1}{0.013} \text{ 0.078 x 0.03082/3 x S1/2 =0.589 x S1/2 (m3/sec)}$$

Table 6.9.3 Discharge capacity CB (B)

S	Qa	Remarks
0.001	0.019	
0.002	0.026	
0.003	0.032	
0.004	0.037	

0.005	0.042	
0.006	0.046	
0.007	0.049	
0.008	0.053	
0.009	0.056	
0.010	0.059	

Comment : It is possible to drain it by assuming the interval of to be 50m in 0.1% in the longitudinal slope

6.9.4. Location of the Catch Basin on Median DS (C)-1 (Future)

(1) Flood

<u>Conditions</u>

Case : Road Surface Drainage (see **Fig. 6.9.7**)
Type of Drainage : Concrete Ditch (see **Fig. 6.9.8**)
Location : L=100m

$$Qd = \frac{1}{3.6 \times 106} C \times I \times A = 0.0503 \text{ (m3/sec)}$$

Where
Qd : Discharge (m3/sec)
C : Runoff coefficient = 0.9
I : Average intensity of storm (mm/hr) = 135 (mm/hr)
A : Catchment area (m2) = 1,490.0 (m2)

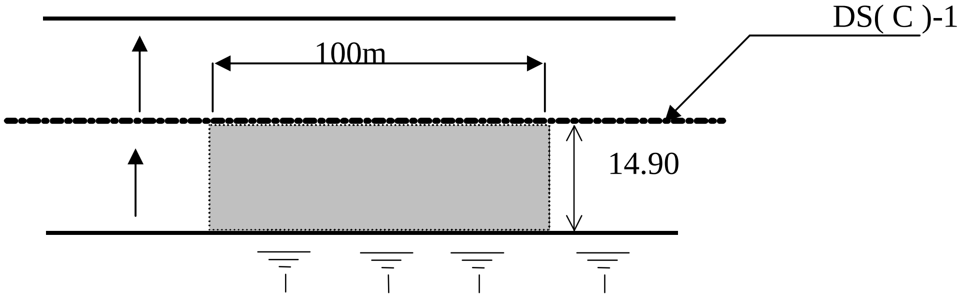


Fig. 6.9.7 Plan of Median Ditches Surface

Table 6.9.4 Discharge capacity DS (C)-1

S=0.001 Qa =0.0503

Lm	Qd	Remarks
50	0.0252	
100	0.0503	
150	0.0755	
200	0.1006	
250	0.1258	
300	0.1509	

(2) Discharge capacity DS (C)-1

$$Qa = \frac{1}{n} A \times R2/3 \times S1/2 = (m3/sec)$$

Where

Qa	: Discharge capacity (m3/sec)
n	: Manning roughness coefficient = 0.015
A	: Cross-sectional area of flow (m2) = 0.073
R	: Hydraulic radius (A/P) (m) = 0.090
S	: Longitudinal gradient = 0.001~0.025

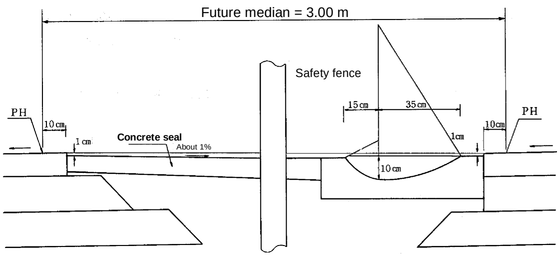


Table 6.9.8 Median Ditches DS (C)-1

$$Qa = \frac{1}{0.015} \quad 0.073 \times 0.0902/3 \times S1/2 = 0.977x\ S1/2$$
(m3/sec)

Table 6.9.5 Discharge capacity DS (C)-1

S	Qa	Remarks
0.001	0.031	50mQd=0.025
0.002	0.044	
0.003	0.054	100mQd=0.050
0.004	0.062	
0.005	0.069	
0.006	0.076	150mQd=0.076
0.007	0.082	
0.008	0.087	
0.009	0.093	
0.010	0.098	
0.015	0.120	200mQd=0.101
0.020	0.138	250mQd=0.126
0.025	0.154	300mQd=0.151

6.9.5. Location of the Catch Basin on Median DS (M)-D

(1) Flood

<u>Conditions</u> When tentative is used
Case : Road Surface Drainage (see **Fig. 6.9.9**)
Type of Drainage : Grouted Stone Riprap (see **Fig. 6.9.10**)
Location : L=100m

Volume of discharge is calculated by Rational method (see **clause 4.4**.) as follows.

$$Qd = \frac{1}{3.6 \times 106} C \times I \times A = 0.0503 \text{ (m3/sec)}$$

Where
Qd : Discharge (m3/sec)
C : Runoff coefficient = 0.9
I : Average intensity of storm (mm/hr) = 135 (mm/hr)
A : Catchment area (m2) = 1,490.0 (m2)

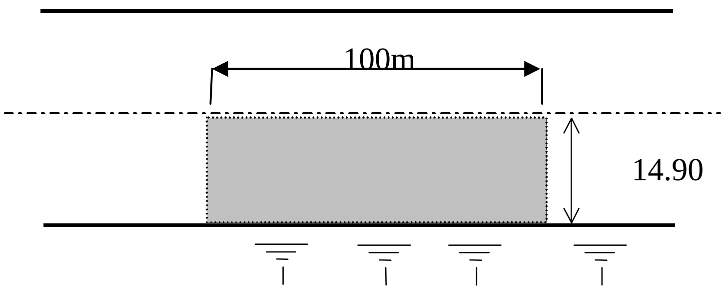


Fig. 6.9.9 Plan of Road Surface (Out of Scale)

Table 6.9.6 Flood volume

S=0.001 Qa =0.493

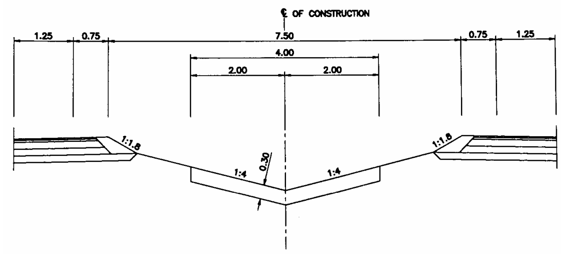
L	Qd	Remarks
50	0.0252	
100	0.0503	
150	0.0755	
200	0.1006	
250	0.1258	
300	0.1509	

(2) Discharge capacity DS (M)-D

$$Qa = \frac{1}{n} A \times R^{2/3} \times S^{1/2} = \ (m^3/sec)$$

Where

Qa	: Discharge capacity (m^3/sec)
n	: Manning roughness coefficient = 0.025
A	: Cross-sectional area of flow (m^2) = 1.000
R	: Hydraulic radius (A/P) (m) = 0.243
S	: Longitudinal gradient = 0.001~0.01



(4-lane Stage)

Fig. 6.9.10 Differences of Median Drainage Facility

$$Qa = \frac{1}{0.025} 1.000 \times 0.243^{2/3} \times S^{1/2} = 15.576 \times S^{1/2} \quad (m^3/sec)$$

Table 6.9.7 Discharge capacity DS (M)-D

S	Qa	Remarks
0.001	0.493	
0.002	0.697	
0.003	0.853	
0.004	0.985	
0.005	1.101	
0.006	1.207	
0.007	1.303	
0.008	1.393	
0.009	1.478	
0.010	1.558	

6.9.6. Location of the Catch Basin on Median DS (M)-C

<u>Conditions</u>

Case : Road Surface Drainage (see **Fig. 6.9.11**)
Type of Drainage : Asphalt Concrete Curb (see **Fig. 6.9.12**)
Location : L=100m

Volume of discharge is calculated by Rational method (see **clause 4.4**.) as follows.

$$Qd = \frac{1}{3.6 \times 10^6} C \times I \times A = 0.0906 \text{ (m3/sec)}$$

Where
Qd : Discharge (m3/sec)
C : Runoff coefficient = 0.9
I : Average intensity of storm (mm/hr) = 135 (mm/hr)
A : Catchment area (m2) = 2,685.0 (m2)

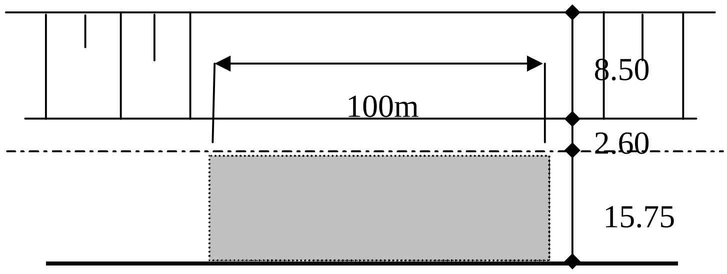


Fig. 6.9.11 Shoulder Ditch on Cut Section DS (M)-C

Table 6.9.8 Flood volume DS (M)-C

S=0.001 Qa

L	Qd	Remarks
50	0.0453	
100	0.0906	
150	0.1359	
200	0.1812	
250	0.2265	
300	0.2718	
400	0.3624	
500	0.4530	
600	0.5436	
700	0.6342	
800	0.7248	
900	0.8154	
1000	0.9060	

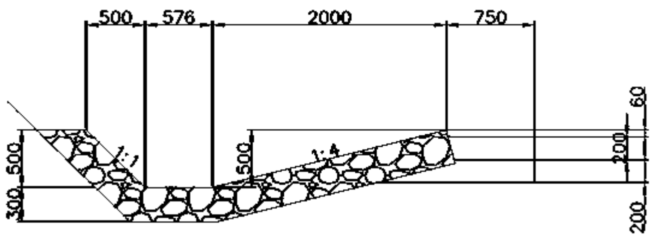
6.9.7. Location of the DS (M)-C

(1) Discharge capacity DS (M)-C

$$Qa = \frac{1}{n} A \times R2/3 \times S1/2 = (m3/sec)$$

Where

Qa : Discharge capacity (m3/sec)
n : Manning roughness coefficient = 0.025
A : Cross-sectional area of flow (m2) = 0.894
R : Hydraulic radius (A/P) (m) = 0.267
S : Longitudinal gradient = 0.001~0.03



(unit: mm)

Fig. 6.9.12 Shoulder Ditch on Cut Section DS (M)-C

$$Qa = \frac{1}{0.025} 0.894x\ 0.2672/3 \times S1/2 = 14.826 \times S1/2 \quad (m3/sec)$$

Table 6.9.9 Flood volume DS (M)-C

S	Qa	Remarks
0.001	0.469	500mQd=0.453
0.002	0.663	
0.003	0.812	
0.004	0.938	1000mQd=0.906
0.005	1.048	
0.006	1.148	
0.007	1.240	
0.008	1.326	
0.009	1.407	
0.010	1.483	

6.9.8. Location of the Catch Basin on DS (C)-D

A-3 Bypass

(1) Flood

<u>Conditions</u>

Case : Road Surface Drainage (see **Fig. 6.9.13**)
Type of Drainage : Asphalt Concrete Curb (see **Fig. 6.9.14**)
Location : L=50m

$$Qd = \frac{1}{3.6 \times 10^6} C \times I \times A = (m3/sec)$$
$$= 0.0169 \text{ (m3/sec. per50m)}$$

Where
Qd : Discharge (m3/sec)
C : Runoff coefficient = 0.9
I : Average intensity of storm (mm/hr) = 135 (mm/hr)
A : Catchment area (m2) = 500.0 (m2)

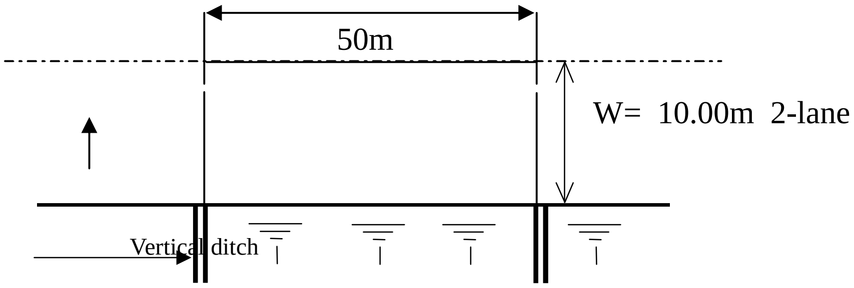


Fig. 6.9.13 Plan of A-3 Bypass Surface

Table 6.9.10 Flood volume DS (M)-C

L	Qd	Remarks
50	0.0169	
100	0.0338	
150	0.0507	
200	0.0676	
250	0.0845	
300	0.1014	
400	0.1352	
500	0.1690	
600	0.2028	
700	0.2366	
800	0.2704	
900	0.3042	
1000	0.3380	

6.9.9. Location of the DS (C)-D

(1) Discharge capacity DS (C)-H

$$Qa = \frac{1}{n} \text{ A x R2/3 x S1/2 = (m3/sec)}$$

Where

Qa	: Discharge capacity (m3/sec)
n	: Manning roughness coefficient = 0.015
A	: Cross-sectional area of flow (m2) = 0.0912
R	: Hydraulic radius (A/P) (m) = 0.108
S	: Longitudinal gradient = 0.001~0.030

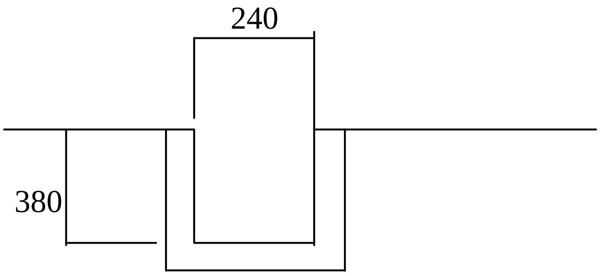


Fig. 6.9.14 Ditch Section DS (C)-D

$$Qa = \frac{1}{0.015} \text{ 0.0912x 0.1082/3 x S1/2 = 1.379x S1/2 (m3/sec)}$$

Table 6.9.11 Flood volume DS (C)-D

S	Qa	Remarks
0.001	0.044	100mQd=0.0338
0.002	0.062	150mQd=0.0507
0.003	0.076	200mQd=0.0676
0.004	0.087	250mQd=0.0845
0.005	0.097	
0.006	0.107	300mQd=0.1014
0.007	0.115	
0.008	0.123	
0.009	0.131	400mQd=0.1352
0.010	0.138	
0.015	0.169	
0.020	0.195	
0.030	0.239	

CHAPTER 7

BRIDGE & STRUCTURE DESIGN

CHAPTER 7 BRIDGE & STRUCTURE DESIGN

7.1 Bridge Design

Finally, the Government of Sri Lanka (GOSL) had requested to apply the Special Term for Economic Partnership (STEP) loan scheme after the submission of draft final report for this Project, and decided using steel girder bridges to satisfy the loan condition.

The steel bridges were applied at main highway viaduct (Viaduct-1: V1) and ramp bridges (Viaduct-5 to 8: V5 to V8), A1 interchange area to consider construction ability, regional characteristics, aesthetics view, etc.

And in this design period, the latest soil investigation result was incorporated for the bridges/viaducts. Length of Viaduct-3 (V3: The 2nd Biyagama Viaduct), pile length and foundation type were changed based on this review.

In this Chapter 7, the previous discussion from clause 7.1.1 to 7.1.8 were remained and changed design such as steel girder design and V3 length were discussed from clause 7.1.9.

Moreover, the bridge/viaduct lists for the final design are shown in **Table 7.1.0**.

Table 7.1.0 Bridge/Viaduct List

- Detailed Design

No.	Category	ID No.	Station	Crossing Object	Class	Length (m)	Span Arrangement (m)	1st Stage Effective Width (m)	Width (m)	Skew Angle (degree)	Type
1	*Highway*	*H9*	*16+169.50 – 16+524.50*	*Kelani River B214, AB10 Road*	*A*	*355.0*	*10@35.5*	*2*10.75*	*31.40*	*68-00-00 at Pier-4*	*PC-I (3 or 4-span continuous)*
2	*Overpass*	*O8*	*9+415.00*	*Gonahena Ihala Biyanwila Road*	*C*	*42.0*	*2@21.0*	*10.00*	*10.90*	*90-00-00*	*PC-I (2-span continuous)*
3		*O9*	*10+210.00*	*B169 Road*	*B3*	*50.0*	*2@25.0*	*13.00*	*13.90*	*62-00-00*	*PC-I (2-span continuous)*
4		*O10*	*11+261.00*	*B401 Road*	*B3*	*44.0*	*2@22.0*	*13.00*	*13.90*	*(Curved)*	*PC-I (2-span continuous)*
5		*O11*	*12+518.921*	*Sapugaskanda – Biyagama Road*	*B2*	*45.0*	*2@22.5*	*21.20*	*22.10*	*71-04-23*	*PC-I (2-span continuous)*
6		*O12*	*14+843.00*	*Jayanthi Mawatha*	*C*	*45.0*	*2@22.5*	*10.00*	*10.90*	*72-00-00*	*PC-I (2-span continuous)*

- Basic Design

No.	Category	ID No.	Station	Crossing Object	Class	Length (m)	Span Arrangement (m)	1st Stage Effective Width (m)	Width (m)	Skew Angle (degree)	Type
1	Highway	V1	8+648.00 – 8+970.00	A1 Interchange (Soft Ground)	A	322.0	18.0+21.0 +3@47.67 +3@46.67	2*10.75 with widening	31.40	90-00-00	Steel-I (2 to 3-span continuous)
2		*V2*	*13+755.00 – 14+070.00*	*(Soft Ground)*	*A*	*315.0*	*9@35.0*	*2*10.75*	*31.40*	*90-00-00*	*PC-I (3-span continuous*3)*
3		V3	15+95.00 – 15+515.00	(Soft Ground)	A	420.0	12@35.0	2*10.75	31.40	90-00-00	PC-I (3-span continuous*4)
4		*V4*	*15+895.00 – 16+169.50*	*Mudun Ela (Soft Ground)*	*A*	*274.5*	*7@35.0+29.5*	*2*10.75*	*31.40*	*90-00-00*	*PC-I (3-span continuous*2 +Simple)*
5	Ramp	V5	0+241.285 – 0+363.785	A1 I.C. Ramp-2	A	122.500	3@40.833	7.00 with widening	7.90, Varies	90-00-00	Steel Box (3-span continuous)
6		V6	0+239.766 – 0+361.835	A1 .C. Ramp-3	A	122.069	35.069+2@43.500	7.00 with widening	7.90, Varies	90-00-00	Steel Box (3-span continuous)
7		V7	0+365.448 – 0+489.070	A1 I.C. Ramp-5	A	123.622	3@41.207	7.00 with widening	7.90, Varies	90-00-00	Steel Box (3-span continuous)
8		V8	0+365.448 – 0+503.019	A1 I.C. Ramp-6	A	137.571	3@45.857	7.00 with widening	7.90, Varies	90-00-00	Steel Box (3-span continuous)
9		*V9*	*0+108.155 – 0+275.655*	*B214 I.C. Ramp-1*	*A*	*167.500*	*2*25.0+3*35.0*	*7.00 with widening*	*7.90, Varies*	*90-00-00*	*PC-I (2 or 3-span rigid*
10		*V10*	*0+73.330 – 0+312.830*	*B214 I.C. Ramp-2*	*A*	*239.900*	*4*29.5+3@35.0*	*7.00 with widening*	*7.90, Varies*	*90-00-00*	*PC-I (3 or 4-span rigid)*

7.1.1. General

Contained within the Outer Circular Highway - Northern Section 1 (OCH-N1) there are a total of 16 bridges/viaducts to be designed which are spread throughout the project, comprising 1 highway bridge, 4 highway viaducts, 6 ramp bridges and 5 highway overpasses.

The highway bridge crossing over the Kelani Ganga (Ganga: River, the second longest river in Sri Lanka – approximately 145km long) is 355m in length.

The Basic Design of the bridges for this section of OCH had not been completed in the previous Study because of the alignment change after the Study by Road Development Authority (RDA). Also during this Study the proposed embankment scheme for the highway was changed from earthworks to viaduct at soft ground locations in consideration of environmental advantages.

Therefore, the design for bridges is included to consider the basis, such as determination of bridge location, length, type, etc. with the examination results.

(1) Scope of Bridge Design

The scope of the bridge design is;

- Selection of bridge/viaduct types
- Planning of bridges/viaducts (location, length, width)
- Initial design and detailed design calculations for general formation
- Drawing of general views and engineering details
- Quantity calculation

There are two (2) levels of design – detailed and basic, and scope for each design was as follows;

Detailed Design: Kelani River Crossing Bridge and all five (5) of the Highway Overpass Bridges were carried out to the detailed design level.

The earlier basic design for these bridges was executed without sufficient on-site topographic and geological data. Thus, before starting the detailed design, all basic design output had to be checked with site data and the planning of bridge/structures confirmed with the RDA again.

Basic Design: Four (4) Viaducts for main highway and six (6) Ramp Bridges were carried out to basic design level.

Some of the geological data (boring data) could not be incorporated with this basic design as the investigation was not finished during this design period due to flooding at site which precluded access.

(2) Design Standards

Primary Bridge Design Manual and the other RDA standards were adopted, and British Standards (BS), British Standard Institution (BSI) and Japanese Design Standard were adopted as sub standards.

For the bridge design of this section, the standard is described in **Appendix 7.1**.

7.1.2. Bridge List

All bridges/Viaducts in this Project are tabulated on **Table 7.1.1**.

Table 7.1.1 Bridge/Viaduct List

- Detailed Design

No.	Category	ID No.	Station	Crossing Object	Class	Length (m)	Span Arrangement (m)	1st Stage Effective Width (m)	Width (m)	Skew Angle (degree)	Type
1	Highway	H9	16+169.50 – 16+524.50	Kelani River B214, AB10 Road	A	355.0	10@35.5	2*10.75	31.40	68-00-00 at Pier-4	PC-I (3 or 4-span continuous)
2	Overpass	O8	9+415.00	Gonahena Ihala Biyanwila Road	C	42.0	2@21.0	10.00	10.90	90-00-00	PC-I (2-span continuous)
3		O9	10+210.00	B169 Road	B3	50.0	2@25.0	13.00	13.90	62-00-00	PC-I (2-span continuous)
4		O10	11+261.00	B401 Road	B3	44.0	2@22.0	13.00	13.90	(Curved)	PC-I (2-span continuous)
5		O11	12+518.921	Sapugaskanda – Biyagama Road	B2	45.0	2@22.5	21.20	22.10	71-04-23	PC-I (2-span continuous)
6		O12	14+843.00	Jayanthi Mawatha	C	45.0	2@22.5	10.00	10.90	72-00-00	PC-I (2-span continuous)

- Basic Design

No.	Category	ID No.	Station	Crossing Object	Class	Length (m)	Span Arrangement (m)	1st Stage Effective Width (m)	Width (m)	Skew Angle (degree)	Type
1	Highway	V1	8+648.00 – 8+970.00	A1 Interchange (Soft Ground)	A	322.0	18.0+21.0 +4@35.75 +4@35.0	2*10.75 with widening	31.40	90-00-00	PC-I (2 to 4-span continuous)
2		V2	13+755.00 – 14+070.00	(Soft Ground)	A	315.0	9@35.0	2*10.75	31.40	90-00-00	PC-I (3-span continuous*3)
3		V3	15+130.00 – 15+445.00	(Soft Ground)	A	315.0	9@35.0	2*10.75	31.40	90-00-00	PC-I (3-span continuous*3)
4		V4	15+895.00 – 16+169.50	Mudun Ela (Soft Ground)	A	274.5	7@35.0+29.5	2*10.75	31.40	90-00-00	PC-I (3-span continuous*2 +Simple)
5	Ramp	V5	0+241.285 – 0+363.785	A1 I.C. Ramp-2	A	122.500	7@17.50	7.00 with widening	7.90, Varies	90-00-00	PC-I (3 or 4-span continuous)
6		V6	0+239.766 – 0+361.835	A1 .C. Ramp-3	A	122.069	6@17.50+17.069	7.00 with widening	7.90, Varies	90-00-00	PC-I (3 or 4-span continuous)
7		V7	0+365.448 – 0+489.070	A1 I.C. Ramp-5	A	123.622	6@17.50+18.622	7.00 with widening	7.90, Varies	90-00-00	PC-I (3 or 4-span continuous)
8		V8	0+365.448 – 0+503.019	A1 I.C. Ramp-6	A	137.571	7@17.50+15.071	7.00 with widening	7.90, Varies	90-00-00	PC-I (3 or 4-span continuous)
9		V9	0+108.155 – 0+275.655	B214 I.C. Ramp-1	A	167.500	2*25.0+3*35.0	7.00 with widening	7.90, Varies	90-00-00	PC-I (2 or 3-span rigid
10		V10	0+73.330 – 0+312.830	B214 I.C. Ramp-2	A	239.900	4*29.5+3@35.0	7.00 with widening	7.90, Varies	90-00-00	PC-I (3 or 4-span rigid)

7.1.3. Selection of Basic Bridge Type

(1) General

Although both concrete and steel bridges have been implemented in Sri Lanka, a concrete bridge is generally more advantageous for reasons of ease of procurement, maintenance and economics and locally available construction expertise than for a steel bridge. In particular, the construction cost of a concrete bridge is generally cheaper than a comparable steel structure.

For the above reasons concrete bridges have generally been adopted for most of the previous bridge projects in Sri Lanka, including those of the RDA, and have been selected for this Project

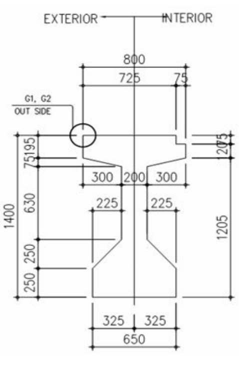
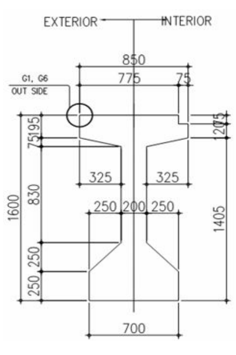
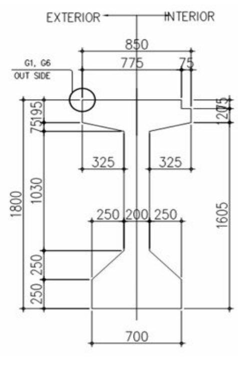
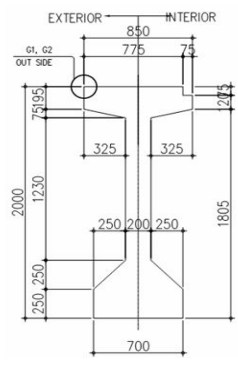
(2) Superstructure

1) PC I-Girder

Bridge span lengths in the OCH-N1 project range from 20 m to 35 m. As prestressed concrete I-section girders (PC-I- Girder) are the most suitable type of superstructure for these span lengths, as determined by previous studies, this type of structure was selected for all bridges.

Applied PC I-Girder sections for this project are shown in **Table 7.1.2** below:

Table 7.1.2 Application of PC I-Girder

No.	1	2	3	4
Cross Section				
Applicable Girder Length (m)	~ 22.5	~ 27.5	~ 32.5	~ 36.5
Area (sq.cm.)	5,770	6,470	6,870	7,270
Weight (kN/m)	13.85	15.53	16.49	17.45
Applied Bridge	O8, O10, O11, O12, V1, V5, V6, V7, V8	O9, V9	---	H9, V1, V2, V3, V4, V9, V10

2) Concrete Panel as Non-structural Formwork

As one of the proposed construction methods, pre cast reinforced concrete panels of 75 mm thickness were used for deck slab support. This method has been used in OCH – Southern Section Project (OCH-S) as temporary formwork and non-structural member.

The panel is used in this design also for the ease of placement and handling available during deck slab construction. The sketch of the proposed deck slab and panel is shown as **Fig. 7.1.1**.

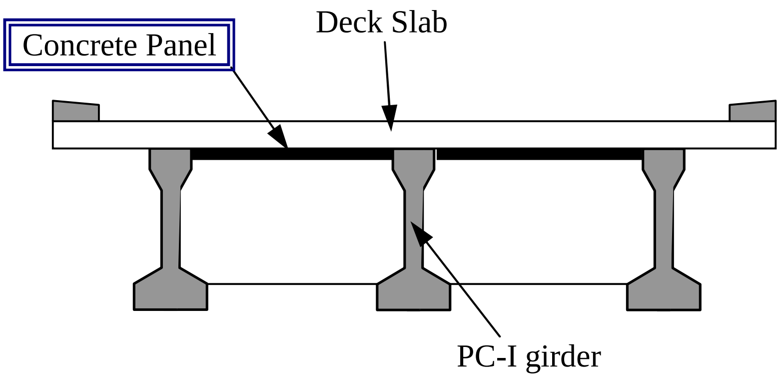


Fig. 7.1.1 Concrete Panel as Non-structural Formwork

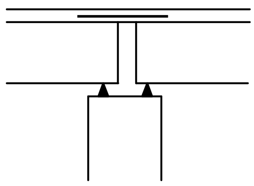
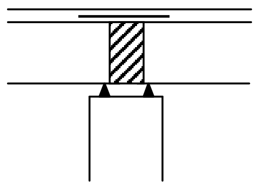
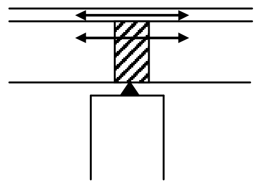
3) Connection of Slabs

Slabs should be connected continuously as much as possible in order to eliminate expansion joints, reduce cost, and increase slab durability. As shown in **Table 7.1.3**, there are basically three connection methods. After comparison, the “RC Connection” method was selected for this project.

Based on Japanese Codes, the RC connection can be applied under the following conditions:

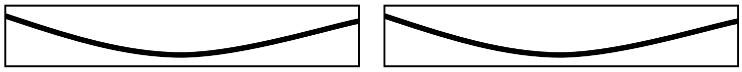
- The grid model analysis with spring constant due to bearing shoe shall be used,
- Span length shall be equal or less than 35 m,
- Each span shall be almost equal in length,
- When the skew angle of bridge is less than 70 degrees, torsional rigidity of members shall be considered,

Table 7.1.3 Comparison of Slab Connection Method

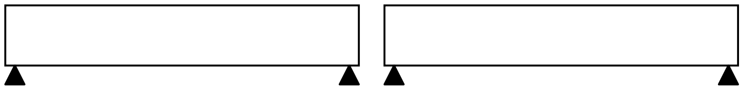
Item	**Slab Connection**	**RC Connection**	**PC Connection**
General View	by steel reinforcement 	by steel reinforcement 	by PC cable 
Continuity of Slab Girder	Continuous Separated	Continuous Continuous	Continuous Continuous
Bearing	Two	Two	One
Structural Behavior	Acts as two separate beams	Depends on the spring effect of the bearings	Clearly a continuous beam
Construction Control	Easier	Easy	Need to replace bearing
Construction Cost	More economical	Economical	Expensive
Maintenance Cost	More	Less	Least
Application to Composite structure	Difficult application	Easily applicable	Easily applicable
Judgment	2	1 (Applied)	3
Reason	- Slab less durable - Needs more maintenance - Not popular	- Very popular - Increases slab durability - Easy to construct	- Difficult construction - Initial cost expensive - Needs much pre-stressing

The construction sequence is as follows:

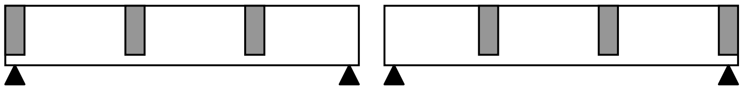
Step-1: Pre-cast PC-I Girder with post-tensioning



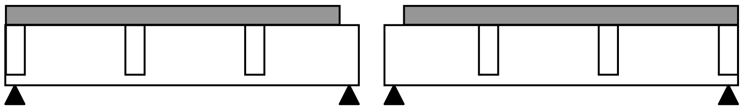
Step-2: Launch PC-I girder on the two bearings



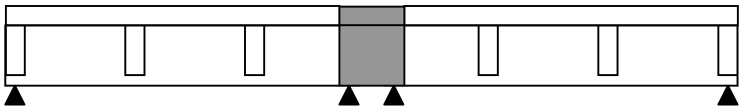
Step-3: Place both intermediate and end diaphragms



Step-4: Pour intermediate slab



Step-5: Pour connection diaphragm and slab



Step-6: Induce pre-stressing in connection diaphragm

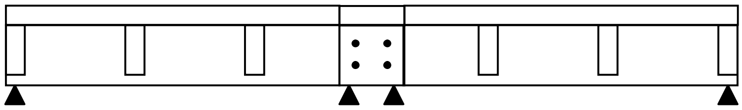


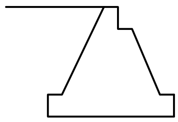
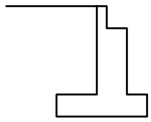
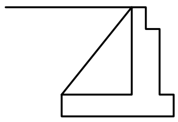
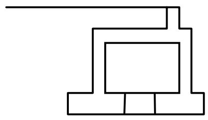
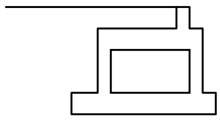
Fig. 7.1.2 Construction Sequence of RC Connection

7.1.4. Substructure and Foundation

(1) Abutment

The reversed T-type abutment was applied because cheaper construction cost and ease of construction are more advantageous than the other possible types for this project. General abutment type depending on the applicable height is shown in **Table 7.1.4**, which refers to Japanese Codes.

Table 7.1.4 Abutment Types

Type and Shape		Applicable Height (m)	Characteristic
Gravity-type		H ≤ 5	- Simple structure - Easy construction - Heavy
Reversed T type		5 < H < 14	- Economic - Easy construction
Counter-forted Buttressed type		10 ≤ H	- Economic - Intricate construction - Difficulty in back filling
Rigid-framed type		10 ≤ H ≤ 15	- Complicate structure - Expensive
Box type		15 ≤ H	- Large–scale structure - Complicated structure - Expensive

The dimensions of abutment (spread foundation) in OCH Project were made uniform.

(2) Pier

Wall type and Column type piers were selected in this design. The reasons for this selection are as follows;

<u>Highway bridge and viaducts</u>

- The wall type pier introduces least interference possible to the river flow thereby minimizing disturbance and restriction to the river current.
- Wall type piers with cantilevered coping have been adopted as the most popular and general shape for viaducts

<u>Ramp bridges</u>

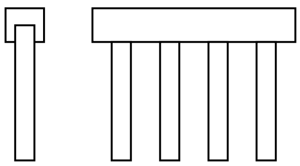
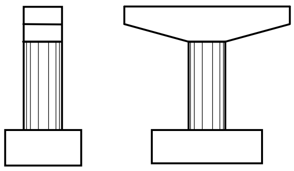
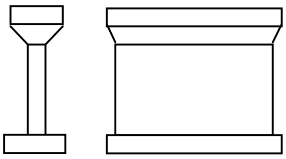
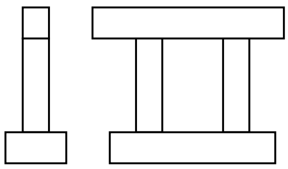
- Column type piers were adopted in consideration of the light weight of upper structure, and to provide attractive appearance.

<u>Overpass bridges</u>

- All piers are located on the median of the OCH and pier thickness shall therefore be minimized.

General pier type is shown in **Table 7.1.5**, with reference to Japanese Codes.

Table 7.1.5 Pier Types

Type	**Figure**	**Characteristics**
Pile bent type pier		- Simple structure with capped pile head - Weak horizontal force and flexible structure - Unsuitable for piers in river where scouring is expected - For light-weight superstructure - Lowest cost
Column type pier		- General construction - Diameter of column is big - Blocks large area of river crossing
Wall type Pier		- General construction - Shape appropriate the direction of the river flow - Pier thickness can be minimized
Rigid framed pier		- Generally used for wide super- structure - Unsuitable for piers in rivers

The dimensions of pier in OCH Project are unified as below:

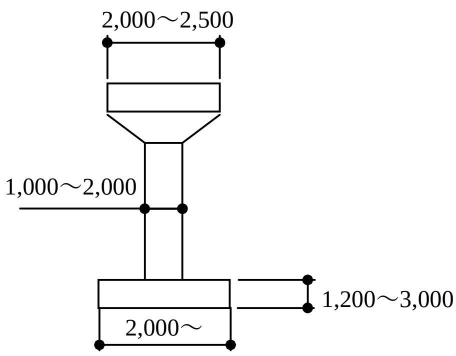


Fig. 7.1.3 General Dimensions of Pier

(3) Foundation

The typical geological profile of low-lying land is of about 5-10m thickness of soft soil such as peat, organic clay, peaty clay, and/or alluvial clay with an N-value of 0-5, overlying an alluvial/residual sand layer with an N-value of 5-20 on top of a stratum of weathered rock. The weathered rock layer has an N-value of over 50 and is adopted as the bearing stratum for bridge foundations.

Foundation type may vary dependent upon the site conditions however spread footing and pile foundation were adopted where appropriate in this project. For piled foundations, Cast-in-site RC piles were selected because of the soil condition and applicability.

Table 7.1.6 Foundation Types

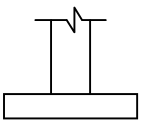
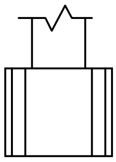
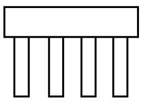
Depth of Soil Stratum	**Foundation Type**	**Remarks**
3.0 m to 4.0 m	Spread Footing 	- Open cut or cofferdam is required for excavation to the bearing stratum
4.0 m to 6.0 m	Caisson Foundation (Cylinder Well, side-by-side caissons) 	- Filled cofferdam is required for piers in rivers. Dewatering should be considered - If bearing stratum is not flat, special treatment is required to set the caisson at level
Deeper than 6.0 m	Pile Foundation 	- Pile type and diameter should be selected based on site conditions

Table 7.1.7 Pile Types

Pile Type	**Applicable Length, Dimension**	**Procurement of Material**	**Characteristics**
RC square pile	5m to 10m ϕ0.35m, ϕ0.45m	Available in Sri Lanka	- Unsuitable for changes in bearing stratum - Applicable to small bearing capacity - Joining is difficult and limited in length - Common for small bridges in Sri Lanka - Economical construction cost
Cast-in-site RC pile	10m to 60m φ0.6m toφ1.5m	Available in Sri Lanka	- Suitable for changes in bearing stratum - Applicable for large bearing forces - No joining necessary and applied for long piles - Common for large bridges in Sri Lanka - Very economical
Steel pile	5m to 60m φ0.3m toφ1.5m	Imported	- Applicable for large bearing forces - Joining to long piles is possible - Uneconomical
PC pile	5m to 25m φ0.3m toφ1.0m	Imported	- Applicable for small bearing capacity - Uneconomical

Comparison of pile diameter

Applied diameter of Cast-in-situ Concrete Bored Pile was determined by cost comparison. Based on RDA practice locally available pile diameters of 1.0 m, 1.2 m and 1.5 m diameter piles were compared.

Comparison results of abutment and pier are shown in **Table 7.1.8** and **Table 7.1.9** respectively.

The most economical diameter – 1.5m was selected and adopted for this Project.

Table 7.1.8 Pile Diameter Comparison for Abutment

(unit: Thousand Rs.)

Item		**Diameter of Cast-in-situ Concrete Bored Pile (m)**		
		1.0	**1.2**	**1.5**
Highway Bridge (Kelani River Crossing Bridge) – Pile Length = 20m				
Pile-cap	Concrete	6,504	7,732	5,781
	Reinforcement	9,062	10,774	8,055
Pile		50,320	52,399	45,953
Total (Rate)		65,886 (1.102)	70,906 (1.186)	59,790 (1.000)
Overpass Bridge – Pile Length = 17m				
Pile-cap	Concrete	1,271	1,133	1,398
	Reinforcement	1,844	1,643	2,027
Pile		9,721	9,898	9,014
Total (Rate)		12,836 (1.032)	12,673 (1.019)	12,439 (1.000)
Viaduct – Pile Length = 10m				
Pile-cap	Concrete	6,249	7,429	5,555
	Reinforcement	9,062	10,774	8,055
Pile		25,160	26,200	22,977
Total (Rate)		40,471 (1.106)	44,403 (1.214)	36,587 (1.000)

Table 7.1.9 Pile Diameter Comparison for Pier

(unit: Thousand Rs.)

Item		**Diameter of Cast-in-situ Concrete Bored Pile (m)**		
		1.0	**1.2**	**1.5**
Highway Bridge (Kelani River Crossing Bridge) – Pile Length = 8m				
Pile-cap	Concrete	4,612	4,382	5,381
	Reinforcement	6,426	6,105	7,497
Pile		27,447	29,111	24,744
Total (Rate)		38,486 (1.023)	39,597 (1.052)	37,622 (1.000)
Overpass Bridge – Pile Length = 17m				
Pile-cap	Concrete	562	345	340
	Reinforcement	815	500	493
Pile		4,860	4,949	4,507
Total (Rate)		6,237 (1.168)	5,794 (1.085)	5,341 (1.000)
Viaduct – Pile Length = 10m				
Pile-cap	Concrete	1,083	1,393	1,244
	Reinforcement	1,571	2,020	1,804
Pile		5,146	5,822	4,419
Total (Rate)		7,801 (1.045)	9,235 (1.237)	7,466 (1.000)

7.1.5. Kelani River Crossing Bridge (Highway Bridge No. 9: H9)

General

The Kelani River, the second longest river in Sri Lanka of approximately 145km length, is located at the end of this Project area (STA.16+320) and beginning of the OCH Southern Section (OCH-S) area. The catchment area of this river is very large (approximately 2,229 sq.km) and drains a very large area of Sri Lanka. OCH-N1 passes through the lowland flood plain area and shall be affected by the flooding of this river, especially during rainy season.

The OCH-N1 alignment at the crossing area is skewed at 65 to 70 degree to this river and on a 2,000m radius curve throughout. The alignment was decided after the previous Studies (feasibility study and basic design) and therefore the bridge length and the other concerned features were revised in this design.

Bridge Length

Based on the previous Studies, the waterway opening for water-flow of Kelani River required by the Hydrological analysis result was 206m. In the previous studies an approach embankment had been adopted between the river-flow area and AB10 Road.

However, following the alignment change, review of the detailed topographic and river survey conducted for this Study and the request for a longer span (wider opening) by both RDA and the Department of Irrigation (DOI), the bridge length was reviewed and revised.

Kelani River Crossing Bridge in previous Studies and this Study are shown in **Fig. 7.1.4**:

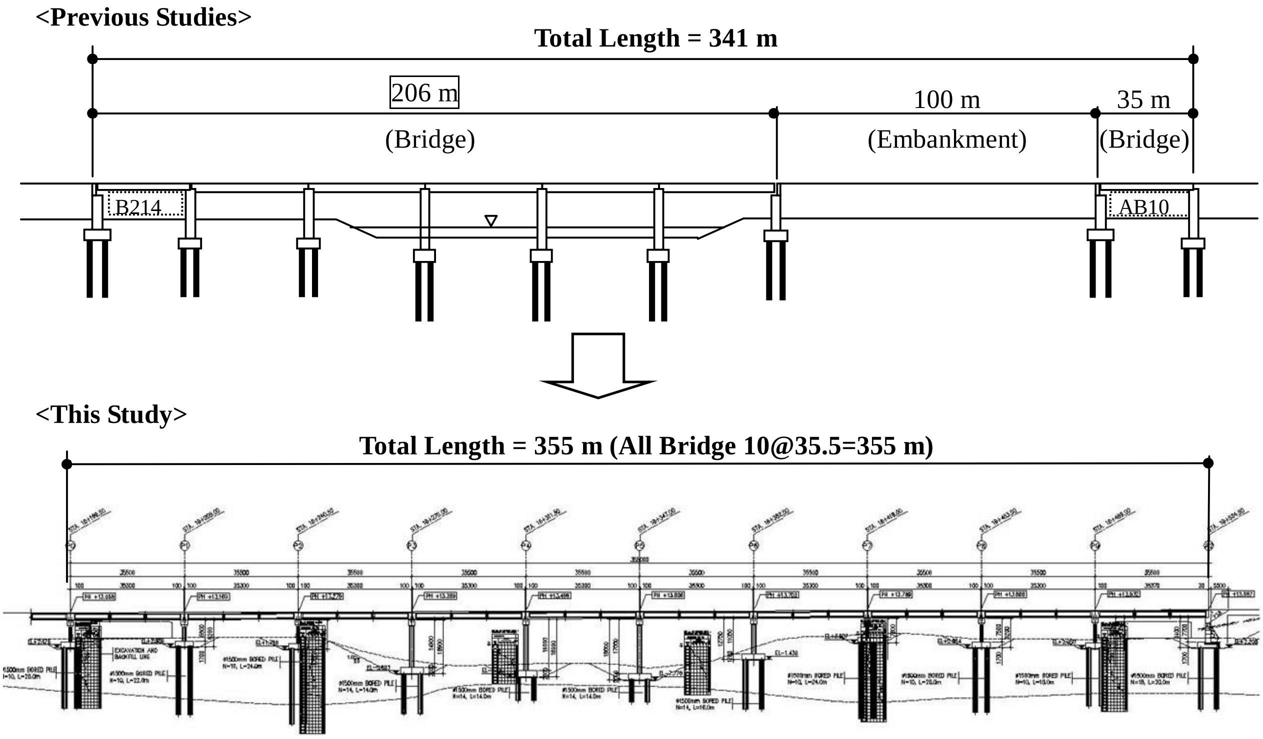


Fig. 7.1.4 Kelani River Bridge

For the Kelani River crossing bridge, a **total length of 355.0m** (10 spans @ 35.5m girder) was decided after consideration and incorporation of the following factors;

- Crossing over both B214 and AB10 Roads
- Minimized hydrological impact
- Safe protection of the highway from flood
- Construction ease and the period to build
- Land acquisition

Following consideration of all matters the RDA and DOI approved the length of Kelani River Bridge at 355.0m.

The comparison result is shown in **Table 7.1.10** and **Figure 7.1.5**.

Table 7.1.10 Comparison of Bridge Length

Alternative	**Construction Cost (Million Rs.)**			**Comments**
	Construction Stage	**Item**	**Cost**	
Original Alternative-1 Bridge + Embankment	1st Stage	PC I-Girder (Superstructure)	731	1. Disturbance to water-flow by embankment and backwater effect during floods are higher compared with Alternative-2, 2. Easy to damage the embankment due to erosion and sliding during high floods unless strong embankment protection is applied.
		Abutment, Pier (Substructure)	530	
		Embankment	80	
		Sub-Total (unit cost: Thousand Rs./sq.m.)	1,341 (163.8)	
	Final Stage (Expansion)	PC I-Girder (Superstructure)	159	
	Total (unit cost: Thousand Rs./sq.m.)		1,500 (147.4)	
Alternative-2 All Bridge	1st Stage	PC I-Girder (Superstructure)	916	1. Adverse impact on hydrology (floods) is minimum, 2. Construction period is shorter than Alternative-1. **<Applied>**
		Abutment, Pier (Substructure)	570	
		Sub-Total (unit cost: Thousand Rs./sq.m.)	1,486 (194.7)	
	Final Stage (Expansion)	PC I-Girder (Superstructure)	199	
	Total (unit cost: Thousand Rs./sq.m.)		1,685 (166.5)	

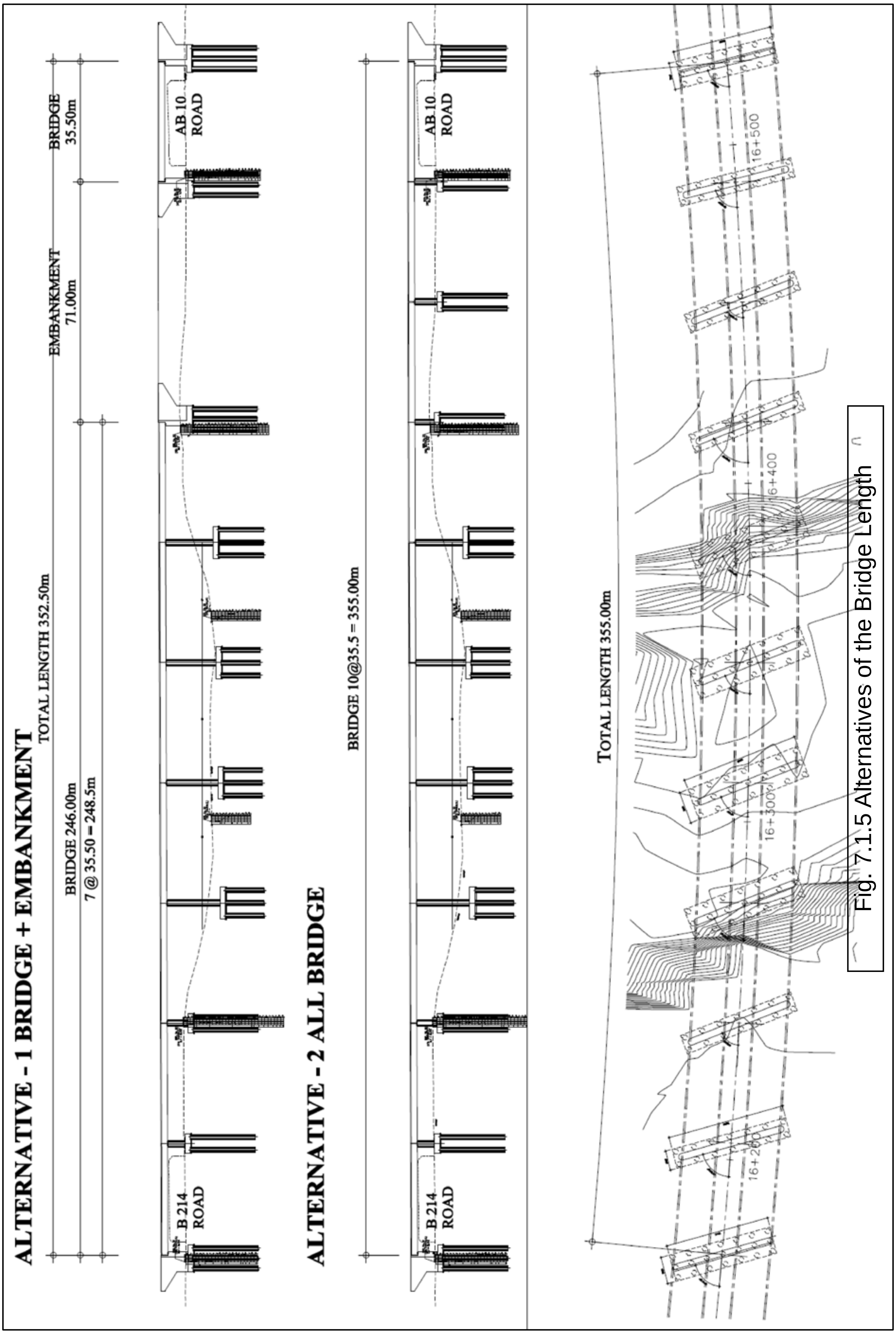


Fig. 7.1.5 Alternatives of the Bridge Length

Clearance of the Bridge

Vertical Clearance

Elevation of Kelani River Crossing Bridge shall satisfy the following clearance requirements;

-B214 Road (Classification B)	:	5.10 m from the road surface
- AB10 Road (Classification A)	:	5.10 m from the road surface
- Navigation clearance (RDA requirement)	:	4.75 m from A.W.L. (= +4.00 m M.S.L.)
- Free board on the river	:	1.20 m from H.F.L.
- High Flood Level (H.F.L.)	:	+8.50 m M.S.L.(100years return period)

Minimum Bridge Span

Minimum bridge spans were decided to satisfy the following;

- Navigation clearance (RDA requirement)	:	6.0 m wide (clear space)
- Standard Span Length (Japanese Code)	:	35.0 m (length between piers)

Standard Span of 35.0m is the length perpendicular to the river flow. Therefore, although required bridge span may exceed 35.0m when considering the skew of the alignment, a “5m alleviation rule” that considers the economic advantage is conformed with.

Bridge Type

Bridge type had been compared and decided in the previous Studies, and Pre-stressed Concrete I-Girder (PC I-Girder) was selected as the most economical scheme with 35m girder length for the applicable span by PC I-Girder.

Therefore, PC I-Girder was used for this bridge in the design.

Span Arrangement

3 and 4 span continuous superstructure type was applied in consideration of smooth ride, structural durability and the maintenance. For Kelani River Bridge a configuration of 3-span, 4-span and 3-span continuous from the beginning of this bridge was adopted following consideration of the following items:

- Structural features due to different pier height at the river bank and water-flow area,
- Simplified and/or uniformity of structure for both design and construction

Scouring Effect and Foundation

Based on the hydrological analysis result (see Chapter-6), the scouring depth of Pier-3 to 6 is 3.6 to 5.5m and Pier-2 & 7 is 3.1 to 3.3m during 100 years return period flood event.

Gabion mat was provided to protect against scouring and the area of mattress is as follows based on the River Code – horizontal length equivalent to twice the scouring depth from the edge of pile-cap shall be required;

- Pier-3, 4, 5 and 6	:	11.0m from the 4 edge of pile-cap
- Pier-2 and 7	:	6.6m from the 4 edge of pile-cap

The protection area for scouring (Pier-3 to 6) is shown in **Fig. 7.1.6**.

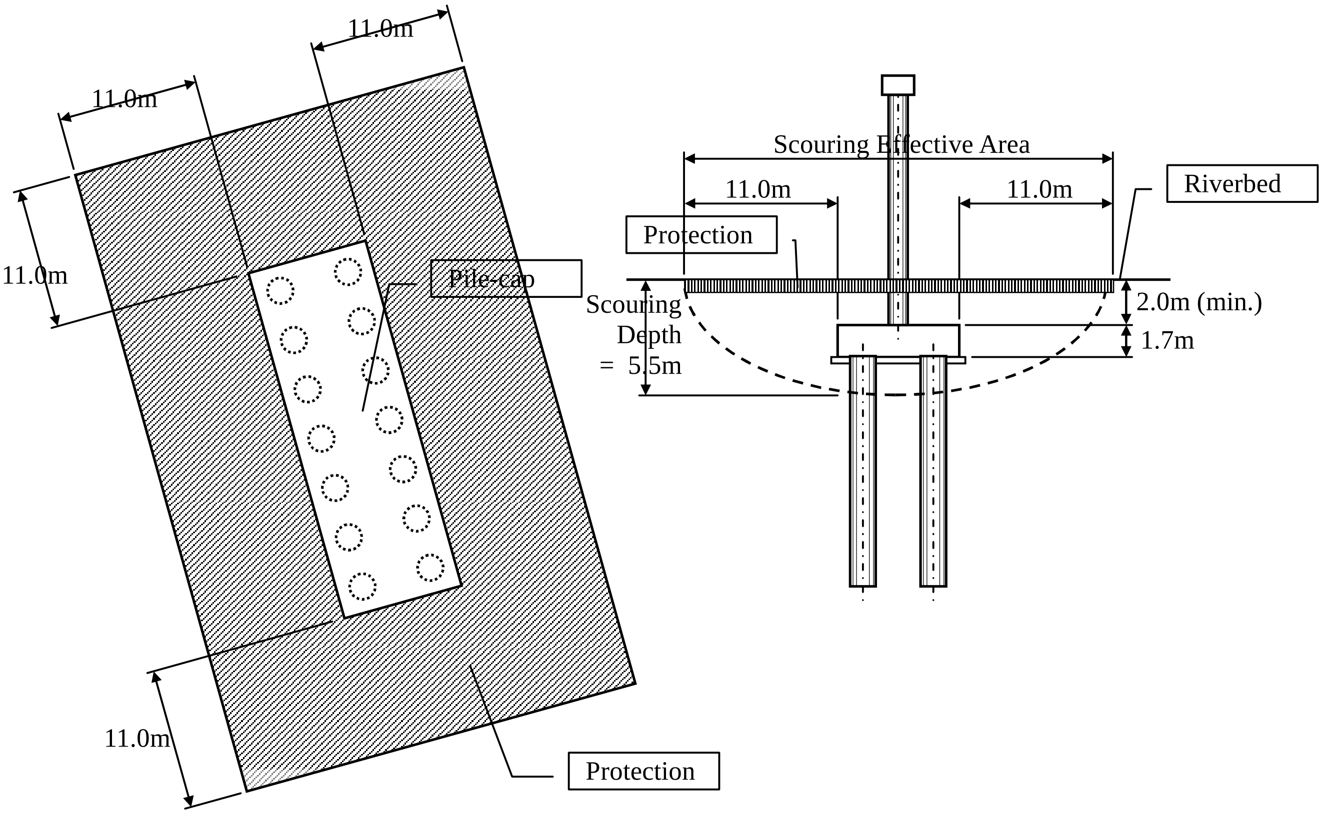


Fig. 7.1.6 Protection Area for Scouring (Pier-3 to 6)

Pier Shape

A wall type pier with oval or rounded edge shape shall be used for piers in the river to minimize the disturbance to the water-flow. The same shape shall also be applied to the piers near the river (or riverbank) for simplification of construction and aesthetic reasons. The general shape is shown in **Fig. 7.1.7**.

Stage Construction

This bridge shall be constructed in 2-time stages for the number of lanes.
For the 1st stage, 4-lanes (2 lanes in each direction) will be constructed. In the future as the 2nd and Final stage, an additional 2-lanes (1 lane in each direction) will be added inside (median side), to finally become 6-lanes operation.

This Staged Construction can be applied to the upper-structures to join girders and slab at inside. However, staged construction of substructures is quite difficult, and for piers in the river it is not permitted to build a new pier immediately adjacent to another (Japanese Code). Therefore, substructures shall be constructed at one time.

The proposed pier arrangement is shown in **Fig. 7.1.8**.

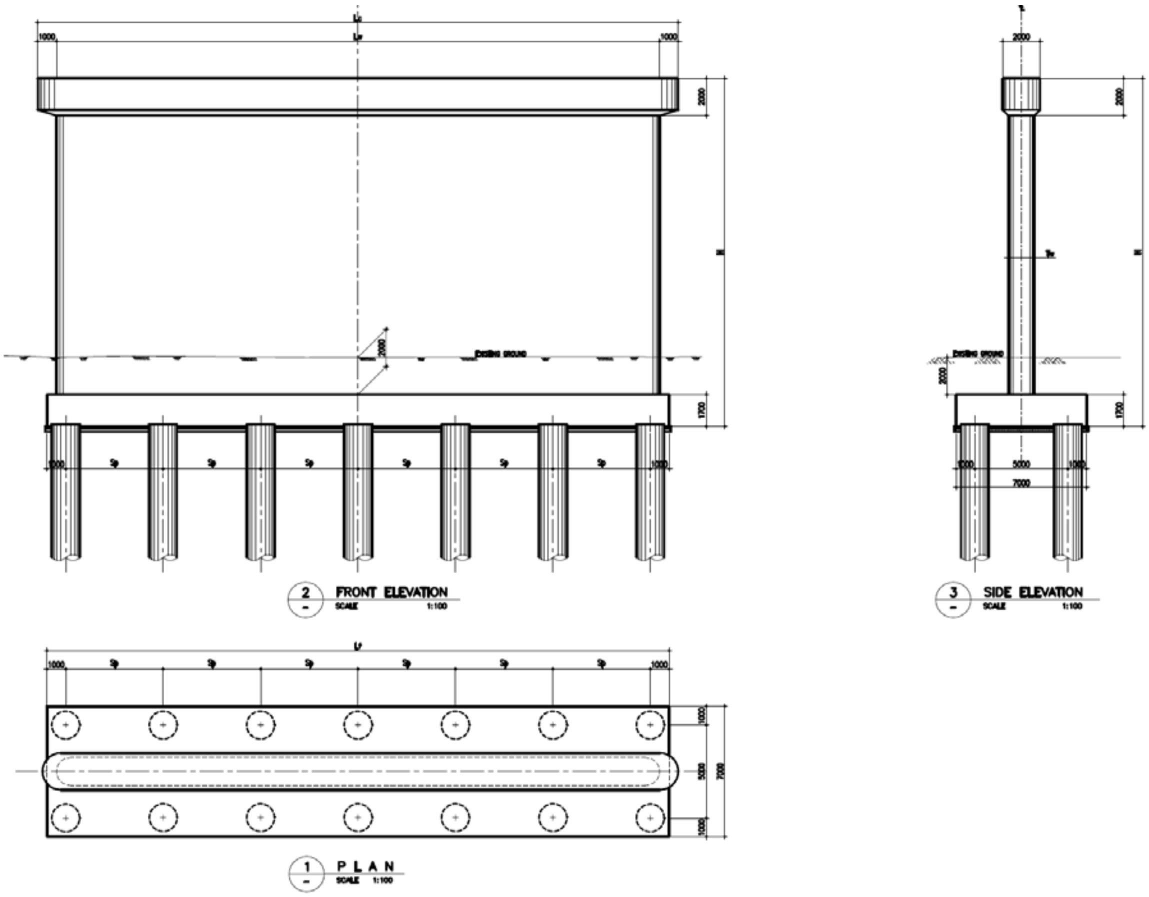


Fig. 7.1.7 General Shape of Pier

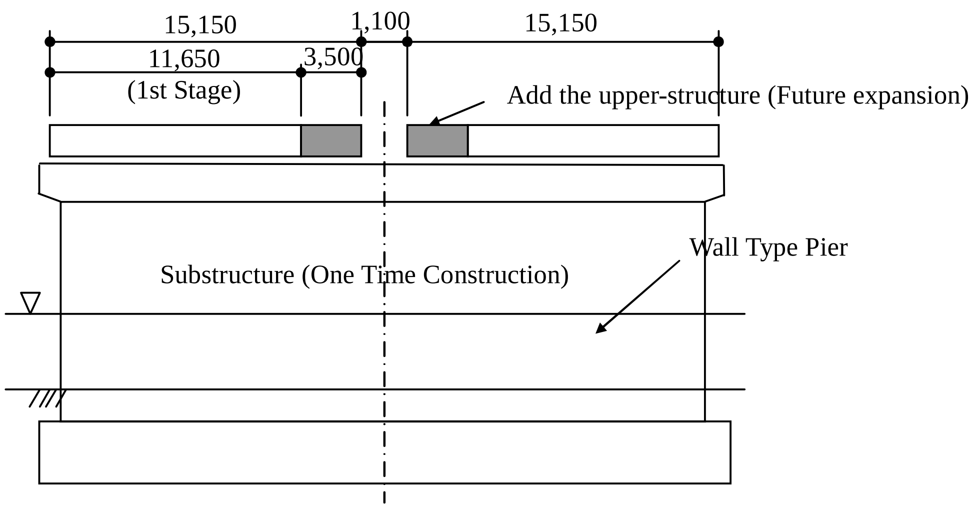


Fig. 7.1.8 Stage Construction of Bridge

7.1.6. Viaducts

All viaducts and ramp bridges/viaduct locations and lengths were decided following consideration of the soil conditions and social and environmental impacts during earthwork construction, not for structural reasons.
Viaduct-1 to 4 (V1 – V4) for main-line and Viaduct-5 to 10 (V5 – V10) for ramps were planned (as shown in **Table 7.1.1**). The main points of decision for those viaducts were;

- Huge volume of embankment and soft ground treatment materials are necessary if A1 Interchange (V1, V5 to V8 area) is planned to be constructed by earthwork scheme. However, those materials do not exist near the area. Therefore the social environmental impacts are disadvantageous, such as high noise, vibration to houses neighboring the construction site and reduced life to the haul road due to high numbers of carrying trucks,
- Flood plain area of Kelani River (V2 to V5, V9 and V10 area) has deep soft soils – comprising peat and/or organic clay/sand. Replacement methods such as soft ground treatment are needed for such types of soils, and the volume of replacement materials is large. Therefore also adverse environmental impacts will occur during construction if earthwork scheme is selected here.

The detailed discussion is mentioned in Chapter 3.

Bridge Type

3 bridge types and 11 alternatives were compared to decide the standard type of Viaduct for main-line. The comparison result is shown in **Fig. 7.1.9** & **7.1.10**, and **Table 7.1.11**.

PC I-Girder, 35m length was selected as the most economical.

This scheme – utilizing pre-fabricated girder is the most suitable in the "shallow soft deposit" soil conditions for this Project.

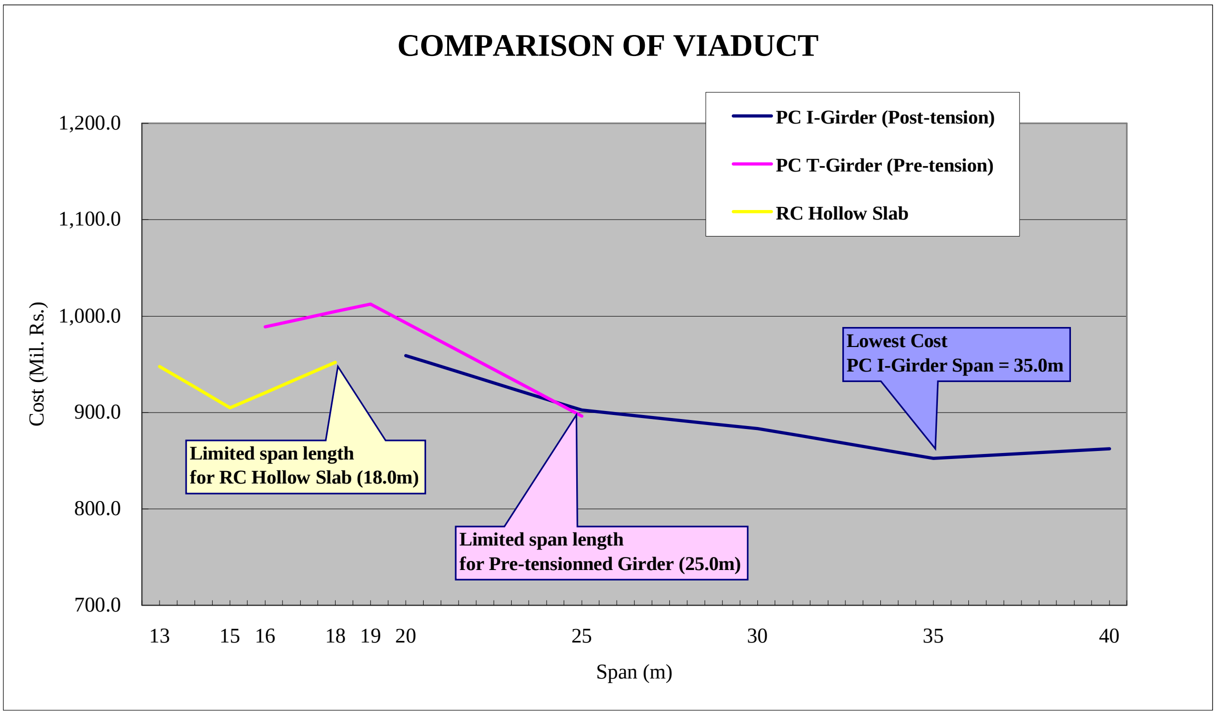
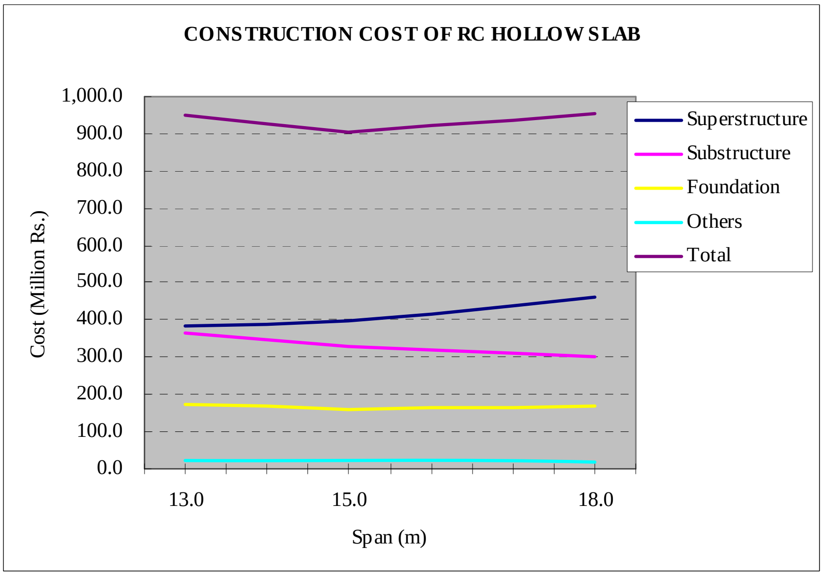


Fig.7.1.9 Type Comparison Result for Viaduct (Main-Line)



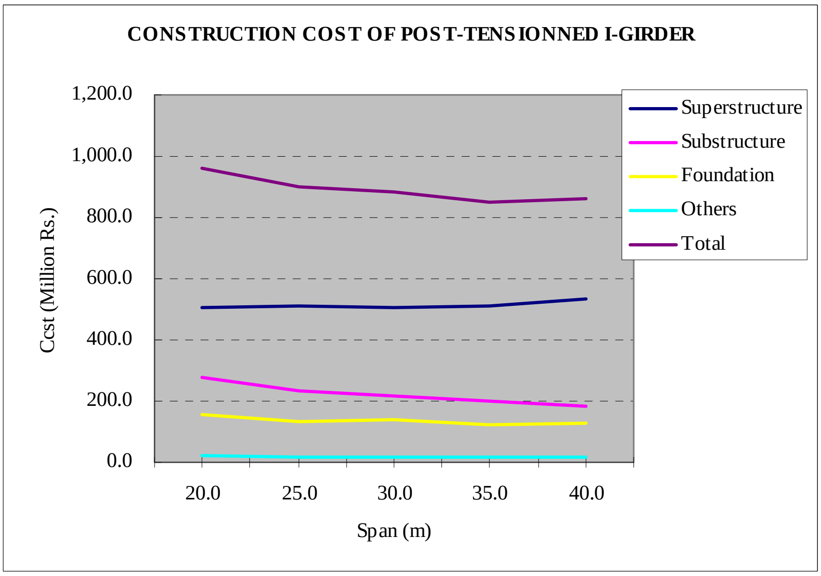


Fig. 7.1.10 Type Comparison Result for Viaduct (Main-Line: Component)

Table 7.1.11 Type Comparison for Viaduct (Main-Line: 1/2)

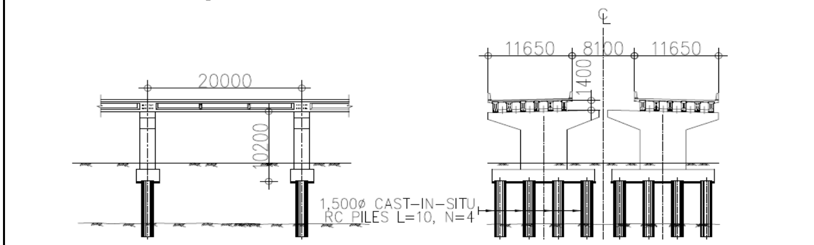
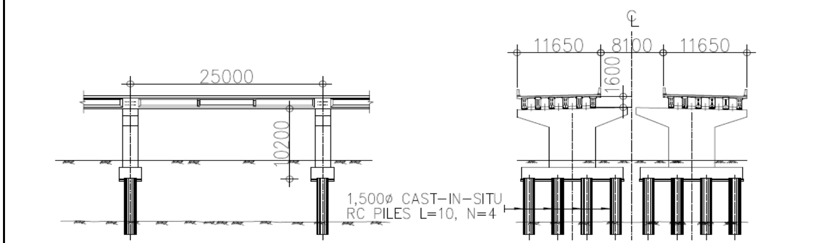
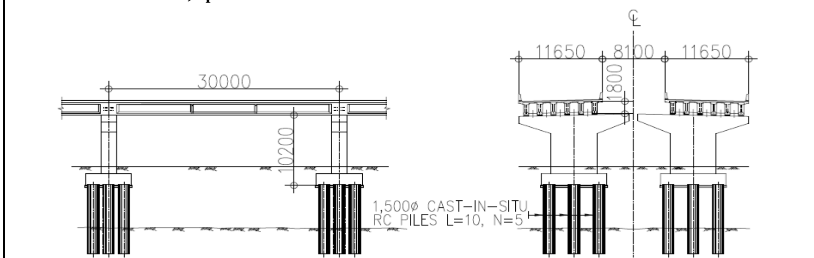
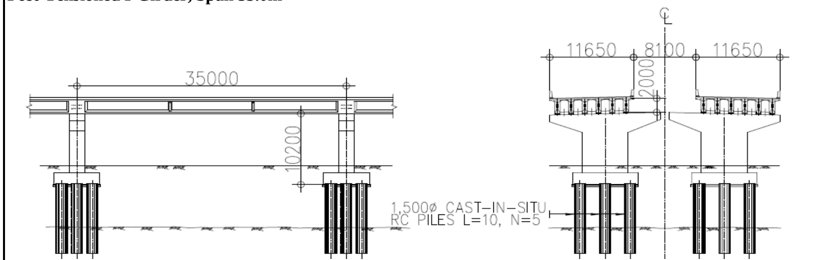
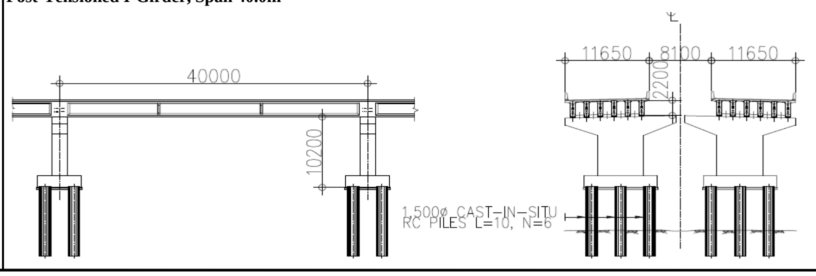
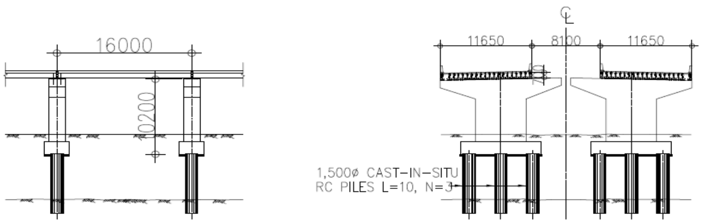
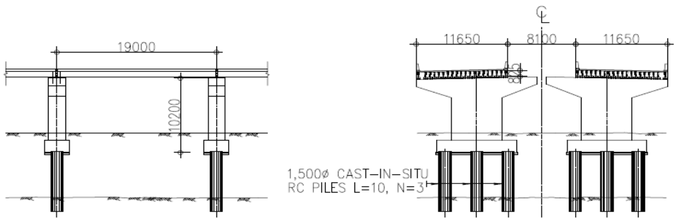
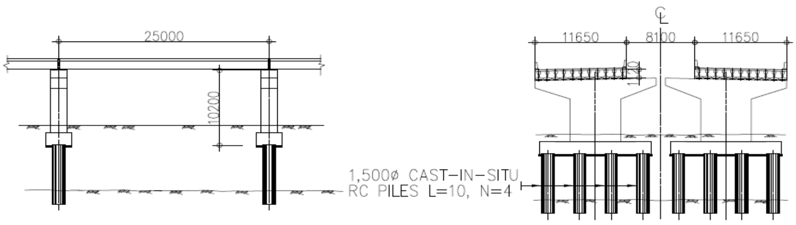
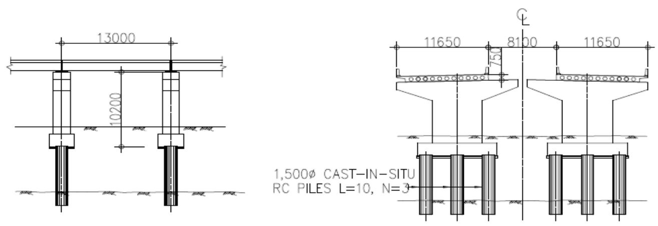
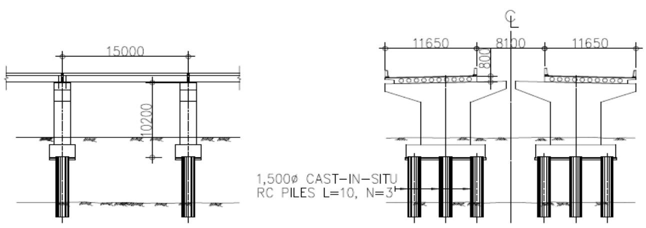
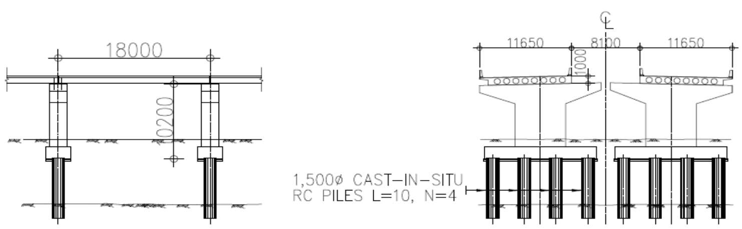
Alternative	**General View**	**Construction Cost <Million Rs., per 300m> (Rate)**		**Judge**
1	**Post-Tensioned I-Girder, Span 20.0m** 	Superstructure	504.6	
		Substructure	277.1	
		Foundation	157.3	
		Others	20.1	
		Total	**959.1**	
		< 148.7 Th/sq.m.>		
			(1.125)	
2	**Post-Tensioned I-Girder, Span 25.0m** 	Superstructure	511.6	
		Substructure	235.9	
		Foundation	135.8	
		Others	19.1	
		Total	**902.4**	
		< 139.9 Th/sq.m.>		
			(1.059)	
3	**Post-Tensioned I-Girder, Span 30.0m** 	Superstructure	507.9	
		Substructure	219.3	
		Foundation	137.6	
		Others	18.6	
		Total	**883.4**	
		< 137.0 Th/sq.m.>		
			(1.036)	
4	**Post-Tensioned I-Girder, Span 35.0m** 	Superstructure	511.8	
		Substructure	197.5	
		Foundation	124.8	
		Others	18.2	
		Total	**852.3**	**Applied**
		< 132.1 Th/sq.m.>		
			(1.000)	
5	**Post-Tensioned I-Girder, Span 40.0m** 	Superstructure	533.9	
		Substructure	184.0	
		Foundation	126.7	
		Others	18.0	
		Total	**862.6**	
		< 133.7 Th/sq.m.>		
			(1.012)	

Table 7.1.11 Type Comparison for Viaduct (Main-Line: 2/2)

Alternative	General View	Construction Cost <Million Rs., per 300m> (Rate)		Judge
6	**Pre-Tensioned Reversed T-Girder, Span 16.0m** 	Superstructure	505.9	
		Substructure	309.0	
		Foundation	152.8	
		Others	21.1	
		Total	**988.8**	
		< 153.3 Th/sq.m.>		
			(1.160)	
7	**Pre-Tensioned Reversed T-Girder, Span 19.0m** 	Superstructure	566.7	
		Substructure	288.9	
		Foundation	136.8	
		Others	20.1	
		Total	**1,012.5**	
		< 157.0 Th/sq.m.>		
			(1.188)	
8	**Pre-Tensioned Reversed T-Girder, Span 25.0m** 	Superstructure	491.4	
		Substructure	249.7	
		Foundation	136.0	
		Others	19.1	
		Total	**896.2**	
		< 138.9 Th/sq.m.>		
			(1.052)	
9	**RC Hollow Slab, Span 13.0m** 	Superstructure	382.5	
		Substructure	367.2	
		Foundation	175.5	
		Others	22.5	
		Total	**947.7**	
		< 146.9 Th/sq.m.>		
			(1.112)	
10	**RC Hollow Slab, Span 15.0m** 	Superstructure	396.5	
		Substructure	328.0	
		Foundation	159.0	
		Others	21.4	
		Total	**904.9**	
		< 140.3 Th/sq.m.>		
			(1.062)	
11	**RC Hollow Slab, Span 18.0m** 	Superstructure	459.6	
		Substructure	303.4	
		Foundation	168.8	
		Others	20.3	
		Total	**952.1**	
		< 147.6 Th/sq.m.>		
			(1.117)	

For the ramp bridge, three (3) kinds of structure were compared and PC I-Girder, 17.5m was selected for economical reasons (see **Table 7.1.12**). The length of 17.5m was decided in consideration of girder arrangement for curve effect.

Table 7.1.12 Cost Comparison for Viaduct Type (Ramp)

Alternative	Construction Cost (Million Rs.)		Judge
1. Post-Tensioned I-Girder, Span 17.5m	Superstructure	62.0	**Applied**
	Substructure	28.5	
	Foundation	20.0	
	Others	4.3	
	Total	**114.8**	
	Unit Cost (Thousand/sq.m.)	**133.9**	
	Rate	**1.000**	
2. Pre-Tensioned Reversed T-Girder, Span 17.5m	Superstructure	61.7	
	Substructure	30.2	
	Foundation	20.0	
	Others	4.2	
	Total	**116.1**	
	Unit Cost (Thousand/sq.m.)	**135.4**	
	Rate	**1.011**	
3. RC Hollow Slab, Span 17.5m	Superstructure	57.8	
	Substructure	31.6	
	Foundation	26.2	
	Others	4.2	
	Total	**119.8**	
	Unit Cost (Thousand/sq.m.)	**139.7**	
	Rate	**1.044**	

Control Point/ Objects

Required major crossing control points and/or objects are;

- A1 Bypass for Viaduct-1
- Re-routed River – Mdun Ela (Kelani River Tributary) for Viaduct-4

A1 Bypass Crossing (Viaduct-1: V1)

A1 Bypass at STA. 8+667.0 for main-line was newly planned and designed in this Project. Viaduct-1 is crossed over this A1 Bypass.

Cross section of A1 Bypass at the bridge location is shown in **Fig. 7.1.10**.

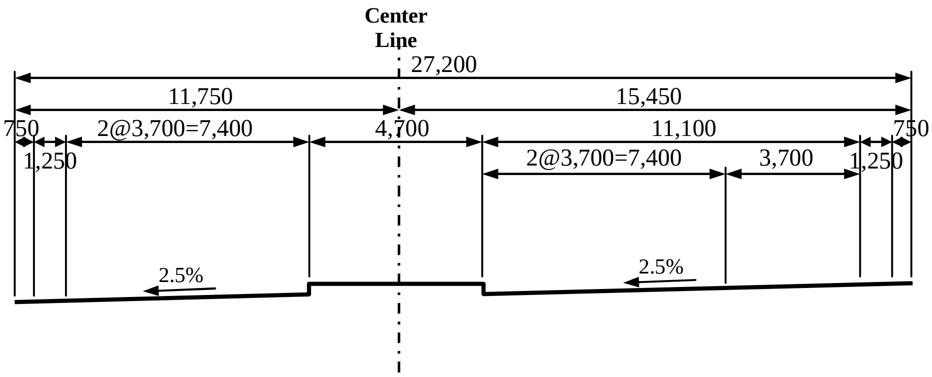


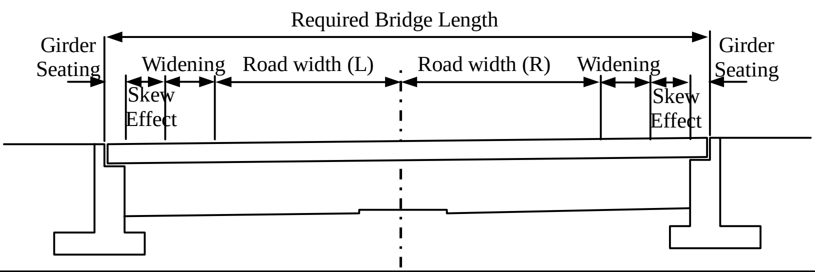
Fig. 7.1.10 Cross Section of A1 Bypass

Required Bridge Length

A1 Bypass has been planned for future expansion by the adding of an additional 3.0m lane in each direction. To allow for this pavement widening in the future, the total required bridge length shall be 38.20 m. as shown in **Table 7.1.13**.

Table 7.1.13 Required Bridge Length

	Left Side (A1 Abutment side)	**Right Side (A2 Abutment side)**	**Total**
Road width	**11.75**	**15.45**	**27.20**
Road widening (Future Expansion)	**3.00**	**3.00**	**6.00**
Girder seating	**1.00**	**1.00**	**2.00**
Additional space for Curve	**1.50**	**1.50**	**3.00**
Total	**17.25**	**20.95**	**38.20**



Type of Superstructure

The type of superstructure for a 38.20 m bridge length can be either;

- PC I-Girder (1 span or 2 span)
- PC Box Girder (1 span)

Required superstructure depth shall not exceed 2.2m, based on A1 Bypass alignment, the required vertical clearance of 5.1m and OCH main-line vertical alignment.

The depths(D) of the above nominated superstructures are;

- PC I-Girder, 1 span, 38.2m length: D=2.5m or greater
- PC I-Girder, 2 span, 18+21m length: D=2.0m
- PC Box Girder, 1 span, 38.2m length: D=3.0m or greater

Therefore, 2 span PC I-Girder bridge was selected at this section.

Span Arrangement

A 2 span (18.0m + 21.0m) bridge was selected to minimize the length and in consideration of the median shape and angle of A1 Bypass. Center pier skew of 5 degrees was adopted to match the crossing angle of the main-line.

Substructure and Foundation

The heights of A1 abutment and P1 pier were determined to be 12.0m and 10.2m respectively. The pile-caps for these structures were embedded 30cm minimum into the existing ground in order not to be projection piles, to follow the Bridge Design Standard.

Cast-in-situ concrete bored pile with 1.5 m diameter was adopted as the foundation because of deep assumed bearing stratum, and the pile length were determined to be 10 ~ 12 m based on the soil investigation result.

Re-routed River – Mudun Ela (Kelani River Tributary) Crossing (Viaduct-4: V4)

The bridge crossing over a tributary river of Kelani River named “Mudun Ela” and a narrow path near the river was planned at STA. 15 + 912.5.

Required Width of River

Mudun Ela crosses the Project at STA. 15 + 900. In the hydrological study, the disturbance effect on the river flow, especially during flood, due to the embankment of OCH was considered, and re-location with conservative section was requested by the Department of Irrigation (DOI).

The planned river section at this point is shown in **Fig. 7.1.11**.

Clearance shall be provided under the bridge for the high flood level (H.F.L.) plus freeboard. The elevation required to meet this requirement is; + 8.20 m (H.F.L.) + 0.60 m (Freeboard) = + 8.80 m M.S.L.

Therefore the elevation of the soffit of the girder elevation shall be at least + 8.80 m M.S.L.

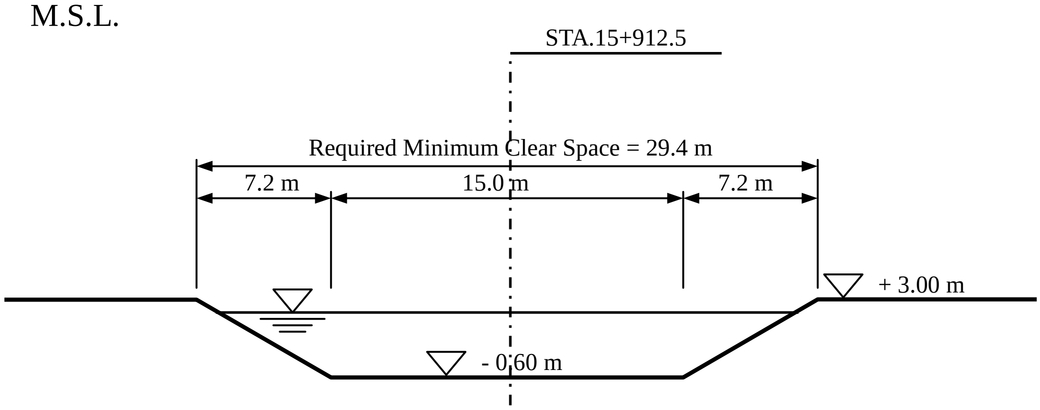


Fig. 7.1.11 Cross Section of Diverted River (Mudun Ela)

Bridge Length

For bridge length a required minimum clear space for the river and path way for maintenance are necessary. Standard span of 35m for viaduct can be applied.

Substructure and Foundation

Based on the soil investigation result, bearing stratum exists at 3 to 5m depth at the right bank side (beginning side), and from 7 to 10m depth in the other side. Therefore spread footing is applied to right bank side – Abutment-1 and piled foundation applied to other side – Pier-1.

Individual Viaduct Characteristics

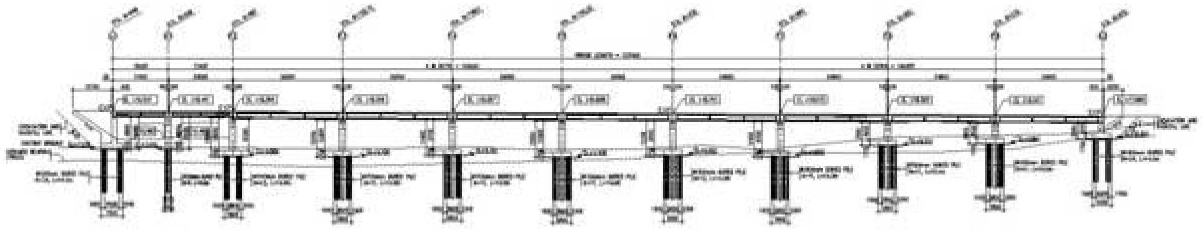
Individually, all viaducts' characteristics are introduced below:

Viaduct-1 (V1: STA. 8+648 – 8+970)

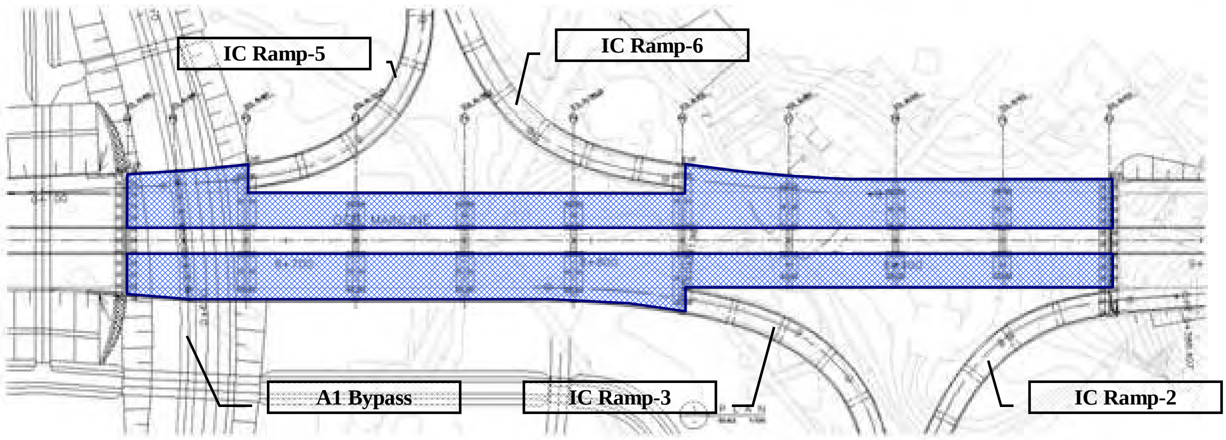
Characteristics:

- Viaduct-1 (V1) is located at/around A1 Interchange area.
- The shape of bridge surface is complicated due to tapering of on/off interchange ramps and the acceleration/de acceleration lanes widening.
- A1 Bypass is crossed by OCH main-line at STA. 8+667. The minimum span length and maximum depth of upper-structure are 39m and 2.0m respectively.
- Based on the results of the economic comparison a uniform 35m span length was used throughout, except for A1 Bypass crossing section.
- 2 span continuous deck upper-structure – monolithic structure was used atop launched girders.
- Span arrangement: (18.0+21.0)+4@35.75+4@35.0=322.0m
- Wall type (T-shape) piers with cantilevered coping and reversed T-type abutments were used.
- Although assumed bearing stratum is not deep (5 to 10m below the existing ground level), pile foundation was applied to ensure structural stability and meet safety considerations due to the lack of soil investigation data.

Plan & Profile:



Profile



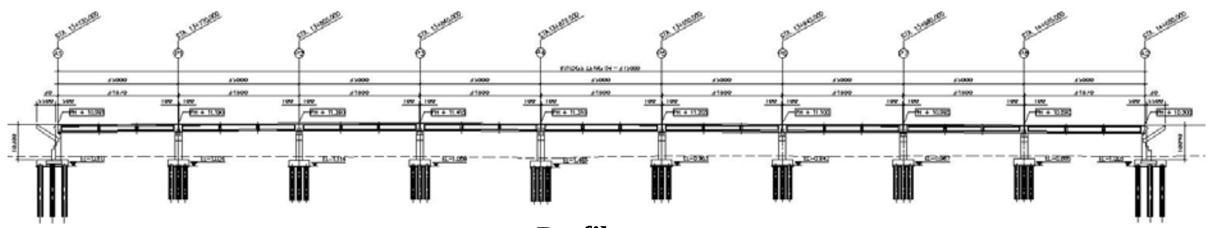
Plan

Viaduct No. 2 (V2: STA. 13+755 – 14+070) & Viaduct No. 3 (V3: STA. 15+130 – 15+445)

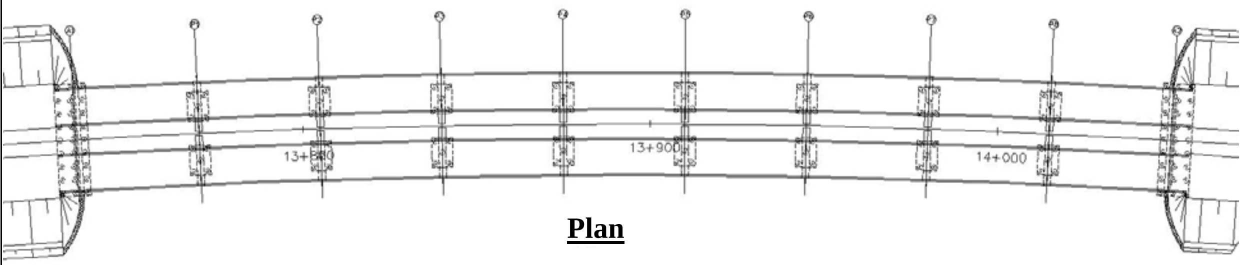
Characteristics:

- Both Viaduct-2 (V2) and Viaduct-3 (V3) locations were determined due to the soil conditions. Soft soil, such as organic clay and/or peat is distributed approximately 5~6 m deep in those areas. Viaduct type was chosen to reduce embankment fill material requirements, minimize filling material for soft soil countermeasure and to reduce the social environmental impacts of heavy equipment/vehicle traffic during construction.
- Based on the economical comparison result, 35m span length was used.
- 3 span continuous upper-structure – monolithic structure after launching girders was used.
- Span arrangement: 9@35.0=315.0m
- Wall (T-shape) type pier with cantilevered coping and reversed T-type abutment were used.
- Assumed bearing stratum for foundation is 10 to 15m below the existing ground level therefore pile foundation was used.
- Both V2 and V3 are located within a curve of 2,000m radius

Plan & Profile:



Profile



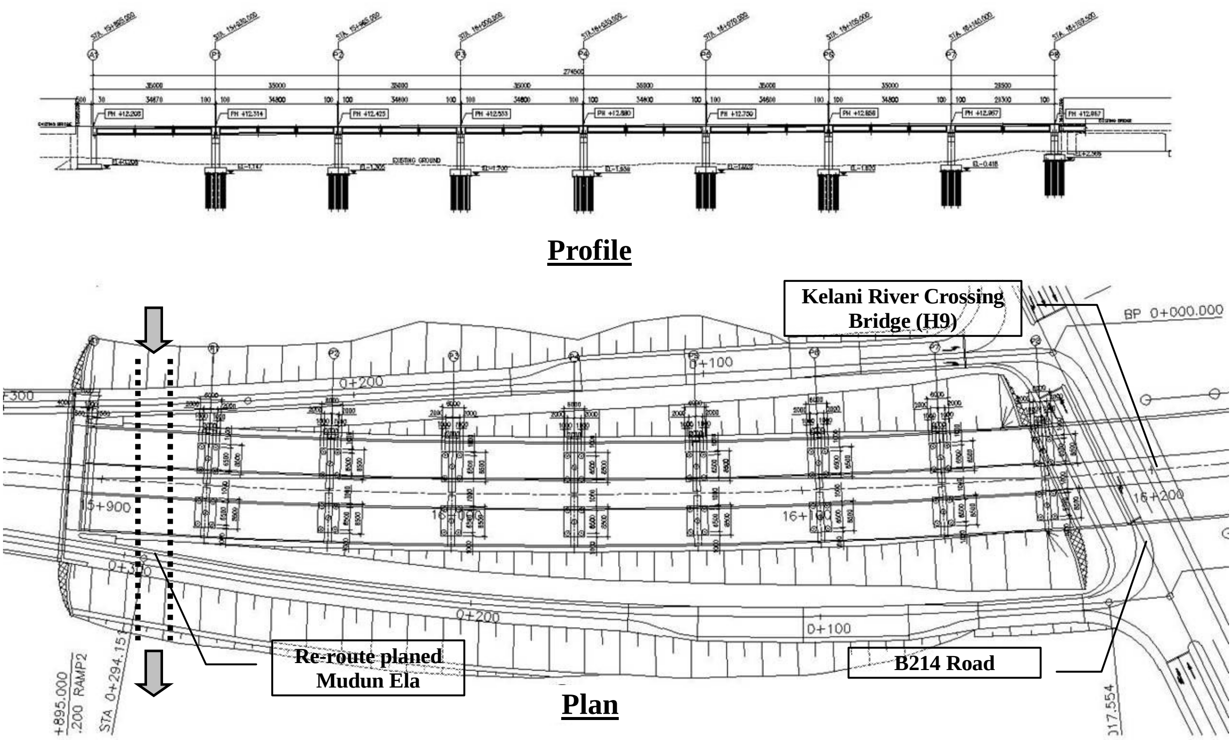
Plan

Viaduct No. 4 (V4: STA. 15+895 – 16+169.5)

Characteristics:

- Re-routing Mudun Ela (Kelani River tributary) is located at the first span of this viaduct.
- V4 connects to the Kelani River Crossing Bridge (H9) above the B214 road.
- The area has rough ground, meandering river and marsh. Soft soil is present throughout this area and assumed bearing stratum is deep (10 to 15m). Therefore pile foundations were applied, however A1 Abutment used spread type because the bearing stratum is shallow.
- Based on the results of the economic comparison a 35m span length was used
- 2 or 3 span continuous upper-structure – monolithic structure after launching girders was used.
- Span arrangement: 7@35.0+29.5=274.5m
- Wall type (T-shape) pier with cantilevered coping and reversed T-type abutment were used.
- Located within a curve of 2,000m radius.

Plan & Profile:



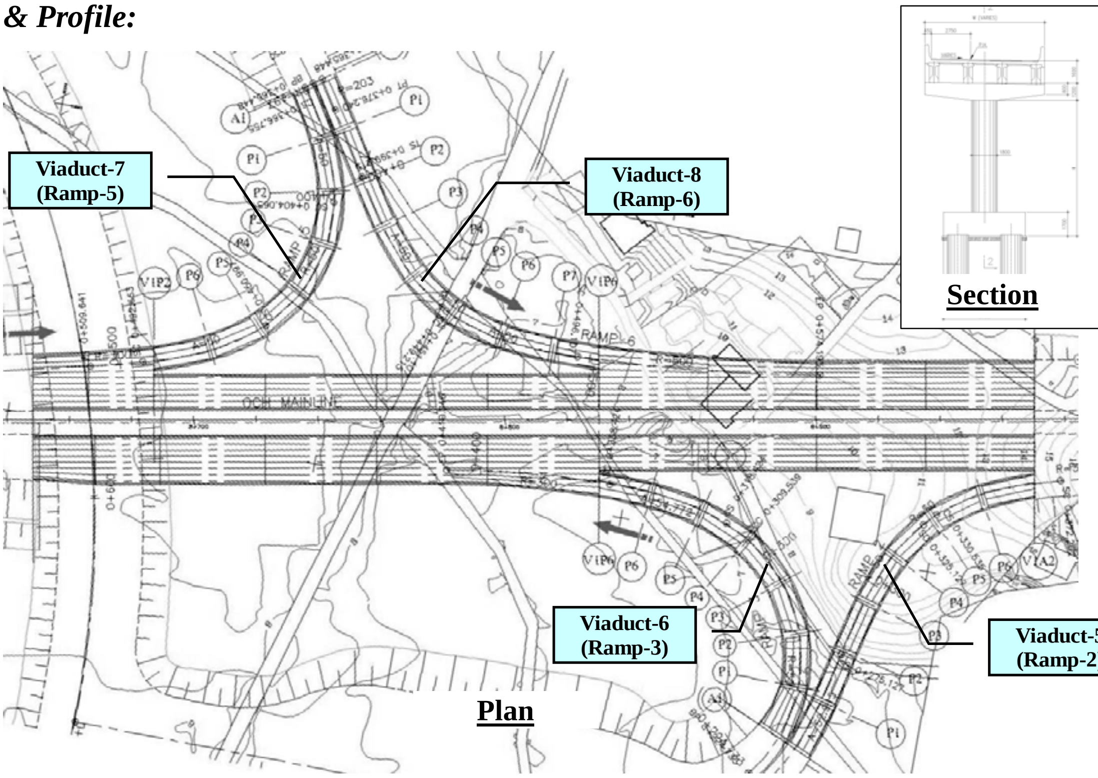
A1 Interchange Ramps

- Viaduct-5 (V5: STA. 0+241.285 – 0+363.785): Ramp-2 Bridge of A1 Interchange
- Viaduct-6 (V6: STA. 0+239.766 – 0+361.835): Ramp-3 Bridge of A1 Interchange
- Viaduct-7 (V7: STA. 0+365.448 – 0+489.070): Ramp-5 Bridge of A1 Interchange
- Viaduct-8 (V8: STA. 0+365.448 – 0+503.019): Ramp-6 Bridge of A1 Interchange

Characteristics:

- To limit the intrusion into surrounding residences all ramps are located in limited space and thus compact shape for ramps was applied. Therefore, all ramp bridges also have sharp curve of 50m radius.
- Based on the results of the economic comparison a PC I-Girder bridge was selected and 17.5m span length was decided in consideration of the sharp curve effect.
- Soil conditions are similar to V1 and assumed bearing stratum is deep. The soil conditions at the V5 area could not be investigated due to opposition by nearby residents. Therefore, pile foundation was applied in all ramp viaducts.
- Monolithic construction to connect substructure and superstructure was applied to meet the structural durability and integrity of the curved bridge.
- Span arrangement: V5: 7@17.50=122.5m
  V6: 6@17.50+17.069=122.069m
  V7: 6@17.50+18.622=123.622m
  V8: 7@17.50+15.071=137.571m
- Circular column with cantilevered coping type (T-shape) pier was used to for aesthetic reasons and reversed T-type abutment was used.

Plan & Profile:



Plan

Section

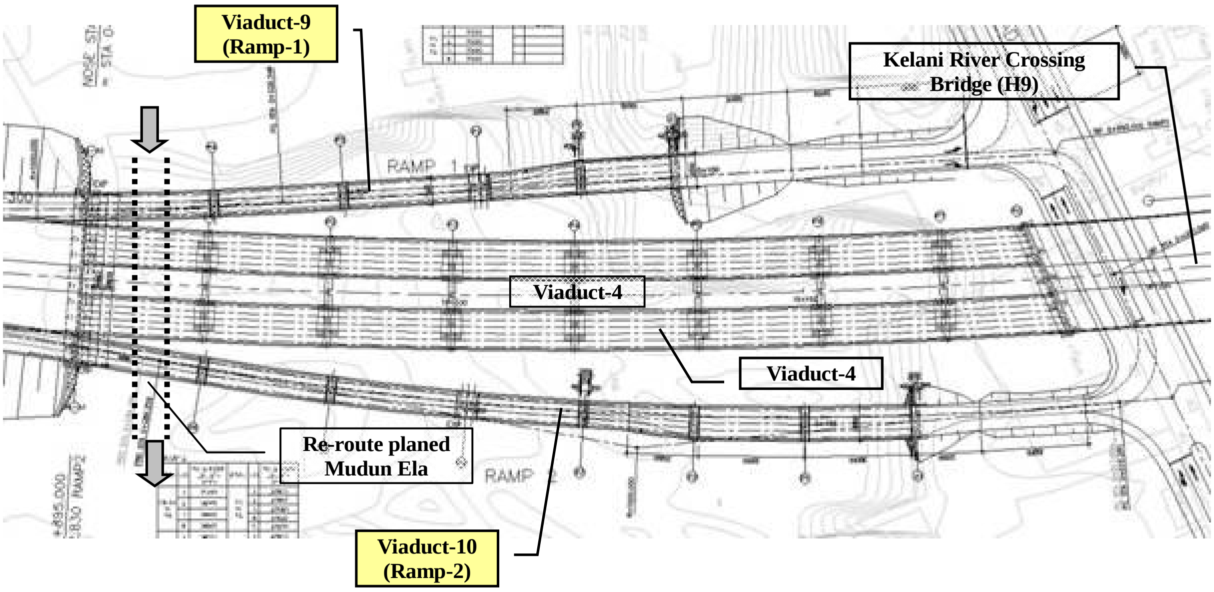
B214 Interchange Ramps

- Viaduct-9 (V9: STA. 0+116.855 – 0+275.655): Ramp-1 Bridge of B214 Interchange
- Viaduct-10 (V10: STA. 0+78.930 – 0+312.830): Ramp-2 Bridge of B214 Interchange

Characteristics:

- The soil conditions are similar to V4 so pile foundation was applied.
- Some spans may be submerged during flood of 50 years return period. Thus the bridge type adopted PC I-Girder without bearing pad, with girder and pier-head directly connected (rigid frame structure).
- The span length followed V4 arrangement for uniformity and the adjusted aesthetic considerations and 35m span length was used.
- Monolithic construction method between substructure and superstructure type was applied to minimize the maintenance and in consideration of structural durability and characteristics. Especially, for maintenance, bearing shoe was not used as possibly up to half of the bridge structure could be submerged during 50 year flood.
- Span arrangement: V9: 2@25.0+3@35.0
  V10: 4@29.5+3@35.0
- Circular column with cantilevered coping type (T-shape) pier was used for aesthetic reasons and reversed T-type abutment was used.

Plan & Profile:



<u>Plan</u>

7.1.7. Overpass Bridges

General

The total number of Overpass Bridges on OCH-N1 is five (5). The bridge lengths range from 42m to 50m, and 2-span continuous type structure (monolithic girders and slabs) was applied to follow the OCH-S design. Also reversed-T type abutment and wall type pier were applied.

In accordance with the basic policy for bridge length developed in the feasibility study for OCH (Year 2000), the minimized length has been decided although this means the abutments will be quite high and relatively costly. This approach was adopted in this Study also.

Individual Characteristics

Overpass Bridge-8 (O8: STA. 9+415)

- Road class is “C” and named “Gonahena Ihala Biyanwila Road”,
- The only straight bridge for OCH main line (crossing angle: 90 degrees),
- The approach is embankment, mass concrete retaining walls were used beside both abutments to reduce/minimize the wing wall length,
- Pile foundation was applied because the assumed bearing stratum is 15-20m depth,
- Based on latest advice (June, 2007) from National Water Supply and Drainage Board (NWS&DB) and Sri Lanka Telecom Ltd. (SLT), it is necessary to carry a water pipe of 110mm dia. PVC at the edge of bridge surface (**Fig. 7.1.14**) and provision is to be made for 2 nos. of 115mm dia. Cables (**Fig. 7.1.15**) under side -walk.

Profile and plan are shown in **Fig. 7.1.17** below;

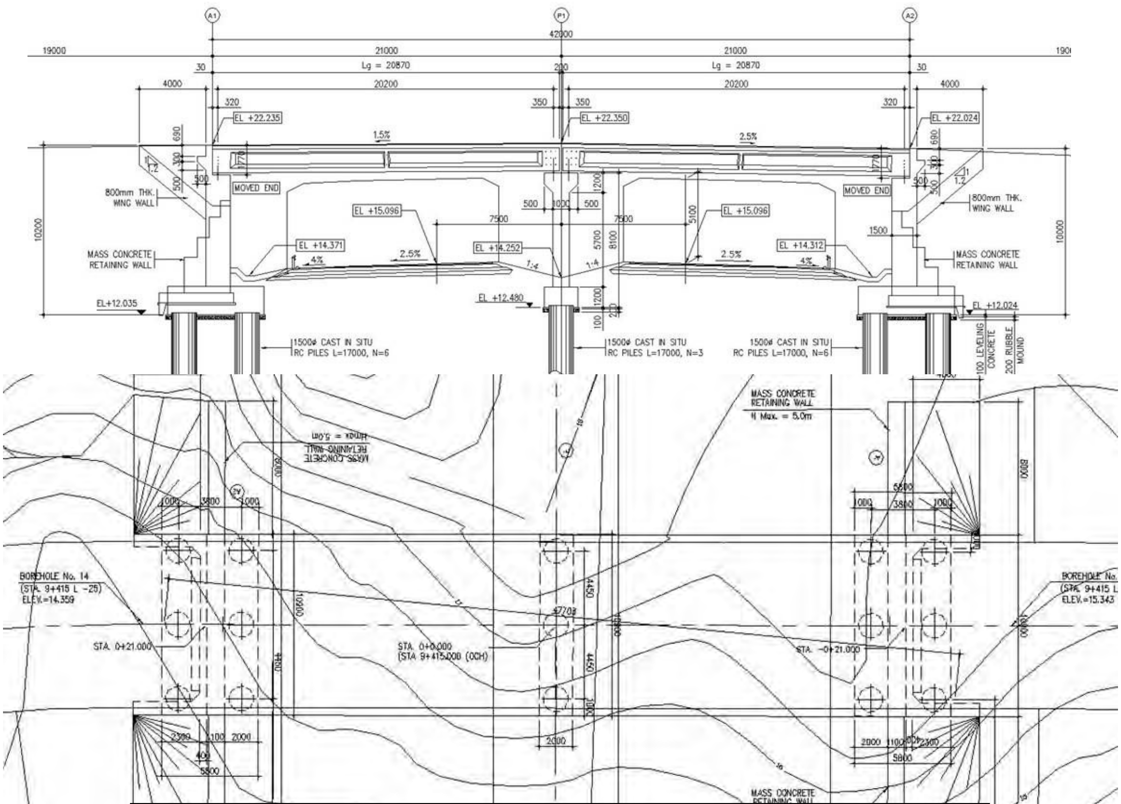


Fig. 7.1.12 Profile & Plan of Overpass Bridge-8 (O8)

Overpass Bridge-9 (O9: STA. 10+210)

- Road class is “B3” and named “B169 Road”,
- Located at a large cut section with differential elevation height between OCH main line and surface of O9 of approximately 15m so that high abutments of 12.5 to 13.5m height were needed. Also long wing walls were needed and the lengths are 14 and 11m,
- O9 has quite steep skew angle of 62 degrees to OCH main line so that bridge length (50.0m) became longer than a 90 degree crossing.
- Assumed bearing stratum is about 10m below the existing ground. However, the cutting depth is also approximately 10m and thus spread foundation was adopted,
- Based on the latest advice (June, 2007) from NWS&DB and SLT, it will be necessary to carry a water pipe of 160mm dia. PVC at the edge of bridge surface (**Fig. 7.1.14**) and to provide for 8 nos. of 115mm dia. Cables (**Fig. 7.1.15**) under side-walk.

Overpass Bridge-10 (O10: STA. 11+261)

- Road class is “B3” and named “B401 Road”,
- Bridge surface shape followed the existing road with 145m radius curve based on the highway design policy of this Study Team. However the straight PC I-girder was used,
- Soil condition is suitable for spread foundation,
- Based on the latest advice (June, 2007) from NWS&DB and SLT, it will be necessary to carry a water pipe of 300mm dia. DI at the edge of bridge (see **Fig. 7.1.13**) and to provide for 4 nos. of 115mm dia. Cables (**Fig. 7.1.15**) under side-walk.

Overpass Bridge No-11 (O11: STA. 12+518.921)

- Road class is “B2” and named “Sapugaskanda – Biyagama Road”,
- Bridge has angle of 71^{o}-04’-23” to OCH main line and the length is 45.0m,
- Assumed bearing stratum exists approximately 7m below the ground which is almost the same level as the OCH road. Therefore spread foundation was used,
- Based on the latest advice (June, 2007) from NWS&DB and SLT, it will be necessary to carry a water pipe of 450mm dia. DI at the edge of bridge (see **Fig. 7.1.13**) and to provide for 8 nos. of 115mm dia. Cables (**Fig. 7.1.15**) under side-walk.

Overpass Bridge No-12 (O12: STA. 14+843)

- Road class is “C” and named “Jayanthi Mawatha”,
- Bridge has angle of 72 degrees to OCH main line and the length is 45.0m,
- Assumed bearing stratum exists deeper than 20m below the ground surface therefore pile foundation was used,
- Based on the latest advice (June, 2007) from NWS&DB and SLT, it will be necessary to carry a water pipe of 110mm dia. PVC at the edge of bridge surface (see **Fig. 7.1.14**) and to provide for 2 nos. of 115mm dia. Cables (**Fig. 7.1.15**) under side-walk.

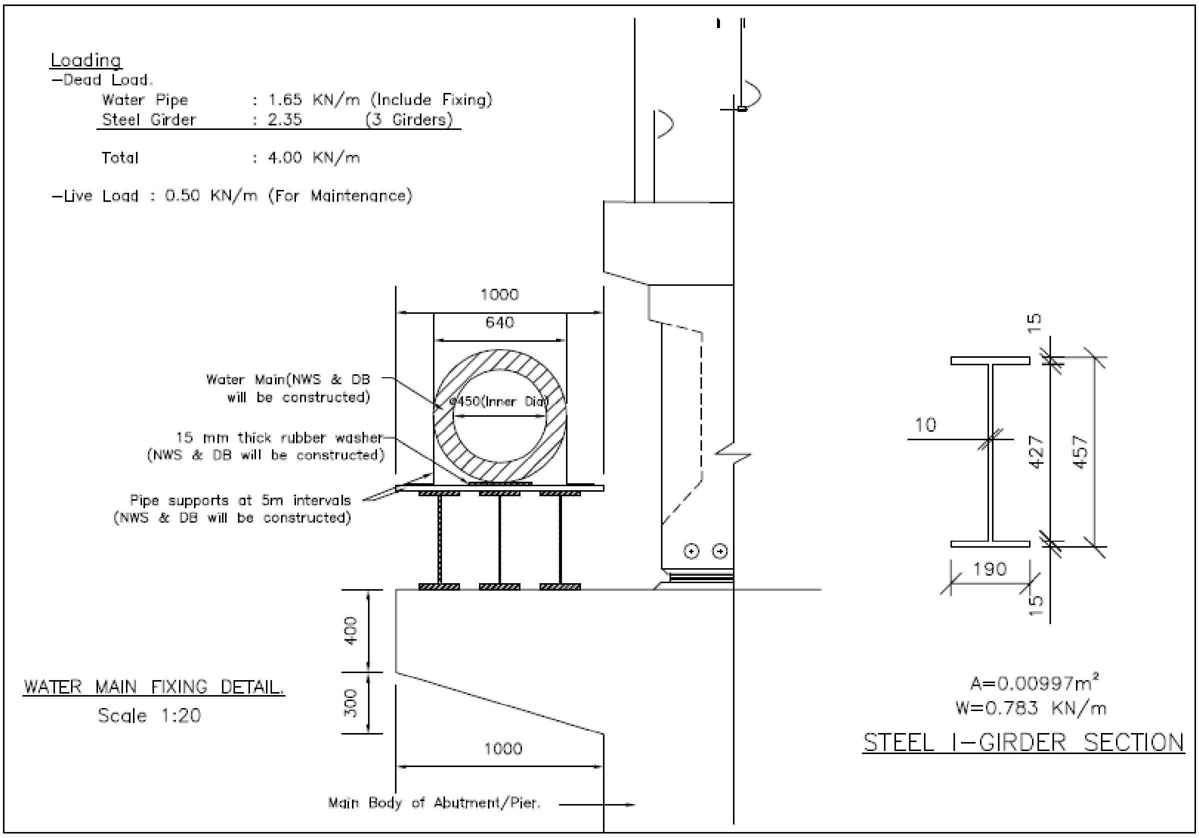


Fig. 7.1.13 Water-Pipe Fixing Details (For Large Pipe)

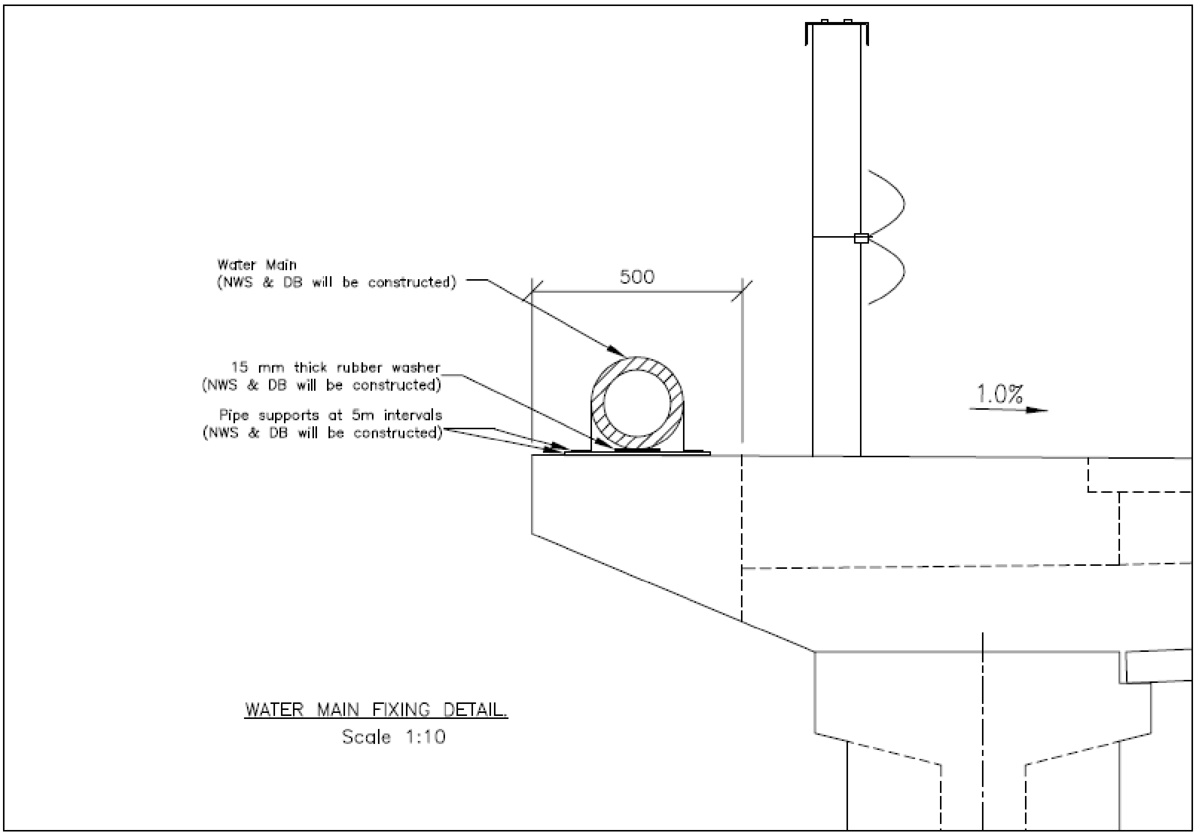


Fig. 7.1.14 Water-Pipe Fixing Details (For Small Pipe)

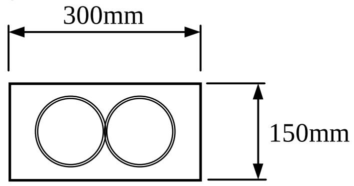
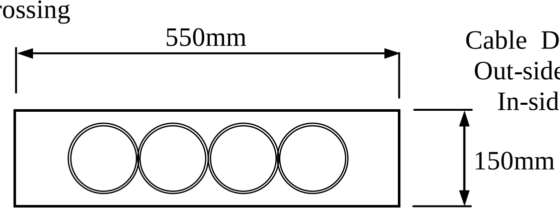
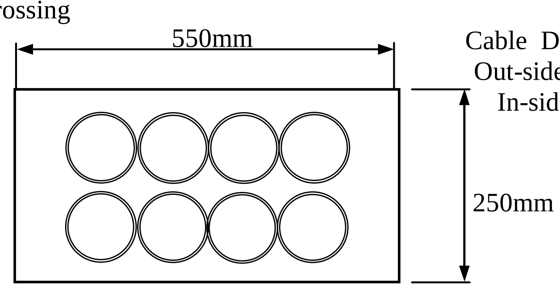
Bridge ID (STA.)	Cable Arrangement	Remarks
O8 (9+415) O12 (14+843)	2 Way Crossing  Cable Diameter Out-side: 115mm In-side: 100mm	
O10 (11+261)	4 Way Crossing  Cable Diameter Out-side: 115mm In-side: 100mm	
O9 (10+210) O11 (12+518.921)	8 Way Crossing  Cable Diameter Out-side: 115mm In-side: 100mm	

Fig. 7.1.15 Telecom Cables and the Application Bridges

7.1.8. Detailed Computation Results

Summary of the detailed computation results for Kelani River Crossing Bridge (H9) and 5 Overpass Bridges of the OCH-N1 are shown in **Tables 7.1.14** to **7.1.21**.

Table 7.1.14 Detailed Computation Results (1/8) – Kelani River Crossing Bridge <Superstructure>

Member	Limit state	Item				Allowable value	Unit	H9					
Girder	SLS	Bending moment (Center of span)	Concrete stress	Immediately after prestressing	Top of girder	-1≦	N/mm^2	10..14					
					Bottom of girder	≦0.5fci=0.5*36=18.0	N/mm^2	5.37					
				At the time of pouring slab concrete	Bottom of girder	≦0.4fcu=0.4*50=20.0	N/mm2	11.15					
				Working stage	Top of slab	≦0.5fcu=0.5*35=17.5	N/mm^2	-1.78					
					Top of girder	≦0.4fcu=0.4*50=20.0	N/mm^2	2.39					
					Bottom of girder	$-0.36\sqrt{fcu}=-0.36\sqrt{50}=-2.55$≦	N/mm^2	6.19					
			PC tendon stress	At the time of prestressing	Centroid of PC tendon	≦1,440	N/mm^2	1276.60					
				Immediately after prestressing		≦1,295	N/mm^2	1137.01					
				Working stage		≦1,110	N/mm^2	1029.99					
	ULS	Bending moment (Center of span)	Moment (M)				kN.m	10188.45					
			Resistant moment (Mr)			M≦Mr	kN.m	17466.87					
		Shear force (1/8 of span)	Shear force (Shu)				kN	1502.92					
			Resistant shear force for compression destruction (Suc)			Shu≦Suc	kN	7791					
			Resistant shear force for diagonal tensile destruction (Sus)			Shu≦Sus	kN	2212.45					
Cross Beam	SLS	Bending moment (Center of span)	Concrete stress	After composition	Plus moment: Top of slab	≦0.5fcu=0.5*35=17.5	N/mm^2	-0.05					
					Plus moment: Bottom of slab		N/mm^2	-0.04					
					Plus moment: Top of cross beam	≦0.4fcu=0.4*35=14.0	N/mm^2	-0.04					
					Plus moment: Bottom of cross beam		N/mm^2	0.70					
					Minus moment: Top of slab	$-0.36\sqrt{fcu}=-0.36\sqrt{35}=-2.13$≦	N/mm^2	0.03					
					Minus moment: Bottom of slab		N/mm^2	0.03					
					Minus moment: Top of cross beam	≦0.4fcu=0.4*35=14.0	N/mm^2	0.03					
					Minus moment: Bottom of cross beam		N/mm^2	-0.03					
			PC tendon stress	At the time of prestressing	Centroid of PC tendon	≦1,440	N/mm^2	1300.74					
				Immediately after the prestressing		≦1,260	N/mm^2	680.84					
				Working stage		≦1,080	N/mm^2	597.82					
	ULS	Bending moment (Center of span)	Plus moment		Moment (M)		kN.m	109.08					
					Resistant moment (Mr)	M (+)≦Mr (+)	kN.m	727.57					
			Minus moment		Moment (M)		kN.m	14.61					
					Resistant moment (Mr)	Mr (-)≦M (-)	kN.m	130.63					
Slab	SLS	Cracking	Crack width		Transverse	≦0.25 (Severe)	mm	-1.194					
			Concrete stress		Transverse	≦17.5	N/mm^2	16.85					
	ULS	Bending moment	Transverse		Lever arm	≦0.95		0.916					
					Moment	≦Mu=82.9	kN.m	49.65					
		Shear force	Ultimate shear stress			≦vc=0.97	N/mm^2	0.708					
		Punching shear	Ultimate shear force			≦Vc=422.1	kN	33.52					

Table 7.1.15 Detailed Computation Results (2/8) – Kelani River Crossing Bridge <Superstructure 1/3>

Member	Limit state	Item				Allowable value	Unit	H9							
								P0	P1	P2	P3	P4	P5	P6	P7
Pile	SLS	Stability analysis	Reaction of pile	Maximum	Reaction (Pmax)	Pmax≦Ra	kN	6772	67712	67712	6732	6732	6732	6732	67712
					Allowable bearing capacity (Ra)		kN	7061	7061	7061	7061	7061	7061	7061	7061
				Minimum	Reaction (Pmin)	Pa≦Pmin	kN	0	0	0	0	0	0	0	0
					Allowable uplift capacity (Pa)		kN	-1825	-1825	-1825	-1825	-1825	-1825	-1825	-1825
			Horizontal displacement			≦1.5	cm	0.3508	0.3508	0.3508	0.0090	0.0090	0.0090	0.0090	0.3508
	ULS	Stability analysis	Reaction of pile	Maximum	Reaction (Pmax)	Pmax≦Ra	KN	8858	8858	8858	9012	9012	9012	9012	8858
					Allowable bearing capacity (Ra)		KN	17475	17475	17475	17475	17475	17475	17475	17475
				Minimum	Reaction (Pmin)	Pa≦Pmin	KN	0	0	0	0	0	0	0	0
					Allowable uplift capacity (Pa)		KN	1825	1825	1825	1825	1825	1825	1825	1825
		Stress analysis	Axial force		Axial load (N)	N≦Nuz	KN	8858	8858	8858	9012	9012	9012	9012	8858
					Maximum axial load (Nuz)		KN	17475	17475	17475	17475	17475	17475	17475	17475
			Bending moment		Design ultimate moment (M)	M≦Mmax	kN.m	1733	1733	1733	910.44	910.44	910.44	910.44	1733
					Maximum bending moment (Mmax)		kN.m	6713	6713	6713	6713	6713	6713	6713	6713
Parapet	SLS	Stress analysis	Back face		Concrete	≦0.5fcu=0.5*30=15.0	N/mm²								
					Reinforcement	≦0.75fy=0.75*460=345.0	N/mm²								
		Cracking	Back face		Crack width	≦0.25 (Severe)	mm								
	ULS	Stress analysis	Bending moment	Back face	Design ultimate moment (M)	M≦Mmax	N.m								
							kN.m								
			Shear force		Shear stress (v)	≦0.75√fcu=0.75√30=4.11	N/mm²								
Wall / Column	SLS	Stress analysis	Concrete			≦0.5fcu=0.5*30=15.0	N/mm²	5.70	5.70	5.70	5.33	5.33	5.33	5.33	5.33
			Reinforcement			≦0.75fy=0.75*460=345.0	N/mm²	138	138	138	134	134	134	134	134
		Cracking	Crack width			≦0.25(Severe)	mm	-0.060	-0.060	-0.060	-0.069	-0.069	-0.069	-0.069	-0.069
	ULS	Stress analysis	Axial force		Axial load (N)	N≦Nuz	KN	50008	50008	50008	67125.45	67125.45	67125.45	67125.45	67125.45
			Bending moment		Design ultimate moment (M)	M≦Mmax	kN.m	26819	26819	26819	57614.51	57614.51	57614.51	57614.51	57614.51
			Shear force		Shear stress (v)	≦0.75√fcu=0.75√30=4.11	N/mm²	0.104	0.104	0.104	0.074	0.074	0.074	0.074	0.074
Pile-cap	SLS	Stress analysis	Toe		Concrete	≦0.5fcu=0.5*30=15.0	N/mm²	7	7	7	11	11	11	11	11
					Reinforcement	≦0.75fy=0.75*460=345.0	N/mm²	111	111	111	184	184	184	184	184
			Heel		Concrete	≦0.5fcu=0.5*30=15.0	N/mm²	7	7	7	11	11	11	11	11
					Reinforcement	≦0.75fy=0.75*460=345.0	N/mm²	111	111	111	184	184	184	184	184
	ULS	Stress analysis	Toe	Bending moment	Design ultimate moment (M)	M≦Mmax	kN.m	33761	33761	33761	75392	75392	75392	75392	75392
				Shear force	Shear stress (v)	≦0.75√fcu=0.75√30=4.11	N/mm²	0.50	0.50	0.50	0.61	0.61	0.61	0.61	0.61
			Heel	Bending moment	Design ultimate moment (M)	M≦Mmax	kN.m	33761	33761	33761	75392	75392	75392	75392	75392
				Shear force	Shear stress (v)	≦0.75√fcu=0.75√30=4.11	N/mm²	0.50	0.50	0.50	0.61	0.61	0.61	0.61	0.61

Table 7.1.16 Detailed Computation Results (3/8) – Kelani River Crossing Bridge <Superstructure 2/3>

Member	Limit state	Item				Allowable value	Unit	H9		
								P8	P9	A2
Pile	SLS	Stability analysis	Reaction of pile	Maximum	Reaction (Pmax)	Pmax≤Ra	kN	67712	67712	4169
					Allowable bearing capacity (Ra)		kN	7061	7061	6155
				Minimum	Reaction (Pmin)	Pa≤Pmin	kN	0	0	1337
					Allowable uplift capacity (Pa)		kN	-1825	-1825	1766
			Horizontal displacement			≤1.5	cm	0.3508	0.3508	0.3841
	ULS	Stability analysis	Reaction of pile	Maximum	Reaction (Pmax)	Pmax≤Ra	KN	8858	8858	5903
					Allowable bearing capacity (Ra)		KN	17475	17475	15187
				Minimum	Reaction (Pmin)	Pa≤Pmin	KN	0	0	1231
					Allowable uplift capacity (Pa)		KN	1825	1825	1766
		Stress analysis	Axial force		Axial load (N)	N≤Nuz	KN	8858	8858	7293
					Maximum axial load (Nuz)		KN	17475	17475	15187
			Bending moment		Design ultimate moment (M)	M≤Mmax	kN.m	1733	1733	1213
					Maximum bending moment (Mmax)		kN.m	6713	6713	6713
Parapet	SLS	Stress Analysis	Back face		Concrete	≤0.5fcu=0.5*30=15.0	N/mm²			5
					Reinforcement	≤0.75fy=0.75*460=345.0	N/mm²			100
		Cracking	Back face		Crack width	≤0.25 (Severe)	mm			0.05
	ULS	Stress Analysis	Bending Moment	Back face	Design ultimate moment (M)	M≤Mmax	kN.m			72.67
				Maximum bending moment (Mmax)			kN.m			198.8
			Shear force		Shear stress (v)	≤0.75√fcu=0.75√30=4.11	N/mm²			0.299
Wall / Column	SLS	Stress analysis	Concrete			≤0.5fcu=0.5*30=15.0	N/mm²	5.70	5.70	6.85
			Reinforcement			≤0.75fy=0.75*460=345.0	N/mm²	138	138	190
		Cracking	Crack width			≤0.25(Severe)	mm	-0.060	-0.060	0.04
	ULS	Stress analysis	Axial force		Axial load (N)	N≤Nuz	KN	50008	50008	-
			Bending moment		Design ultimate moment (M)	M≤Mmax	kN.m	26819	26819	2305.41
					Maximum bending moment (Mmax)		kN.m	-	-	3254.17
			Shear force		Shear stress (v)	≤0.75√fcu=0.75√30=4.11	N/mm²	0.104	0.104	0.299
Pile-cap	SLS	Stress analysis	Toe		Concrete	≤0.5fcu=0.5*30=15.0	N/mm²	7	7	4
					Reinforcement	≤0.75fy=0.75*460=345.0	N/mm²	111	111	74
			Heel		Concrete	≤0.5fcu=0.5*30=15.0	N/mm²	7	7	8
					Reinforcement	≤0.75fy=0.75*460=345.0	N/mm²	111	111	279
	ULS	Stress analysis	Toe	Bending moment	Design ultimate moment (M)	M≤Mmax	kN.m	33761	33761	26563
					Maximum bending moment (Mmax)		kN.m	-	-	73468
				Shear force	Shear stress (v)	≤0.75√fcu=0.75√30=4.11	N/mm²	0.50	0.50	0.36
			Heel	Bending moment	Design ultimate moment (M)	M≤Mmax	kN.m	33761	33761	35149
					Maximum bending moment (Mmax)		kN.m	-	-	48564
				Shear force	Shear stress (v)	≤0.75√fcu=0.75√30=4.11	N/mm²	0.50	0.50	0.36

Table 7.1.17 Detailed Computation Results (4/8) – Kelani River Crossing Bridge <Superstructure 3/3>

Member	Limit state	Item					Allowable value	Unit	H9					
									A2					
WING	SLS	Stress analysis	Section*		a	Concrete	$\leq$0.5fcu=0.5*30=15.0	N/mm^2	3.60					
						Reinforcement	$\leq$0.75fy=0.75*460=345.0	N/mm^2	83.00					
					b	Concrete	$\leq$0.5fcu=0.5*30=15.0	N/mm^2	5.90					
						Reinforcement	$\leq$0.75fy=0.75*460=345.0	N/mm^2	99.00					
		Cracking	Section*		a	Crack width	$\leq$0.25 (Severe)	mm	-0.18					
					b				0.11					
	ULS	Stress analysis	Bending moment	Section*	a	Design ultimate moment (M)	M$\leq$Mmax	kN.m	1810					
						Maximum bending moment (Mmax)		kN.m	5380					
					b	Design ultimate moment (M)	M$\leq$Mmax	kN.m	365.67					
						Maximum bending moment (Mmax)		kN.m	870.28					
			Shear force	Section*	a	Shear stress (v)	$\leq 0.75\sqrt{fcu}=0.75\sqrt{30}=4.11$	N/mm^2	1.679					
					b			N/mm^2	0.572					

Table 7.1.18 Detailed Computation Results (5/8) – Overpass Bridge <Superstructure>

Member	Limit state	Item					Allowable value	Unit	08	09	010	011	012	
Girder	SLS	Bending moment (Center of span)	Concrete stress	Immediately after prestressing		Top of girder	-1≦	N/mm^2	5.28	6.65	5.71	5.99	6.52	
						Bottom of girder	≦0.5fci=0.5*36=18.0	N/mm^2	3.63	1.91	3.32	3.17	2.75	
				At the time of pouring slab concrete		Bottom of girder	≦0.4fcu=0.4*50=20.0	N/mm2	6.22	7.50	6.76	7.07	7.47	
				Working stage		Top of slab	≦0.5fcu=0.5*35=17.5	N/mm^2	-1.49	-2.10	-1.62	-1.73	-2.37	
						Top of girder	≦0.4fcu=0.4*50=20.0	N/mm^2	-0.41	2.21	0.30	0.62	0.75	
						Bottom of girder	-0.36√fcu=-0.36√50=-2.55≦	N/mm^2	4.64	2.61	4.26	4.10	3.83	
			PC tendon stress	At the time of prestressing		Centroid of PC tendon	≦1,440	N/mm^2	1276.60	1276.60	1276.60	1276.60	1276.60	
				Immediately after prestressing			≦1,295	N/mm^2	1055.14	1073.12	1056.78	1064.52	1061.08	
				Working stage			≦1,110	N/mm^2	960.29	988.33	963.90	972.55	971.61	
	ULS	Bending moment (Center of span)	Moment (M)					kN.m	3167.59	4407.04	3136.89	3290.47	3614.12	
			Resistant moment (Mr)				M≦Mr	kN.m	4777.93	5545.36	4741.52	4747.51	4805.78	
		Shear force (1/8 of span)	Shear force (Shu)					kN	751.33	856.09	825.05	831.44	893.28	
			Resistant shear force for compression destruction (Suc)				Shu≦Suc	kN	5167.50	6307	5167.50	5167.50	5167.50	
			Resistant shear force for diagonal tensile destruction (Sus)				Shu≦Sus	kN	1382.83	1582.97	1382.83	1385.45	1385.25	
Cross Beam	SLS	Bending moment (Center of span)	Concrete stress	After composition	Plus moment	Top of slab	≦0.5fcu=0.5*35=17.5	N/mm^2	0.24	0.07	0.27	0.19	0.27	
						Bottom of slab		N/mm^2	0.18	0.05	0.20	0.15	0.20	
						Top of cross beam	≦0.4fcu=0.4*35=14.0	N/mm^2	0.18	0.05	0.20	0.15	0.20	
						Bottom of cross beam		N/mm^2	0.61	0.73	0.63	0.76	0.58	
					Minus moment	Top of slab	-0.36√fcu=-0.36√35=-2.13≦	N/mm^2	0.03	0.04	0.16	0.04	0.04	
						Bottom of slab		N/mm^2	0.02	0.03	0.12	0.03	0.03	
						Top of cross beam	≦0.4fcu=0.4*35=14.0	N/mm^2	0.02	0.03	0.12	0.03	0.03	
						Bottom of cross beam		N/mm^2	-0.03	-0.04	-0.16	-0.04	-0.04	
			PC tendon stress	At the time of prestressing		Centroid of PC tendon	≦1,440	N/mm^2	1300.74	1300.74	1300.74	1300.74	1300.74	
				Immediately after the prestressing			≦1,260	N/mm^2	680.64	709.57	709.57	753.87	680.64	
				Working stage			≦1,080	N/mm^2	598.48	626.97	627.51	670.93	598.52	
	ULS	Bending moment (Center of span)	Plus moment		Moment (M)			kN.m	95.84	101.52	103.41	85.68	102.92	
					Resistant moment (Mr)		M (+)≦Mr (+)	kN.m	389.69	543.94	404.10	425.58	389.71	
			Minus moment		Moment (M)			kN.m	8.87	13.51	36.78	9.70	10.41	
					Resistant moment (Mr)		Mr (-)≦M (-)	kN.m	77.23	79.03	91.93	91.93	91.93	
Slab	SLS	Cracking	Crack width		Transverse		≦0.25 (Severe)	mm	uncracked	uncracked	uncracked	uncracked	uncracked	
			Concrete stress		Transverse		≦17.5	N/mm^2	12.651	12.046	8.172	10.424	12.457	
					Longitudinal			N/mm^2						
	ULS	Bending moment	Transverse		Lever arm		≦0.95		0.94	0.94	0.95	0.94	0.94	
					Moment		≦Mu=82.9	kN.m	33.366	33.866	18.89	26.164	32.343	
			Longitudinal		Lever arm		≦0.95							
		Shear force	Ultimate shear stress				≦vc=0.97	N/mm^2	0.678	0.678	0.678	0.678	.678	
		Punching shear	Ultimate shear force				≦Vc=422.1	kN	79.448	71.215	37.107	84.848	62.922	

Table 7.1.19 Detailed Computation Results (6/8) – Overpass Bridge <Superstructure 1/3>

Member	Limit state	Item				Allowable value	Unit	O8		O9		O12			
								A1 & A2	P1	A1 & A2	P1	A1 & A2	P1		
Pile	SLS	Stability analysis	Reaction of pile	Maximum	Reaction (Pmax)	Pmax≤Ra	kN	4899	4487.07	5525	5414.14	4899	4785.88		
					Allowable bearing capacity (Ra)		kN	7633	7633	7633	7633	7633	7633		
				Minimum	Reaction (Pmin)	Pa≤Pmin	kN	238	0	384	0	238	0		
					Allowable uplift capacity (Pa)		kN	-2161	-2161	-2161	-2161	-2161	-2161		
			Horizontal displacement			≤1.5	cm	0.450	0.1954	0.22	0.1726	0.450	0.1658		
	ULS	Stability analysis	Reaction of pile	Maximum	Reaction (Pmax)	Pmax≤Ra	KN	7267	6147.12	7310	7532.19	7267	6704.24		
					Allowable bearing capacity (Ra)		KN	18970	18970	18970	18970	18970	18970		
				Minimum	Reaction (Pmin)	Pa≤Pmin	KN	10	0	-151	0	10	0		
					Allowable uplift capacity (Pa)		KN	-2161	-2161	-2161	-2161	-2161	-2161		
		Stress analysis	Axial force		Axial load (N)	N≤Nuz	KN	7267	6147	7310	7532	7267	6704		
					Maximum axial load (Nuz)		KN	18970	18970	18970	18970	18970	18970		
			Bending moment		Design ultimate moment (M)	M≤Mmax	kN.m	2160	1712	1139	1515	2160	1811		
					Maximum bending moment (Mmax)		kN.m	5966	5966	5966	5966	5966	5966		
Parapet	SLS	Stress Analysis	Back face		Concrete	≤0.5fcu=0.5*30=15.0	N/mm^2	3	-	4	-	3			
					Reinforcement	≤0.75fy=0.75*460=345.0	N/mm^2	54	-	75	-	54			
		Cracking	Back face		Crack width	≤0.25 (Severe)	mm	0.03	-	0.04	-	0.03			
	ULS	Stress Analysis	Bending Moment	Back face	Design ultimate moment (M)	M≤Mmax	kN.m	38.67	-	53.95	-	38.67			
				Maximum bending moment (Mmax)			kN.m	198.8	-	198.8	-	198.8			
			Shear force		Shear stress (v)	≤0.75√fcu=0.75√30=4.11	N/mm^2	0.25	-	0.306	-	0.25			
Wall / Column	SLS	Stress analysis	Concrete			≤0.5fcu=0.5*30=15.0	N/mm^2	4.9	3.56	9.6	5.03	4.72	4.14		
			Reinforcement			≤0.75fy=0.75*460=345.0	N/mm^2	153	158	225	222	147	185		
		Cracking	Crack width			≤0.25(Severe)	mm	-0.13	-0.272	-0.15	-0.045	0.12	-0.176		
	ULS	Stress analysis	Axial force		Axial load (N)	N≤Nuz	KN	-	12019.42	-	20460		12736		
			Bending moment		Design ultimate moment (M)	M≤Mmax	kN.m	1484	5611.31	3985	-	1414	6474		
					Maximum bending moment (Mmax)		kN.m	2546	-	4669	9359.5	2546	-		
			Shear force		Shear stress (v)	≤0.75√fcu=0.75√30=4.11	N/mm^2	4.108	0.072	4.108	0.061	4.108	0.073		
Pile-cap	SLS	Stress analysis	Toe		Concrete	≤0.5fcu=0.5*30=15.0	N/mm^2	4	-	6	-	4			
					Reinforcement	≤0.75fy=0.75*460=345.0	N/mm^2	100	-	100	-	133			
			Heel		Concrete	≤0.5fcu=0.5*30=15.0	N/mm^2	7	-	12	-	7			
					Reinforcement	≤0.75fy=0.75*460=345.0	N/mm^2	232	-	325	-	237			
	ULS	Stress analysis	Toe	Bending moment	Design ultimate moment (M)	M≤Mmax	kN.m	13989	-	44928	-	13694			
					Maximum bending moment (Mmax)		kN.m	25025	-	66620	-	25025			
				Shear force	Shear stress (v)	≤0.75√fcu=0.75√30=4.11	N/mm^2	0.32	-	0.01	-	0.32			
			Heel	Bending moment	Design ultimate moment (M)	M≤Mmax	kN.m	9774	-	39810	-	9998			
					Maximum bending moment (Mmax)		kN.m	16269	-	43312	-	16270			
				Shear force	Shear stress (v)	≤0.75√fcu=0.75√30=4.11	N/mm^2	0.32	-	0.01	-	0.32			

Table 7.1.20 Detailed Computation Results (7/8) – Overpass Bridge <Superstructure 2/3>

Member	Limit state	Item			Allowable value		Unit	O10		O11			O11	
					Abutment	Pier		A1&A2	P1	A1&A2	P1			
STABILITY	SLS	Tipping*	Eccentricity (e=M/V)		≤ea=1.233	≤ea=0.5	-	0.35	0.431	0.69	0.455			
		Sliding	Safety factor (f)		fa=1.5≤	fa=1.5≤	-	2.3	-	1.98	-			
		Ground reaction	Reaction (qmax)		≤qa=1000.0	≤qa=1000.0	kN/m²	292	266.34	341	343.25			
		Bearing	Vertical force (V)		≤Qa=25,884 (n=2.5)	≤Qa=384,130 (n=2.5)	kN	22828	6529.04	29924	18579			
		The ratio of the resisting moment to a rotation moment			2.0≤	2.0≤	-	3.93	-	3.22	-			
	ULS	Tipping*	Eccentricity (e=M/V)		≤ea=7.4	≤ea=3.0	-	0.28	0.431	0.54	0.455			
		Sliding	Safety factor (f)		fa=1.0≤	fa=1.0□	-	2.0	-	1.76	-			
		Ground reaction	Reaction (qmax)		≤qa=1,500.0	≤qa=1,500.0	kN/m²	407	464.03	444	507.68			
		Bearing	Vertical force (V)		≤Qa=60,438 (n=1.0)	≤Qa=218,995 (n=1.0)	KN	33394	16509.35	42602	28975			
		The ratio of the resisting moment to a rotation moment			1.0≤	1.0≤	-	3.79	-	3.15	-			
			Back face	Concrete	≤0.5fcu=0.5*30=15.0	≤0.5fcu=0.5*30=15.0	N/mm²	3	-	3	-			
				Reinforcement	≤0.75fy=0.75*460=345.0	≤0.75fy=0.75*460=345.0	N/mm²	65	-	65	-			
			Back face	Crack width	≤0.25 (Severe)	≤0.25 (Severe)	mm	0.03	-	0.03	-			
			Back face	Design ultimate moment (M)	M≤Mmax	M≤Mmax	kN.m	45.94	-	46.16	-			
			Maximum bending moment (Mmax)				kN.m	198.8	-	198.8	-			
			Shear force	Shear stress (v)	≤0.75√fcu=0.75√30=4.11	≤0.75√fcu=0.75√30=4.11	N/mm²	0.25	-	0.25				
WALL (COLUMN)	SLS	Stress analysis	Concrete		≤0.5fcu=0.5*30=15.0	≤0.5fcu=0.5*30=15.0	N/mm²	6.3	4.32	6.3	3.16			
			Reinforcement		≤0.75fy=0.75*460=345.0	≤0.75fy=0.75*460=345.0	N/mm²	193	159	195	105			
		Cracking	Crack width		≤0.25(Severe)	≤0.25(Severe)	mm	0.01	-0.259	0.01	-0.306			
	ULS	Stress analysis	Axial force	Axial load (N)	N≤Nuz	N≤Nuz	KN	-	13730	-	24428			
			Bending moment	Design ultimate moment (M)	M≤Mmax	M≤Mmax	kN.m	1895	6970.39	1908	11803.55			
				Maximum bending moment (Mmax)			kN.m	2546	-	2283	-			
			Shear force	Shear stress (v)	≤0.75√fcu=0.75√30=4.11	≤0.75√fcu=0.75√30=4.11	N/mm²	4.108	0.069	4.108	0.071			
FOOTING	SLS	Stress analysis	Toe	Concrete	≤0.5fcu=0.5*30=15.0	≤0.5fcu=0.5*30=15.0	N/mm²	6	3.76	7	4.76			
				Reinforcement	≤0.75fy=0.75*460=345.0	≤0.75fy=0.75*460=345.0	N/mm²	155	159.53	183	177.61			
			Heel	Concrete	≤0.5fcu=0.5*30=15.0	≤0.5fcu=0.5*30=15.0	N/mm²	1	3.76	3	4.76			
				Reinforcement	≤0.75fy=0.75*460=345.0	≤0.75fy=0.75*460=345.0	N/mm²	9	159.53	69	177.61			
	ULS	Stress analysis	Toe / Bending moment	Design ultimate moment (M)	M≤Mmax	M≤Mmax	kN.m	747	939.62	827	1026.49			
				Maximum bending moment (Mmax)			kN.m	1339	-	1339	-			
			Toe / Shear force	Shear stress (v)	≤0.75√fcu=0.75√30=4.11	≤0.75√fcu=0.75√30=4.11	N/mm²	0.683	0.546	0.713	0.596			
			Heel / Bending moment	Design ultimate moment (M)	M≤Mmax	M≤Mmax	kN.m	519	939.62	114	1026.49			
				Maximum bending moment (Mmax)			kN.m	1339	-	1339	-			
			Heel / Shear force	Shear stress (v)	≤0.75√fcu=0.75√30=4.11	≤0.75√fcu=0.75√30=4.11	N/mm²	0.327	0.546	0.129	0.596			

Table 7.1.21 Detailed Computation Results (8/8) – Overpass Bridge <Superstructure 3/3>

Member	Limit state	Item					Allowable value	Unit	O8	O9	O10		O11		O12
									A1 & A2	A1 & A2	A1	A2	A1	A2	A1 & A2
WING	SLS	Stress analysis	Section*		a	Concrete	≦0.5fcu=0.5*30=15.0	N/mm^2	7.30	3.50	6.70	7.50	5.30	8.30	2.10
						Reinforcement	≦0.75fy=0.75*460=345.0	N/mm^2	121.00	84.00	112.00	126.00	87.00	130.00	41.00
					b	Concrete	≦0.5fcu=0.5*30=15.0	N/mm^2	2.70	1.70	2.60	3.00	2.20	3.40	0.90
						Reinforcement	≦0.75fy=0.75*460=345.0	N/mm^2	51.00	37.00	52.00	63.00	43.00	64.00	21.00
		Cracking	Section*		a	Crack width	≦0.25 (Severe)	mm	0.15	-0.17	0.13	0.15	0.09	0.17	-0.14
					b				-0.08	-0.16	-0.09	-0.08	-0.13	-0.03	-0.75
	ULS	Stress analysis	Bending moment	Section*	a	Design ultimate moment (M)	M≦Mmax	kN.m	500.46	1097.00	426.99	782.02	383.03	628.92	365.82
						Maximum bending moment (Mmax)		kN.m	799.48	5195.00	808.99	1236.00	808.99	808.99	2033.00
					b	Design ultimate moment (M)	M≦Mmax	kN.m	161.51	247.43	149.97	251.24	124.18	202.18	117.84
						Maximum bending moment (Mmax)		kN.m	791.43	1568.00	670.29	935.52	670.99	719.43	1368.00
			Shear force	Section*	a	Shear stress (v)	≦0.75√fcu=0.75√30=4.11	N/mm^2	0.695	0.231	0.629	0.739	0.563	0.815	0.354
					b			N/mm^2	0.402	0.981	0.367	0.428	0.329	0.427	0.206

7.1.9 Design Changes

Based on bridge design for our scope in this Project, the following two (2) items were studied and reviewed:

(1) Application of steel girder
(2) Review of foundation due to the latest soil investigation

(1) Application of Steel Girder

STEP Loan Scheme

GOSL has decided to apply the STEP JBIC Loan Scheme to consider the following advantaged conditions;

- Interest rate of repayment has quit low and 10 years grace period,
- To exploit advantaged Japanese technology, and
- The technical transfer

Application of Steel Girder

Generally, steel structures are the following advantages and disadvantages;
Advantages:

- ➢ Handling at the construction (erection, joint, etc.) is easy,
- ➢ Flexibility of fabrication (shape availability) is well,
- ➢ Construction period at site is short

Disadvantages:

- ➢ Costly than concrete girder,
- ➢ Frequent maintenance, especially for protecting of corrosion (painting, etc.) needed

In this Project, viaducts of main-line and ramp bridges at A1 Interchange were applied to use steel girder to consider:

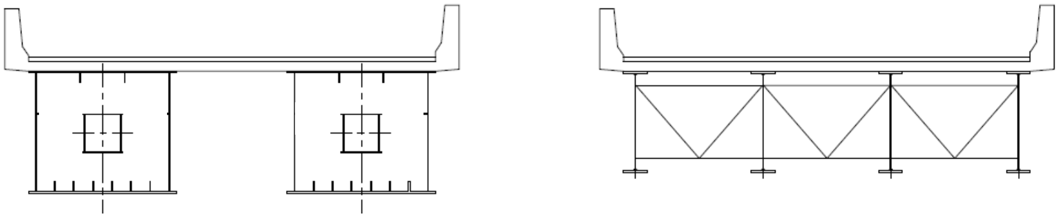
- Regional condition --- to minimize the site work and not to disturb long period for the residents around the area,
- Arterial road characteristics --- not to make bad affects for traffic flow during construction to A1 road as the arterial highway

Steel Girder Type

Generally, most popular and familiar steel girder has the following two (2) types:

- □ I-section plate girder
- □ Box-section plate girder

Those typical cross sections are shown in **Fig. 7.1.16**.



Box-Section Plate Girder I-Section Plate Girder

Fig. 7.1.16 Typical Cross Section of Plate Girder

And depend on structural mechanism between girder and deck slab, both composite and non-composite types can be used. In this Project, non-composite type was selected to consider the ease of future expansion and maintenance, etc. as mentioned on Design Standard.

Relationship between bridge type and applicable span length for steel bridge in Japanese practice is shown in **Table 7.1.22**.

Table 7.1.22 Relation Between Bridge Type and Applicable Span Length for Steel Bridge

	Bridge Type	Span Length (m) 0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 250 500 1,000	Achievement of Maximum Span (m)
Plate Girder	Simple Composite H Girder Bridge		–
	Simple Non Composite I Girder Bridge		–
	Simple Composite I Girder Bridge		–
	Simple Non Composite Box Girder Bridge		–
	Simple Composite Box Girder Bridge		–
	Continuous Non Composite I Girder Bridge		–
	Continuous Non Composite Box Girder Bridge		140
	Continuous Girder Bridge with Steel Deck		300
	Rigid Frame Bridge		234
Truss	Simple Truss Bridge		227
	Continuous Truss Bridge		548
Arch	Langer Girder Bridge		150
	Adversed Langer Girder Bridge		140
	Lohse Girder Bridge		383
	Adversed Lohse Girder Bridge		200
	Langer Truss Bridge		156
	Trussed Langer Girder Bridge		175
	Neilsen Bridge		305
	Arch Bridge		518
	Cable Stayed Bridge		890
	Suspension Bridge		1990

Most suitable span length   Applicable span length

Viaduct-1 (V1: A1 Interchange Viaduct)

Determination of Span Arrangement

Control points for V1 are:

a. Limited superstructure depth (D) above the A1 Bypass: Dmax. = 2.2m
b. Locations of Pier-2 and Pier-5 are fixed based on the geometrical reason for approach to the Interchange Rampways

Therefore, Abutment-1 and 2, Pier-1, 2 and 5 were decided the location. And 3 different span arrangement alternatives were nominated (shown in **Fig. 7.1.17**);

- Alternative-1: (18.0+21.0) + 3@47.667 + 3@46.667
- Alternative-2: (18.0+21.0) + 4@35.750 + 4@35.0
- Alternative-3: (18.0+21.0) + 2@71.5 + 2@70.0

And compared for a series of this viaduct. The comparison result is shown in **Table 7.1.23**.

Alternative-1 was selected as the most economical and minimized number of piers.

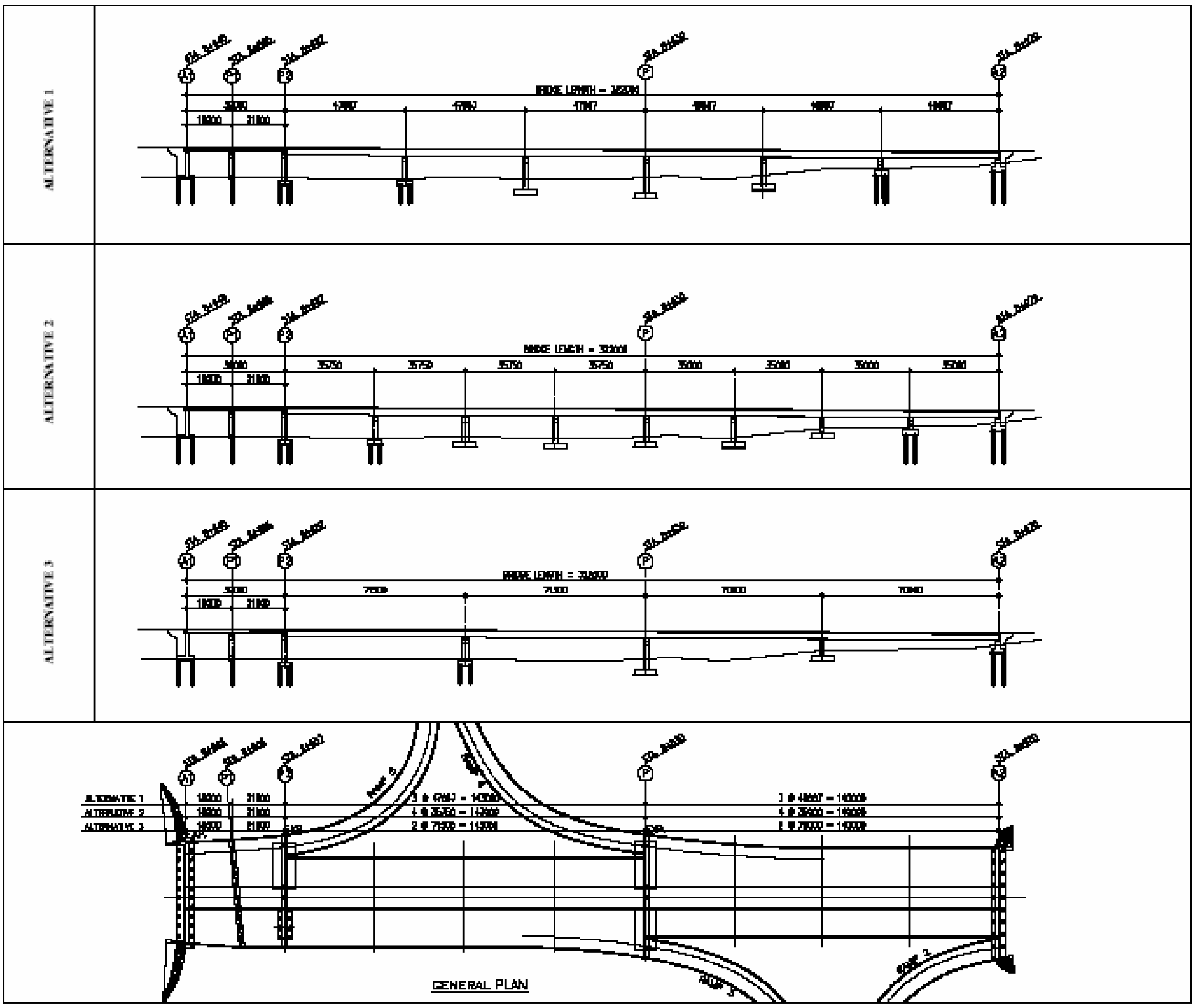


Fig. 7.1.17 Comparison for Span Arrangement of V1

Table 7.1.23 Cost Comparison for Viaduct-1

Alternative	Construction Cost (Million Rs.)		Judge
1. (18.0+21.0) + 3@47.667 + 3@46.667 m	Superstructure	1,702.4	**Applied**
	Substructure	348.8	
	Foundation	97.5	
	Others	9.5	
	Total	**2,158.2**	
	Unit Cost (Thousand/sq.m.)	**243.8**	
	Rate	**1.000**	
2. (18.0+21.0) + 4@35.750 + 4@35.0 m	Superstructure	1,595.7	
	Substructure	438.0	
	Foundation	122.9	
	Others	10.2	
	Total	**2,166.8**	
	Unit Cost (Thousand/sq.m.)	**244.8**	
	Rate	**1.004**	
3. (18.0+21.0) + 2@71.5 + 2@70.0 m	Superstructure	3,843.3	
	Substructure	298.5	
	Foundation	72.1	
	Others	8.7	
	Total	**4,222.6**	
	Unit Cost (Thousand/sq.m.)	**477.1**	
	Rate	**1.957**	

* 2@71.5 and 2@70.0 bridges are adopted box girder as the applicable type.

Girder Type Selection

I-section plate girder was selected for V1, because:

- The span ranged from 35m to 47m was selected as the most reasonable for the construction cost in the above “Determination of Span Arrangement”,
- Basically, I-section plate girder is the most economical and popular method for the ranged span length in the Japanese and/or international practices,
- V1 is shaped almost straight and simple though it has widening for ramps,
- Actual bridge data of 47m span on Japanese practice, unit weight of steel for box type is approx. 350 ~ 420 kg/sq.m, and for I-type is 250 ~ 300 kg/sq.m.

Viaduct-5 to 8 (V5 to V8: A1 Interchange Ramps)

Girder Type Selection

Box section plate girder was selected for A1 Interchange ramp bridges to consider:

- ✓ Adequate girder type for the curved bridge,
- ✓ Good stability for the torsional rigidity due to curve effect,
- ✓ Stable and safety erection by closed box section,
- ✓ Rationalized shape followed the geometrical structure

The span arrangement was also considered about the radius of curvature (Rmin=50m), and the lengths of each ramp bridges were decided. Span arrangement of all ramp bridges is:

➢ V5 (A1 IC Ramp-2):	3@40.833	=	122.500m
➢ V6 (A1 IC Ramp-3):	35.069+2@43.500	=	122.069m
➢ V7 (A1 IC Ramp-5):	3@41.207	=	123.622m
➢ V8 (A1 IC Ramp-6):	3@45.857	=	137.571m

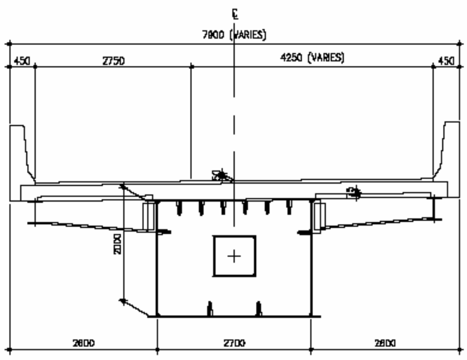
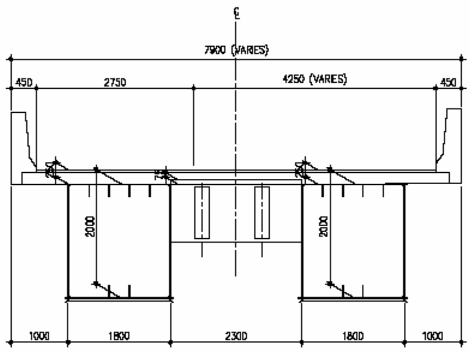
Comparison of Steel Box Girder

Single box (1-box) with bracket and stringer type and double box (2-box) type can be applied for these ramp bridges because the structure width is only 7 to 8.5m, narrow.

Comparison result, based on V5 (A1 IC Ramp-2 bridge: 3@40.833m) of these types is shown in **Table 7.1.24**.

Finally, single box type was applied for the ramp bridges.

Table 7.1.24 Comparison Result of Box Girder (Number of Box)

	Alternative-1 Single Box Type (1-Box)	Alternative-2 Double Box Type (2-Box)
Section		
Characteristics	- Less man-hour for fabrication than 2-Box - Short period for girder erection - Generally economical for these ranged span length - Lot of bracket, 2 stringer needed - Aesthetically good - Structural aesthetics good - Approx. steel weight: 309.6 ton	- Absolutely and physically much needed number of webs and width of flanges for this span and road width (unreasonable) - Longer erection period than 1-Box - Aesthetically good - Structural stability good - Approx. steel weight: 319.7 ton
Judge	Applied	

(2) Review of Foundation due to the Latest Soil Investigation

Based on the latest soil investigation data (mainly boring log) and the engineering considerations, the foundation of all bridges was revised.

Major revisions are:

1. Length of Viaduct-3 (V3: The 2nd Biyagama Viaduct) were changed from 315m to 420m, to avoid peat and organic clay layers,
2. In the shallow bearing stratum area, pile type foundation was changed to spread type foundation,
3. Embedded length of the pile tip into the hard rock (Br) was checked and adjusted the limitation with 2 times of diameter or 4m, due to the rock properties.

(3) Individual Viaduct Characteristics

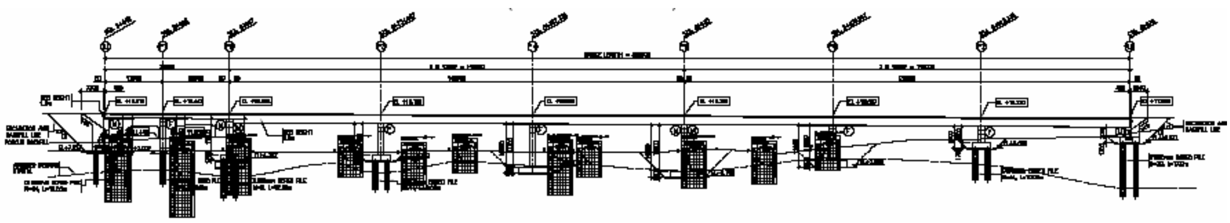
Changed design bridges are individually introduced below:

Viaduct-1 (V1: STA. 8+648 – 8+970)

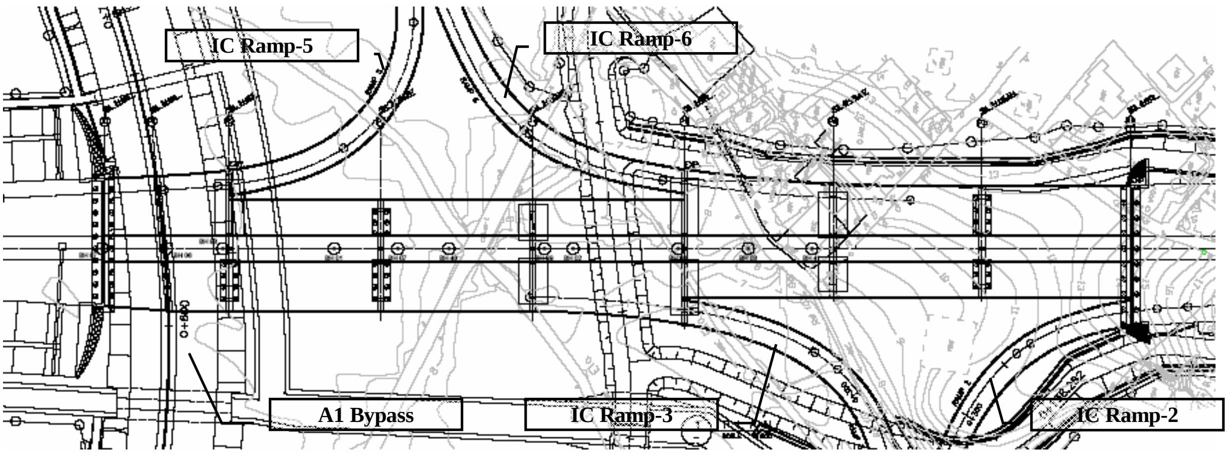
Characteristics:

- Viaduct-1 (V1) is located at/around A1 Interchange area.
- The shape of bridge surface is complicated due to tapering of on/off interchange ramps and the acceleration/de acceleration lanes widening.
- A1 Bypass is crossed by OCH main-line at STA. 8+667. The minimum span length and maximum depth of upper-structure are 39m and 2.0m respectively.
- Based on the comparison result, I-section plate girder with 47m span length was used, except for A1 Bypass crossing section.
- Span arrangement: (18.0+21.0) + 3@47.667 + 3@46.667 = 322.0m
- Wall type (T-shape) piers with cantilevered coping and reversed T-type abutments were used.
- Based on the assumed bearing stratum, Pier-2 (east bound), Pier-4, Pier-5 and Pier-6 were used spread foundation and rest of the piers and abutments were used pile foundation due to the latest soil investigation data.

Plan & Profile:



Profile



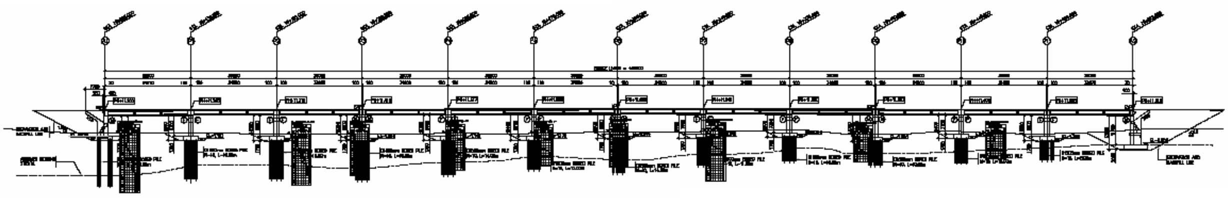
Plan

Viaduct No. 3 (V3: STA. 15+130 – 15+445)

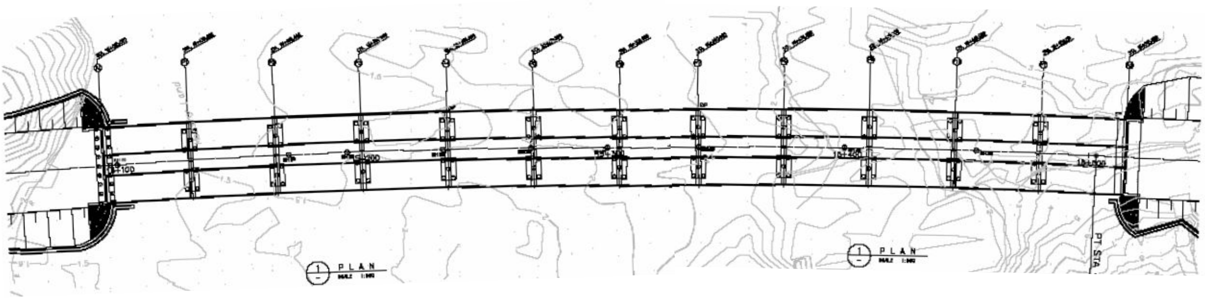
Characteristics:

- Viaduct-3 (V3) location was determined due to the soil conditions. Soft soil, such as organic clay and peat are distributed approximately 5~6 m deep in this area. Viaduct type was chosen to reduce embankment fill material requirements, minimize filling material for soft soil countermeasure and to reduce the social environmental impacts of heavy equipment/vehicle traffic during construction.
- Based on the economical comparison result, 35m span length was used.
- 3 span continuous upper-structure – monolithic structure after launching girders was used.
- Span arrangement: 12@35.0 = 420.0m
- Wall (T-shape) type pier with cantilevered coping and reversed T-type abutment were used.
- Assumed bearing stratum for foundation is 10 to 15m below the existing ground level therefore pile foundation was used.
- This viaduct is located within a curve of 2,000m radius

Plan & Profile:



Profile



Plan

A1 Interchange Ramps

- Viaduct-5 (V5: STA. 0+241.285 – 0+363.785): Ramp-2 Bridge of A1 Interchange
- Viaduct-6 (V6: STA. 0+239.766 – 0+361.835): Ramp-3 Bridge of A1 Interchange
- Viaduct-7 (V7: STA. 0+365.448 – 0+489.070): Ramp-5 Bridge of A1 Interchange
- Viaduct-8 (V8: STA. 0+365.448 – 0+503.019): Ramp-6 Bridge of A1 Interchange

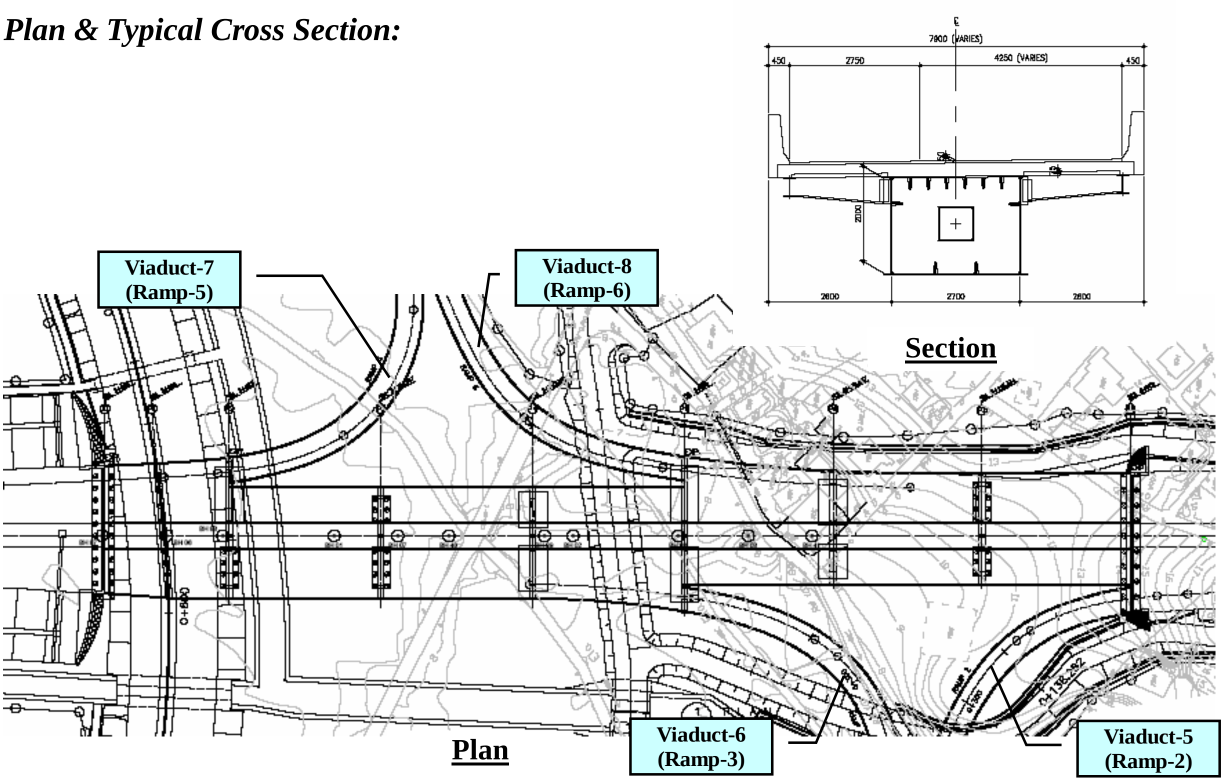
Characteristics:

- To limit the intrusion into surrounding residences all ramps are located in limited space and thus compact shape for ramps was applied. Therefore, all ramp bridges also have sharp curve of 50m radius.
- Based on the comparison result a Steel Box Girder bridge was selected and 40m to 45m span length was decided in consideration of the structural stability.
- Soil conditions are similar to V1. Pile and spread foundation types were applied.
- Continuous superstructure was applied to meet the structural durability and integrity of the curved bridge.
- Span arrangement:

V5:	3@40.833	=	122.500m
V6:	35.069+2@43.500	=	122.069m
V7:	3@41.207	=	123.622m
V8:	3@45.857	=	137.571m

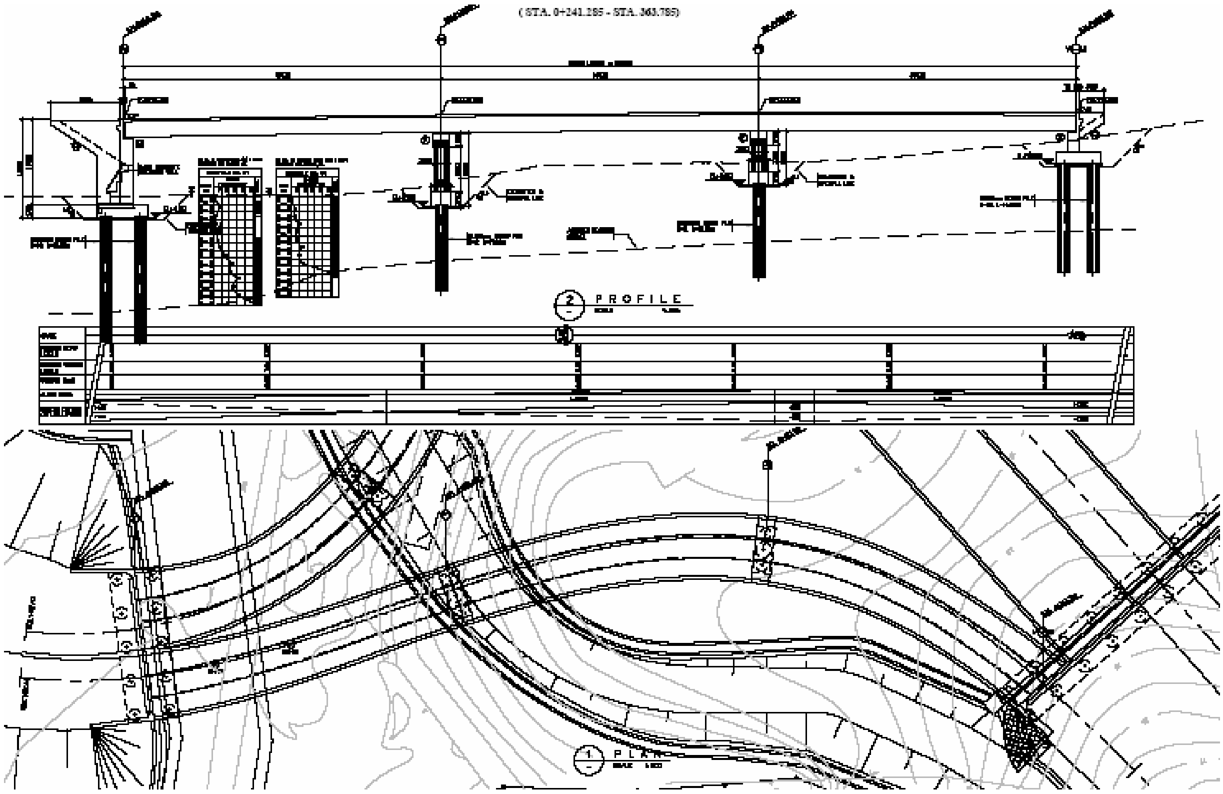
- Circular column with cantilevered coping type (T-shape) pier was used to for aesthetic reasons and reversed T-type abutment was used.

Plan & Typical Cross Section:

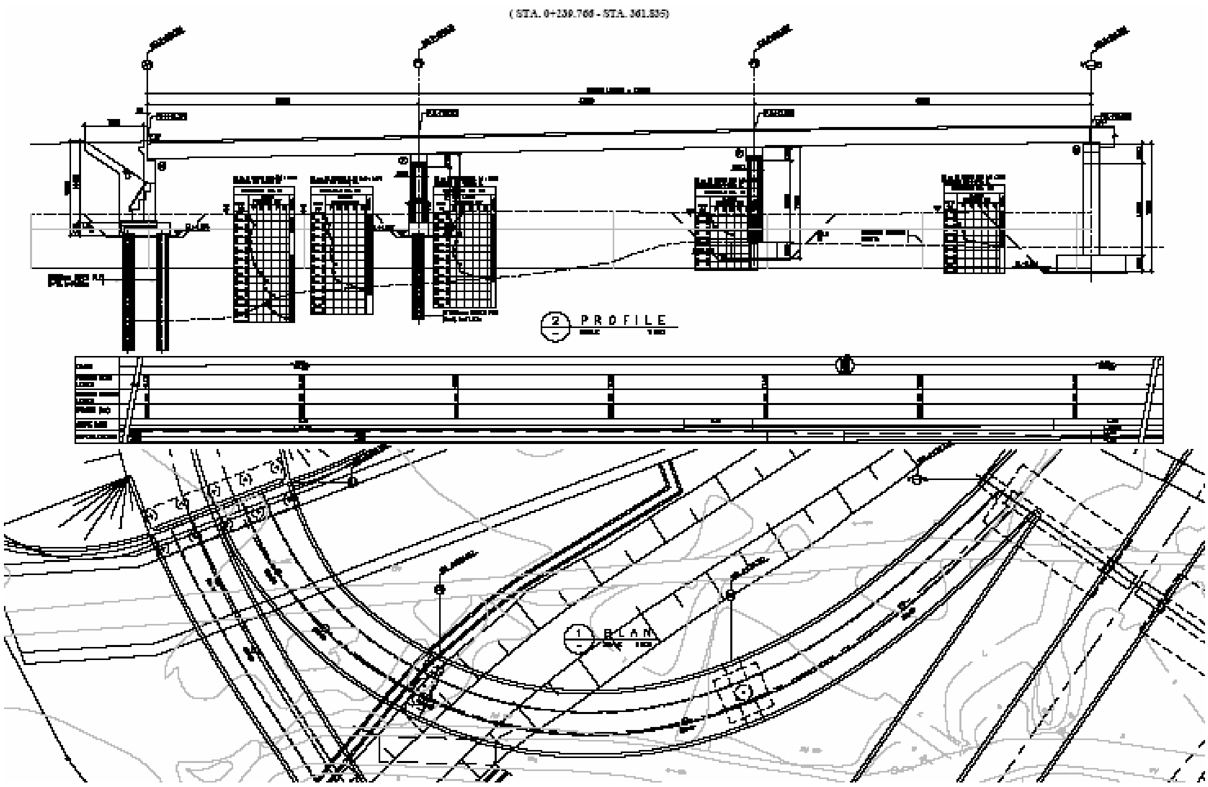


Plan & Profile:

Viaduct-5 (V5: A1 IC Ramp-2)

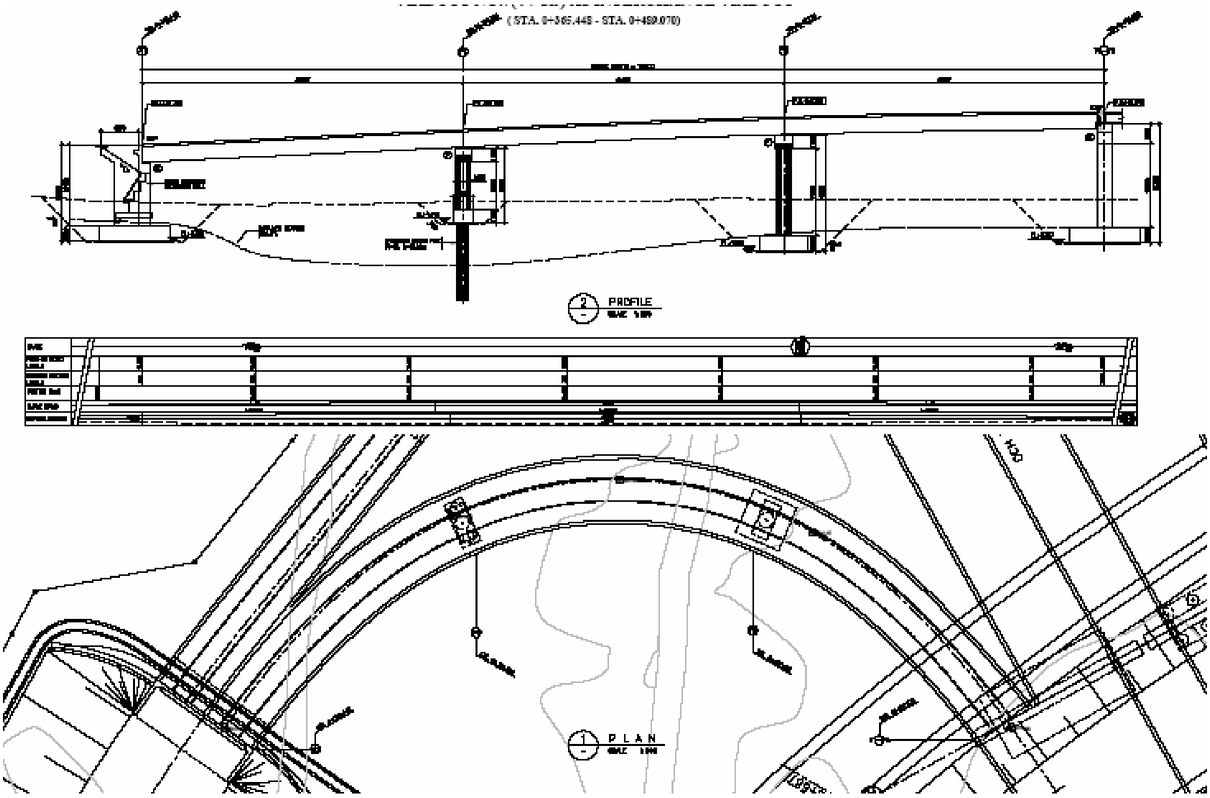


Viaduct-6 (V6: A1 IC Ramp-3)

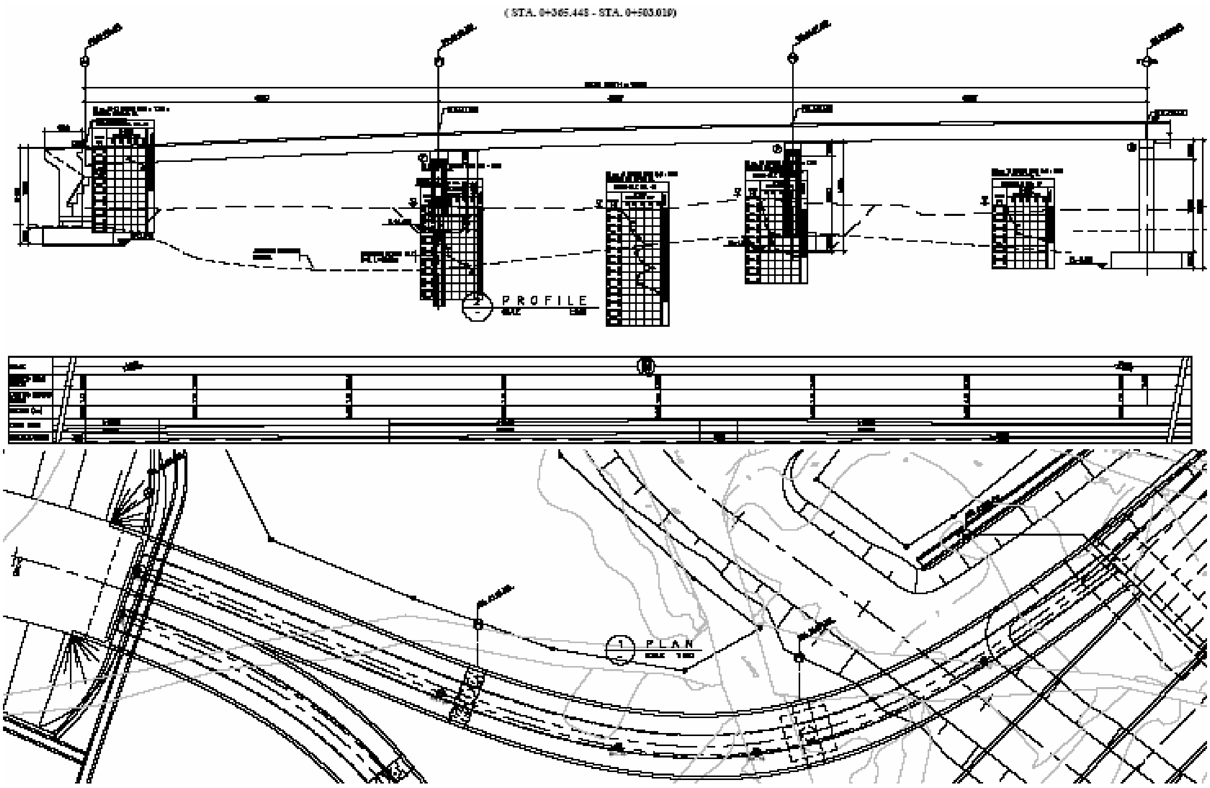


Plan & Profile:

Viaduct-7 (V7: A1 IC Ramp-5)



Viaduct-6 (V6: A1 IC Ramp-3)



7.2. Structure Design

7.2.1. Box Culvert

(1) Required Clearance in Width and Height

1) Underpass

Following descriptions explain about under-pass (box culverts for road), which has side-ditches on both sides. List of underpass culverts is in **Table 7.2.1**.

Table 7.2.1 List of Underpass Culvert

No	Crossing Object	Station	Road Class	Skew (deg)	Required Clearance (m)		Length (m)
					Vertical (track clearance)	Horizontal (drain width excluded)	
1	Main Expressway	10 + 917.000	D	90	4.80	6.50	35.40
2	Main Expressway	13 + 327.000	D	82	4.80	6.50	39.20
3	Main Expressway	14 + 619.000	D	90	4.80	6.50	35.80
4	Main carriageway	15 + 840.000	E	90	4.50	5.90	50.00
5	A1-Ramp-1	0 + 136.297	C	79	5.10	10.00	41.30
6	A1-bypass	0 + 285.000	D	90	4.80	6.50	27.50
7	A1-bypass	1 + 293.000	D	82	4.80	6.50	26.00

<u>Track-Clearance</u>

In the study of this Report(Chapter3.2.2 Clearance), the box-culverts of approach road crossing the Main Carriageway, Ramp-way and A1-bypass are classified as C, D orE. And adding the space of side-ditches, Details of culvert inner dimensions for C-class, D-class and E-class roads are decided respectively as shown in **Fig. 7.2.1, Fig. 7.2.2** and **Fig. 7.2.3.**

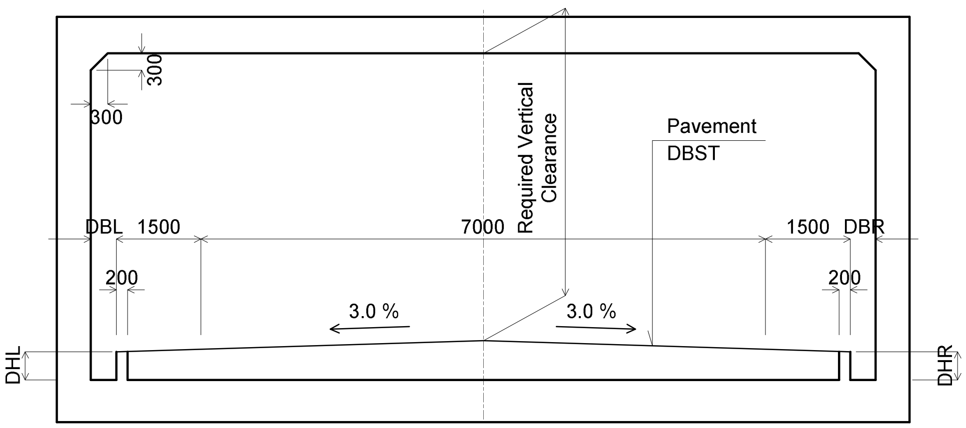


Fig. 7.2.1 Detail of Box Culvert Internal Clearance for Class-C Road

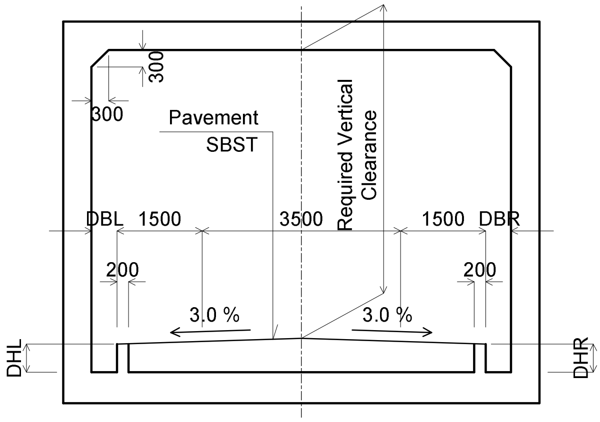


Fig. 7.2.2 Detail of Box Culvert Internal Clearance for Class-D Road

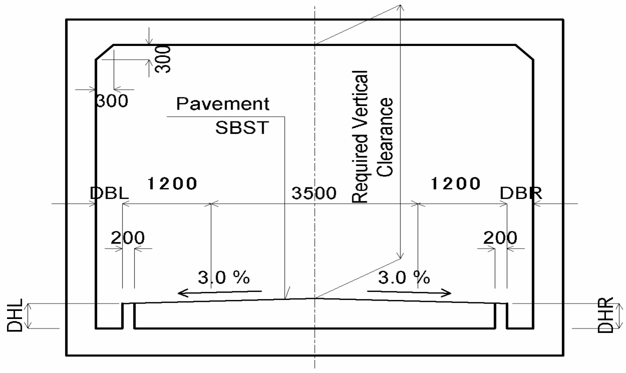


Fig. 7.2.3 Detail of Box Culvert Internal Clearance for Class-E Road

Fundamentally, minimum track-clearance (Hc) for all minor roads is 5.1m in the design, taking double-decker bus into account. However, 4.8m of track-clearance is also acceptable where the track-clearance is restricted by the vertical alignment of OCH main highway. However the track-clearance of E-class road may be applied 4.5m where it is obviously used as only local area and is smaller traffic density expected no more increasing in future.

Pavement and Cross-fall

Pavement type of DBST and SBST are selected to use respectively for Class-C, Class-D and Class-E roads.

For these pavement types, cross-fall shall be 3.0 % from the carriageway center to both sides.

Drainage (Side-ditch)

Minimum width of side ditch (DBL and DBR) along the underpass culverts shall be 0.45m together with the minimum depth (DHL and DHR) of 0.30m.

These dimensions shall be determined based on the hydrological analysis and drainage design. In respect of drainage depth, it shall also be adjusted to existing roadside elevation.
Drainage wall with 200 mm thickness shall be considered as follows:

- If ditch wall height (DHL/DHR) is equal to or less than 0.80 m, the wall shall be within the shoulder as shown in **Fig. 7.2.1, Fig. 7.2.2** and **Fig. 7.2.3**.
- If ditch wall height (DHL/DHR) is greater than 0.80 m, safety fence shall be installed on top of ditch wall. The wall shall therefore be outside of the shoulder.

2) Drainage Culverts

Inner dimensions of drainage culverts shall be decided as the result of hydrological analysis in the "Chapter-6 Hydrology & Drainage Design".
List of drainage culverts is in **Table 7.2.2**.

Table 7.2.2 List of Drainage Culvert

No.	Location	Station	Skew Angle (degree)	Cell	Inner Width (m)	Inner Height (m)	Length (m)
Drainage							
8	Main Line	9+340.0	90-00-00	1	3.25	2.00	40.8
9		9+760.0	70-00-00	2	3.25	3.00	44.2
10		10+530.0	90-00-00	1	3.25	2.00	51.4
11		10+690.0	75-00-00	1	3.25	2.00	58.4
12		11+740.0	90-00-00	1	3.25	2.00	46.4
13		11+935.0	90-00-00	1	3.25	2.00	37.9
14		12+350.0	90-00-00	2	3.25	3.00	35.5
15		12+735.0	90-00-00	1	3.25	3.00	33.7
16		13+210.0	90-00-00	2	3.25	3.00	56.4
17		13+550.0	90-00-00	1	3.25	2.00	58.8
18		14+340.0	90-00-00	1	3.25	2.00	55.3
19		15+585.0	90-00-00	1	3.25	2.00	54.8
20	A1 IC Ramp-4	0+025.0	90-00-00	3	3.25	3.00	35.5
21	A1 Bypass	0+575.0	90-00-00	3	3.25	2.00	40.4
22		0+675.0	90-00-00	1	3.25	3.00	32.7
23		0+990.0	90-00-00	1	3.25	3.00	36.5
24		1+312.0	90-00-00	1	3.25	3.00	47.4
25	Ap Rd 8+805	0+055.0	90-00-00	5	3.25	3.00	11.5
26		0+252.0	90-00-00	1	2.00	1.50	12.2
27	Ap Rd 13+327	0+040.0	90-00-00	1	3.25	2.00	10.0
28		0-040.0	90-00-00	2	3.25	2.00	11.4
29	Ap Rd 0+102 A1 BP	0+230.0	33-00-00	1	3.25	2.00	22.4
30	Ap Rd 0+285L A1 BP	0+107.0	90-00-00	1	2.00	1.50	6.0
31		0+208.0	90-00-00	1	2.00	1.50	6.0
32	Ap Rd 1+294 A1 BP	0-025.0	90-00-00	1	3.25	2.00	9.1
33		0+030.0	90-00-00	3	3.25	2.00	8.8
34	Fr Rd Ramp-2	0+280.0	60-00-00	1	2.00	1.50	9.4
35	Fr Rd Ramp-6	0+515.0	58-00-00	1	2.00	1.50	10.6
Irrigation							
36	Main Line	9+726.0	53-00-00	1	3.25	2.0	51.7
37		9+876.0	69-00-00	1	3.25	2.0	41.3
38		10+497.5	60-00-00	1	3.25	2.0	57.7
39		12+250.0	56-00-00	1	3.25	2.0	42.1
40		12+395.0	84-00-00	1	3.25	2.0	35.7
41	A1 Bypass	0+500.5	78-00-00	1	3.25	2.0	38.5
42		0+634.0	86-00-00	1	3.25	2.0	40.1
43		1+665.0	77-00-00	1	3.25	2.0	48.2

(2) Soil Cover on Concrete Culvert

It is commonly said that at least pavement thickness from the road surface to the top of sub-grade or 500 mm, whichever is smaller, shall be ensured above the concrete box culvert.

In this Project, pavement structure of Outer Circular Highway comprises the followings:

Asphaltic Concrete Wearing Course	t =	40 mm
Asphaltic Concrete Binder Course	t =	85 mm
Aggregate Base Course	t =	225 mm
Sub-base Course	t =	175 mm
Total	t =	525 mm

Therefore, minimum soil cover of 500 mm is considered for box culvert.

(3) Foundation

The spread foundation is used for both road and drainage box structures. In this study, the countermeasure for the soft ground is executed for the embankment prior to the culvert construction.

To improve the foundation ground for culvert construction site, two methods, namely "replacement of soft soil with suitably graded sand, gravel or stone" and "soft soil improvement together with preloading method", are mainly to be carried out as they are made on the embankment section. Culvert shall be constructed after the design settlement is achieved.

To ensure the enough bearing capacity for each box culvert, ultimate bearing capacity of foundation ground should be confirmed by testing.

(4) Wing Wall Type for Road Culvert

Wing wall for road culvert is of return type with maximum cantilever length of 8 m.

In case slope toe affect to the crossing roads and waterway with 8 m cantilever length of wing wall, retaining wall of wet masonry or plain concrete structure shall be used at the slope toe, according to the practices in Sri Lanka.

The outline of structure is illustrated in **Fig. 7.2.4**.

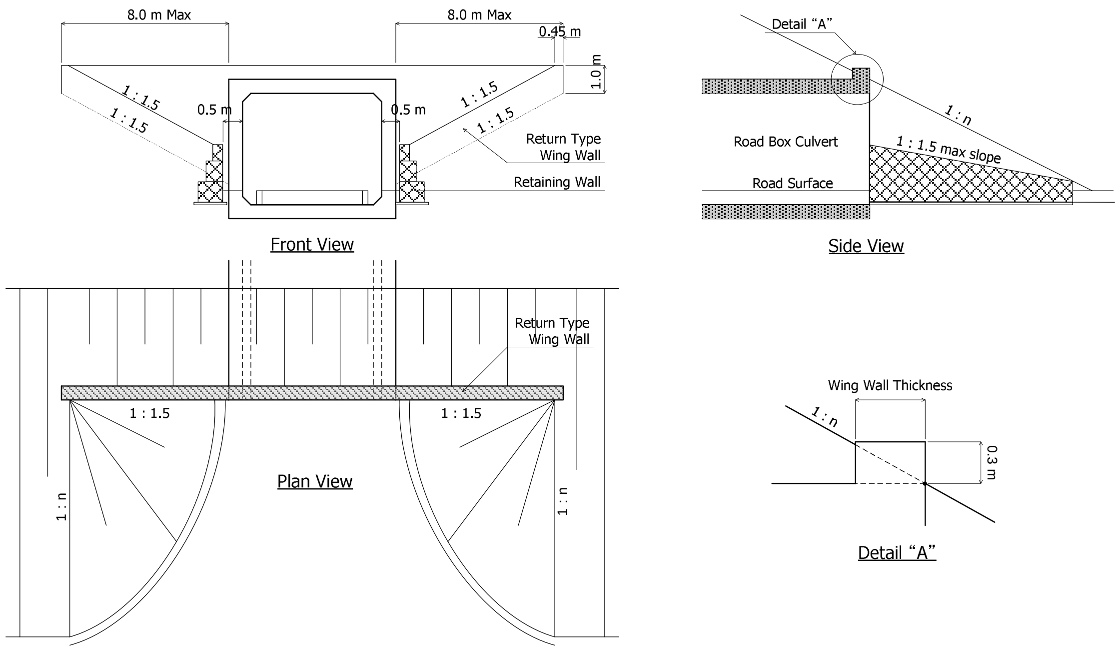


Fig. 7.2.4 Application of Return Type Wing Wall

(5) Inlet/Outlet Structure with Scour Protection for Drainage Culvert

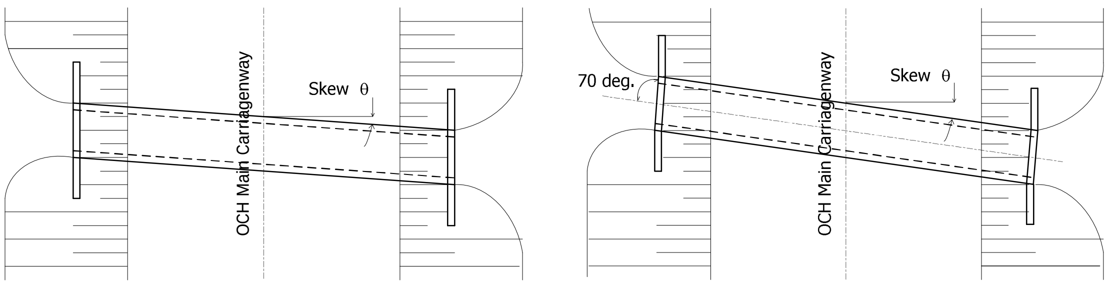
Inlet and outlet structures are of U-shaped retaining wall. At inlets and outlets locations, the bed is easy subject to scour. Therefore, curtain wall and bed protection by gabion box is recommended to use in this Project.

(6) Skew

Generally, it is recommended the skew be 20 degree or smaller, according to the Japanese Specifications. In this case, wing wall is set in parallel with the OCH centerline.

In case the skew is greater than 20 degree, appropriate treatment on wing wall construction shall be made.

In this Project, wing wall shall be set with 70 degree between culvert centerline and wing wall direction as shown in **Fig. 7.2.5**.



Skew = 20 deg. Skew > 20 deg.

Fig. 7.2.5 Setting of Wing Wall

(7) Longitudinal Joint of Box Culvert

In general, longitudinal joints shall be provided in the perpendicular direction to the wall elements at 10 – 15 m intervals. Water-stop shall be installed on longitudinal joints to prevent water leakage.

(8) Approach Slab

Box culvert is constructed in the embankment section in this Project in order to ensure the existing or detour crossing roads and drainage openings even after completion of Outer Circular Highway.

It is said that the weight of box culvert is smaller than embankment material and almost equal settlement will occur both in box culvert portion and embankment portion.

In general, approach slab is installed between structure and embankment portions to provide smooth ride quality even after differential settlement occurs.

In this Project, existing ground is in poor condition and soft soil improvement work is carried out to prevent future settlement of embankment. Even though, residual settlement after completion of the Project will remain, and there may be differential settlement between structure and embankment portions. Such amount of differential settlement will be analyzed by Soft Soil Improvement Study.

Approach slab is to be installed when such differential settlement is big enough to give adverse effect to smooth ride quality. Considering of influence of truck load, approach slab should be added to the culverts especially located on which cover fill least than 1m.

List of the culverts taken approach slab is shown in **Table 7.2.3**.

Table 7.2.3 List of the culverts taken approach slab

Road/ Drainage	Location	Station	Minimum Cover (m)	Remarks
Road	Main Expressway	10 + 917.000	0.7	
	Main Expressway	14 + 619.000	0.5	
	A1-Ramp-1	0 + 136.297	0.6	
	A1-bypass	0 + 285.000	0.9	
	A1-bypass	1 + 293.000	0.6	
Drainage	Main Expressway	12 + 735.000	Less 1.0	1 Cell
	Ap Rd 8+805	0 + 252.000	0.6	1 Cell
	Ap Rd 13+327	0 + 040.000	Less 1.0	1 Cell
	Ap Rd 0+102 A1 BP	0 + 230.000	0.7	1 Cell
	Ap Rd 0+285L A1 BP	0 + 107.000	0.6	1 Cell
		0 + 208.000	0.6	1 Cell
	Ap Rd 1+294 A1 BP	0 - 025.000	0.7	1 Cell
	Fr Rd Ramp-2	0 + 280.000	0.7	1 Cell
	Fr Rd Ramp-4	0 + 515.000	0.8	1 Cell
	Ap Rd 1+294 A1 BP	0 - 030.000	0.6	3 Cell
	Ap Rd 8+805	0 + 055.000	0.5	5 Cell

7.2.2. Pipe Culverts

(1) Introduction

Pipe culverts are installed under the embankment in order to ensure the existing drainage openings and irrigation canals even after the Project implementation.

According to the experience in Sri Lanka, concrete pipes are used for this purpose. Bedding types are in general granular bed and plain concrete bed, or 360° (plain or reinforced) concrete surrounded type for pipes subject to heavy loadings when the soil cover is very small or very big.

Installation conditions of pipes are classified into three (3) types; narrow trench condition, positive projection condition (wide trench/embankment) and negative projection condition.

(2) Minimum Diameter of Pipe Culvert

Minimum diameter of pipe culvert crossing the OCH main highway shall be 900 mm taking the maintenance works into account.

However, for those crossing the minor roads, smaller diameter of 450 mm is to be used.

(3) Soil Cover on Concrete Pipe Culvert

It is commonly said that soil cover depth of at least 600 mm shall be provided above the concrete pipe crown from the experience/practice in Sri Lanka.

(4) Concrete Pipe Internal Diameter and Dimensions

According to BS 5911, pipe culvert nominal diameter (internal diameter) DN and wall thickness used in the Project is summarized in **Table 7.2.4**.

Table 7.2.4 Pipe Dimensions

Nominal Diameter DN (mm)	Wall Thickness t (mm)	Outside Diameter Bc (mm)	Remarks
450	70	590	Class-L, -M, -H
600	86	772	
900	130	1160	
1200	125	1450	
1500	120	1740	

(5) Bedding Type

As described in "Design Standard", the following five (5) types are used.

Type B: 180° Granular Bed

Type S: 360° Granular Bed

Type A: 120° Plain Concrete Cradle

Type C-1: 360°Reinforced Concrete Surrounded Type

Type C-2: 360°Plain Concrete Surrounded Type

The bedding granular material shall have the following characteristics:

Table 7.2.5 Pipe Bedding Granular Material[1]

Nominal Diameter (DN)	Max Particle Size mm	Class of Bedding	Suitable Materials	
			Either aggregate to BS 882	Or having a CF value:
450	20	S	10, 14 or 20 mm nominal single size; or 14 to 50 mm graded; or 20 to 5 mm graded	0.15 max.
		B		0.30 max.
Over 450	40	S	10, 14, 20 or 40 mm nominal single size crushed rock; or 14 to 5 mm graded; or 20 to 5 mm graded; or 40 to 5 mm graded.	0.15 max.
		B		0.30 max.

The concrete used for Type A, C-1 and C-2 bedding shall be monolithic and have a minimum 28-day cube strength of 20 MPa.

(6) Inlet/Outlet Structure for Drainage Pipe Culvert

Inlet/outlet structure types for pipe culverts are determined in consideration of the existing site conditions and detour plan of drainage ditch, irrigation canal and so on.

Inlet/outlet structures used in the Project are in **Table 7.2.7**.

(7) Selection of the Combination of Pipe Class and Bedding Class

According to the design method described in "Chapter-3 Design Standard", applicable pipe culverts by soil cover depth, pipe strength class and bedding class is summarized in **Table 7.2.6**.

Table 7.2.6 Combination Types and Applicable Range of Cover Depth (m)[2]

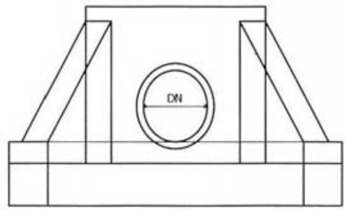
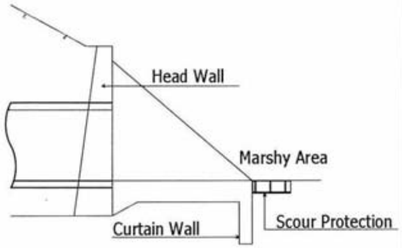
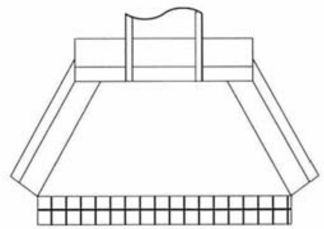
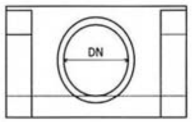
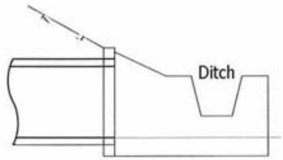
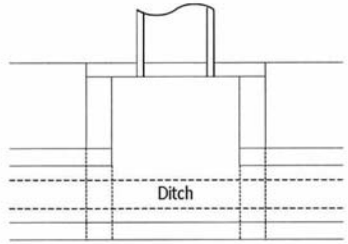
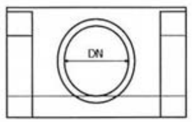
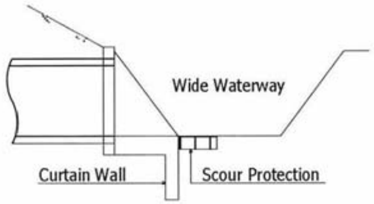
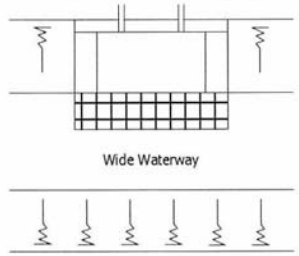
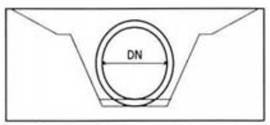
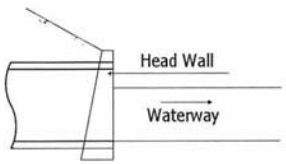
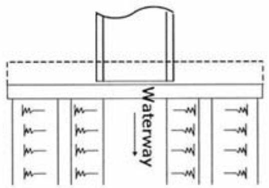
Pipe Strength Class	L	L	M	M	M	H	L
Bedding Class	C-1	C-2	B	S	A	A	C-1
Bedding Factor			1.9	2.2	2.6	2.6	
Pipe Diameter 0.45	- 0.5	- 0.9	- 3.0	- 3.9	- 4.9	- 5.9	> 5.9
Pipe Diameter 0.60	- 0.5	- 0.9	- 2.9	- 3.8	- 4.8	- 5.8	> 5.8
Pipe Diameter 0.90	- 0.5	- 0.9	- 3.0	- 3.9	- 4.9	- 6.6	> 6.6
Pipe Diameter 1.20	- 0.5	- 0.9	- 2.8	- 3.7	- 4.7	- 6.3	> 6.3
Pipe Diameter 1.50	- 0.5	- 0.9	- 2.8	- 3.5	- 4.5	- 6.2	> 6.2

Note: "C-1" means reinforced concrete surrounded type and "C-2" means plain concrete surrounded type

1 Source: "A Guide to Design Loadings for Buried Rigid Pipes, (Transport and Road Research Laboratory Overseas Unit; UK)"

2 Source: "A Design Manual for Small Bridges (Transport and Road Research Laboratory Overseas Unit; UK)", "Depth of Cover Charts (from ARC Pipes Company Catalogue) "

Table 7.2.7 Type of Inlet/Outlet Structure for Pipe Culvert

	Front View	Side View	Plan	Comments
Type-1				- This type is used when the water is flown in or out from/to the marshy area. - U-shape RC retaining wall structure. - Scour bed protection is installed if necessary.
Type-2				- This type is used when the water is flown in or out from/to the ditch at the toe of embankment. - U-shape RC retaining wall structure.
Type-3				- This type is used when the water is flown in or out from/to the wide waterway at the toe of embankment. - U-shape RC retaining wall structure. - Scour bed protection is installed if necessary.
Type-4				- This type is used when the pipe culvert is installed along the waterway crossing OCH alignment. - Only head wall structure.

7.2.3. Retaining Wall

(1) Introduction

This section of the report describes retaining walls and their design. Retaining walls are used where the R.O.W. is restricted by adjacent land use or where the toe of an embankment requires adjustment due to an adjacent structure's wing wall.

(2) Selection of Retaining Wall Type

1) General

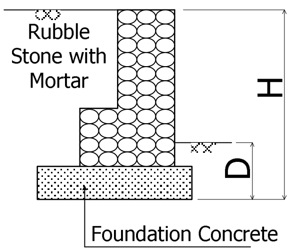
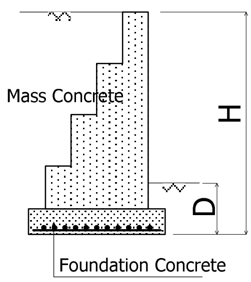
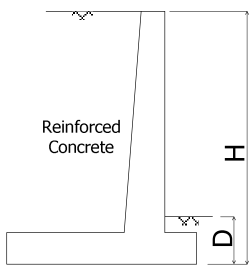
The principal types of retaining wall generally in use are as follows:

- Rubble Masonry Walls
- Gravity Concrete Walls
- Reinforced Concrete Walls

2) Characteristics and Considerations of Each Type of Wall

Retaining wall types above are applied depending on the required height from the foundation to the top of structure and foundation ground conditions.

Table 7.2.8 Type of Retaining Wall and Characteristics

Retaining Wall Type	Typical Cross Section	Characteristics
Rubble Masonry Walls (RDA Standard)		- Limited to heights of less than 2.9 m for retained soil. - Generally used at the toe of embankments to connect wing-walls and tipped fills smoothly. - Spread foundation is used. - Minimum embedded depth D is 90 cm. - Minimum berm width of 1 m is required at the top of masonry walls.
Gravity Concrete Walls (RDA Standard)		- Limited to heights of less than 5 m. - No reinforcement in wall. - Minimum embedded depth D is 1.5 m. - Spread foundation is used. - Stepped type is conventionally used for normal highway in Sri Lanka
Reinforced Concrete Walls		- Limited to heights of bigger than 5 m. - Reinforcement concrete walls consist of "T-shaped" and "L-shaped" walls. The selection of the appropriate type of wall is dependent on highway geometry. - Minimum embedded depth D is 1.5 m. - A pile foundation may be used for walls whose foundation is in poor condition.

CHAPTER 8

CONSTRUCTION PLAN

CHAPTER 8 CONSTRUCTION PLANNING

8.1. General

Construction planning is mainly comprised of establishing a construction method and preparing a construction time schedule. The results of this work are utilized in estimating construction costs and establishing a project implementation schedule.

8.2. Construction Area

The route of the OCH Northern Section -1 has been planned mainly along the low land such as marsh and paddy fields in order to reduce demolition of existing buildings and houses and resettlement. For that the embankment work of the OCH Northern Section -1 has become huge volume for the construction of a highway with total length of 8.3km., because High embankment is planned to avoid damage to road structures from floods and also to allow for the clearance for the underpass structures. Therefore, the viaduct has been planned on soft ground area considering environment impact around construction area to reduce of huge hauling volume of dumping and borrow material.

Northern section 1 starts from Sta. 8+200 and ends at Sta. 16+500, between the Interchange with route AA001 Bypass (Kadawatha) and the route AB010 (Kaduwela). However, between existing AA001 (around Sta. 8+200) and AA001 Bypass will not be constructed until the construction of the Northern Section 2 starts.

8.3. Major Work Items

Quantities of major construction works are as shown in **Table 8.3.1**. Layout of structures, bridges, box culverts are shown in **Fig. 8.3.1**.

Table 8.3.1 Quantities of Major Construction Works

Item			Unit	Volume
Road Works	Soft Soil Treatment works	GCP	m	382,000
		Band Drain	m	532,000
	Earth works	Cutting & Filling	m^3	607,000
		Embankment (Borrow material)	m^3	800,000
	Pavement works	Asphalt Concrete	Ton	61,000
		Aggregate Base	m^3	56,000
		Sub base	m^3	47,000
		Selected Material	m^3	38,000
Bridge Works	Kelani River Bridge Viaduct	PC Bridge	m	1,365
		Metal Bridge	m	322
	Ramp Brodges	PC Bridge	m	407
		Metal Bridge	m	506
	Over pass Brodge	PC Bridge	m	226
Box culvert		Underpass, Drainage	no.	27
Interchange		A!-IC, B214-IC	no.	2

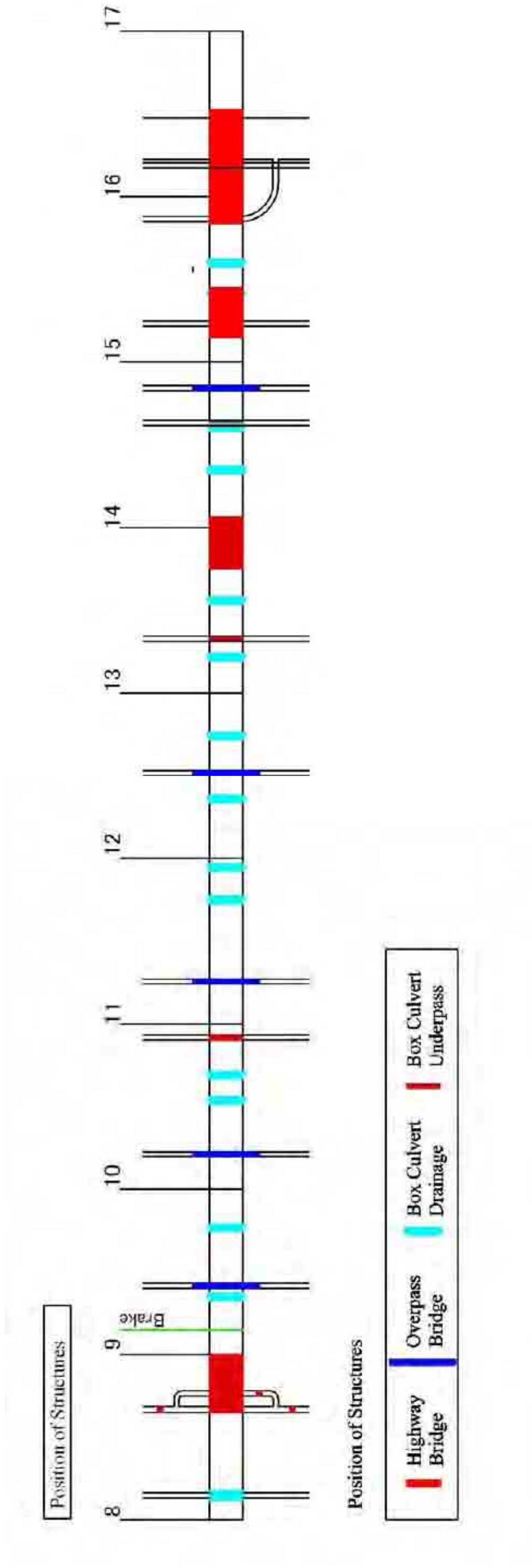


Fig.8.3.1 Layout of Structure of Northern Section - 1

8.4. Highway Construction

8.4.1. General

In the northern section 1 section of OCH, a large quantity of embankment materials is to be hauled from several borrows pits. The construction work should be carried out in such a manner to use main roads (AA001, Sapugaskanda Biyagama Road, BB214 and AB010) to transport material from borrow pits or suppliers' material to the intersection of the OCH. At the job site, construction of temporary road under the OCH as a pilot shall be used as much as possible for transportation of material. Where local roads are used for transportation, pavement strengthening/repairing of local roads will be necessary during the construction period.

Highway construction will be executed after land acquisition is completed as shown below flowchart.

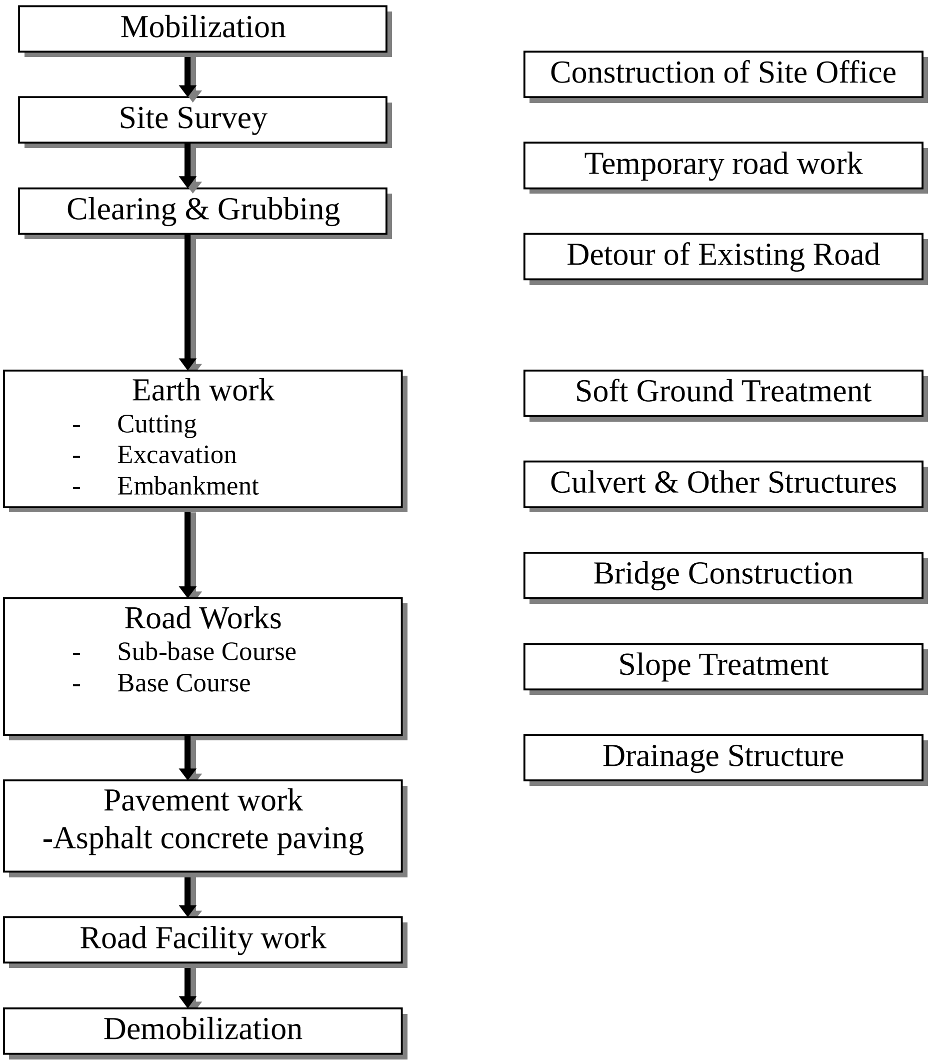


Fig. 8.4.1 Highway Construction Flow

8.4.2. Borrow Material Plan

The large quaintly of borrow material shall be hauled from several borrow pits. The existing borrow pits are shown in **Fig. 2.2.16** and **Table 2.2.13** of **Chapter 2**. The borrow pits are located at distances between 13 km and 25 km (average 20km) from OCH.

8.4.3. Quarry Plan

There are several operating quarry sites around the OCH route and are located at distances between 2 km and 18 km (average 15km) from OCH. The quarry sites are shown in **Fig.2.2.16** and **Table 2.2.14** of **Chapter 2**.

8.4.4. Disposal Plan

It is necessary disposal area for the reuses less excavated soil such as soft soil. Candidate sites of disposal are shown in **Fig.2.2.16** and **Table 2.2.15** of **Chapter 2**. The candidate disposal sites are located at distances between 11 km and 27 km (average 25km) from OCH

8.4.5. Materials Transportation Plan

Transportation of the Borrow and Quarry materials from there's pits or sites to OCH project site shall be transported via main roads, Route AA001, Sapugaskanda Biyagama Road and BB241 to OCH in order to minimize disturbance to public traffic. Route AA001 road shall be used minimally in consideration of its heavy volume of public traffic, therefore cut soil between A1-IC and Sapugaskanda Biyagama Road will be used for the compacted sandy soil and filling material of A1-IC Ramps and A1 Bypass to be avoided traffic interference of the AA001 road. (refer to **Fig.8.4.2**).

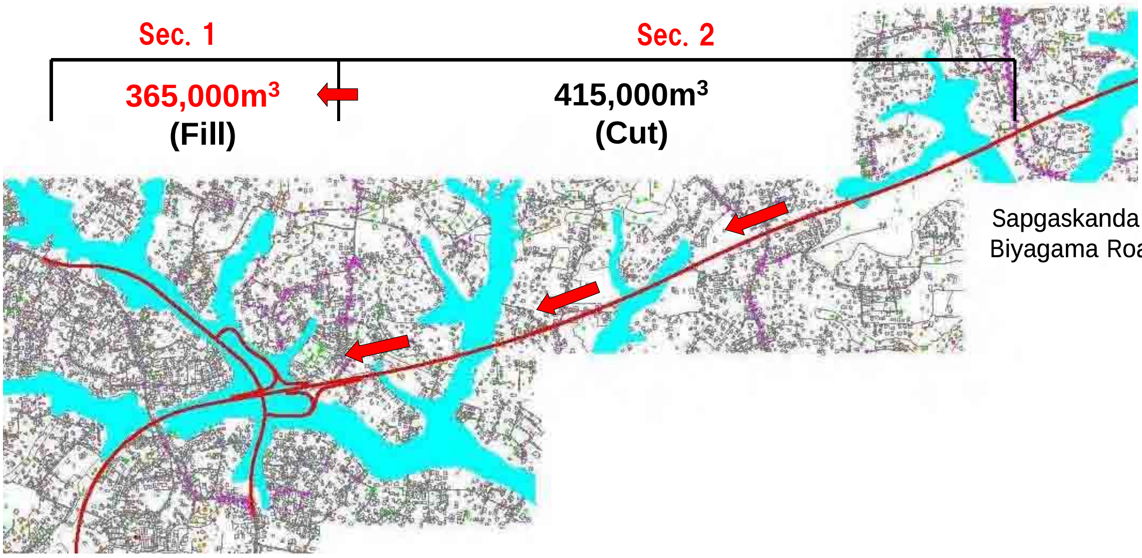


Fig. 8.4.2 Haulage plan of the material for A1IC

8.4.6. Pavement Works Plan

There are several companis producing and supplying asphalt concrete and their details and capacity are given in **Table 8.4.1**, and **Fig.8.4.3**.

Table 8.4.1 Asphalt Suppliers

No.	Supplier	Address	Max capacity (ton /hour)
1	International Construction Consortium Ltd.	Madapatha	60
2	Maga Engineering	Homagama	60
3	Fumihiko Engineering Construction Pvt.Ltd.	Paliyagoda	100

Fig. 8.4.3 Location of Asphalt and Concrete Plants

8.4.7. Soft Soil Mitigation Plan

The Gravel Compaction Pile (GCP) method will be mainly used for the soft ground improvement in the OCH Northern Section – 1 Project.

Soft soil improvement working flow is shown as **Fig. 8.4.4** below:

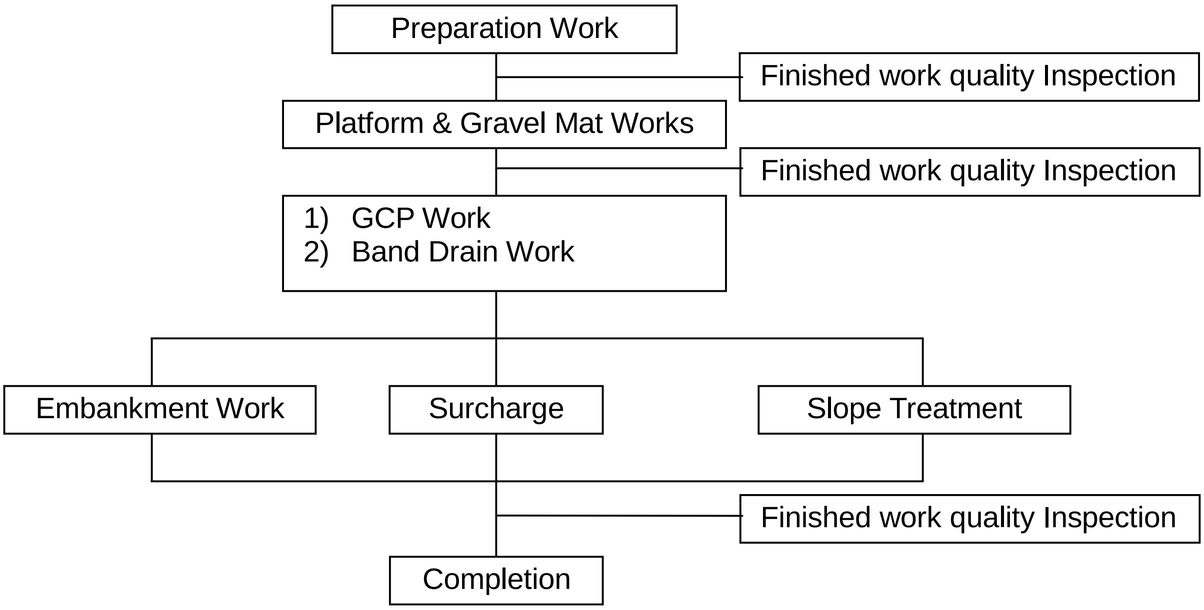


Fig. 8.4.4 Flowchart of Soft Soil Improvement Work

(1) Platform and Gravel Mat

To do the improvement of the soft ground, the Platform and gravel mat should be made before GCP work. The Platform will be made by sandy soil and the gravel mat will be made by quarry material on the soft ground to be improved workability of heavy equipment for GSP work. Platform and gravel mat materials shall be speared by bulldozer.

(2) Gravel compaction Pile

Working flowchart of Gravel Compaction Pile is shown as **Fig. 8.4.5**.

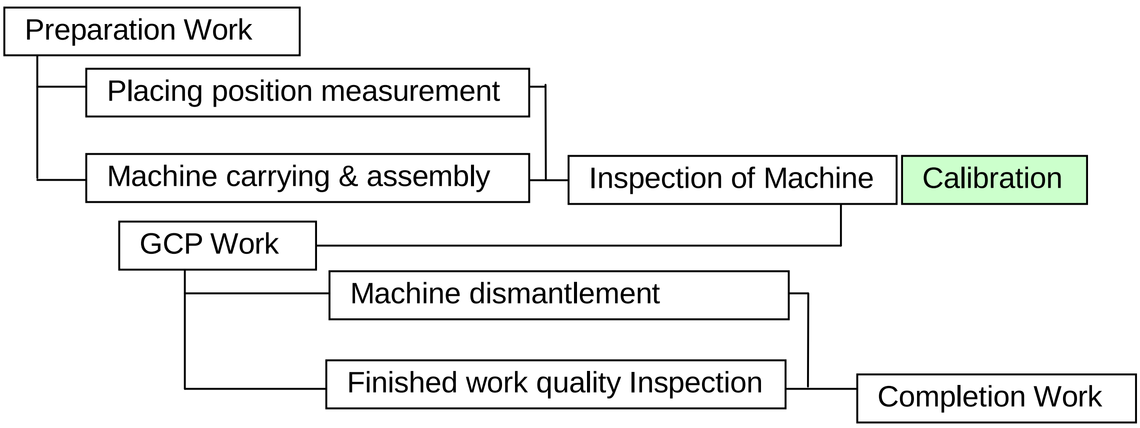


Fig. 8.4.5 Flowchart of Gravel compaction Pile

8.5. Bridges

Pre-stressed Concrete (PC) I girder bridges are selected for main highway bridges (exclude A1-IC main highway and ramps viaducts) and overpass bridges in the Northern Section 1. Because span length of all bridges equal or less than 35.75m (including .Kelani Bridge). A1-IC main highway and ramps viaducts will be constructed by steel. Steel I girder for main highway and steel box girder for ramps will be produced in Japan. Abutments are selected reversed T type, piers are selected wall type for Kelani River Bridge and reversed T type for other bridges, and foundation is pile foundation is designed by geological conditions.

8.5.1. Concrete Procurement

There are several companies produce and supply ready-mixed concrete in the Colombo region, the details of the companies and their capacities are given in **Table 8.5.1**.

Table 8.5.1 Concrete Suppliers

No.	Suppliers	Address	Capacity (m^3/hour)
1	Sanken Lanka (PVT) Ltd	Colombo 14 & Peliyagoda	20
2	Informax Construction (PVT) Ltd	Colombo 10	60 & 90
3	Tudawa Brothers	Colombo 5	25
4	Sunbee Ready Mix	Battaramulla	60
5	Maga Engineering	Gothatuwa	60
6	International Construction Consortium	Bokundara	50
7	Keangnam Ready Mix	Malabe	120
8	Oru Mix Lanka Pvt. Ltd.	Wellampitiya	20
9	Tokyo Cement Co.	Peliyagoda	120

8.5.2. Foundation Works

(1) Excavation

Excavations shall be carried out in accordance with the most practical methods at each bridge location. In general, foundation works for concrete structures shall not be programmed during wet seasons. The general method of excavation will be an open cut excavation.

For the foundations in the Kelani River will be required sheet pile cofferdam and temporary bridge and landing stage to be constructed substructure.

Rough arrangement drawing of temporary bridge and landing stage are shown as **Fig.8.5.1**.

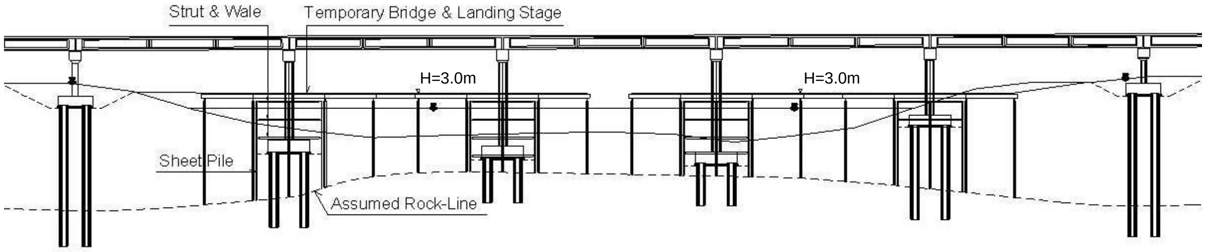


Fig. 8.5.1 Temporary Bridge & Landing Stage Plan

(2) Bored Piles

In consideration of the geological conditions, All-Casing Method will be adopted for the execution of bored pile. The working flow of All-Casing Method is shown as **Fig. 8.5.2**.

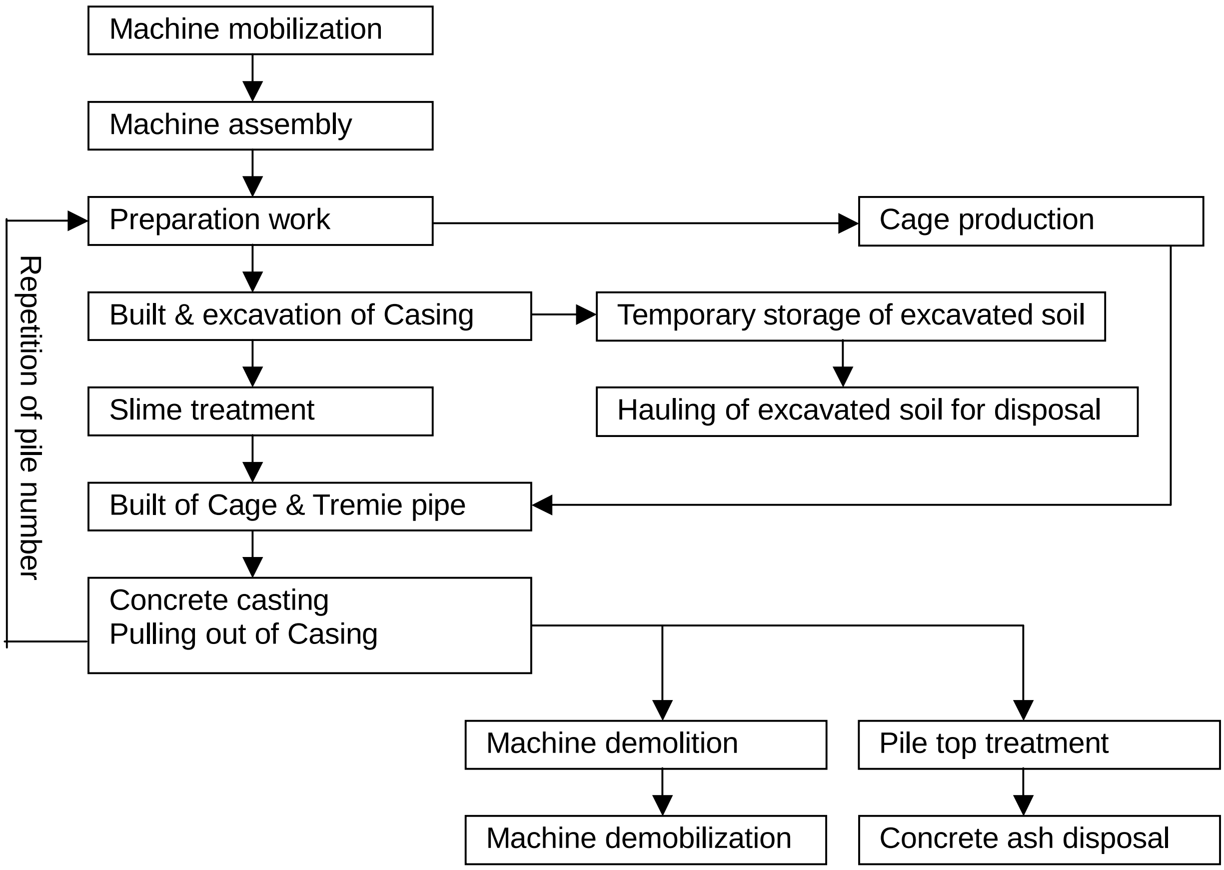


Fig. 8.5.2 Flowchart of Bored Pile Work

8.5.3. Sub-structure

The progressive completion of the foundation works will enable the subsequent works to proceed with the construction of the reinforced concrete substructures. The construction of these substructures shall be in conventional reinforced concrete construction up to the superstructure.

(1) Formwork

Ordinary water proofing quality plywood or coated plywood will be used for formwork as allowed by the Specification. Form ties with plastic cones will be used for securing of forms in place. Lift height will be determined in accordance with the construction joints approved by the Engineer. Generally, the walls and piers shall be 1 to 5 lifts.

(2) Steel Reinforcement

All reinforcement will conform to the requirements of BS 4449 Grade 460, Type 2 and will be purchased from local manufacturers or imported. Cutting and bending bars shall be carried out on site at a bar bending yard and shall be carried out in accordance with the specifications and the code of practice. All bars will be securely fixed with annealed wire and spacer blocks (mortar or concrete) made with the same quality of placed concrete and will be tied to reinforcement to ensure required concrete cover.

(3) Concreting

In general, concrete shall be placed in accordance with the requirements of Technical Specifications and in a manner to suit the various conditions which may be encountered, using any or combination of the following techniques: concrete pump cars, cranes equipped with skip buckets of appropriate sizes, bins, chutes, or manually. In all methods used, slump values specified shall be maintained during placing and care shall be given to prevent segregation of the constituents. Compaction of concrete shall be by electric, mechanical or pneumatic drive, immersion type vibrators or formwork mounted type in accordance with the Technical Specifications. When concrete has sufficiently hardened, construction joint surfaces will be green cut by wire brush and water jets.

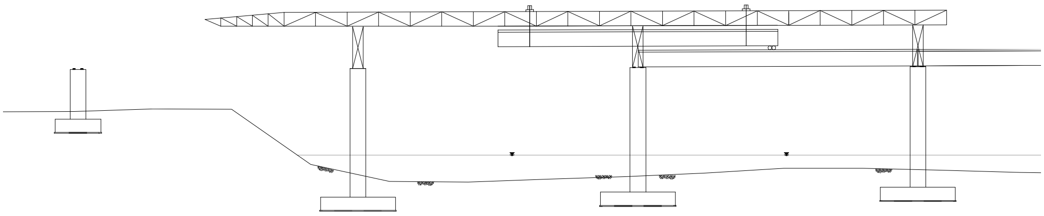
8.5.4. Superstructure

Initially, land near to the bridge approaches will be reserved for the beam casting activities. The land will be levelled by cutting and/or filling.

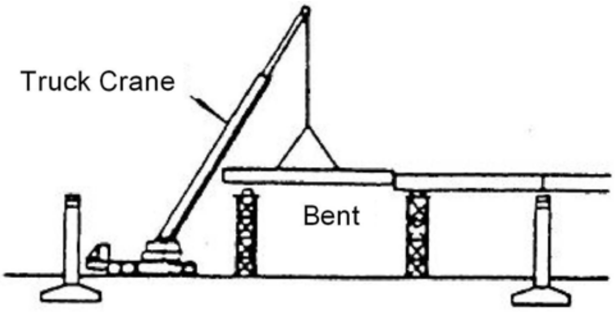
Post-tensioning of the PC I-girders will be carried out in one step after they have reached adequate strength and prior to their erection. The pre-stressing of the beams will be in accordance with the requirements of the detailed design and the specification requirements.

As for the pre-cast post-tensioned I girder-bridge, the I-girders shall be constructed in an area offset from each bridge abutment under the supervision of the specialised staff. The PC I-girders will be erected by erection girder for V2, V3, V4 highway viaducts which are located in mash area and H9 (Kelani River Bridge) on Kelani River (refer to Fig.10.4.4 for Kelani River Bridge Construction Plan).

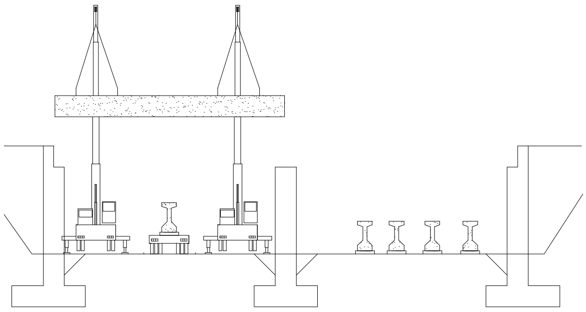
The steel I-girders of V1 highway viaduct and steel box girder bridges of V5 to V8 ramps viaducts will be erected by truck crane and vent and PC I-girders of V9 and V10 ramps viaducts will be erected by truck crane. (rerfer to **Fig.8.5.3**)



(a) Erection Girder Method



(b) Truck Crane and Bent Method



(c) Mobile Crane Method

Fig. 8.5.3 Erection of PC I-Girder

The crosses beams will then be cast-in-site and post-tensioned after the required time and strength have been achieved. The construction of the deck slab is on the 75mm thick pre-cast reinforced concrete panel (RC panel) that is placed in between the PC I-girders as shown on **Fig. 8.5.3**.

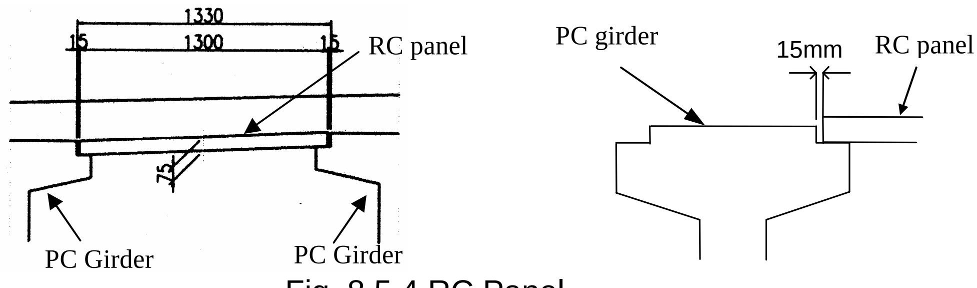


Fig. 8.5.4 RC Panel

For the cantilevered slabs, steel brackets shall be installed on the side of PC I-girder, or shall provide the support from bottom for formwork and the scaffolding for the slab construction.

The slab shall have a broom finish to enhance adherence between the concrete and the wearing course to follow. Finishing works including the approach slabs, reinforced concrete barrier and other miscellaneous construction shall be completed following the slab construction.

8.6. Diversion Road Plan

Diversion roads for crossing roads should be provided during the construction of Overpass bridges and Underpass box culverts and a sufficiently wide diversion road is required for the public traffic. Existing crossing roads that needed diversion road are listed in **Table 8.6.1**.

Table 8.6.1 Diversion Road List

No.	Location	Station	Road Name	Remark
1	OCH Main Carriageway	09+415	Gonahena-Mawatha Road	Overpass (O8)
2		10+210	Kadawatha-Nawaramandiya Road (B169)	Overpass (O9)
3		11+261	Siyambalape-Galwalkada Road (B401)	Overpass (O10)
4		12+518	Sapugaskanda-Biyagama Road	Overpass (O11)
5		14+843	Layanthi-Mawatha Road	Overpass (O12)
6	A1 Bypass	01+293	Bandaranayaka-Mawatha Road	Box culvert

8.7. Diversion Channels Plan

The OCH Northern Section – 1 which is mainly passing through the marshy area divides the flood plain into two parts on the left and the right sides. Under the existing conditions, shallow flow can be seen in main watercourse and both sides get, simultaneously inundated with heavy rainfall. According to the OCH Northern Section -1 alignment, main watercourse shall be realigned at some locations as the existing waterway will be filled by road embankment. Therefore, drainage canals have been designed with hydrological calculations. However, during the construction it is required to provide temporary waterways in small scale at some locations in order to have a continuous water flow from upstream to down stream. Related chart is shown in **Fig. 8.7.1**.

Fig.8.7.1 Diversion Route Plan

8.8. Construction Schedule

8.8.1. Construction Period

By adopting viaduct and gravel compaction pile method for soft ground treatment countermeasure, the hauling volume of unsuitable soil disposal and borrow material was reduced. Therefore, the possible construction period shall be set at 3 years. The construction time schedule is planned as shown in **Table 8.8.3** and the schedule for required heavy vehicle and machineries are shown in **Table 8.8.4**.

8.8.2. Kelani River Bridge Construction

To complete the construction work of the Kelani river bridge in one year or one year half, the surface level of temporary bridge and landing stage will be set 3.0m MSL height because almost 8 months can be used for four piers construction where in the river. Between December and May, installation of temporary work, bored pile work, concrete casting for piers and related works shall be completed. From middle of July to August, temporary facilities will be removed.

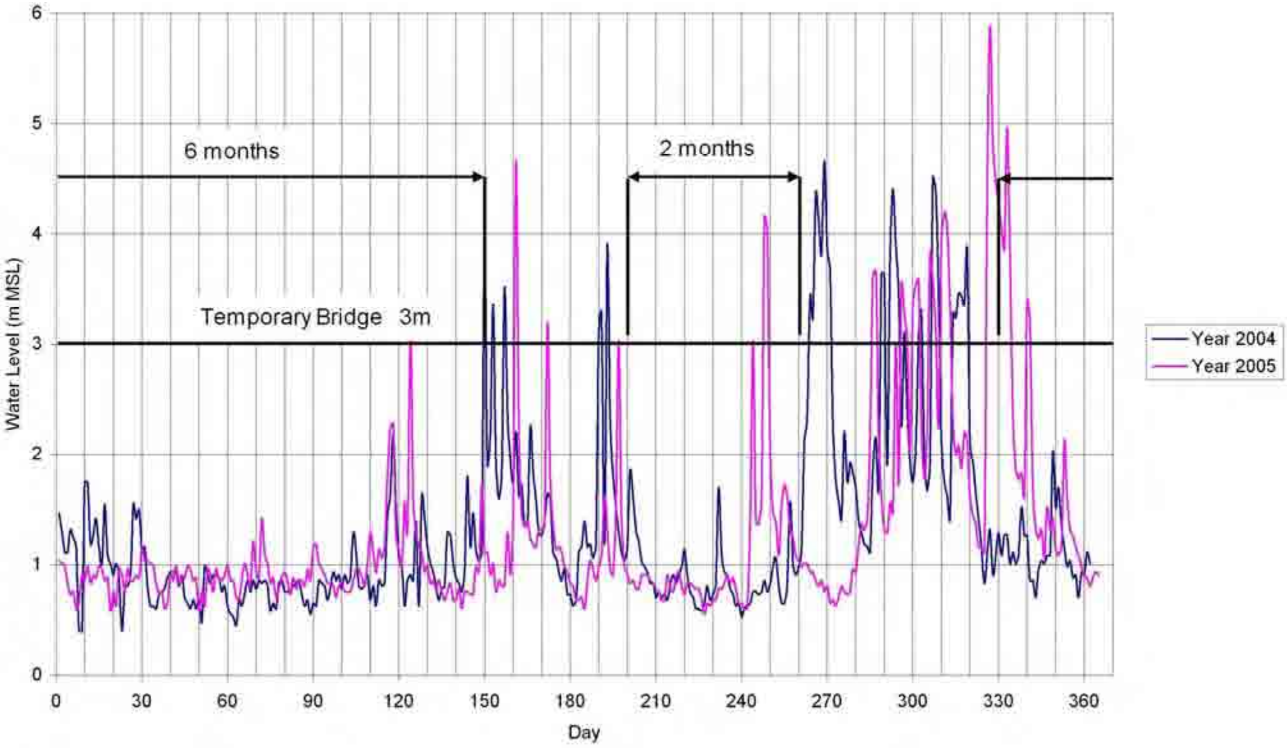


Fig. 8.8.1 Daily Water Level of Kelani River

8.8.3. Effective Working Days

Working days ratio for the project is estimated according to rainy days (daily rainfall over 10mm), including Sundays and holidays. The following table shows the data for the above factors.

Table 8.8.1 Working Days Rate

Month		1	2	3	4	5	6	7	8	9	10	11	12
Rainy Day (Over 10mm/day)		5.60	2.80	6.20	8.60	13.00	9.20	5.00	6.20	7.40	16.40	12.60	6.80
Holiday	Sunday	4	4	4	5	4	4	5	4	5	4	4	5
	National Holiday	2	2	1	3	3	1	1	1	1	1	2	2
	Other Holiday				4								
Dates of Overlapping Holiday & Rainy Day		1.08	0.60	1.00	3.44	2.94	1.53	0.97	1.00	1.48	2.65	2.52	1.54
Monthly Dates		31	28	31	30	31	30	31	31	30	31	30	31
Number of Days Worked		20.48	19.80	20.80	12.84	13.94	17.33	20.97	20.80	18.08	12.25	13.92	18.74
Rate of Opreration		0.66	0.71	0.67	0.43	0.45	0.58	0.68	0.67	0.60	0.40	0.46	0.60
Average of Operation Rate		0.58											

* Rainfall data : Average between 2002 & 2006 (Homagama)

Rainly Season Average (4, 5, 10, 11) : 0.434

Dry Season Average (1, 2, 3, 6, 7, 8, 9, 12) : 0.646

However, the earthwork can not do during few days after the rainy day. Therefore, the working ratio for the earthwork shrinks than above ratio. Then, in consideration of these, its value was found with 0.41.

Table 8.8.2 Construction Schedule

Work Items	Unit	Qty.	1												2												3											
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36
1 Preparatory Works	l.s.	1																																				
2 Soft Ground Treatment Works	m	382,000																																				
3 Earthworks (Cut & Fill)	m3	600,000																																				
(Embankment)	m3	778,000																																				
4 Pavement Works	ton	60,000																																				
5 Bridge Works (V1)	l.s.	1																																				
(V5 ~ V8)	l.s.	1																																				
(V2)	l.s.	1																																				
(V3)	l.s.	1																																				
(V4)	l.s.	1																																				
(V9 ~ V10)	l.s.	1																																				
(Kelani River)	l.s.	1																																				
(Over Bridges)	l.s.	1																																				
6 Box Culvert Works	nr.	27																																				
7 Ancillary Works	l.s.	1																																				

Table 8.8.3 Preliminary Schedule for required Heavy Vehicle and Machineries

No.	Equipment	Capacity	Required No. (estimate)
01	All Casing Excavation Machine		5
02	Truck Crane	120t	16
03	Special Truck	85t	1
04	Crawler Sand Pile Machine for GCP		15
05	Erection Girder		4
06	Bent		12
07	Band Drain Machine		5
08	Backhoe	0.8m3	35
09	Bulldozer	21t	15
10	Ripper Bulldozer	32t	1
11	Tire Roller	20t	10
12	Asphalt Paver	2.4-6.1m	2
13	Crawler Crane	50t	6
14	Crawler Crane	80t	2
15	Vibration Hammer	60kw	2
16	Generator	200kva	8
17	Air Compressor	10-11.5m3	8
18	Pre-stressing pump/jack	7S12.7	12
19	Pre-stressing pump/jack	15S12.7	4
20	Pre-stressing pump/jack	1S21.8	2
21	Pre-stressing pump/jack	12S15.2	4
22	Pre-stressing pump/jack	1S28.6	2
23	Dump Track	10t	150

Note: Small equipment such as Generator 20Kva, Road Marker, Water Bowser, etc. is not included in above table.

CHAPTER 9

ENVIRONMENTAL STUDY

CHAPTER 9 ENVIRONMENTAL STUDY

9.1. Background

An Environmental Impact Assessment (EIA) was carried out in the Feasibility Study (FS) for the OCH by JICA in 2000 (2000 EIA). The environmental study was conducted on the alignment selected in the FS as preferable trace based on the concept of urban sustainability, which consists of the three factors of economic, social and environmental sustainability. The EIA detailed the environmental impacts of implementing the preferred alignment, recommended specific and practical mitigatory measures for reducing or avoiding adverse environmental impacts.

The EIA report was approved by the Central Environmental Authority (CEA) in February 2001 with three years validity. The validity was extended until May 2007 based on the request from RDA. Note that RDA has requested CEA for the additional three years extension for the approval (see **Appendix 9.1**). In the conditional letter of the approval issued to the RDA, CEA has recommended a deviation of the trace based on the findings of the 2000 EIA. The recommended deviation, from station 17+000 to station 21+800, a distance of 4800m, was through marsh and paddy lands except for a length of 500m in intermittent high ground (herein after " Kaduwela deviation").

It was decided that fresh approval is required for the above deviations as stipulated in section 23 EE of the National Environmental Act at the Monitoring Committee of the CEA held in March 2003. The CEA requested RDA to prepare the Supplementary Environmental Impact Assessment (SEIA) report for the deviation route.

The SEIA study for Kaduwela deviation has been conducted by JICA Study Team in the Detailed Design for the Southern Section of the OCH based on the Terms of References (TOR) prepared by the CEA. The purpose of the SEIA is to ensure that the potential environmental consequences of the proposed deviation of the OCH are recognized early for required mitigatory action, which will be taken into consideration in project planning and design. The report was approved by CEA in July 2005.

In the meantime, the SEIA study for Biyagama deviation has been also conducted by RDA in July 2005 based on the decision at the Monitoring Committee of the CEA held in March 2003. Note that the reason why the SEIA study for Biyagama deviation was conducted by RDA is due to the new JICA Guidelines for Environmental and Social Considerations (April 2004), which stipulates that necessary environmental clearance for proposed projects are under the responsibilities of project proponents. The supplementary EIA for Biyagama deviation has been conducted by a local consulting company. The SEIA report was opened to public through public inspection stipulated in National Environmental Act. The inspection was conducted form August 25th 2006 to September 28th 2006 for 30 working days after public notice at newspaper (see **Appendix 9.2**). The report was approved by CEA in February 2007 (see **Appendix 9.3**).

The SEIA report for Biyagama deviation was reviewed by the JICA Study Team in the Detailed Design for Northern Section 1 in terms of compliance with the JICA Guidelines.

In addition to those SEIA, CEA has requested for a Supplementary Environmental

Impact Assessment (SEIA) for the deviation of layout of Kadawatha Interchange. The SEIA study has been conducted by RDA. The TOR for the SEIA is referred to **Appendix 9.4**. The draft of the SEIA report has been already prepared by a local consultanting company and it is expected to obtain approval from CEA by the end of September 2007.

The brief history of the OCH is shown in **Fig. 9.1**.

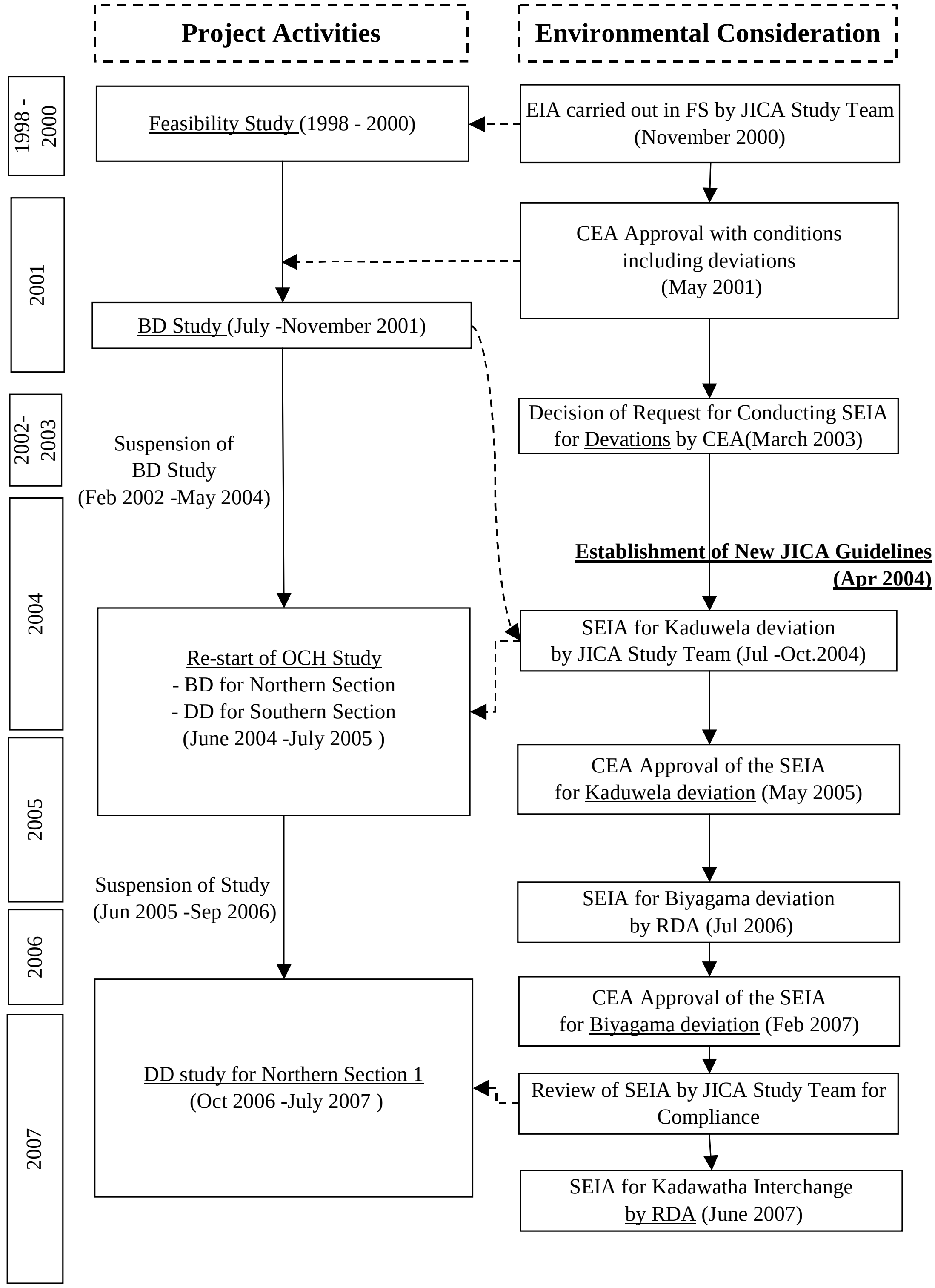


Fig. 9.1.1 Brief History of the OCH

Subsequent to the findings of the 2000 EIA and the SEIA for Biyagama deviation, further studies have been conducted on the specific environmental impacts of the project and the required mitigation measures for the Northern Section 1. The Environment Management Plan(EMP) including the Environmental Monitoring Plan for the Northern Section 1 of the OCH Project has been formulated based on these additional considerations and EIA findings. The EMP for the Northern Section 1 of the OCH presents the implementation details of the environmental protection measures recommended for the pre-construction, construction and operational phases of the Project.

Presently, Resettlement Implementation Plan (RIP) for the Northern Section 1 of the OCH Project has been preparing under the responsibility of RDA. Since involuntarily resettlement is considered critical issue for the Project, the review of the preparation activities has been conducted in the course of the environmental study.

9.2. SEIA for the Biyagama Deviation

9.2.1. General

The deviation to the road trace at Biyagama had been considered in the D/D to mitigate the social issues of the project. However, as detailed studies had not been conducted to assess whether the proposed deviation at this location mitigates social and other environmental impacts CEA determined that fresh approval is required for the above deviations as stipulated in section 23 EE of the National Environmental Act. CEA requested RDA to prepare the Supplementary Environmental Impact Assessment (SEIA) Report for the deviation route.

The purpose of the SEIA is to ensure that the potential environmental consequences of the proposed deviation of the OCH are recognized early for required mitigatory action, which will be taken into consideration in project planning and design.

9.2.2. Summary of the SEIA

(1) Existing Environment and Site Description

The physical environment of the proposed deviation was studied in detail by using available data and field surveys including sample analysis. The proposed deviated trace traverses mostly on marshy swamps and agricultural plots, which serve as flood regulating territory (see **Fig. 9.2**). The project area is in the western coastal wet zone, which receives more than 2500 mm rainfall from the bimodal rainfall pattern of the country. The low land swamps and the low land paddy fields play a significant role in preventing and controlling flooding in the area. There are significant numbers of drainage paths that cross the proposed deviation. The soils in the project area have been identified and the general geology and mineral resources of the area have also been described. The water quality data in the 2000 EIA was supplemented using the data available at the CEA for the Kelani River, which was used for the Supplementary Environmental Impact Assessment Study for the Kaduwela deviation in 2004.

Four main ecological habitats have been identified in the project area. These include Home gardens, cultivated lands, road reservations and aquatics and associated areas. Distribution of floral and faunal species in the project area has been identified with respect to the above habitats. Terrestrial and aquatic flora found in the proposed trace with regard to taxonomic (native, introduced, endemic) and conservation status (abundant and threatened) have been listed. All the faunal species including invertebrates and vertebrates (gastropods, dragonflies, butterflies, fish, amphibians, reptiles, birds and mammals) have been also listed with their taxonomic and conservation status.

Fig. 9.2.1 Deviation Route and Previous Route

Important land use sites, utility facilities, industrial sites, educational and religious places, burial grounds and recreational areas affected by the road trace have been enumerated and tabulated. The affected transport, irrigation canals, drinking water, electricity and telephone facilities, hospitals and health facilities, markets and other institutions have been identified. The general socio-economic conditions have been examined and described. The socio-economic profile in the study area including demographic characteristics social infrastructure, cultural ties income generation and employment has been summarized under each sub heading while the data compiled through surveys are tabulated. Other infrastructure facilities such as roads have been identified and listed.

(2) Anticipated Environmental Impacts

It was anticipated that the physical, ecological and social aspects of the environment in the project area would be affected during the pre-construction, construction and operation phases of the project. The impacts anticipated due to the different project implementation phases and different project activities are:

- Surface run-off and future flood water levels
- Irrigation and flood protection works
- Inundation levels in flood plains
- Future urbanization of the sub- catchments
- Water quality changes
- Noise and vibration levels
- Air quality changes
- Loss of habitat
- Fauna and flora
- Fisheries and aquatic life
- Community severance
- Changes in property values
- Changes in accessibility and demand for schools, religious and business institutions
- Impacts on transportation
- Impacts on special social groups
- Impacts on households and businesses to be relocated
- Impacts on employment opportunities
- Impacts on accessibility
- Fragmentation of agricultural lands
- Loss of agricultural land and production
- Changes in economic and socio-economic situation

(3) Measures to Mitigate Negative Impacts

In a development project of this nature, the negative environmental impacts cannot be totally avoided. It is generally accepted that such impacts would be mitigated to the extent possible and / or necessary to prevent environmental disaster. Therefore, mitigatory actions for all the significant adverse impacts have been recommended.

In order to mitigate and / or minimize the unavoidable adverse impacts of the proposed deviation, the mitigatory measures given below are proposed in the following project implementation stages.

- Pre-construction (Design Stage)
- Construction Stage
- Post-project Implementation Stage

The deviation of the route trace is by itself one of the mitigation measures to minimize the negative impacts on the communities and their properties, which is considered a critical issue in implementing the project. An adequate compensation program for the families affected by the project is also proposed. An effective compensation and income restoration program will be implemented to satisfy the affected people. Adequacy, timeliness and reliability are the main criteria adopted to fulfill an effective compensation program.

The impact on the drainage pattern will be avoided or minimized to an acceptable level by providing bridges, culverts and other drainage structures of the required capacity where necessary.

(4) Conclusions and Recommendations

- The flood level in the area encompassing the deviated trace will increase slightly due to the construction of OCH. However, further increase of flood level and changes in flow regime could be minimized if sufficient cross drainage culverts / bridges of required sizes, based on accurate hydrological computations are provided at proper locations.
- The construction of the road embankment in the deviated trace will cause drainage problems in the drainage of excess water from the paddy fields due to non-availability of drainage canals on either side of the road embankment. This aspect needs to be carefully studied in consultation with related stakeholders.
- The deviation route of the OCH does not pass though any natural forest, scrubland or any protected area.
- If the mitigation measures recommended are implemented, the impacts on the ecology of the wetland would not be significant.
- During the study some endemic and threatened floral and faunal species were identified in the project area. However taking into consideration the beneficial social impacts, it can be said that the total adverse environmental impacts are not significant.
- Except for a few houses in the deviated section, no significant and sensitive religious, historical or cultural places are affected due to construction of the proposed road.
- Since the road trace has been mostly designed to run through abandoned paddy lands people have no serious objections for the road in these locations.
- The most important decisions that the community need are final conclusions on the road right of way, the exact no of houses that will be demolished and the time frame of the project implementation, as this information is required by them to plan their own property development activities such as construction of new houses, selling of their lands etc.

- As much as possible the new road should not disturb the existing road network. At locations where the existing roads are disturbed suitable engineering solutions will be worked out to allow the local people to continue using these local roads. Installation of facilities to cross the new road at critically important points will provide opportunities for local people to maintain the same relations with communities whose residences will be physically separated due to construction of new road.

9.2.3. Examinations on the Compliance with the JICA Guidelines

(1) JICA Environmental Social Guidelines

JICA established the new guidelines for environmental and social considerations in March 2004. The objectives of the guidelines are to encourage the recipient governments to take appropriate considerations of environmental and social factors as well as to ensure that JICA's support for and examination of environmental and social considerations are conducted accordingly. The guidelines outline JICA's responsibility and procedures, and requirements for the recipient governments to facilitate achievement of the objectives.

The guidelines consist of three chapters, namely, "I. Basic Matters", "II. Process of Environmental and Social Considerations" and "III. Procedures of Environmental and Social Considerations". "I. Basic Matters" describes seven JICA's basic principles as below including policies and objectives.

A wide range of impacts to be addressed is covered.

- Measures for environmental and social considerations are implemented at an early stage
- Follow-up activities are carried out after cooperation projects are terminated
- JICA is responsible for accountability when implementing cooperation projects
- JICA asks stakeholders for their participation
- JICA discloses information
- JICA enhances organizational capacity

When JICA considers either the selection of proposed projects or the support for and examination of environmental and social considerations, JICA examines how the recipient governments meet the requirements that JICA requires as mentioned below.

EIA documents

- EIA documents must be written in official languages or in languages familiar to people within the host countries
- Impacts to be assessed and examined in terms of environmental and social considerations include impacts on natural environment as well as social considerations
- Multiple alternatives must be examined to avoid or minimize adverse impacts and to choose a better project option in terms of environmental and social considerations
- In addition to the direct and immediate impacts of projects, derivative, secondary and cumulative impacts are also to be examined and assessed to a reasonable extent

Accessibility to EIA documents

- It is requested that EIA documents be made open to local stakeholders including local people. In addition, EIA documents should be available for public reading at all times, and the making of copies of these for the local stakeholders should be permitted.

Compliance with Laws, Standards and Plans

- Projects must comply with laws, ordinances and standards relating to environmental and social considerations established by the governments that have jurisdiction over the project site
- Projects must, in principle, be undertaken outside protected areas that are specifically designated by laws or ordinances of the governments for conservation of nature or cultural heritage

Social Acceptability

- For projects with a potentially large environmental impact, sufficient consultations with local stakeholders, such as local residents, must be conducted via the disclosure of information from an early stage where alternatives for project plans may be examined. The outcome of such consultations must be incorporated into the contents of project plans.
- Appropriate consideration must be given to vulnerable social groups, such as women, children, the elderly, the poor and ethnic minorities.
  Involuntary Resettlement and loss of means of livelihood are to be avoided where feasible, exploring all viable alternatives. People to be resettled involuntarily and people whose means of livelihood will be hindered or lost must be sufficiently compensated and supported by project proponents, etc., in a timely manner.

Indigenous Peoples

- When projects may have adverse impact on indigenous peoples, all of their rights in relation to land and resources must be respected in accordance with the spirit of relevant international declarations and treaties. Efforts must be made to obtain the consent of indigenous peoples after they have been fully informed.

Monitoring

- In cases where sufficient monitoring is deemed essential for appropriate environmental and social considerations, such as projects for which mitigation measures should be implemented while monitoring their effectiveness, project proponents must ensure that project plans include feasible monitoring plans.

(2) Examination on Compliance with the “JICA Guidelines”

With regards to the SEIA report, the compliance with the JICA Guidelines for Environmental and Social Considerations was examined based on the requirements mentioned above.

EIA documents

The SEIA addresses not only ecological and hydrological aspects but also social aspects. The SEIA conducts examination of multiple alternatives through reviewing the FS report of OCH (2000). The SEIA report is prepared in English first, then consequently in Sinhara and Tamil.

Accessibility to EIA documents

The accessibility to the SEA report was ensured through public inspection.

Compliance with Laws, Standards and Plans

The SEIA was implemented under the National Environmental Act (NEA) of Sri Lanka.

Social Acceptability

The SEIA was informed to public-by-public notice before implementation and the report was opened to the public through public inspection.

Involuntary Resettlement

The deviation route was proposed to alleviate social impact caused by involuntary resettlement. People to be resettled involuntarily and people whose means of livelihood will be hindered or lost will be compensated and supported by the project based on the resettlement implementation Plan.

Indigenous Peoples

No indigenous peoples were identified at the project site.

Monitoring

Although the SEIA report doesn't contain any monitoring plan, the environmental management Plan (EMP) including environmental monitoring plan has been established in the course of the detailed design activities for Northern Section 1 that covers the deviation route.

The summary of the examination on the conformity with the JICA Guideline is shown in the **Table 9.1**.

Table 9.2.1 Summary of the Examination on the Compliance with the JICA Guideline

Items	Stipulations in the Guideline	SEIA
EIA documents	Written in official languages or in languages familiar to people within the host countries	The report was prepared in English first, then consequently in Sinhala and Tamil.
	Impacts to be assessed and examined in terms of environmental and social considerations including impacts on natural environment as well as social considerations	The SEIA addresses not only ecological and hydrological aspects but also social aspects.
	Examination of multiple alternatives	The SEIA conducts examination of multiple alternatives through reviewing the FS report of OCH (2000).
Accessibility to EIA documents	Open to local stakeholders and available for public reading at all times, and possible for making of copies of these for the local stakeholders	The accessibility to the SEA report was ensured through public inspection.
Compliance with Laws, Standards and Plans	Projects must comply with laws, ordinances and standards relating to environmental and social considerations established by the governments that have jurisdiction over the project site	The SEIA was implemented under the National Environmental Act (NEA) of Sri Lanka.
Social Acceptability	Sufficient consultations with local stakeholders via the disclosure of information from an early stage	The SEIA was informed to public-by-public notice before implementation and the report was opened to the public through public inspection.

Items	Stipulations in the Guideline	SEIA
Involuntary Resettlement	Avoid where feasible and sufficient compensation and support in a timely manner to affected People	The deviation route was proposed to alleviate social impact caused by involuntary resettlement. People to be resettled involuntarily and people whose means of livelihood will be hindered or lost will compensated and supported by the project based on the resettlement implementation Plan.
Indigenous People	When projects may have adverse impact on indigenous people, all of their rights in relation to land and resources must be respected in accordance with the spirit of relevant international declarations and treaties. Efforts must be made to obtain the consent of indigenous people after they have been fully informed.	No indigenous people were identified at the project site.
Monitoring	In cases where sufficient monitoring is deemed essential for appropriate environmental and social considerations, such as projects for which mitigation measures should be implemented while monitoring their effectiveness, project proponents must ensure that project plans include feasible monitoring plans.	The environmental management Plan (EMP) including environmental monitoring plan will be established in the course of the detailed design activities for Northern Section 1 that covers the deviation route.

(3) Conclusions

The SEIA addresses broad environmental impacts not only of ecological and hydrological aspects but also social aspects as required in the Guidelines. The SEIA conducts examination of alternatives for the alignment. The report is prepared in English, Sinhala and Tamil so that the local people could easily understand the environmental impacts caused by the project. Easy accessibility to the SEIA report was ensured through public inspection. The public notice of the SEIA study before implementation and information disclosure through public inspection has contributed in ensuring the social acceptability. With respect to involuntary resettlement, the project-affected people (PAPs) were identified through the socio-economic study and the SEIA proposes to compensate and support the PAPs through the project, based on the resettlement implementation plan (RIP). The SEIA report doesn’t contain a monitoring plan. However, the environmental management Plan (EMP) that will be established in the course of the detailed design activities for the Northern Section I will include an environmental monitoring plan.

Considering the results of the above examination of the SEIA, it could be concluded that the SEIA report is acceptable in terms of compliance with the JICA Guidelines.

9.3. Environmental Management Plan

Subsequent to the findings of the 2000 EIA and the SEIA for Biyagama deviation, further studies have been conducted on the specific environmental impacts of the project and the required mitigation measures for the Northern Section 1. The Environment Management Plan (EMP) including the Environmental Monitoring Plan for the Northern Section 1 of the OCH Project has been formulated based on these additional considerations and EIA findings.

The draft of the EMP has been finalized through close consultation with the Environmental and Social Division (ESD) of the RDA. The initial comments from the ESD on the draft of the EMP are shown in **Appendix 9.5**.

The EMP for the Northern Section I of the OCH presents the implementation details of the environmental protection measures recommended for the pre-construction, construction and operational phases of the Project.

RDA and the contractors should execute the environmental management plan with proper planning, coordination and management of all recommended environmental protection measures and activities

Note that the EMP will be included in the tender documents with the condition that bidders for construction of the Northern Section 1 of the OCH. An Environmental Management Action Plan (EMAP) will be submitted based on this EMP, when tendering their bid documents.

9.3.1. Pre-Construction Stage

In the pre-construction stage the Project related environmental issues are mainly the social impacts caused by land acquisition activities.

In addition, it is necessary to establish a base line for monitoring changes in ambient air quality, water quality and noise levels as a result of the OCH. Although the 2000 EIA has established the air and water quality and noise levels before the project, need to be updated since the EIA was conducted in early 2000.

Further, the potential hydrological impacts will have to be mitigated by including the recommendations of the Hydrological Study to be conducted by the RDA.

(1) Social Impacts

The objectives of the EMP with regard to social impacts are:

- To ensure that adverse impacts on community is avoided, mitigated or compensated
- To ensure better living conditions for project affected persons (PAPs)
- To ensure smooth and timely land acquisition

The mitigation measures recommended are:

- Prepare an inventory of PAPs and extent of damage to properties after the final ROW is identified
- Formulate a comprehensive Resettlement Action Plan (RAP), based on the national policy on resettlement and conduct validation of the RAP
- Conduct public consultations on the compensation package and relocation process
- Implementation of RAP plan to the satisfaction of the affected families

(2) Hydrological Impacts

The objective of the EMP in managing hydrological impact is:

- To minimize hydrological and drainage impacts such as flood levels and fluctuations of ground water table

The mitigation measures recommended are:

- Incorporating into the design the mitigation measures such as adequate cross drainage structures, recommended in the detailed Hydrological Study conducted as part of the detailed design

9.3.2. Construction Stage

(1) Negligence of Contractor and Workers on the Environmental Requirements

The objective of the EMP with regard to conduct orientation for contractor and workers is:

- To ensure that the contractor and the workers understand the environmental requirements and implementation of mitigation measures

The mitigation measure proposed in this regard is:

- Conducting special briefing and/or on-site training for the contractors and workers on the environmental requirement of the project

(2) Impact on Water Quality

The objective of the EMP with regard to water quality management is:

- To ensure adverse impacts on water quality caused by construction activities are minimized

The mitigation measures proposed in this regard are:

- Proper construction management including, training of operators and other workers to avoid pollution of water bodies by the operation of construction machinery and equipment
- Storage of lubricants, fuels and other hydrocarbons in self contained enclosures
- Proper disposal of solid waste from construction activities and labour camps
- Cover the construction material and spoil stockpiles with a suitable material to reduce material loss and sedimentation
- Avoid stockpiling of material close to water bodies
- Stripped material shall not be stored where natural drainage will be disrupted
- Servicing of vehicles and equipment at properly managed and equipped workshops, where waste oil is collected and disposed of at approved locations
- Sanitation arrangements at work sites and temporally accommodation facilities, such that no raw sewage could be released into drains or water bodies

(3) Impact on Air Quality/Dust

The objective of the EMP in relation to air quality management is:

- To minimize the air borne particulate matter released to the atmosphere

The mitigation measures recommended to achieve this objective are:

- All heavy equipment and machinery shall be fitted in full compliance with the national and local regulations
- Stockpiled soil and sand shall be slightly wetted before loading, particularly in windy conditions
- Fuel efficient and well maintained haulage trucks shall be employed to minimize exhaust emissions
- Vehicles transporting soil, sand and other construction material shall be covered
- Exposed areas shall be dampened
- Procurement of high efficient plant, maintenance of them in good conditions, and operated by trained and qualified personnel
- Selection of sites for materials extraction shall be away from residential areas in order to reduce the impacts from dust

(4) Noise/ Vibration

The objective of the EMP in relation to noise and ground vibration management is:

- To minimize noise level increases during construction operations
- To minimize/avoid the ground vibration due to construction operations

The mitigation measures recommended in this regard are:

- All heavy equipment and machinery shall be fitted in full compliance with the national and local regulations
- As a rule, the operation of heavy equipment shall be conducted in daylight hours
- Hammer-type pile driving operations shall be avoided during night and close to residential areas
- Construction equipment, which generates excessive noise shall be enclosed

- Well maintained haulage trucks will be used with speed controls
- Where blasting is required controlled blasting shall be employed under the supervision of Geological Surveys and Mines Bureau (GSMB) / Central Environmental Authority (CEA)
- Taking reasonable measures to minimize noise / vibration near noise sensitive areas such as schools and places of worship

(5) Sedimentation of Streams and Water Bodies

The objectives of the EMP in relation to sedimentation of streams and water bodies are:

- To minimize soil erosion
- Minimize carry over of eroded soil particles by surface run-off

The mitigation measures recommended include:

- Back-fill should be compacted properly in accordance with design standards
- In the short-term, either temporary or permanent drainage works shall protect all areas susceptible to erosion
- Measures shall be taken to prevent ponding of surface water and scouring of slopes.
- Newly eroded channels shall be backfilled and restored to natural contours
- Use of silt traps where earthworks are carried out adjacent to water bodies, proper disposal of waste and excavated material, and re-use of suitable material where possible

(6) Handling and Storage of Construction Materials

The objective of the EMP in relation to handling and storage of construction material management is:

- To minimize contamination of the immediate surroundings

The mitigation measures recommended includes:

- Hazardous materials shall be stored on impervious ground under cover. The area shall be constructed as a spill tray to avoid spread of accidental spills
- Safe ventilation for storage of volatile chemicals shall be provided
- Access to areas containing hazardous substances shall be restricted and controlled
- Blasting material used in construction activities shall be kept under lock and key and issues shall be closely monitored

(7) Spoil and Construction Waste Disposal

The objectives of the EMP in relation to spoil and construction waste disposal management are:

- To minimize the generation of spoil and construction waste
- To optimize the reuse of spoil and construction waste
- To ensure safe and proper disposal of spoil and construction waste

The mitigation measures include:

- Estimating the amounts and types of spoil and construction waste to be generated by the project
- Investigating whether the waste can be reused in the project or by other interested parties

- Identify potential safe disposal sites close to the project and investigate the environmental conditions of the disposal sites and select the most suitable and safest sites
- As soft ground treatment is envisaged it should be ensured that adequate disposal sites are identified and approval for agreed sites are obtained in advance
- If material to be disposed from soft ground treatment includes peat adverse impacts of the materials such as acidity shall be managed to avoid contamination of the environment of the disposal site
- Unsuitable excavated materials should be systematically carried away from the areas prone to erosion
- Incorporate reuse of waste materials and use of designated disposal sites
- Used oil and lubricants shall be recovered and reused or removed from the site in full compliance with the national and local regulations
- Oil wasted shall not be burned

(8) Work Camp Operation and Location

The objective of the EMP in relation to office and work camp operation and location is

- To ensure that the operation of office and work camps does not adversely affect the surrounding environment and residents in the area

The mitigation measures recommended in this regard are:

- Identify location of work camps in consultation with Grama Niladaris (GNs). The location shall be subjected to approval by the RDA
- Wherever possible, camps shall not be located near settlements or near drinking water supply intakes
- Cutting of trees shall be avoided and removal of vegetation shall be minimized.
- Water and sanitary facilities shall be provided for employees
- Solid waste and sewage shall be managed according to the national and local regulations. As a rule solid waste must not be dumped, buried or burned at or near the project site, but shall be disposed of to the nearest sanitary landfill or site having and complying with the necessary permits
- The Contractor shall organize and maintain a waste separation, collection and transport system
- The Contractor shall document that all liquid and solid hazardous and non-hazardous waste are separated, collected and disposed of according to the given requirement and regulations
- Water logging and ponding in the office and work camp sites shall be avoided.
- At conclusion of the project, all debris and waste shall be removed. All temporary structures, including office buildings, shelters and toilets, shall be removed.
- Exposed areas shall be planted with suitable vegetation
- The RDA supervising engineer shall inspect and report that the camp has been vacated and restored to pre-project conditions

(9) Loss of Vegetative Cover of the Areas for Temporally Yard

The main objectives of the EMP in relation to loss of vegetative cover are:

- To minimize the loss of vegetation cover due to land clearing and construction related operations and undertakings
- To restore the loss of vegetation cover due to construction related operations and

undertakings
- To avoid loss of top soil which may be used in reestablishing vegetative cover

The mitigation measures recommended in this regard are:
- Minimize cutting trees and other important vegetation during construction
- Stockpile and cover the top soil for future use
- Landscaping the road verges
- Planting of trees/shrubs/ornamental plants to contribute to the aesthetic value of the area and to compensate for the lost capability of the area to absorb carbon dioxide
- At conclusion of the project, all debris and waste shall be removed. All temporary structures, including office buildings, shelters and toilets, shall be removed

(10) Safety Precautions for the Workers

The objective of the EMP in relation to safety precautions of the workers is:
- To ensure safety of workers

The mitigation measures recommended include:
- Providing adequate warning signs
- Providing workers with skull guard or hard hat or other protective devices
- The Contractor shall instruct his workers in health and safety matters, and require the workers to use the provided safety equipment
- Establish all relevant safety measures as required by law and good engineering practices
- Arranging for regular safety checks of vehicles and material, and allocation of responsibility for checking
- Ensuring that material extraction operations, especially blasting, are supervised and carried out by trained and experienced workers

(11) Traffic Condition

The objectives are:
- To minimize disturbances to vehicular traffic and pedestrians during haulage of construction materials, spoil and equipment & machinery
- To mitigate damage to the road network used during haulage of construction materials, spoil and equipment & machinery

The mitigation measures recommended are:
- Formulation and implementation of a construction related traffic management plan
- Installation of traffic warning signs, and enforcing traffic regulations during transportation of materials, equipment & machinery
- Conducting awareness programmes on safety and proper traffic behavior in densely populated areas near the construction sites
- Assign traffic control personnel

(12) Impact on Wetlands

The objective of the EMP in relation to wetlands is:
- To ensure that damage to wetlands and its ecosystem is minimized during construction

The mitigation measures recommended are:

- Avoid disposal of wash water, solid waste as discarded packing etc., on wetlands
- Avoid temporary structures or stockpiling on wetlands

(13) Social Impacts

The objectives of the EMP in relation to social impact management are:

- To ensure minimum impacts from construction labor force
- To ensure minimum impact on public health
- To ensure damage to property from construction activities and haulage of construction materials, spoil and equipment and machinery, is mitigated
- To adverse impacts of blasting and quarrying operations on the community are minimal

The mitigation measures proposed include:

- Conflicts with local community shall be avoided
- Potential for spread of vector borne and communicable diseases from labour camps shall be avoided
- Competition with locals for resources will be avoided
- A condition survey of properties along the haulage roads and located close to the construction works should be undertaken prior to construction and all damages to such properties will be repaired / compensated
- Quarrying operations should be subject to GSMB approval
- Other blasting activities should be carried out with CEA approval

(14) Hydrological Impacts

The objective of the EMP is to avoid hydrological impacts of temporary structures such as pilot road

The mitigation measures proposed are:

- Provide adequate cross drainage structures when the pilot road is constructed to ensure that hydrological problems are avoided

(15) Impacts on Flora and Fauna

The mitigation measures proposed include:

- All hot mix plant, crushing plant, workshops, depots and temporary workers accommodation facilities shall be sited in approved locations
- Toxic and hazardous materials required for construction, including asphalt will be properly stored and secured and sited in approved locations

Vehicles and equipment shall be maintained in good operable condition, ensuring no leakage of oil or fuel and the fitting of proper exhausted baffles

(16) Damage to Land, Visual Impact and Collection of Stagnant Water from Borrow Pit

The mitigation measures proposed include:

- Choice of borrow sites avoid cultivatable land, obtaining agreements with land owners and appropriate permits, siting borrowing pits away from residential areas, site restoration by landscaping landscaping

(17) Compaction of soil by construction plant and equipment

The mitigation measures proposed include:

- Limiting movement and stationing of plant and vehicles to the Right of Way, and specific sites where appropriate leases have been negotiated, and/or clearances obtained

(18) Disruption to traffic damage to motor roads, increase dust and noise nuisance from haulage of materials

The mitigation measures proposed include:

- Choice of quarried and borrow sites that are served by roads of adequate capacity for heavy trucks
- Repair damages to roads caused by haulage of construction materials, spoil and equipment and machinery
- If local road are used for transportation, approval shall be taken from local authorities

(19) Severance of utilities during construction, where relocation is necessary and as a result of accidental damage

The mitigation measures proposed include:

- Inform the public on timing and duration of any disruption to water, poser, telecommunications or other services
- Reduction of risk of accidental damage by ensuring that vehicles and equipment are operated by trained and licensed personnel, and that operations are adequately supervised

9.3.3. Operation Stage

In order to achieve sustainability of the development works, it is necessary to ensure the effectiveness of mitigation measures even after construction, as some adverse environmental impacts may result in the operation of the Project facilities.

(1) Air Quality

The objective of the EMP in relation to air quality management is

- To minimize air pollution from road usage

The mitigation measures proposed are:

- Strict enforcement of the Vehicular Exhaust Emission Standards subsequent to an awareness program
- Implementing the Vehicle Emissions Testing (VET) Program
- Provision of a vegetative barrier to arrest the spread of air borne particles to residential areas

<u>The first two mitigation measures should be considered in the national context.</u>

(2) Noise / Vibration

The objective of the EMP in relation to noise / ground vibration emission management is:

- To minimize and / or avoid the noise level enhancement resulting from road traffic
- To avoid and/or minimize the ground vibration resulting from the vehicles

The mitigation measures recommended for this purpose are:

- Expedite the establishment of standards and regulations for noise level / vibration emanating from vehicles / mobile sources
- Strict enforcement of the regulations, subsequent to an awareness program
- In sensitive areas such as schools, places of worship, hospitals and libraries, sound barriers including tree linings shall be employed

<u>The first two mitigation measures should be considered in the national context.</u>

The complete Environmental Management Plan for the Northern Section 1 of OCH is summarized as **Table 9.2**.

Table 9.3.1 Summary of the Environmental Management Plan for OCH for the Northern Section 1

Project Activity and Potential Environmental Impacts	Mitigation Measures	Approximate Location	Cost Estimate	Institutional Responsibility	
				Implementation	Supervision
<Pre-Construction Stage>					
Social Impacts	● Prepare the inventory of losses of PAPs and extent of damage to properties after the final ROW is identified ● Check whether impacts on PAP can be avoided/minimized through changes in the design or construction method ● Formulate a comprehensive Resettlement Implementation Plan (RIP), based on the national policy on resettlement and conduct validation of the RIP ● Conduct public consultations in groups or individually on the special compensation package and relocation process ● Implementation of RIP plan to the satisfaction of the affected families	Throughout the project road		RDA (PMU)	RDA & Ministry of Land
Hydrological Impacts	● Incorporating into the design the mitigation measures recommended in detailed Hydrological Study conducted as part of the detailed design			Design Consultant	RDA (PMU)
<Construction Stage>					
Negligence of Contractor and Workers on the Environmental Requirements	● Conducting special briefing and / or on-site training for the contractors and workers on the environmental requirement of the project	Throughout the project site	Engineering cost	Contractor	RDA (PMU)
Impacts on Water Quality	● Proper construction management including as training of operators and other workers to avoid pollution of water bodies by the operation of construction machinery and equipment ● Storage of lubricants, fuels and other hydrocarbons in self contained enclosures ● Proper disposal of solid waste from construction activities and labour camps ● Cover the construction material and spoil stockpiles with a suitable material to reducing material loss and sedimentation ● Avoiding stockpiling of materials close to water bodies ● Stripped material shall not be stored where natural drainage will be disrupted ● Servicing of vehicles and equipment at properly managed and equipped workshops, where waste oil is collected and disposed of at approved locations ● Sanitation arrangements at work sites and temporally accommodation facilities, such that no raw sewage could be released into drains or water bodies	Throughout the project site (workcamps, storehouses etc.)	Engineering cost	Contractor	RDA (PMU)
Impacts on Air Quality / Dust	● All heavy equipment and machinery shall be fitted in full compliance with the national and local regulations ● Stockpiled soil and sand shall be slightly wetted before loading, particularly in windy conditions ● Fuel efficient and well maintained haulage trucks shall be employed to minimize exhaust emissions ● Vehicles transporting soil, sand and other construction material shall be covered ● Spraying of bare areas with water ● Procurement of high efficient plant, maintenance of them in good conditions, and operated by trained and qualified personnel ● Selection of sites for materials extraction shall be away from residential areas in order to reduce the impacts from dust	Throughout the project site	Engineering cost	Contractor	RDA (PMU)

Project Activity and Potential Environmental Impacts	Mitigation Measures	Approximate Location	Cost Estimate	Institutional Responsibility	
				Implementation	Supervision
Noise / Vibration	● All heavy equipment and machinery shall be fitted in full compliance with the national and local regulations ● As a rule, the operation of heavy equipment shall be conducted in daylight hours ● Hammer-type pile driving operations shall be avoided during nighttime and build up areas ● Construction equipment, which generates excessive noise shall be enclosed ● Well maintained haulage trucks will be used with speed controls ● Controlled blasting should be employed under approval and supervision of GSMB / CEA ● Taking reasonable measures to minimize noise/vibration near noise sensitive areas such as schools and places of worship	Throughout the project site and along the haulage road	Engineering cost	Contractor	RDA (PMU)
Sedimentation of Streams and Water Bodies	● Back-fill should be compacted properly in accordance with design standards. ● In the short-term, either temporary or permanent drainage works shall protect all areas susceptible to erosion. ● Measures shall be taken to prevent ponding of surface water and scouring of slopes. Newly eroded channels shall be backfilled and restored to natural contours. ● Use of silt traps where earthworks are carried out adjacent to water bodies, shall be properly disposal of waste and excavated material, and re-use of suitable material where possible	Throughout the project site	Engineering cost	Contractor	RDA (PMU)
Handling and Storage of Construction Materials	● Hydrocarbons and hazardous materials shall be stored on impervious ground under cover. The area shall be constructed as a spill tray to avoid spread of accidental spills. ● Safe ventilation for storage of volatile chemicals shall be provided. ● Access to areas containing hazardous substances shall be restricted and controlled. ● Blasting material used shall be stored safely and issues monitored closely	Storage areas	Engineering cost	Contractor	RDA (PMU)
Spoil and Construction Waste Disposal	● Estimating the amounts and types of spoil and construction waste to be generated by the project. ● Investigating whether the waste can be reused in the project or by other interested parties. ● Identify potential safe disposal sites close to the project and investigate the environmental conditions of the disposal sites and prepare recommendation of most suitable and safest sites. ● Provide adequate disposal sites and obtain approval ● Designated disposal sites to be used. ● Incorporate reuse of waste materials and use of designated disposal sites in the detailed design. ● Unsuitable excavated materials should be systematically carried away from the areas prone to erosion. ● Used oil and lubricants shall be recovered and reused or removed from the site in full compliance with the national and local regulations. ● Oil wasted shall not be burned.	Throughout the project site and workers camps	Engineering cost	Contractor	RDA (PMU)

Project Activity and Potential Environmental Impacts	Mitigation Measures	Approximate Location	Cost Estimate	Institutional Responsibility	
				Implementation	Supervision
Work camp operation and Location	● Identify location of work camp and office sites in consultation with Grama Niladaris (GNs). The location shall be subject to approval by the RDA. If possible, camps shall not be located near settlements or near drinking water supply intakes. ● Cutting of trees shall be avoided and removal of vegetation shall be minimized. ● Water and sanitary facilities shall be provided for employees. ● Solid waste and sewage shall be managed according to the national and local regulations. As a rule solid waste must not be dumped, buried or burned at or near the project site, but shall be disposed of to the nearest sanitary landfill or site having and complying with the necessary permits. ● The Contractor shall organize and maintain a waste separation, collection and transport system. ● The Contractor shall document that all liquid and solid hazardous and non-hazardous waste are separated, collected and disposed of according to the given requirement and regulations. ● Water logging and ponding shall be avoided ● At conclusion of the project, all debris and waste shall be removed. All temporary structures, including office buildings, shelters and toilets, shall be removed. ● Exposed areas shall be planted with suitable vegetation. ● The RDA supervising engineer shall inspect and report that the camp has been vacated and restored to pre-project conditions.	Workers camps	Engineering cost	Contractor	RDA (PMU)
Loss of Vegetation Cover of the Areas for Temporary Yard	● Minimize cutting trees and other important vegetation during construction. ● Stockpile and the top soil for future use ● Landscaping the road verges ● Planting of trees/shrubs/ornamental plants to contribute to the aesthetic value of the area and to compensate for the lost capability of the area to absorb carbon dioxide.	Temporary Yards	Engineering cost	Contractor	RDA（PMU/ESD）
Safety and Precaution for the Workers	● Providing adequate warning signs ● Providing workers with skull guard or hard hat and other safety devices. ● The Contractor shall instruct his workers in health and safety matters, and require the workers to use the provided safety equipment. ● Establish all relevant safety measures as required by law and good engineering practices ● Training and briefing of works on safety precautions, their responsibility for their safety and the safety of others, ensuring that vehicles and equipment operators are properly licensed and trained, arranging for provision of first aid facilities, rapid availability of trained paramedical personnel, and emergency transport to nearest hospital with accident and emergency facilities ● Allocation of responsibilities to ensure that these arrangements are kept in place ● Arranging for regular safety checks of vehicles and material, and allocation of responsibility for checking ● Ensuring that material extraction operations, especially blasting, are supervised and carried out by trained and experienced workers	Throughout the project site	Engineering cost	Contractor	RDA（PMU/ESD）
Traffic Condition	● Formulation and implementation of a construction related traffic management plan ● Installation of traffic warning signs, and enforcing traffic regulations during transportation of materials and equipment & machinery ● Conducting awareness programmes on safety and proper traffic behavior in densely populated areas near the construction sites ● Assign traffic control personnel	Throughout the project site and roads connecting the sites	Engineering cost	Contractor	RDA（PMU/ESD）

Project Activity and Potential Environmental Impacts	**Mitigation Measures**	**Approximate Location**	**Cost Estimate**	**Institutional Responsibility**	
				Implementation	**Supervision**
Impacts on Wetland	● Avoid disposal of wash water, solid waste as discarded packings etc., on wetlands ● Avoid temporary structures or stockpiling on or close to wetlands	Wet land areas	Engineering cost	Contractor	RDA（PMU/ESD）
Social Impacts	● Conflicts with local community should be avoided ● Potential for spread of vector borne and communicable diseases from labour camps shall be avoided ● Competition with locals for resources will be avoided ● Quarrying/blasting operations to be under GSMB/CEA supervision	Neighborhood of local communities	Engineering cost	Contractor	RDA（PMU/ESD）
Hydrological Impacts	● Provide adequate cross drainage structures	Along the project road	Engineering cost	Contractor	RDA（PMU/ESD）
Impacts on Flora and Fauna	● All hot mix plant, crushing plant, workshops, depots and temporary workers accommodation facilities shall be site in approved locations ● Toxic and hazardous materials required for construction, including asphalt will be properly stored and secured and sited in approved locations ● Vehicles and equipment shall be maintained in good operable condition, ensuring no leakage of oil or fuel and the fitting of proper exhausted baffles	Throughout the project site	Engineering cost	Contractor	RDA（PMU/ESD）
Damage to Land, Visual Impact and Collection of Stagnant Water from Borrow Pit	● Choice of borrow sites avoid cultivatable land, obtaining agreements with land owners and appropriate permits, siting borrowing pits away from residential areas, site restoration by landscaping	Borrow pit site	Engineering cost	Contractor	RDA（PMU/ESD）
Compaction of soil by construction plant and equipment	● Limiting movement and stationing of plant and vehicles to the Right of Way, and specific sites where appropriate leases have been negotiated, and/or clearances obtained	Along the project road	Engineering cost	Contractor	RDA（PMU/ESD）
Disruption to traffic damage to motor roads, increase dust and noise nuisance from haulage of materials	● Choice of quarried and borrow sites that are served by roads of adequate capacity for heavy trucks. Where minor roads are used, provision for repair and restoration ● Repair damages to roads caused by haulage of construction materials, spoil and equipment and machinery ● If local road are used for transportation, approval shall be taken from local authorities	Borrow and quarry site	Engineering cost	Contractor	RDA（PMU/ESD）
Severance of utilities during construction, where relocation is necessary and as a result of accidental damage	● Inform the public on timing and duration of any disruption to water, poser, telecommunications or other services ● Reduction of risk of accidental damage by ensuring that vehicles and equipment are operated by trained and licensed personnel, and that operations are adequately supervised	Throughout the project site	Engineering cost	Contractor	RDA（PMU/ESD）
< Operation Stage>					
Air quality / Dust	● Strict enforcement of the regulations subsequent to an awareness programme ● Provision of a vegetative barrier to arrest the spread of air borne particles to residential areas	Throughout project road section	To be determined	RDA	CEA
Noise / Vibration	● Strict enforcement of the regulations subsequent to an awareness programme ● In sensitive areas such as schools, places of worship, hospitals and libraries, sound barriers including tree linings will have to be employed.	Throughout project road section	To be determined	RDA	CEA

9.4. Environmental Monitoring Action Plan

The mitigation measures proposed in the environmental management plan will be carried out by the responsible agencies. Among the environmental parameters considered in the environmental management plan, the items mentioned below were prioritized for inclusion in the monitoring plan:

- Groundwater levels
- Water quality (surface water and ground water)
- Air quality
- Noise levels
- Social impacts

The sampling locations proposed for monitoring included in this EMP have been determined through consultation with CEA. Environmental monitoring results will be submitted to ESD / CEA periodically by the monitoring agencies / contractor.
As mentioned later, the baseline data of existing water quality, air quality and noise levels will be established before commencement of the construction. Note that PMU should make great effort for not exceeding the baseline data during construction phase.

9.4.1. Ground Water Level

(1) The objective for monitoring ground water level is:

- To assess the negative impact on ground water level caused by construction activities especially in the adjacent areas where cutting are planned. Cutting might induce decreasing ground water levels.

(2) Parameters to be monitored are:

- Ground water level

(3) Monitoring Location

No.	Locations
1	Well at Ms. Premawathie/s house; 645/2, Ihalabiyanwila, Kadawatha
2	Well at Mr. W. Wimalasena's house; 391/14, Galwala Handiya, Makola South, Makola

Sampling locations is referred to in **Fig. 9.3**.

(4) Frequency

- Two times during dry and wet seasons before construction (Pre-construction)
- Every 3 months and on complaints during construction
- Two times with an interval of six months during operation stage for one (1) year

(5) Responsible agency

Independent laboratory under contract to RDA

9.4.2. Water Quality (surface and ground water)

(1) The objective for monitoring water quality is:

- To avoid contamination of water by construction and related activities as accidental oil spills, disposal of solid waste, spoil, construction material and domestic wastewater

(2) Parameters to be monitored are:

- pH
- Electrical Conductivity (EC)
- Dissolved Oxygen (DO)
- Biochemical Oxygen Demand (BOD)
- Chemical Oxygen Demand (COD)
- Suspended Solids (SS)
- Nitrate
- Phosphate
- Chloride
- Oil / Grease
- Zinc
- Lead
- Total coliform
- E. coliform

(3) Monitoring Locations

<Surface Water>

No.	Locations
1	Stream opposite Ele Kade (proprietor – Mr Bertram), Pragathi Mawatha, Ihala Biyanwila
2	Water body at edge of marsh at Bandaranayake Mawatha, off Dewamitta Place; down stream of Kottunna Tank
3	Kelani River; north end of crossing

<Ground Water>

No.	Locations
1	Well at Ms. Premawathie's house; 645/2, Ihalabiyanwila, Kadawatha
2	Well at Mr. W. Wimalasena's house; 391/14, Galwala Handiya, Makola South, Makola

Sampling locations are referred to in **Fig. 9.3**.

(4) Frequency

- Two times during dry and wet seasons before construction (Pre-construction)
- Every 3 months and on complaints during construction
- Two times with an interval of six months during operation stage for one (1) year

(5) Responsible agency

- Independent laboratory under contract to RDA

9.4.3. Air quality

(1) The objective of monitoring air quality is:

- To minimize the air pollution in both construction and operation phases

(2) The following Parameters have been selected for monitoring air quality:

- Carbon monoxide
- Sulphur dioxide
- Nitrogen dioxide
- Ground level ozone
- pM10 - particulate matter

(3) Monitoring Locations

No.	Locations
1	At or near Ele Kade (proprietor – Mr Bertram), Pragathi Mawatha, Ihala Biyanwila
2	At or near Mr W Wimalasena`s house; 391/14, Galwala Handiya, Makola South, Makola

Samplings locations are referred to **Fig. 9.3**.

(4) Frequency

- One time for 24 hour monitoring at dry weather condition before construction
- Suspended Particulate Matter (SPM) for 8 hours quarterly and on complains during construction
- Every six months during operation stage for one (1) year

(5) Responsible agency

- Independent laboratory under contract to RDA

9.4.4. Noise / Ground Vibration

(1) The objective of noise and ground vibration level monitoring is:

- To minimize the noise emission
- To minimize and /or avoid ground vibration

(2) Parameters

- For major interchanges - 24 hours measurements L_{Aeq} and L_A 90, 15 min. (Day/Night)
- For other locations - Three (03) hours measurements L_{Aeq} and L_A 90, 15 min. (Day/Night)

(3) Monitoring Locations

<Noise for 24hrs>

No.	Locations
1	Stream opposite Ele Kade (proprietor – Mr. Bertram), Pragathi Mawatha, Ihala Biyanwila

<Noise for 3hrs>

No.	Locations
1	Well at Mr. W. Wimalasena’s house; 391/14, Galwala Handiya, Makola South, Makola

Sampling locations are referred to **Fig. 9.3**.

(4) Frequency

- Back ground noise levels (before construction) - One time with identification of noise barriers requirement locations. Every six months during construction
- During construction, noise levels (one (01) hour LAeq for day time and 5 minutes for night time) and ground vibration should be carried out quarterly and on complains from residents
- Every six months during operation stage for one (1) year

(5) Responsible agency

- Independent laboratory under contract to RDA

9.4.5. Social Impacts

Monitoring of social impacts will be carried out based on the comprehensive Resettlement Implementation Plan (RIP) to be prepared by the RDA by the end of the Detailed Designs.

The objectives of monitoring social impacts are:
- To ensure that PAPs are settled in a similar or a better environment
- To ensure that PAPs are adequately compensated
- To avoid adverse direct and indirect impacts of resettlement of PAPs
- To identify residual adverse impacts of relocation

The items to be monitored are:
- Payment of compensation and resettlement
- Adverse social effects of relocation as disruption of cultural ties, access to social infrastructure etc.
- Impact on income levels and sustainability
- Impact on female headed families and disadvantaged PAPs
- Availability of employment opportunities for daily wage earners
- Potential conflict situations

Monitoring will commence as soon as relocation is started in the pre-construction phase and continue till the third year after resettlement.

Monitoring will be assigned to an organization with institutional capabilities to conduct social impact monitoring as detailed above. Responsible agency should be RDA.

9.4.6. Baseline Data

The baseline conditions of existing water quality, air quality and noise levels should be established before commencement of the construction as requested by CEA, Environmental Monitoring and Assessment (EM&A) Div. Note that it is necessary to collect the baseline data for water quality (ground water and surface water) for dry and rainy season respectively.

The monitoring activities are summarized as below:

Items	Parameters	Responsible		Frequency
		Construction	Operation	
Ground water levels	- Ground water level	PMU/ESD	RDA SLLRDC CEA	Two times during dry and wet period before construction; Quarterly and on complains during construction; Every six months during operation stage for 03 years
Water quality	pH, EC, DO, BOD, COD,SS, Nitrate, Phosphate, Chloride, Oil/Grease, Zinc, Lead, Total coliform, E. coliform	PMU/ESD	RDA/CEA	Two times during dry and wet period before construction; Quarterly and on complains during construction; Every six months during operation stage for one year
Air quality / Dust	- Carbon monoxide - Sulphur dioxide - Nitrogen dioxide - Ground level ozone - pM10	PMU/ESD	RDA/CEA	One time 24 hour monitoring at dry weather before construction; Eight (8) hours SPM quarterly and on complains during construction; Every six months during operation stage for 03 years
Noise / ground vibration	Mean sound level (Leq (24)) Day-night sound level (Ldn)	PMU/ESD	RDA/CEA	Back ground noise levels (before construction) - One time with identification of noise barriers requirement locations; Every 03 months during construction, During construction, noise levels (one (01) hour LA eq for day time and 5 minutes for night time) and ground vibration should also be carried out on complains from residents; Every six months during operation stage for 03 years
Social Impacts	- Payment of compensation - Adverse social effects of resettlement - Impact on income levels and	PMU/ESD & Ministry of Land	RDA	Commencing Pre-construction phase continue till the third year after resettlement

Items	Parameters	Responsible		Frequency
		Construction	Operation	
	sustainability - Impact on vulnerable families and disadvantaged PAPs - Availability of employment opportunities for daily wage earners, share croppers and Ande farmers. - Potential conflict situations			

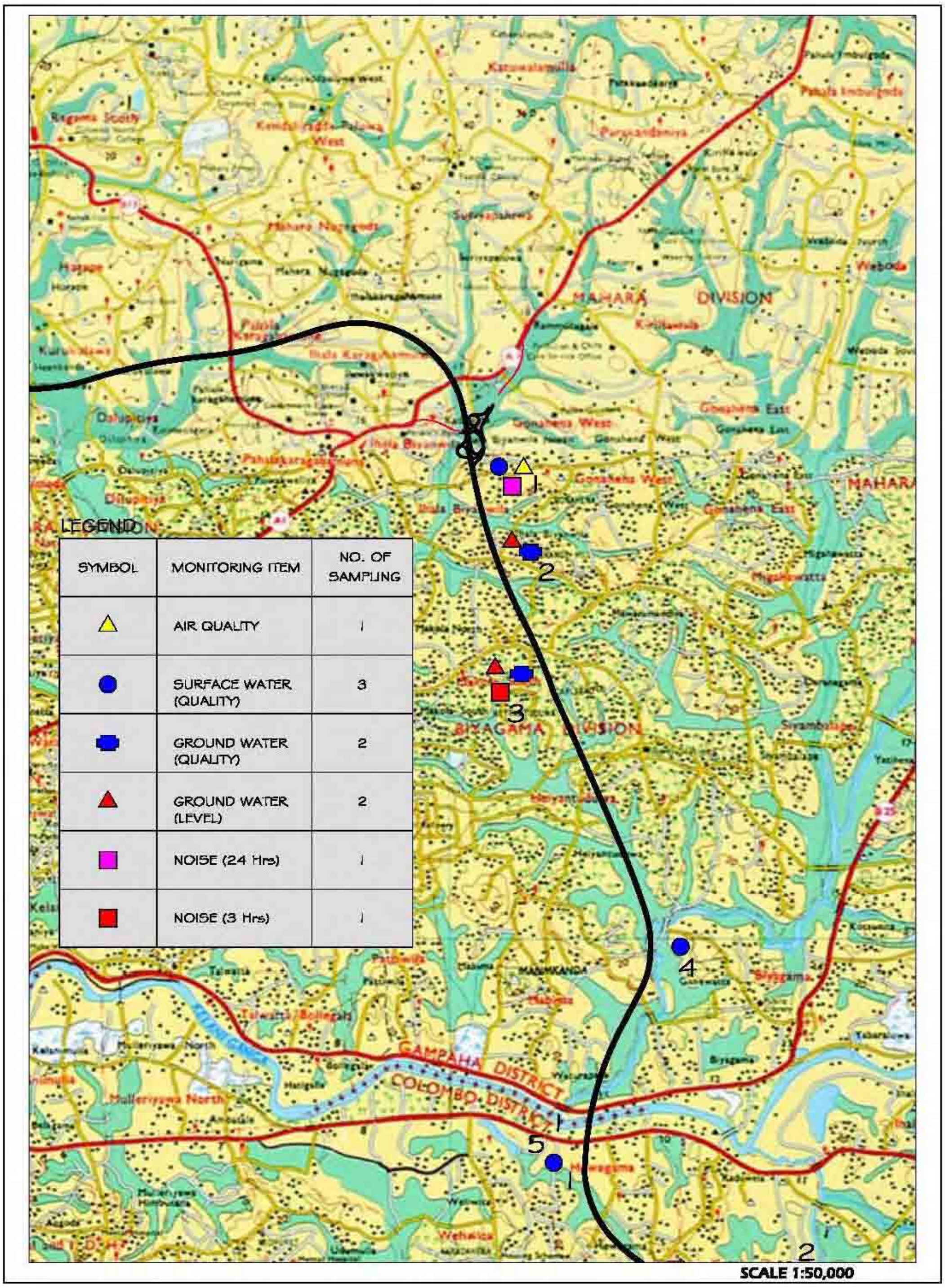


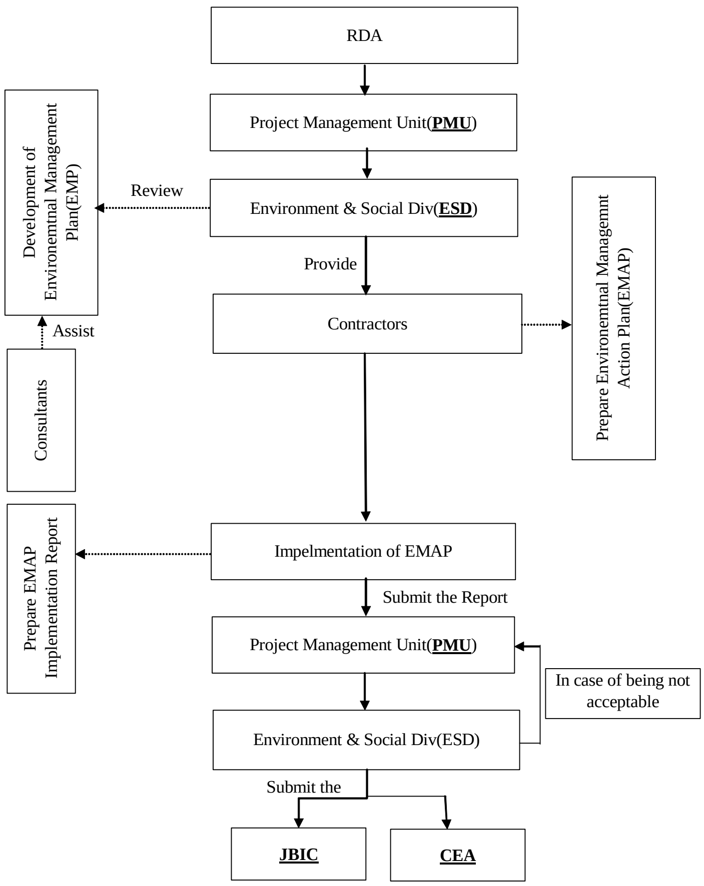
Fig. 9.4.1 Sampling Locations for each Monitoring Parameters

9.5. Responsibilities For Implementation Of EMP

The Environmental and Social division (ESD) of RDA has a roll of supervising every road development project in terms of environmental and social management. The contractors will prepare Environmental Management Action Plan (EMAP) based on this EMP. Under supervising of the ESD, the PMU for the Project, with assistance of the Environmentalist of the Project Implementation Consultant (PIC), will

- Carry out Compliance Monitoring: the day-to-day inspection on compliance of mitigation measures of the contractors
- Prepare Environmental Monitoring Reports
- Address grievances from affected people

The general workflow for environmental management for the Project is as shown below.



9.6. Land Acquisition

9.6.1. Land Acquisition Procedure in Sri Lanka

Land Acquisition in Sri Lanka is carried out under the provisions of the Land Acquisition Act (LAA) of 1950. The procedure prescribed in the Act includes many steps and several different government agencies; therefore it is a time-consuming process. **Fig 9.4** shows the flow of land acquisition process.

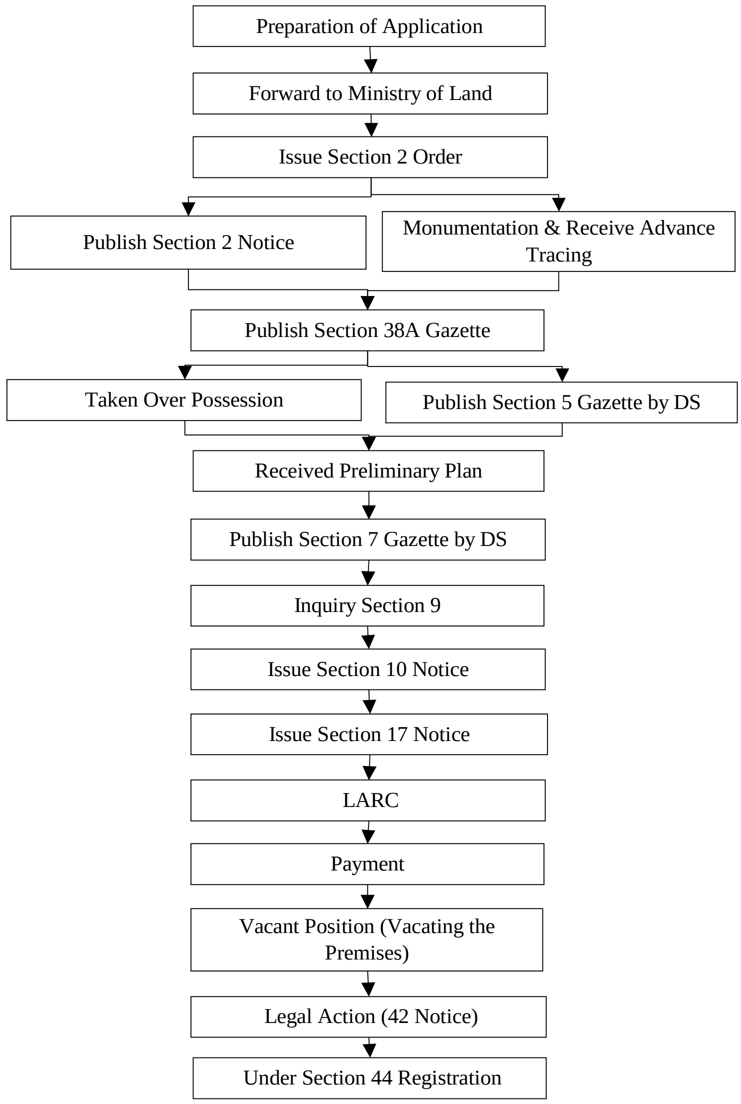


Fig 9.6.1 Land Acquisition Procedures

9.6.2. Progress of Land Acquisition for Northern Section 1

The OCHP has been given attention to the completion of topographic survey of the non-surveyed section in the Northern Section 1: Kaduwela to Kadawatha Section (up to A1 Interchange) by community participation. An NGO, National Forum of Peoples Organization (NFPO), was appointed in April 2006 to conduct public consultation and facilitate the topographic survey in 1.28 km of the road trace where public opposition was blocking the land surveyor’s work.

All of the procedures (including payment and vacating of premises) will be completed by the end of November 2008. Presently, monumentation has been completed and advance tracing is proceeding forward and Lay Out Map (to confirm land owner and land boundary) is expected to be completed by at end of March 2008.

The detail of the current stepwise progress of land acquisition is shown in **Appendix 9.6**.

9.6.3. Resettlement Implementation Plan (RIP)

The RIP (Resettlement Implementation Plan) for the OCH was submitted to the JBIC and to the Ministry of Lands in March 2005. Their report consisting of 14 Chapters, laid down the National Involuntary Resettlement Policy. The RIP included the Socio-Economic features of the APs. The households were interviewed from Maharagama to Kadawatha living within the 100-120m corridor.

Since a considerable progress already had been made by the OCH project, the OCHP prepared a RIP as addendum for the initial RIP for the first 12.9km (Maharagama to Kaduwala) in August 2006. The addendum was intended to continue and update the actions taken by the OCHP, in terms of the scope of the RIP. The IOL and SES have provided all details of the APs, their affected lands, buildings etc. The addendum covered all topics and the gaps that were there in the initial RIP.

OCHP intends to prepare a similar addendum for the Northern Section 1, too, which will commence after collection of IOL data and is expected to be completed by the end of February 2008.

CHAPTER 10

PROJECT COST ESTIMATE

CHAPTER 10 PROJECT COST ESTIMATES

10.1. General

Project cost is estimated for the detailed design of the Northern Section - 1. Note that these estimates will be utilized in the economic analysis in this report. To arrive at total Project cost, unit rates were prepared applying the Sri Lanka Highway Schedule of Rates (HSR), the Japanese Civil Work Estimation Standard, the Japanese Cost Estimation for Bridge Erection and the Japanese Steel Bridge Manual. When these four were not applicable, consultations were held with relevant contractors to obtain unit-cost information. In addition, costs were compared with those of recent major road construction projects funded by international donor agencies in Sri Lanka to confirm their reliability. Note that cost is composed of:

- Construction Cost
- Land Acquisition and Resettlement Cost
- Cost for Relocation of Public Utilities (electricity, telephone lines and water supply)

The basic assumptions and methods for estimating Project costs are as follows:

1) Private contractor(s) carry out all construction work.
2) The unit cost of each cost component is estimated by applying the HSR for fiscal year 2006, the Japanese Civil Work Estimation Standards, the Japanese Cost Estimation for Bridge Erection and the Japanese Steel Bridge Manual.
3) When the HSR and Japanese standards are inadequate for providing the unit cost for a particular item, then interviews are held with local contractors to gather the necessary data to determine the appropriate cost.
4) Land acquisition cost is based on market prices and on data from the Sri Lanka Land Acquisition Department.
5) The special machine (Bore Pile machine, GCP machine, Girder erection facilities) is supplied from outside of Sri Lank.
6) The steel girder will be produced in Japan, and it is carried to Sri Lanka by marine transportation.
7) The GCP execution labor cost is used foreign labor unit price
8) Engineering service for Northern Section – 1 was already contracted, so that it is not included in this estimation.
9) Physical contingency is estimated to be 10% of the total cost for construction and engineering services and includes an allowance for price escalation for labor, material and equipment.
10) Currency exchange rate: Rs. 1 = JPY 1.099 (average for April 2007)
11) Taxation: Construction Works: 15% VAT

10.2. Procurement

10.2.1 Labor Force

Local contractors carry out most of the general road work in Sri Lanka and hence there is an ample labor pool with skills in civil works, except for when specific skills in fields such as pre-stressed concrete are required. Note that there are several post-tensioned concrete bridges in this Project, but there are some local construction firms that have the capacity to undertake this work.

10.2.2 Supply of Machinery and Materials

As it is difficult to procure special machinery and high-grade materials to ensure the quality required for bridges in Sri Lanka, most of these materials and machinery will have to be imported.

(1) Construction Materials

Major materials required for the Project and their availability in Sri Lanka are as shown in the table below.

Table10.2.1 Availability of Material in Sri Lanka

Material	Production & Supply in Sri Lanka	Availability in Sri Lankan Market	Price for Cost Estimation
Cement	Sufficient	Available	Market price
Reinforcement bar	Diameter (D=32mm)	Available	Market price
Asphalt	Sufficient	Available	Market price
Shaped steel	Not produced	Only small size Available	Market price
PC strand cable	Not produced	Not available	Imported price
Bridge bearing	Not produced	Not available	Imported price
Expansion joint	Not produced	Not available	Imported price

(2) Ready-mixed Concrete and Asphalt

There are eight companies that can produce and supply ready-mixed concrete in the Colombo region. According to a survey of these plants, it was found that a sufficient volume of ready-mixed concrete for the Project is readily available. As for hot-mixed asphalt, there are several suppliers in the Colombo area and a survey of these plants also shows that they have the capacity to supply the required quantity for the Project.

(3) Construction machinery

Construction machinery includes those owned by private and public companies. Private contractors generally use their own machinery, while some companies lease. Heavy construction machinery such as 50-ton mobile cranes, together with large numbers of dump trucks and backhoes required for the Project, shall be imported to ensure the smooth implementation of the OCH’s construction work.

Though, following special machinery procurement is considered from foreign countries.

- Crawler Sand Compaction Machine for Gravel Compaction Pile
- Truck Crane (160t) for Girder Erection
- Erection Girder for Girder Erection
- Special Truck for Girder Hauling
- All Casing Excavation Machine for Bore Pile Foundation

10.3. Unit Cost

All unit prices below are as of April, 2007. In case of transforming into the present, those unit prices should be considered the escalation rate of from April, 2007 to the present.

10.3.1 Labor

Table 10.3.1 shows the labor unit rates used in estimating construction cost, and includes allowances for social benefits, insurance, etc, and are based on an eight-hour workday.

Table 10.3.1 Unit Rates for Labor

Classification	Unit	Unit Rate (Rs)
Senior Engineer (20 years experience)	month	91,000
Engineer (10 years experience)	month	60,060
Junior Engineer (5 years experience)	month	40,950
Skilled Labor (technician)	day	670
Semi-Skilled Labor	day	569
Unskilled Labor	day	522

10.3.2 Materials

Table 10.3.2 indicates the unit costs for major construction materials. The cost for imported materials is based on the CIF for Colombo, including port handling and clearance charges and import duties. The cost of local materials is based on market prices in the Colombo area.

Table 10.3.2 Unit Rates for Major Materials

Item	Unit	Unit Rate (Rs)
Portland Cement	Ton	11,000
Ready-Mixed Concrete (Grade 30)	m^3	8,160
Reinforcement Steel Bar (Grade 460)	Ton	83,490
Hot-Mixed Asphalt	Ton	4,200
Petrol	Liter	97
Diesel	Liter	65

10.4. Quantity of Major Works

Highway construction consists of earthwork including soft soil treatment work, pavement, bridges road structure and other works. These major work items and their quantities are described in Chapter 6.

10.5. Cost Estimate for Compensation

10.5.1 General

The total number of affected houses within approximately 40 to 110 meters of the right of way (ROW) of the OCH for the Northern Section 1 was counted using 1:1000 drawings prepared by the JICA Study. The counted houses were classified as one/two-storied houses and one/two storied commercial buildings respectively through supplemental field survey. Regarding affected land within the ROW, the areas was calculated with CAD based on the above same drawings. The results were summarized in each land use category, i.e. garden, paddy, coconut and marsh.

10.5.2 Cost Estimate for Affected Structures

The information on preliminary compensation rates for houses was obtained through interviews to private sectors in November 2005 in the Detailed Design for southern section by JICA Study Team. The unit cost was 3000Rs/sq.m for a two-storied house/commercial structure and 2000Rs/sq.m for a single-storied house/commercial structure respectively.

The compensation unit costs for the Northern Section 1 have been decided as 3000Rs/sq.m for a two-storied house/commercial structure and 2000Rs/sq.m for a single-storied house/commercial structure respectively by adding 10percent/year escalation. The table below indicates the cost estimates for compensation of the Northern Section 1.

Table 10.5.1 Cost Estimate for Houses & Other Structures for the Northern Section 1

Types	No.	Amount (Rs.)
<Houses>		
Two storied	20	128,351,520
Single storied, large	216	522,676,800
<Commercial Buildings>		
Two storied	13	66,318,120
Single storied, large	19	68,786,760
GRAND TOTAL		**786,133,080**

Note: Compensation means replacement cost for houses and other structures

10.5.3 Cost Estimates for Land Compensation

The information on compensation rates for land was obtained from the land acquisition officer of OCHP, who have been involved the land acquisition process for several years. The table below contains compensation estimates for the different categories of land for the Northern Section 1.

Table 10.5.2 Compensation (Replacement Cost) for Land for the Northern Section 1

Type of Land	Area (ha)	Amount (Rs.)
Garden	30.42	1,688,255,488
Paddy Field	22.92	33,496,205
Coconut Trees	3.3	60,284,400
Marsh	17.64	644,449,504
Total	**74.28**	**1,846,485,597**

10.6. Total Project Cost

The estimated cost for OCH Northern -1 Project is comprised of construction cost (such as earthwork, pavement work, and road structures, traffic facilities, bridges, culverts and drainage), land acquisition and compensation costs, administration costs and physical and price escalation contingencies. However, the Engineering Services for the tender assistance and supervising of Northern Section – 1 Project is not included in this cost, because their service already was contracted with Southern Section.

The estimated cost for the detailed design of OCH Northern Section – 1 is shown below:

Table 10.6.1 Total Project Cost for the OCH Northern Section 1

Unit: million Rs.

No.	Work Item	Cost
A	Construction Cost	
1	Preliminary & General	1,223
2	Earthworks (including soft soil treatment work)	4,991
3	Base & Sub-base Course	347
4	Pavement	381
5	Structure :Bridge	9,491
	:Box Culvert	588
	:Retaining Wall	100
6	Drainage	679
7	Incidental	368
8	Facilities	155
9	Day works	15
10	Provisional Sum	374
A	Construction Cost Total	18,712
B	Contingency (10% of A)	1,871
	Total of A + B	20,583
C	Administration Cost for RDA (0.5% of A)	103
D	Land Acquisition & Resettlement	2,829
	Total Project Cost	23,515

CHAPTER 11

ROAD MAINTENANCE & OPERATION

CHAPTER 11 ROAD MAINTENANCE & OPERATION

11.1. Background & Objective

Timely and effective road maintenance is vital for all roads, but it is especially true for expressways such as the OCH where the combination of high travel speeds and bad maintenance could possibly cause fatalities or grievous injury. To manage road maintenance properly, the factors of organization, funding, and capacity (both private and governmental) in Sri Lanka should be considered.

Accordingly, each of the above factors is examined from a broad perspective, with existing international experience taken into account. Based on this, a preliminary operation and maintenance (O&M) plan for the OCH is prepared. Note that the analysis in this chapter is based on information collected from interviews carried out with international donors as well as with national, provincial, and local government authorities, together with information and data contained in existing donor and government documentation.

11.2. Organizational & Regulatory Framework

11.2.1. Overview

An overview of the Sri Lankan road network and the organizations responsible for its maintenance are as described in Table 11.2.1.

Table 11.2.1 Overview of Road Network & Entities in Sri Lanka

Category	Class	Length (km)	Function	Administrative Entity	Implementation Entity
National Roads	A	4,192	Inter-provincial trunk road connecting major cities & ports	Ministry of Highways & Road Development	Road Development Authority (RDA)
	B	7,510	Intra-provincial arterial road connecting major urban areas		
Provincial Roads	C	8,457	Major feeder road & road for connecting settlements with markets, etc.	Ministry of Provincial Councils & Local Government	Provincial Road Development Authority (PRDA) of relevant Provincial Council
	D	5,756	Minor feeder road & road for connecting settlements with markets, etc.		
Municipal, Town, Village Roads	E	75,069	Local road to provide access to specific locations	Ministry of Provincial Councils & Local Government	In-house Road Maintenance Unit of Municipal/ Urban Council or Local Authority
	Unclassified				
Other Roads	Unclassified		Plantation, forest, irrigation, roads	Ministry of Rural Development (MORD), Private Company, Agricultural Cooperative	For MORD Village Development Society; otherwise, usually contracted out
Total	-	100,984	-	-	-

Sources: Road Development Authority, 2006.

As the above table indicates, the road network can be basically divided into three tiers. The first tier consists of national roads maintained by the Road Development Authority (RDA), which comes directly under the control of the Ministry of Highways and Road Development (MOHRD). The second tier consists of provincial roads under the overall control of nine provincial councils maintained by either an autonomous Provincial Road Development Authority (PRDA) or a maintenance department contained within a provincial council. The third tier consists of municipal, town, and village roads that are looked after by a maintenance unit usually operating from an office of the relevant local authority. Provincial, municipal, and urban councils, as well as the local authorities, all come under the jurisdiction of the Ministry of Provincial Councils and Local Government (MOPCLG). It should be noted that there is little or no interaction regarding the exchange of information or technology between the entities involved in executing road maintenance at these different levels of government. The chart in Fig. 11.2.1 depicts the hierarchal relationships just described.

Note that there are a number of roads that have a specific limited function, such as those for plantations, forests, irrigation, etc. For large corporate or cooperative operated plantations, they will usually contract out to a private firm to maintain the roads that they own and/or use regularly. As for irrigation and forestry, the maintenance of these roads comes under the administration of the Ministry of Rural Development (MORD), and villagers will carry out the necessary work free of charge upon provision of the necessary materials from the MORD. Note that it is difficult to know the actual length of these roads and they are not considered in this report.

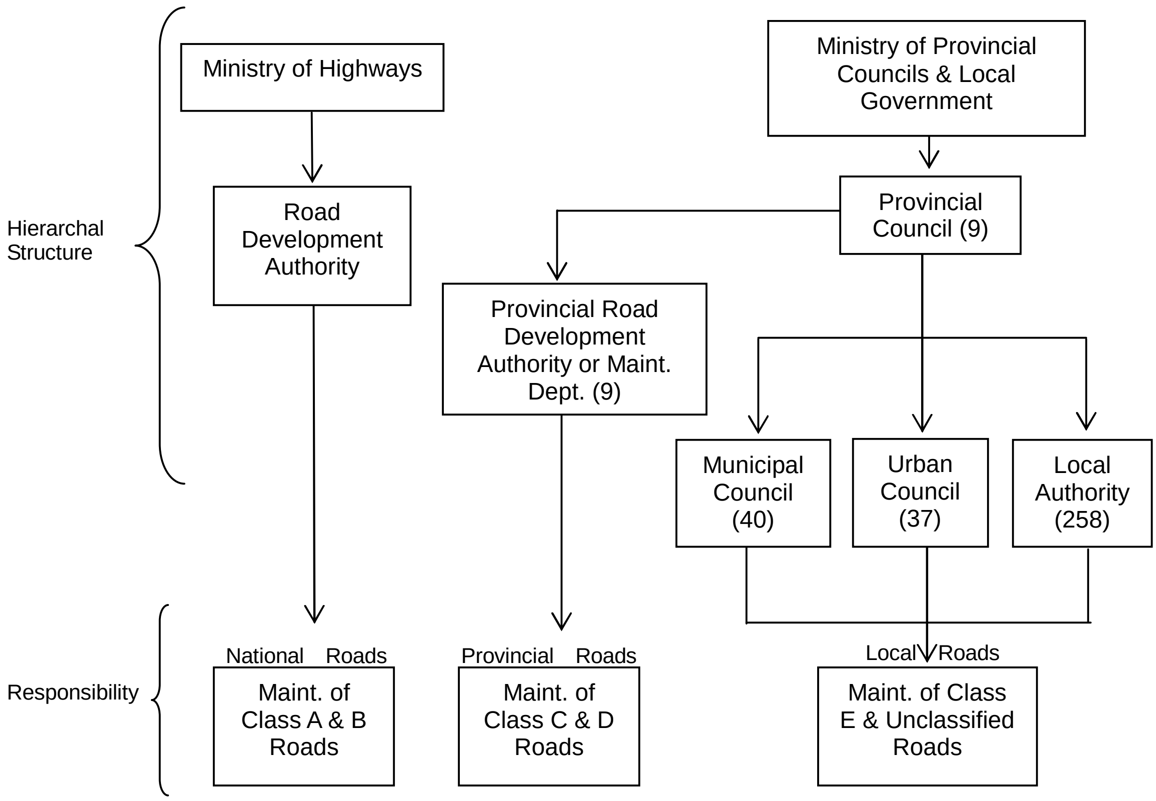


Fig.11.2.1 Road Maintenance Hierarchy in Sri Lanka

(Note: The values in parentheses represent the number of entities)

11.2.2.Organizational Structure for National Roads Maintenance

The 11,702 km of Class A and B road are defined as national roads and are maintained by RDA, which was established in 1981 under the Road Development Act (No. 73). RDA's current functions were put into place in 1986 when it became the successor to the now defunct Department of Highways and has a present a total of about 9,900 employees. Until December 2003, RDA had contracted out its maintenance works to an organization known as the Road Construction and Development Company (RCDC), which was established in 1987 as a limited liability company and 100% owned by RDA. RCDC was disbanded in December 2003 and RDA currently executes its maintenance work either via force account or contract. In RDA, the entity actually responsible for the upkeep of Class A and B roads is the Department of Maintenance Management and Construction. The line of command for the Department is given in Fig. 11.2.2. Note that in addition to the director of Maintenance & Construction there are 13 other directors that also report to the general manager, who acts as the CEO for RDA and reports to the chairman of the Board. Note that RDA's contract laborers were made permanent employees in early 2007.

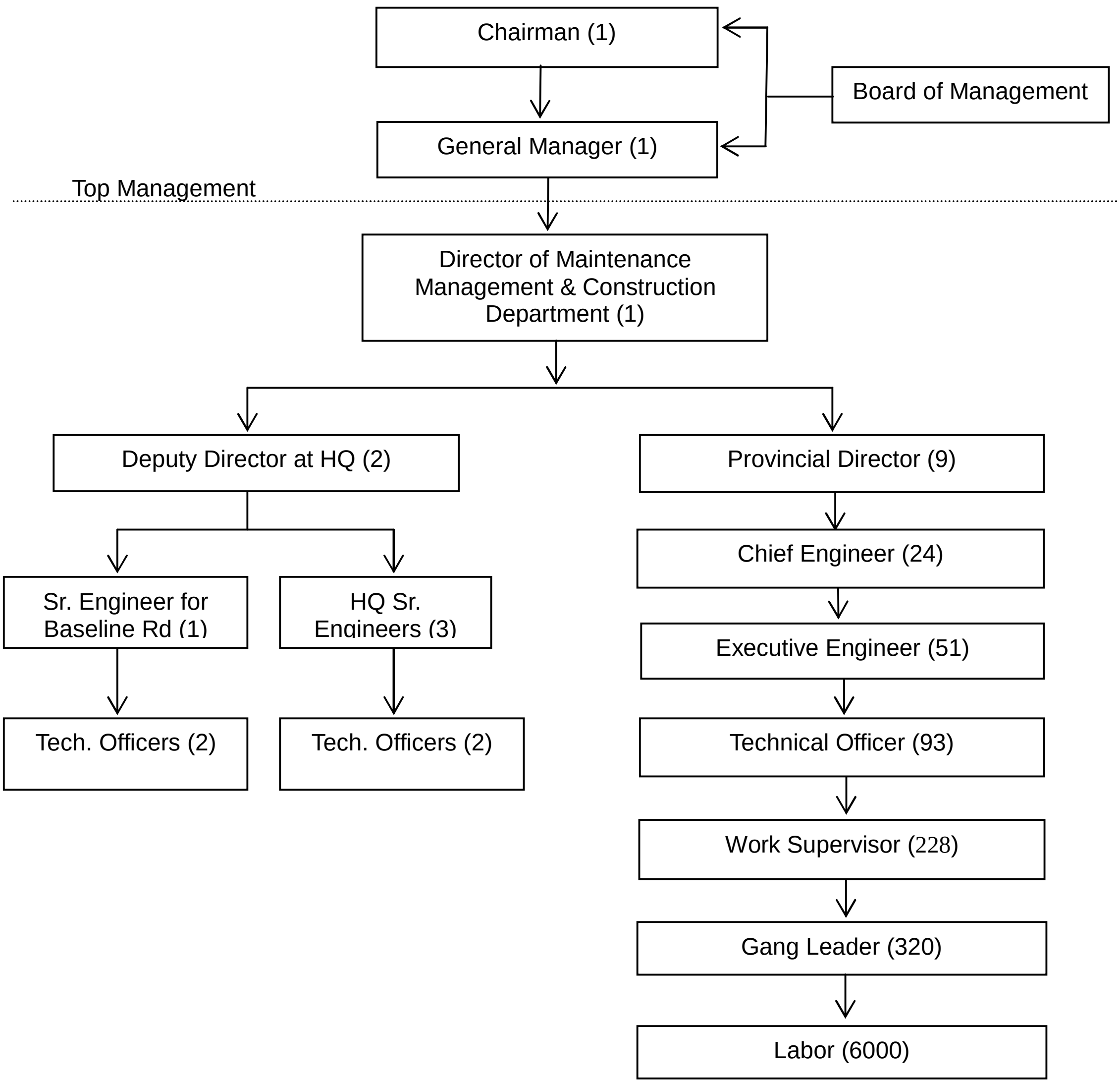


Fig.11.2.2 Line of Command for RDA Department of Maintenance Management & Construction

As the above figure indicates, the director of the Department is the person directly in charge of road maintenance, with two deputy directors and their staff assisting him at RDA headquarters, as well as nine provincial directors (PD) supporting him in administering maintenance in each of Sri Lanka's provinces. Under the PDs, there are 24 chief engineers (CE) in charge of maintenance in the districts of the provinces. The actual execution and supervision of maintenance in the field then takes place from the offices of the 51 executive engineers (EE), with each CE having on average two to three EEs working for him. Technical officers are stationed at the offices of both the EEs and CEs, while work supervisors and gang leaders are stationed at the offices of the EEs.

Note that the top managerial structure of RDA is in the process of transition, and is being re-engineered in order to rationalize the number of people reporting to the general manager from the present 14 directors to five additional general managers (AGM) and is as indicated in Fig. 11.2.3. That is, the current functions and staff of the present directors are to be reorganized to come under the five AGMs, with the organizational setup from provincial directors downward to remain essentially the same. The function of having audits regarding RDA's finance, performance, and the safety of its roads reported directly to the chairman has also been added. Eventually, the AGM for PMUs is to be phased out and future projects would come under either the AGM for Planning & Programming or the AGM for Asset Development.

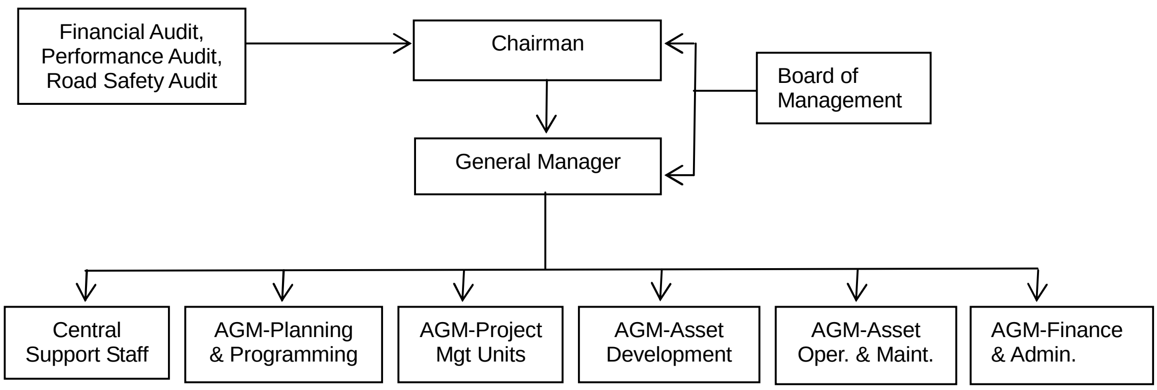


Fig.11.2.3 Proposed Organization for RDA's Top Management

11.2.3.Organizational Structure for Provincial Roads Maintenance

Provincial roads are defined here as Class C and D roads and link up the interior of a province with the national road network, as well as linking up settlements with markets and other intra-provincial facilities. These 14,213 km of road are maintained by the nine PRDAs of the provincial councils, which were established in 1987 under the Provincial Councils Act (No. 42). Depending on the province, the type of entity responsible for maintaining provincial roads varies. That is, there can be an autonomous road authority, a department dedicated to road improvement and maintenance contained within a provincial council, and a general maintenance department also coming directly under a provincial council responsible for the upkeep of all a province's assets (including roads). As Table 11.2.2 indicates, three provinces have autonomous road authorities, two provinces dedicated road improvement and maintenance departments, and three general maintenance departments.

Table11.2.2 Type of Entity Executing Road Maintenance by Province

Province	Type of Entity Executing Road Maintenance	Comments
1. Western	Authority	An autonomous entity that executes road improvement & maintenance
2. Southern		
3. North Central		
4. North Western	Dedicated road maintenance department	An entity coming directly under a provincial council that executes road improvement & maintenance
5. Central		
6. Sabaragamuwa		
7. Uva	General maintenance department	An entity coming directly under the provincial council that executes various types of maintenance, such as irrigation & buildings, as well as road improvement & maintenance
8. Eastern		
9. Northern		

For reference, the line of command for the PRDA of Western Province (PRDA-WP) is given below.

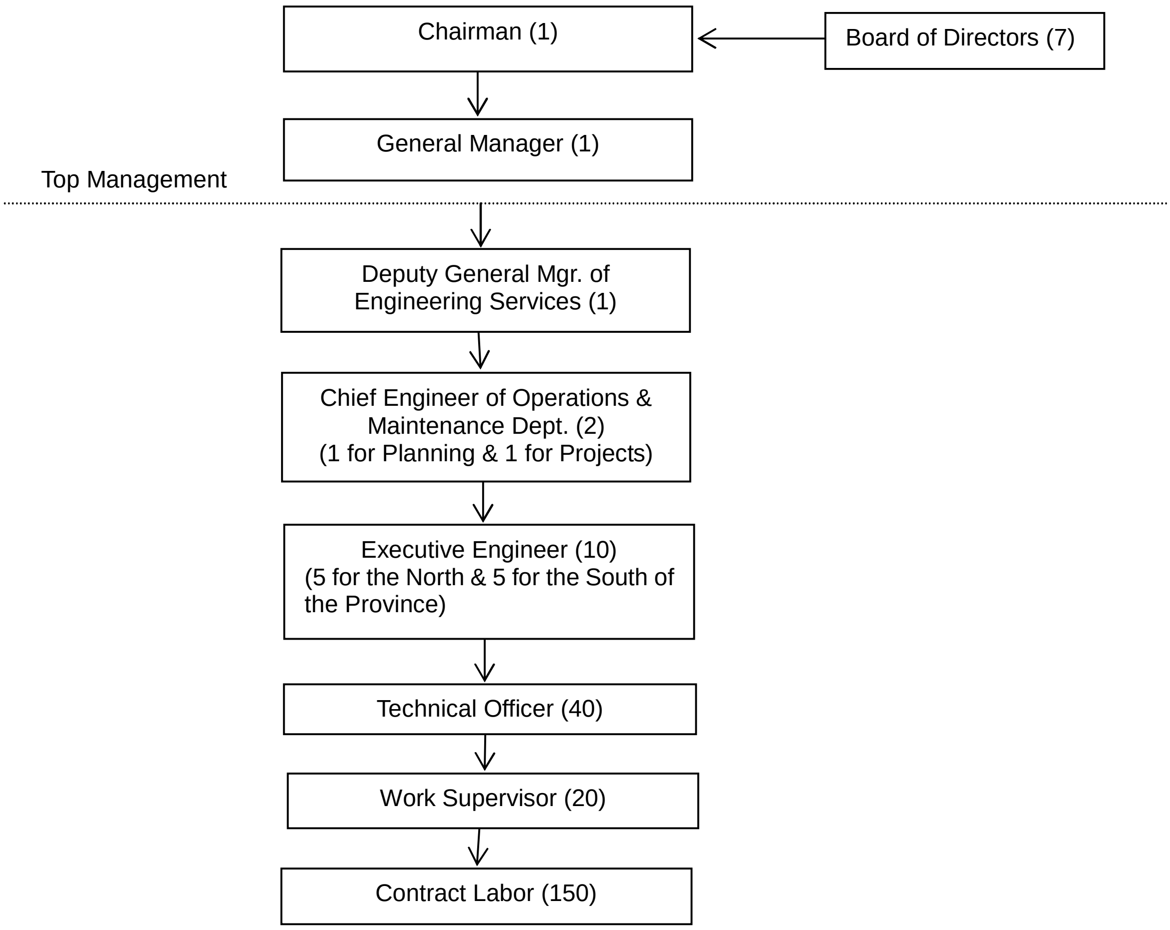


Fig.11.2.3 Line of Command for PRDA of Western Province

As Fig. 11.2.3 indicates, the two chief engineers (CE) of the Operations and Maintenance Department are directly responsible for administering the work of ten executive engineers and their offices for the north and south of the province, which are distributed throughout the province’s three districts in order to oversee the execution of maintenance work on site. These offices have technical officers and work supervisors stationed there and manual labor (about 150 workers on average) is contracted on a day-to-day basis. The CE is ultimately responsible to a deputy general manager, who oversees the entire Engineering Division. Above the deputy general manager is the top management for the province, which consists of a general manager, chairman and a board of directors. The chairman is one of the seven board members and is the head of the PRDA-WP, while the general manager is in charge of the execution of the powers, functions, and duties of the authority. Note that re-engineering has been taking place as part of ADB's Road Sector Development Project (RSDP) and as a result PRDA-WP has added a separate department for quality control and human resource development, which according to PRDA-WP managers has greatly improved the performance of the organization.

11.2.4. Organizational Structure for Local Roads Maintenance

Local roads are defined here as Class E, municipal, town, and village roads and provide connections to specific locations within an area. Note that the exact length of unclassified roads is hard to assess and can vary due to the definition of what constitutes a road. Local roads account for the vast majority of the road network (i.e., more than 70%) and are maintained by the relevant local government, which also comes under the control of a provincial council. In Sri Lanka, there are three types of local government: (1) municipal council, (2) urban council, and (3) local authority. The way that road maintenance is carried out varies from place to place and it seems that provincial councils have not given guidance on appropriate institutional arrangements.[1]

For reference, the road maintenance institutional setup for the Colombo Municipal Council (CMC) is given in Fig. 11.2.4. As the figure indicates, the director of Works is responsible for the overall conduct and performance of the maintenance of roads coming under the authority of the CMC, which totals approximately 500 km. This director is one of eight directors attached to the Division of Engineering Services, which is one of a total of three divisions that execute work and provide services for the CMC. Then, there are three superintendent engineers that assist the director in the administration of CMC's road maintenance and who are also stationed at city hall. One superintendent is in charge of three district engineers for the north and another superintendent of three district engineers for the south of the city. The remaining superintendent is responsible for supervising the work of the central road maintenance unit (CRMU). Note that the CMC is divided into six districts for maintenance purposes and that each district has a district engineer in charge of managing work on site. The CRMU, on the other hand, is responsible for maintaining the most important roads of the city regardless of their location. There are a total of 15 senior technical engineers, 35 technical officers, and 35 overseers that provide support to the district engineers and the CRMU. On average, there are at any one time a total of 1,000 manual laborers. Finally, above the director of Works is the top management of the CMC, which is comprised of the deputy municipal commissioner (DMC) for Engineering Services (and who is one of three DMCs) and the municipal commissioner (MC). The DMC meets with the directors and superintendent engineers every two weeks to check and plan strategy, who then reports to the MC. The MC is then responsible for reporting to the political representatives of the CMC regarding the city's performance in providing services to the citizens of the city.

[1] *Report and Recommendation of the President to the Board of Directors on a Proposed Loan and Technical Assistance to the Democratic Socialist Republic of Sri Lanka for the Road Sector Development Project*, RRP: SRI 31280, ADB, November 2002

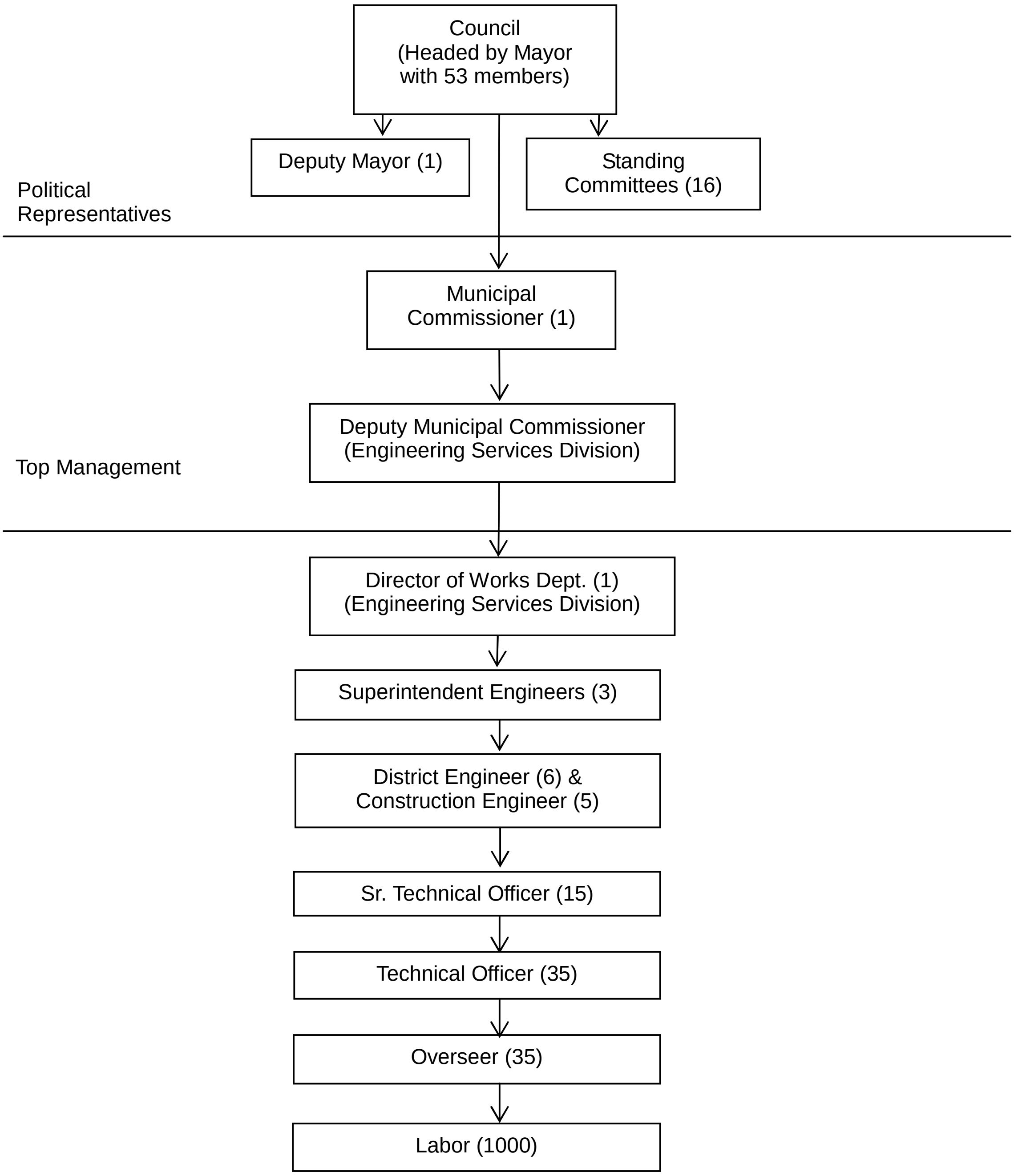


Fig.11.2.4 Line of Command for CMC Department of Works

11.3. Road Maintenance Funding

11.3.1. Overview of Current Status

Funding is one of the most important issues for road maintenance. Almost without exception, the indications are that at all levels of government funding is insufficient to satisfy maintenance needs. Table 11.3.1 shows that in the case of the RDA actual expenditures have exceeded the budget for seven of ten years between 1996 to 2006 by a margin of 1.13 to 2.48 times, with the shortage in funding met by the reallocating of monies internally. This indicates a chronic lack of money that is being dealt with by last minute stopgap measures.

Table11.3.1 Budget & Actual Expenditure for Road/Bridge Maintenance in RDA

Year	Item	Total (Rs. millions)	Ratio (B/A)
1996	Budget (A)	575	2.21
	Actual Expenditure (B)	1,269	
1997	Budget (A)	572	2.18
	Actual Expenditure (B)	1,248	
1998	Budget (A)	560	2.48
	Actual Expenditure (B)	1,388	
1999	Budget (A)	1,825	0.78
	Actual Expenditure (B)	1,416	
2000	Budget (A)	1,940	0.92
	Actual Expenditure (B)	1,794	
2001	Budget (A)	860	1.76
	Actual Expenditure (B)	1,514	
2002	Budget (A)	900	1.86
	Actual Expenditure (B)	1,671	
2003	Budget (A)	1,620	1.13
	Actual Expenditure (B)	1,834	
2005	Budget (A)	2,123	1.25
	Actual Expenditure (B)	2,650	
2006	Budget (A)	3,478	0.84
	Actual Expenditure (B)	2,933	

Source: Department of Maintenance Management & Construction, RDA.

In the case of provincial and local roads as well, the situation is serious. For example, in the case of the PRDA-WP, the difference between the amounts requested for road maintenance and the actual funds received are large and are as shown in Table 11.3.2. As the table indicates, only 40% to 50% of the funding needed is made available. On the other hand, in the case of the CMC, a request for Rs. 205 million for the year 2005 was made for road maintenance but only about 49% of that was allocated. In 2007, all of the allocated funds for road maintenance will have been expended by the end of July, meaning that for the remaining 6 months of the year no maintenance work will be possible. Given that the Western Province is the richest province in Sri Lanka, it can be said that this is indicative of the difficulty in securing sufficient monies for the upkeep of roads.

Table11.3.2 Comparison of Funds Requested & Received for Road Maintenance in the PRDA-WP

Year	Funding Requested for Road Maint.	Funds Actually Received for Road Maint.	Ratio (B/A)
2005	Rs. 250 million	Rs. 120 million	0.48
2006	Rs. 300 million	Rs. 120 million	0.40
2007	Rs. 400 million	Rs. 200 million	0.50

Source: Provincial Road Development Authority of Western Province.

11.3.2.Existing Sources of Revenue for Road Maintenance

Previously, all money for road maintenance for Class A and B roads came from the consolidated account of the Ministry of Finance (MOF) of the Central Government via the Ministry of Highways. However, with the establishment of a Road Maintenance Trust Fund (RMTF) in 2006 as part of the World Bank's Road Sector Assistance Project (RSAP), monies for maintenance for RDA now also come from the RMTF, with the Government of Sri Lanka (GOSL) to allocate funds as agreed upon in a loan covenant with the World Bank for the implementation of the RSAP. As Table 11.3.3 indicates, the GOSL is to provide US$ 30 million in 2006 and afterwards to increase that amount by US$ 4 million annually till the year 2010, which will total US$ 46 million.[2]

Table11.3.3 Tranches of Funds from Gov't to RMTF

Year	Funds to be Allocated to RMTF from MOF (US$ million)
2006	30
2007	34
2008	38
2009	42
2010	46

As for provincial and local roads, the vast majority of funds come from the finance commission of a provincial council, which also receives monies from MOF's general account via the MOPCLG, as well from local taxes on items such as property and rents from public facilities. In addition to finance commission funds, there are also two small discretionary sources of money managed by provincial council politicians known as the District Development Council for Improvement and the Decentralized Budget.

As the above discussion makes clear, a dedicated fund for road maintenance has only been set up recently, and partially explains the reason for maintenance being ill funded, as it has to compete for money with other uses that may seem more urgent then the staid concept of filling in potholes or pavement overlaying. On the other hand, road sector revenue (see Table 11.3.4) would be more than sufficient to fund the current maintenance budget of the entire classified road network, which is about 27,000km in total length. In fact, the tax on petrol and diesel fuel alone may be enough to cover the maintenance costs of all classified roads, meaning that there is not necessarily a lack of money but a mechanism to ensure that the money required for maintenance is made available. In fact, if the concept of "user pays" is applied, road users seem to be paying more than their fair share. On the other hand, it can be understood to a certain extent that the GOSL as a result of revenue shortfall may have no choice but to use road-users charges income for purposes other than roads.

Table11.3.4 Road Sector Revenue (2006)

Type of Tax or Fee	Rs. (billions)
Excise Duty on Petroleum*	14.1
Vat on Petroleum Sales**	12.0
Excise Duty on Motor Vehicle Imports***	20.7
Excise Duty on Spare Parts Imports*	0.5
Motor Vehicle License***	3.0
Total	50.3

Source) *: Sri Lanka Customs Dept. **: Ceylon Petroleum Corp. ***: Dept. of Fiscal Policy

[2] According to the World Bank's 2005 Project Information Document (Report No: AB1713), about US$ 45 million is needed annually for RDA to address the needs of a stable network and the government only allocates on average about 35% of the funds required for the national road network, a large portion of which is used to pay staff salaries.

11.3.3.Need for Road Maintenance Funding Mechanism

As the previous discussion indicates, there is a need for dedicated sources of funding to meet the costs of road maintenance in Sri Lanka. The GOSL recognized this as early as 2000 and, with the assistance of ADB, incorporated into its 2002 National Road Policy its intention to establish a sustainable road maintenance funding mechanism, which is also contained in the GOSL's economic reform program known as "Regaining Sri Lanka" announced in December 2002. The GOSL also stated around this time that it would consider applying a Rs. 1.0 and 0.5 per liter cess on gasoline and diesel fuel, respectively, for any road maintenance fund eventually set up.[3] An ADB consultant (OPUS International of New Zealand) was then engaged in September 2004 to design the road maintenance fund and was to have finished by April 2005. Although the cess was introduced, it seems that nothing came of the road fund and it was not until the World Bank, as part of an agreement with the GOSL to re-engage in the transport sector in Sri Lanka, that an intermediate road fund (IRF) was established in 2006. The structure of the IRF is indicated in Fig. 11.3.1.

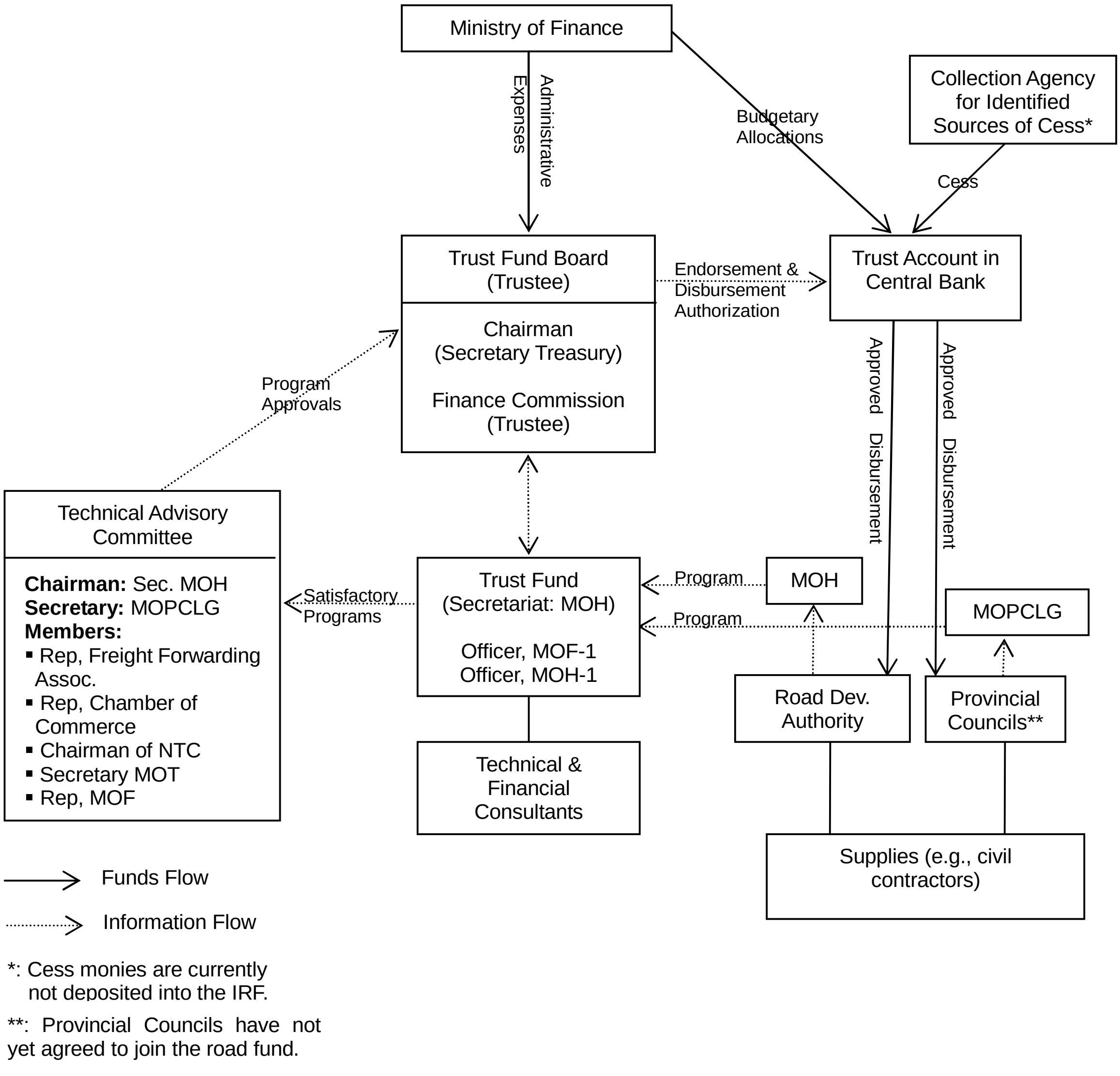


Fig. 11.3.1 Intermediate Road Fund Structure

[3] *Country Strategy and Program 2004-2008*, ADB, CSP: SRI 2003-19, 2003

As the above figure indicates, the IRF is managed by the MOF and input from a technical advisory committee determines which road maintenance projects are to receive money. This determination is based on input from the RDA and the provincial councils, which is examined initially by the Trust Fund with the aid of technical and financial consultants. The account for the road fund is located in the Central Bank and is overseen by the secretary of Treasury and a Finance Commission. This is an interim setup it is not the best practice in terms of road fund operation, which is represented by second generation road funds that have been established in other countries. Some of the current major weaknesses are as follows:

- The road fund is dependent on allocations from the consolidated revenue fund, as cess monies (taxes on gasoline and diesel) still have not been officially earmarked, making it vulnerable to the cash flow problems.
- The provincial councils still have not joined, leaving out of consideration important feeder and collector roads that form part of the core road network.
- The road fund is not governed by an independent body, which should be preferably established by parliament, making it politically weak in regards to making important decisions regarding road maintenance management.
- The monies distributed to the RDA go into a general account and therefore can be used for any purpose. (World Bank monitors road fund monies to ensure intended use.)

According to the World Bank, the above weaknesses are to be addressed within three years after the establishment of the IRF, or mid-2009. At that time, a second generation road fund will be set up.

11.4. Performance & Capacity for Road Maintenance

11.4.1.Overview

It has often been stated in technical publications that for every additional $1 spent on maintenance in developing countries road users save $3.[4] This has and is the crux of the argument for maintaining roads properly. On the other hand, the condition of Sri Lanka's even most important roads have something to be desired as shown in Table 11.4.1. For example, the table indicates that in 2005 only 18.4% of Class A and B roads were in good condition, while 51.8% were in poor to bad condition. This is also substantiated by the World Bank, which says that more than 50% of all national roads have a poor or bad surface condition.[5] As for provincial roads, only 8.0% were in good condition, while 40.0%, 36.0%, and 16.0% were in fair, poor, and bad condition, respectively, indicating a serious lack of maintenance. As for local roads, in 2005, only a mere 5.0% were in good condition with 30% in fair condition. On the other hand, 65% of these roads were in either poor or bad condition. Furthermore, according to a road sector note by the GOSL in 2005, only 10 to 15% of the rural road network provides all-weather access.

Overall, only 38.9% of Sri Lanka's roads are in fair to good condition, while 61.1% are in poor to bad condition.

[4] *Road Funds and Road Maintenance: An Asian Perspective*, ADB, July 2003
[5] Project Information Document, Appraisal Stage, Report No AB1713, 30 August 2005.

Table11.4.1 Condition of Road Surface of Sri Lankan Roads (2005)

Type of Road	Good (%)	Fair (%)	Poor (%)	Bad (%)
National Road	18.4	29.8	35.4	16.4
Provincial Road	8.0	40.0	36.0	16.0
Local Road	5.0	30.0	40.0	25.0
Total	7.2	31.7	38.7	22.4

Source: Final Report on Road Sector Master Plan (ADB TA 4315-SL), 2005.

Although the lack of money as previously described is one of the reasons for the poor performance just cited, interviews with government agencies and a review of extant material also suggests that there are institutional and capacity problems as well. These problems make it difficult or impossible to carry out many of the following management tasks that are required in order to ensure proper road maintenance:

(1) **Inventory:** recording of the basic characteristics of each section of the road network.
(2) **Inspection:** examination of the road and the measurement of its condition.
(3) **Determination of maintenance requirements:** analysis of defects and specification of the maintenance activities needed to correct them and delay further deterioration.
(4) **Resource estimation and work prioritization:** costing of the maintenance program to define an overall budget and the prioritization of work to ensure high-priority work is carried out.
(5) **Work scheduling and performance:** supervision of work to ensure that it is carried out as intended.
(6) **Monitoring**: checking of the quality and effectiveness of the work.

Below, the institutional and capacity problems facing the maintenance of national and provincial/local roads are briefly examined.

11.4.2. National Roads

From 1989 until 1994 the RDA, the organization responsible for national roads, outsourced nearly all of its maintenance and construction work to its subsidiary RCDC. In fact, it was the policy of the GOSL to have all road maintenance works for national highways contracted out to this organzation.[6] Then, from 1995, all of routine maintenance began to be carried out by RDA via force account, while in the case of periodic maintenance and minor improvement work about 50% went to RCDC and the remainder to small-scale private contractors. As for major construction work, all of this went to RCDC automatically. Unfortunately, it seems that RDA and RCDC usually did not avail themselves of contracts in their business dealings. As a result of this lax business relationship and the incentive therefore not to perform, the RDA has experienced the following shortcomings[7]:

[6] *The Second Road Improvement Project in Sri Lanka*, Project Performance Audit Report, ADB, June 2000.
[7] Reference material same as that of footnote 1

(1) Insufficient capacity to systematically plan, program, and prioritize road development (including maintenance).
(2) Lack of pre-construction processes required for advance preparation and planning of complex projects.
(3) Lack of experience, especially at the provincial level, in the use of employer-contractor contracts needed for effective management and control of civil works.
(4) Lack of systems at the provincial level for road maintenance management including planning, budgeting, programming, and controlling.
(5) Lack of a technical audit function.

In response the GOSL, as part of its agreement with ADB, started to reform and re-engineer the road sector. As a result the RCDC, which was to be privatized over a period of five years between 2003 and 2008, was suddenly dissolved in December 2003 as part of a push towards privatization by the government administration then in power. This resulted in RDA having to contract out with little notice a large portion of its periodic maintenance and all of its construction/improvement work to private independent firms, which has been done relatively smoothly.

In addition to organizational changes, there have also been changes in work methods. For example, the RDA introduced in 2003 a software tool known as RMBEC (Road Maintenance Budgeting and Expenditure Control) for the planning, prioritization, and management of maintenance work. This tool is an EXCEL-based program that uses Visual Basic.[8] The modules that compose this program are as shown in Figure 11.4.1, and the program has been installed in RDA's provincial, district, and divisional maintenance offices throughout the country. Note that all maintenance work programs and budgets are prepared using RMBEC, although there are sometimes glitches with staff sometimes not being sufficiently trained in the use of the software due to personnel changes.

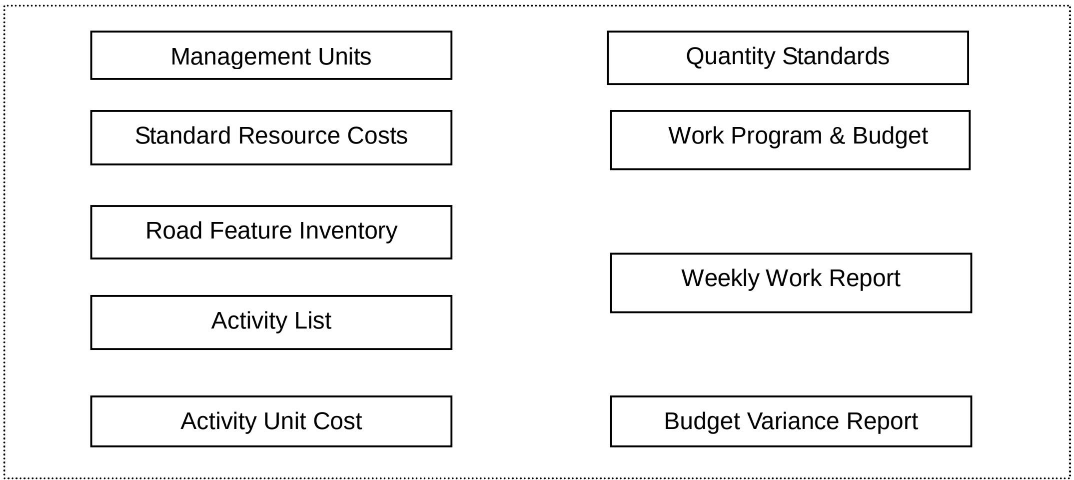


Fig.11.4.1 Modules Comprising RMBEC

The above indicates that RDA's planning, work prioritization, and management capabilities have improved and should continue to do so under the road sector's reform program. However, it should be noted here that the RMBEC system is dependent on

[8] This was carried out with the assistance of an ADB TA known as *Re-engineering of Road Sector Institutions*.

reliable data being available in a timely manner. The collection of this data is no easy task, as it requires scores of men in the field to assess and monitor thousands of kilometers of road. This task is made harder by the fact there are at present no "pocket-sized" booklets for engineers and technicians for the purposes of maintenance inspection, evaluation, and execution. It is recommended that such booklets be prepared and made widely available so that even technicians can carry out inspection and evaluation. Outputs should also be standardized so they can be fed into RMBEC quickly and smoothly. Note that the current specifications for road maintenance have also not been updated since 1989 and have been ignored by some as being outdated. According to ICTAD, which is in charge of this updating work, new road maintenance specifications should be ready for publication in November 2007.

11.4.3. Provincial & Local Roads

The provincial councils, municipal/urban councils, and local governments oversee an existing road network that is vast in scope. Therefore, it can be said that their overriding duty is to maintain or improve existing roads and not to construct new ones. In fact, from the perspective of road density, Sri Lanka's 1.5 km per sq km is significant larger than the worldwide mid-value of 0.2 km per sq km.[9]

Unfortunately, even to a greater degree than the RDA, the PRDAs and maintenance units responsible for roads that come under the municipal/urban councils and local governments do not have adequate capacity to do this. Especially, for those roads under the jurisdiction of local governments, there is essentially no operations and maintenance system in place.[10] In addition to the problems previously described as affecting RDA, other issues that confront PRDAs and local maintenance organizations are as follows[11]:

(1) Insufficient or no road inventory data.
(2) Insufficient computerization in regards to the planning, budgeting, prioritization of road maintenance.
(3) Lack of coordination.
(4) No standard format for maintenance work programs and reporting.
(5) Insufficient expertise in utilizing the private sector to do work.

The above depicts a problematic situation regarding road maintenance at the provincial and local levels. However, the GOSL with the assistance of ADB is trying to remedy this situation in regards to provincial roads via the RSDP. That is, under the provincial component of the RSDP[12], the ADB will assist provinces with institutional reform to improve road management, carry out road inventory surveys, build up road maintenance computer databases, and assist with the improvement of about 1000 km of road in North Central, North Western, Western, and Uva provinces. This work will also take into account the results of ADB's "Southern Provincial Roads Improvement Project", which is the predecessor to this effort. That is, the results and methodologies of ADB's work

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[9] http://www.worldbank.org/transport/roads_ss.htm
[10] *Southern Provincial Rural Road Development Project*, Project Completion Report, ADB, March 2004.
[11] This is based on ADB literature and the Consultant's interviews. In one particular interview by the Consultant, it was mentioned that management positions are left vacant for extended periods of time due to wrangling over appointees, which leaves the organization with insufficient capacity to do its work, and that this was causing serious problems with capacity.
[12] RSDP consists of three components: (1) assist RDA to plan, develop, test, and document procedures and road management systems; (2) assist provincial project management units with improvement of about 1000km of road; (3) provide consulting services for detailed design of national roads selected by feasibility study (see http://www.adb.org./Documents/Profiles/LOAN/31280013.ASP)

with the Southern Provincial Council, which focused on establishing capacity for efficient and effective management and on developing institutional, financial, and private sector contractor resources for sustainable road maintenance, will be utilized as a reference point. Despite this, time will be required to bring the desired and necessary changes, and for that reason a sector approach is being taken by ADB in cooperation with other international donors. There is also the problem that once a project is completed that the improvements or recommendations made will not continue to be carried out. For example, it seems that Southern Province reduced its investment for road maintenance after the above-mentioned ADB project was completed and, together with other provinces receiving ADB assistance, is not executing road condition surveys annually as expected. In interviews with provincial officials, it was clearly stated that there is a lack of desire to adhere to best practices for road maintenance if there is insufficient money to actually do the work required. Another possible reason for the problems with continuity could be that the data and information requirements may be too onerous.

The preceding effort, however, is aimed at the provincial level, the results of which can perhaps be easily transferred to the municipal or urban level, and does not seem to consider roads that are managed by the local governments (known in Sri Lanka as Pradeshiya Sabhas). These roads account for the majority of the country's road network in terms of length and, given the small resources available to local governments, their sheer volume makes it physically impossible for local government by itself to maintain. Given this, community participation would seem to be a necessary pre-condition in order to realize effective maintenance for these roads. However, past experience has shown that it is not easy to sustain such an effort. In response to this, Practical Action[13] has put together the following program and it seems to have been successful in enlisting such participation on a sustainable basis:

(1) The introduction of low-cost equipment and tools simple to use.
(2) Tools light enough so that women and senior members of a community can participate.
(3) A pilot project that offers a 3 to 5 day training period that continues throughout construction/maintenance.
(4) Introduction of systematic community participation via formation and strengthening of a "village development society" that includes the establishment of work rosters, payment schedules, etc most suitable to the particular community. Note that village technicians are selected by these societies and all key decisions regarding work are also made there.
(5) Encouragement to learn and practice methodical systems so as not to waste time and effort.
(6) Establishment of close ties between the community and local government authority directly responsible for the road by keeping the chairman and technical officer of the authority informed of work progress as well as having them as participate in the work monitoring meetings.
(7) Promotion of synergy between the community and the local government authority where for example the former will provide the necessary labor and the latter access to materials.

[13] NGO formerly known as the Intermediate Technology Development Group (ITDG). Its mission is to assist poor people using technology to realize sustainable livelihoods. Its office is located at 5, Lionel Edirisinghe Mawatha, Kirulapone, Colombo 05.

Note that Practical Action has also produced a *Technical Manual on Community Based Rural Road Construction and Maintenance*, which is based on more than a decade of experience, and it is utilized to implement cost-effective approaches for rural road construction and maintenance in a simple and effective manner.

11.5. Private Sector Participation in Road Maintenance

11.5.1. Overview

Depending on the organization, the level of private sector participation (PSP) in road maintenance at present will vary greatly in Sri Lanka (see Table 11.5.1). For example, the CMC does not contract out any of its work to the private sector. On the other hand, the PRDA of Western Province contracts out 50% of its routine maintenance and 100% of its periodic maintenance. As for RDA, all of its routine maintenance is by force account while 90% of its periodic maintenance is outsourced. It is hard to draw any conclusions just from these three examples, except to say that all of them handle much or all of their routine maintenance in-house with basically only one organization handling its periodic maintenance (i.e., CMC). Note, however, that worldwide private contractors undertake the majority of periodic maintenance while the majority of routine maintenance is by force account.[14]

Table11.5.1 Percentage of Force Account & Contracted Out Work by Organization and Type of Maintenance

Organization	Routine Maintenance		Periodic Maintenance*	
	Force Account	Contracted Out	Force Account	Contracted Out
RDA	100%	0%	10%	90%
PRDA of Western Province	50%	50%	0%	100%
CMC	100%	0%	100%	0%

*: In the case of RDA this includes minor improvement works.

Although the above indicates that PSP in road maintenance in Sri Lanka is already relatively high, consideration could be given to outsourcing more routine maintenance, and thereby creating a more competitive environment that could reduce costs for this type of work as well.

Another important consideration is the capability of contractors. In an interview with a CMC official, it was stated that it was difficult to hire contractors of sufficient capability, with the example of being unable to find a qualified contractor to raise the level of manholes cited. In addition, the CMC official noted that as city workers are permanent employees they have detailed knowledge on what must be done and therefore their quality of work is higher than that of a private-sector company. For these reasons, CMC has continued to use the force account system for all its road works and to retrain their workers as needed for handling new works.

In regards to PSP, perhaps one of the main questions here is the modality of PSP and not just PSP itself.

[14] *A Review of Contract Maintenance for Roads*, Prof. K. Madelin, Transport Research Laboratory, UK, 1999.

11.5.2. Importance of Effective PSP Modality

PSP does not itself necessarily guarantee desirable results. That is, the most important thing is not privatization but ensuring the existence of competition as a way to achieve greater efficiency, and it is therefore important to consider the method of PSP. Usually, however, PSP and a competitive environment will go hand in hand. The full range of PSP modalities, which cover both construction and maintenance, are as shown in Table 11.5.2.

Table11.5.2 Types of PSP Modalities

Type of PSP	Description
Performance-Based/Maintenance Management Contracts	Private sector maintains existing road under performance specifications and receives payments from government.
Turnkey Contracts	Private sector designs and constructs a new road to government specifications and receives a fixed payment on completion.
Operation (tolling) and Maintenance Contracts	Private sector maintains road to agreed standards and collects tolls from users that finances the maintenance.
Rehabilitation, Maintenance and Operation	Private sector undertakes significant rehabilitation works to bring existing road to agreed standards, maintains it to those standards, and collects tolls to finance both rehabilitation and maintenance.
BOT	Private sector undertakes and finances design, construction, tolling, and maintenance, usually of large infrastructure projects. The private sector can also bear much of the risk depending on the negotiated concession agreement.
Corridor Management Contracts	Private sector undertakes new construction and maintenance (or rehabilitation) and operation of existing facilities and allows government and private sector to consider roads on a corridor or network basis.

Source: *Developing Best Practices for Promoting Private Sector Investment in Infrastructure: Roads*, ADB, 2000.

PSP in the form of performance-based contracting (PBC) has been a particularly successful method in regards to carrying out road maintenance in a cost-effective manner. Unlike the traditional way of contracting out maintenance work, which is based on a schedule of unit prices and estimates of quantities, PBC pays the contractor on how well he manages to comply with performance standards and not on the amount of work he does. The number of performance indicators in a performance contract can range from 20 to 100. Typical indicators would include the IRI (i.e., the international roughness index), the absence of potholes, the control of cracks and rutting, and the absence of siltation of drainage structures.

Note that a US pilot study involving federal agencies has shown that savings from 8% to 23% are possible with the introduction of PBC (see Table 11.5.3). In Australia and New Zealand as well, cost reductions between 10% and 20% have been possible. In fact, PBC has spread to European, African, and Asian countries and is supported by the World Bank, the European Bank for Reconstruction and Development and the Asian Development Bank.[15]

[15] *Implementing Performance-based Road Management and Maintenance Contracts in Developing Countries*, GTZ, Germany, November 2004.

Table11.5.3 Savings from the Use of Performance-Based Contracts by the US Federal Government Agencies

Type of Contract (in terms of price)	Number of Contracts	Average Savings
< US$ 1 million	8	15%
US$ 1 – 5 million	6	12%
US$ 5 – 10 million	6	23%
> US$ 10 million	4	8%

Source: Based on information from *Performance-Based Contracting for the Highway Construction Industry*, Final Report, Battelle, February 2003.

11.6. Stance of International Donors on Road Maintenance in Sri Lanka

Most, if not all, donors would agree that stable funding and private sector participation are two of the most important factors for sustainable and efficient road maintenance. Usually for this to occur in developing countries, institutional reform and capacity building are necessary. In response to this, the ADB and GOSL came to an agreement in 2002 to reform the road sector via the Road Sector Development Project (or RSDP). The framework of the RSDP serves as a roadmap for the sector's reform, and based on that the World Bank and the Japan Bank for International Cooperation (JBIC) agreed to coordinate their activities in November 2004 with the ADB. That is, the World Bank is to address the issue of sustainable road maintenance funding and JBIC the issue of private sector development in the road construction industry, while the ADB is to focus on institutional strengthening and capacity development.

In the case of realizing stable funding, reform would be carried out to enable the implementation of a road fund with dedicated road-user charges to be used for road maintenance that would managed by an independent body, which would be established by an act of Parliament. This is considered best practice and is substantiated by ADB's *Road Funds and Road Maintenance* report of 2000, where it clearly states that there is only a small chance of a developing country consistently allotting sufficient revenue to meet road maintenance needs. Note that there are 12 countries in Asia (including Sri Lanka) that either have introduced or are planning to introduce a road fund. On the other hand, as stated in 11.3.3, the current road fund in Sri Lanka is an interim setup and it is expected with the support of the World Bank's Road Sector Assistance Project to take three years before it is in accordance with best practice. Note that one of the World Bank's aims is to increase the percentage of national roads in good and fair condition from the current 48% to 60% by 2010, and to prevent the surface condition of these roads from deteriorating any further via the establishment of a sustainable road maintenance funding mechanism.[16]

As for the promotion of PSP in road works (including maintenance), JBIC commissioned a study from November 2006 to June 2007 to examine this issue in detail and to recommend proposed action frameworks (PAFs).[17] As shown in the Table 11.6.1, four PAFs have been put forward by the study: (1) increase outsourcing of civil engineering works by public sector; (2) strengthen technical and administrative capabilities of managers/supervisors of the private sector; (3) support measures to promote construction industry; and (4) expand and stabilize construction material supply. After a careful review

[16] Reference material same as that of footnote 5
[17] The name of the study is *Development of Domestic Private Sector on Civil Works and Engineering in the Road Sector*, and it was executed by Oriental Consultants of Tokyo, Japan.

of previous studies by other donors (especially the ADB), these PAFs were selected and their contents finalized based on feedback from two workshops that included both public and private stakeholders of the road sector, as well as donor representatives from, JBIC, ADB and World Bank. As the contents of the PAFs indicate, in order to promote PSP in the road industry, it necessary to take a comprehensive approach that considers the strengths and weaknesses of the road administrators and private firms, the business environment, and the demand and supply for materials.

Table11.6.1 Proposed Action Frameworks to Facilitate PSP in the Road Sector (1)

Name of PAF	PAF Projects	Description of Project	Responsible Entity
1. Increase outsourcing of civil engineering works by public sector	1) Outsourcing of road maintenance works	- Routine maintenance executed by force account to be outsourced in pilot projects via performance-based road maintenance to be introduced via ADB funded projects.	RDA
	2) Outsourcing of planning and design works for domestic funded projects	- Planning and design works for local funded projects have been executed by in-house engineers and more of this work shall be outsourced to local private consulting firms in order to strengthen capacity.	RDA
2. Strengthen technical & administrative capabilities of managers/supervisors of the private sector	1) Execution of training needs survey	- Current Tertiary and Vocational Education Commission training needs surveys cover the whole construction industry. Accordingly, it is necessary to execute road-sector specific needs surveys.	ICTAD, NCASL, ACESL
	2) Support to training institutes for managerial level course for middle level employees	- Some courses have been conducted by ICTAD and private training institutes. However, it is necessary to hold courses focusing more on road-sector topics.	ICTAD, private institutions (ACTA)
	3) Promotion of project OJT opportunities	- The project aims at having donor or government funded projects provide more OJT opportunities for domestic private firms. A training component shall complement the construction activities and consist of such things as construction planning.	RDA supported by donors
	4) Promotion of advanced managerial course for high level employees & owners	- Some courses by ACTA targeting high- level employees & owners will commence from this April.	ACTA
	5) Revision of Construction Industry Guarantee Fund Levy Act	- Present act does not describe levy objectives or the mechanism for its allocation. At present, levy funds go to the consolidated account. Revision of the Act is to rectify this and ensure the levy is only for construction industry development purposes.	MOF, MOH, MOCH

Table11.6.1 Proposed Action Frameworks to Facilitate PSP in the Road Sector (2)

Name of PAF	PAF Projects	Description of Project	Responsible Entity
3. Support measures to promote construction industry	1) Establishment or resumption of dialogue mechanism for construction industry	- A dialogue mechanism similar to that established in ADB's re-engineering action plan (i.e., Private-Public Partnership) shall be resumed and its dialogue activities monitored.	MOH, MOCH
	2) Enforcement of Construction Industry Authority Act	- A draft Construction Industry Authority Act is under examination by the government and dialogue with relevant organizations shall be monitored.	MOCH
	3) Revision of Construction Industry Guarantee Fund Levy Act	- The present act does not describe the objectives of the levy or the mechanism for determining its allocation. At present, Levy funds go to the consolidated account. Revision of the Act is to rectify this and ensure the Levy is only for construction industry development purposes.	MOF, MOH, MOCH
	4) Strengthening advocate associations	- NCASL to receive funds from a French agency to build a DB system, establish a laboratory & strengthen staff capacity.	Donors
4. Expand & stabilize construction material supply	1) Revision of Ceylon Petroleum Corporation Establishment Law	- Present law allows CPC to monopolize the import of oil products and it shall be revised to open the market to other domestic and foreign investors.	MOPPRD
	2) Support to expand aggregates :relaxatio n of EIA requirements for quarry site	- It takes about one year to get approval for a new quarry site from the CEA, and the project aims at relaxing regulations or EIA procedures for environmentally non-sensitive area.	CEA
	3) Promotion of research on marginalized materials	- Quarrry site survey has been conducted under ADB's re-engineering action plan and the project aims at studying marginalized construction materials to increase supplies.	RDA, ICTAD, NCASL
	4) Establishment of dissemination system on prices & source for materials	- An Aggregate Supply Information Unit is to be established via ADB funding and its activity will be monitored.	RDA
	5) Support to increase no. of facilities for sand exploitation	- Various attempts have been made for producing sand to complement exhausted river sand, including exploitation of sea sand and producing sand by crushing stone. The project will provide support to these new activities.	RDA, SLLRDC

Source) Based on information from *Development of Domestic Private Sector on Civil Works and Engineering in the Road Sector*, Final Report, JBIC Pilot Study for Knowledge Assistance, Oriental Consultants, June 2007

Note) ICTAD: Institute for Construction Training & Development; NCASL: National Construction Assoc. of Sri Lanka; ACESL: Assoc. of Consulting Engineers of Sri Lanka; ACTA: Advanced Construction Training Academy; MOF: Ministry of Finance; MOCH: Ministry of Construction & Housing; MOPPRD: Ministry of Petroleum & Petroleum Resources Development; CEA: Central Environmental Authority; SLLRDC: Sri Lanka Reclamation & Development Corp.

Finally, ADB has been carrying out institutional strengthening and capacity building at the national and provincial levels to enable road agencies to become better professional managers so that they can program, contract, and supervise maintenance work to be carried out by the more efficiently. This has been carried out as part of a re-engineering program for RDA and a capacity strengthening program for provincial road entities as shown in Table 11.6.2.

Table11.6.2 ADB Plans for Strengthening RDA & Provincial Road Entities

No	Name of Plan	Progress
1.	Re-engineering Plan for RDA	
1.1	Strengthening of Planning & Programming	- Data Collection Unit established & trained and data collected for two cycles completed - Inventory & condition data collected & inter-district transport model for core network completed - Economic-based planning manual completed but not comprehensive in nature; training needed & benchmarking with international organization required - HDM-4 calibrated and in operation; training of more staff necessary & benchmarking with international organization required - Draft Final Report for 50-year Road Sector Master Plan by RDA completed & Final Report under preparation
1.2	Strengthening of Pre-Construction Process	- Various guidelines (design, environment, resettlement, & tender) prepared & under RDA review - Based on the above review, RDA is to adopt a preconstruction process - RDA has adopted bidding process for private firms, but training is required
1.3	Strengthening of Construction Management	- Construction management manual prepared & approved by RDA - New construction management procedures introduced but training is required; Updating of procedures in a timely fashion also necessary
1.4	Strengthening of Road Maintenance Management	- Director of Planning of RDA making preparations for Maintenance Management Unit to implement RMMS (road maintenance management system) - Although the RMBEC (road maintenance budgeting & expenditure control) system is operational, it needs to be improved to handle HDM-4 data - Road condition of RDA roads reviewed by provincial offices for RMMS; However, BMS (bridge maintenance system) cannot be included in RMBEC
1.5	Development Plan for Technical Auditing	- Technical audit manual & procedures prepared - Delay in establishment of Performance Audit Division due to reluctance of professionals to join - Name of division to be changed to Quality Assurance Division
1.6	Strengthening of Land Acquisition & Resettlement Process	- A draft Social Safeguard Compliance Manual prepared; Final version to be used as a guide for resettlement planning of road projects - Environment & Social Division established & capacity building underway - Introduction of system to generate data for resettlement action plans, etc.
2.	Strengthening Plan for Provincial Road Provision	
2.1	Establishment of Inter-Provincial Coordination Committee (IPCC)	- IPCC members to be appointed by Provincial Councils and Ministry of Provincial Councils & Local Government (MOPCLG) and a report on the mandate and details of operation of the IPCC to be prepared and approved by the MOPCLG
2.2	Criteria & Targets for Institutional Development Agreed	- New structures will divide responsibilities between headquarters and divisional level branches - For a Provincial Road Development Authority, the new structure is expected to have about 200 staff - The World Bank's Road Economic Decision (RED) Model is to be introduced to planning and programming functions - Based on introduction of new road management techniques & software, objective road maintenance estimates will be produced - 20% of road maintenance to be executed as performance-based contracts
2.3	Institutional & Development Program of road sector for four Provincial Councils	- Institutional development of road authorities and departments for the Western, North Central, North Western, and Uva provinces currently being carried out

Source: Based on information provided in July 2007 from the Colombo Office of ADB, and from the *Report and Recommendation of the President to the Board of Directors on a Proposed Loan and Technical Assistance to the Democratic Socialist Republic of Sri Lanka for the Road Sector Development Project*, RRP: SRI 31280, ADB, November 2002

11.7. Operation & Maintenance (O&M) of the OCH

11.7.1. Operational Aspects

(1) General

Currently there are no expressways in Sri Lanka. With the completion of the Southern Highway, CKE and OCH, there will be a total of approximately 172 km of such roads. From the viewpoint of realizing effective integration and economies of scale, it would seem to make sense to have one organization manage this network. On the other hand, given that the CKE is to be constructed under a PPP scheme and the Southern Highway (SH) and OCH with government-guaranteed loans from international donors, this may prove hard to do, as the CKE would have its O&M scheme determined by the organization contracted to carry out its construction. It is therefore suggested at present that consideration be given to having the SH and OCH managed by a single organization.

(2) Organizational Structure

There are basically four possible options for establishing an entity to manage the SH and OCH and they are: (1) an operating division within an existing government agency such as the RDA, (2) a public corporation or authority, (3) a private corporation, and (4) a joint public-private venture. Since the Government of Sri Lanka (GOSL) is guaranteeing the entire amount of the loans for construction of the SH and OCH, Options (3) and (4) are not appropriate. In addition, as the private sector of Sri Lanka possesses no experience with the management of roads, Option (3) is not a viable choice. As for Option (1), there would be disadvantages such as: i) the expressway network could not be operated as a separate corporate entity with more effective governance and efficiency; ii) a lack of comprehensive and separate financial management and reporting; and iii) limitations in the ability to promote institutional improvements in the road sector and foster PSP. Based on this, Option (2), which would result in a public corporation (hereafter referred to as SLEA (or the Sri Lanka Expressway Authority)) being established as a legal entity with a board of directors, professional management team, qualified staff, and a separate accounting and reporting system, seems to be the most desirable operational structure for the management of the expressway network. In fact, the Cabinet of the GOSL granted its approval in June 2007 to the Ministry of Highways and Road Development to establish such an entity, and directed the Legal Draftman's Department to draft an "Expressway Authority Act".[18] The advantages of this option are as follows:

- Development of a corporate framework and work environment focused on the efficient operation of a modern infrastructure.
- Development of good governance practices.
- Separate and transparent financial management and accounting.
- Demonstration of a commitment to effective corporate development and potential future private sector involvement
- Commitment to new structures that reflect international best practice and promote future economic development.

In achieving the above, it is assumed that the following would also occur:

[18] *Establishment of an Expressway Authority*, Daily News.lk, 29 June 2007, http://www.dailynews.lk/2001/pix/PrintPageasp?REF=/2007/06/29/pol04.asp

- No direct management intervention by the Government.
- Independent auditing of annual statements of accounts.
- Independent determination of salary and employment categories and contracts by the expressway public corporation.
- Authority of the expressway public corporation to encumber assets or revenue streams against borrowings.
- The GOSL is to act like a normal shareholder with responsibility for appointing directors, supervisory board, auditors and raising the initial capital base.
- A Board of Directors (BOD) of the expressway public corporation is to exert authority on key decisions regarding the control of investments, executive compensation and bonuses, major company agreements, replacement or reassignment of company executives the merger or acquisition of other companies and the sale or encumbering of assets.

(3) Practical Operational Aspects

The operation of the systems comprising SLEA will be in compliance with all systemic and legal requirements designated by the GOSL. The day-to-day operations of the expressway will be directed from a control center with two to three staff manning it and will function 24 hours a day 365 days a year. The center will be responsible, amongst other things, for activities such as the following:

- Preparation and revision (as necessary) of O&M manuals.
- Control and monitoring of expressway services and equipment.
- Operation and monitoring of management and control systems.
- Communication with motorists regarding expressway conditions via signage and radio.
- Provision of a 24-hour towing service.
- Provision of security to prevent illegal entry onto the expressway.
- The collection of traffic data to assist with the proper maintenance and management of the expressway.
- The operation and maintenance of weigh stations to be placed at strategic locations.
- Timely response to traffic incidents and emergencies via an effective liaison with local authorities, police, hospitals, fire departments, etc.
- The collection and safekeeping of toll revenue should the expressway become a toll facility.
- Maintaining of records and reports of expressway operations.
- Continuous improvement of expressway operation via periodic reviews.
- Staff training.

As for the maintenance of the expressway, it is recommended that SLEA outsource all of its maintenance work to a private contractor under a performance-based contract scheme. As briefly described earlier, this is very cost-effective and is being promoted by organizations such as the World Bank and ADB as well as a way to promote greater private sector participation in the road sector. In addition, this type of contract will shift most of the risk of maintaining the expressway in good condition onto the contractor, leaving the SLEA to focus on management and operations (see Figure 11.7.1).

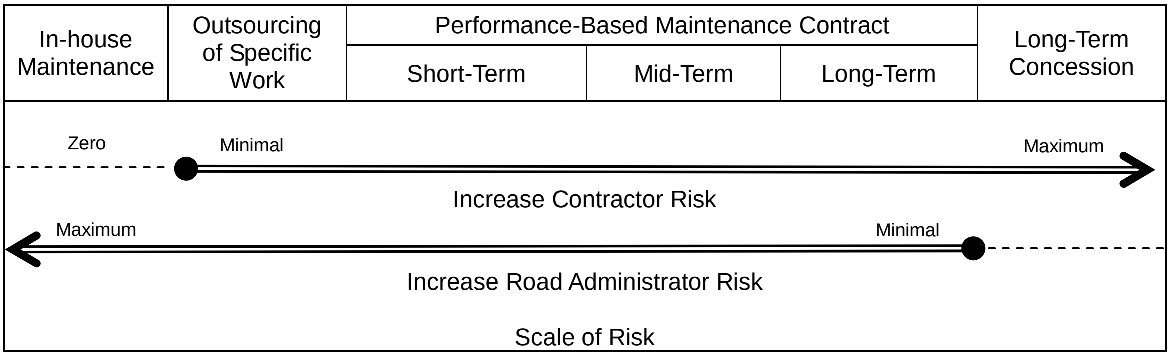

Fig.11.7.1 Relationship between the Scale of Risk & the Type of Contract

11.7.2. Maintenance Aspects

(1) General

The purpose of maintenance is to ensure the preservation of the assets of SLEA as well as the safety of those who use it and will include, but not be limited to, the facilities and work indicated in Table 11.7.1.

Table11.7.1 Basic Types of Facility Maintenance & Objectives

Type of Facility Maintenance	Facilities Concerned	Objective
Pavement Maintenance	• Surface and underlying layers of carriageway.	• To provide a safe roadway surface, preserve the capital investments of the government, maintain a satisfactory riding quality to road users.
Shoulder Maintenance	• Portion of road adjacent to carriageway used for the accommodation of stopped vehicles, emergency use, and lateral support of base and surface courses	• To ensure a smooth transition between the carriageway and shoulder for safety reasons.
Roadside Maintenance	• Areas between the outside edges of shoulders and right-of-way boundaries.	• Removal of trash to provide a safe and attractive right-of-way. • Vegetation management carried out in an environmentally sensitive manner to ensure a safe and attractive right-of-way.
	• Drainage facilities (e.g., ditches, gutters, side drains, outlets, irrigation ditches)	• To ensure normal runoff is collected and removed form roadway and/or right-of-way.
	• Guardrails & fencing	• To replace or repair when unable to function as intended to ensure driver safety.
	• Rest & parking areas (includes restrooms, restaurants, gas stands, etc.)	• To ensure comfortable and convenient places for road users to rest.
Bridge Maintenance	• Any structure erected over a depression or obstruction, such as water, a highway, or railway, usually 6m or more in length.	• To provide safe reliable passage over a depression or obstruction by preserving bridges as close as possible to their original condition.
Traffic Operation Maintenance	• Highway signs (e.g., regulatory, warning signs)	• To ensure that any missing or damaged (warning / regulatory) signs are replaced or repaired as soon as practical in order to ensure driver safety.

The timing and work of the types of maintenance that would be carried out for the above-mentioned facilities are as shown in Table 11.7.2.

Table11.7.2 Types of Maintenance

Type of Maintenance	Description	Timeframe
Routine Maintenance	Consists of work such as grass cutting, drainage clearing, road sweeping, culvert maintenance, road sign maintenance, pot-hole repair, patching, edge repair, and crack sealing.	Required on a daily basis or at frequent intervals during a one-year period.
Periodic Maintenance	Resealing (surface dressing, slurry dealing, etc.), road surface marking	Required at intervals of several years.
Urgent Maintenance	Consists of work such as the removal of debris and other obstacles, the placing of warning signs and diversion works, etc.	Executed as needed to deal with emergencies and problems requiring immediate attention.

It is important that the above work be carried out in the most cost effective and efficient manner possible. As mentioned previously, a performance-based maintenance (PBM) contract seems to be the most suitable method for achieving this and is described below in detail.

(2) Performance-based Maintenance

In traditional maintenance work contracts, the contractor is responsible for executing work that is usually defined by the road administrator or employer, with the contractor being paid on the basis of unit prices for different work items. This is known as a contract based on “inputs” to works and is also the practice commonly used here in Sri Lanka. The problem with this type of contract is that it gives the wrong incentive to the contractor, which is to carry out the maximum amount of work in order to maximize profits. This is inefficient and it has been observed that this type of agreement can result in large amounts of money being spent with insufficient and/or unsatisfactory results.

On the other hand, in the case of PBM contracts, contractors compete with each other by proposing a fixed monthly lump-sum fee per km of road. Note that contractors are not paid directly for inputs or physical work but for outputs, which means maintaining a road to pre-defined standards as defined in the contract with the road administrator. A fundamental feature of the PBM contract is that the contractor is responsible for designing and carrying out actions he believes necessary to comply with service quality levels stated in the contract. The service quality levels are defined from a road user’s perspective and may include factors such as average travel speeds, riding comfort, safety features, etc. If the service quality is not achieved in any given month, the contractor can be fined or payment suspended. Under a PBM contract, there is therefore a strong financial incentive for the contractor to be efficient. That is, in order to maximize profits, the contractor must reduce his activities to the smallest possible volume of intelligently designed interventions, which nevertheless ensure that pre-defined service levels are achieved and maintained over time.[19] The benefits that can be expected from the implementation of a PBM contract are as follows[20]:

- Increased likelihood of actually satisfying required needs

[19] *Performance-Based Management and Maintenance of Roads (Output-based Service Contract), World Bank, Washington D.C., February 2002.*
[20] *Office of Acquisition Management, US Department of Commerce, Seven Steps to Performance-Based Services Acquisition, Benchmark Version, January 2002.*

- Focus on intended results, not process
- Better value and enhanced performance
- Less performance risk
- No detailed specification or process description needed
- Contractor flexibility in proposing solution
- Better competition: not just contractors but solutions
- Contractor buy-in and shared interests
- Shared incentives permit innovation and cost effectiveness
- Surveillance: less frequent but more meaningful
- Variety of solutions to choose from

(3) Measures for Performance-based Maintenance

As the preceding indicates, the measures for evaluating the proper execution of a PBM contract are crucial if it is to be successful. Sample operational performance measures (or contract standards and response times) for expressway pavement maintenance are as shown Table 11.7.3.

Table 11.7.3 Sample of Operational Performance Measures for Expressway Pavement Maintenance[21]

Type of Damage	Contract Standard	Response Time
Potholes	No more than 3 potholes within any continuous 10km section with a diameter greater than 70mm.	48 hours
Surface Damage	Surplus and/or loose chip does not create potential traffic hazard or create public complaints.	2 hours
	Surface has a texture comparable to surrounding surface texture	1 month
Cracking	Total area of cracking within any continuous km is less than $10m^2$	1 month
Heaves and Shoves	No heaves or shoves greater than 20mm	2 weeks
Other Pavement Failure	No pavement failure greater than $5m^2$	2 weeks
Depressions & Rutting	No ponding greater than 30mm in depth at any location.	
Edge Break	No more than 2m of edgebreak with any continuous km less than 0.5m wide.	1 month
	No more than 2m of edgebreak with any continuous km greater than 0.5m wide.	2 weeks

Note that the contract standard is the minimum level of service a contractor must adhere to in order to be in compliance with his PBM contract at any point in time. The objective of the contract standard is to ensure a measurable minimum level of service that is expected by road users on a daily basis. Response time is the time in which a contractor must rectify any particular defect. Of course, if a particular defect poses a potential safety hazard, then said defect must be remedied as soon as possible and no later than 24 hours.

(4) Maintenance Manuals

To carry out the demanding requirements of a PBM contract, it is important that excellent maintenance manuals be available. At present, there are no up-to-date pocket-sized manuals for use in the field to carry out the inspection, evaluation, and execution of maintenance. The current manual in use was prepared in 1989 and is a large and

[21] *Based on Table 5.1 in the State Highway Maintenance Contract Proforma Manual SM032 of Transit New Zealand:* *http://www.transit.govt.nz/technical_information/content_files/Amendment68_PDFFile.PDF*

unwieldy A4-sized document. It is therefore recommended that three new user-friendly manuals for field use be prepared, which would be divided up into those for inspection, evaluation, and execution (see Fig. 11.7.2). From the viewpoint of efficiency, it is important that the contents of the manuals should contain graphics and be easy to understand and apply so that technicians can carry out the work required.

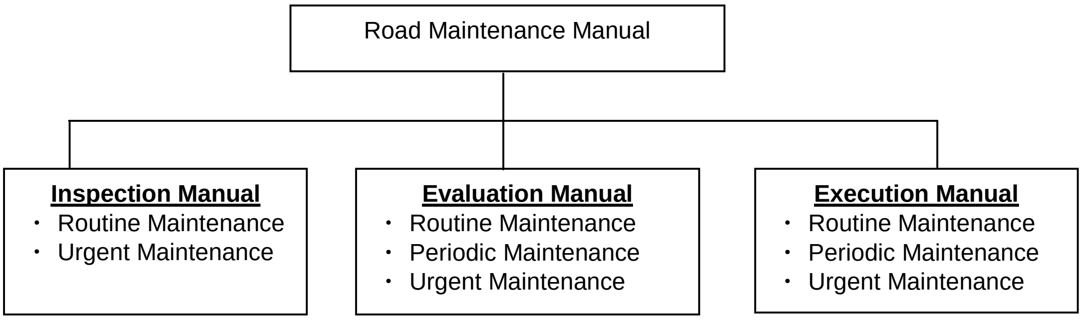

Fig.11.7.2 Recommended Maintenance Manuals for Road Maintenance

(5) Considerations Regarding the Implementation of PBM

In order for the implementation of PBM contracts to be successful, the following should be kept in mind:

- A PBM contract is mainly a management contract and traditional construction and maintenance contractors may have difficulty in its execution. Therefore, consultants with experience in managing contractors may be better suited. In this case, the consultant could sub-contract out to a contractor. Example of criteria for awarding a PBM contract (for a US Federal Highway Administration road project) is as shown in the table below.

Table11.7.4 Evaluation Criteria & Weights for Awarding PBM Contract in Washington, D.C.

Evaluation Criteria	Weight of Criteria
Technical	20%
Staffing/Quality Control/Quality Assurance/ Management	15%
Past Performance	15%
Cost*	50%

*: The extent to which proposed costs reflect potential costs to be incurred over the term of the contract.

- The longer a PBM contract is the better the results should be. That is, a long-term contract permits a contractor to try new technologies and optimize resources. Consideration should then be given to having a PBM contract at least include a single periodic maintenance application in order to maximize benefits. Note that in general, PBM contracts are between 4 and 10 years in duration.
- On the other hand, where there is little or no experience with PBM contracts, a gradual approach using short-term contracts is suggested initially. That is, begin with routine maintenance and allow for a transition period. Performance indicators should also be easy to understand and monitor.
- Establish performance measures before starting the bidding process.
- Ensure that the description of performance measures and their targets are clear.

- The road administrator should properly monitor contractor performance and apply penalties for non-compliance in order to ensure satisfactory results.
- The road administrator and contractor should closely cooperate withy each other.

11.7.3. Cost Considerations

In order for SLEA to function properly, it is important that costs be taken into account accurately. Here, O&M costs for the OCH portion of SLEA are considered. Note that it is assumed that the OCH will consist at minimum of an expressway that will extend from Rt. A4 to Rt. A1, as this is the most economically viable OCH option. Given that there are no existing expressways in Sri Lanka, unit costs for the routine maintenance of the road and bridges of the OCH (see Table 11.7.5) are calculated by referring to information in RDA's Highway Schedule of Rates for Western Province. As for periodic maintenance, it is estimated that it would cost approximately Rs. 265 million in 2007 prices and consist of an overlay (including the carriageway and shoulder). Finally, operation costs are calculated using actual operational costs incurred by the maintenance unit of the Baseline Road (see Table 11.7.6).

Table11.7.5 Annual Routine Maintenance Costs for 4-Lane OCH from Rt. A1 to Rt. A4 (2007 Prices)

Work Item	Annual Unit Cost in Rs.1)	Unit of Measurement2)	Quantity	Annual Routine Maint. Cost (Rs.)
1. Spot Patching	245,133	ln-km	79.8	19,561,613
2. Clearing of Vegetation	66,780	Km	23.4	1,562,652
3. Repair of Guardrail	453,600	Km	23.4	10,614,240
4. Maint. of Signals	147,368	light	4	589,472
5. Maint. of Street Lighting	4,524	Light	273	1,235,052
6. Shoulder Maintenance	179,285	Km	23.4	4,195,269
7. Road Furniture	65,744	Km	23.4	1,538,410
8. Edge Repair	146,758	Km	23.4	3,434,137
9. Maint. of Steel Bridges	992	m^2	12,400	12,300,800
10. Maint. of Concrete Bridges	974	m^2	43,600	42,466,400
Total				97,498,045

1) Unit costs are based on RDA's Highway Schedule of Rates for 2007 for Class A roads.
2) All data measured in km and ln-km includes ramp length.

Table11.7.6 Annual Operational Costs for OCH from Rt. A1 to Rt. A4

Work Item	Annual Unit Cost Rs.1)	Unit Measurement2)	Quantity	Annual Operation Cost In Rs. 3)
1. Overhead incl. Office Staff Salaries & rent	418,950	Km	23.4	10,403,430
2. Maint. of Vehicles incl. Salary of Drivers	219,450	Km	23.4	5,135,130
3. Electricity Bill for Street Lights	44,460	Light	273	12,137,580
4. Electricity Bill for Traffic Signals	194,560	Signal Set	4	778,240
Total				28,454,380

1) Unit costs are estimated by referring to costs for Baseline Road O&M unit.
2) All data measured in km includes ramp length.
3) Tolling is not considered so the cost of operating tolling facilities and collecting toll revenue are not considered.

As the above tables indicate, the annual O&M cost for OCH (from Rt. A1 to Rt. A4) would be approximately Rs.126 million at current prices in years with no periodic maintenance. In the case of years with periodic maintenance, total annual O&M cost would be about Rs.382 million.

11.7.4. Potential of Utilizing Existing Revenue Sources & Level of Sufficiency

(1) Existing Revenue Sources & Level of Sufficiency

An examination of past funding has been shown to be insufficient to meet the needs of road maintenance (see Section 11.3.2), and it has been estimated that only about 26% of the money needed to properly maintain the entire road network of Sri Lanka is being provided.[22]

As for the concept of a road fund, which has only been recently implemented in Sri Lanka (see Section 11.3.3), there is no guarantee that this will be applied to expressways. In fact, there seems to be an understanding within the GOSL, as well as by the donors, that the expressways should be able to pay for their own operation and maintenance costs without relaying on the road fund. Finally, even if the current interim road fund is a success, it is highly questionable that it would be sufficient to meet all of the all needs of road maintenance in Sri Lanka given the severe financial climate of the country.

(2) Tolling

Given the above, it is recommended that a tolling scheme be introduced to cover the O&M costs of SLEA. It is also suggested that this be done from the beginning of operation, as it would be extremely difficult from a social viewpoint to introduce tolls later if users are accustomed to using facilities free of charge. Although it was originally recommended in the feasibility studies for the OCH and SH that they should be non-toll roads in order to achieve social and developmental goals, it is imperative that there are sufficient funds to maintain them in adequate condition as expressways. As tolling seems to be the only realistic option for realizing this, it should done with a view to maximizing SLEA performance with the goal of minimizing tolls so there is no significant impact on the goals of the OCH and SH. Below, a brief examination of whether or not it is realistic to expect that the tolling revenue of the OCH (from Rt. A1 to Rt. A4) could cover its O&M cost is considered.

First, it is should be determined what toll level is acceptable. From an international perspective, the average toll for passenger cars ranges from US$0.03 to $0.08 per km, and although there are variations between countries it is not dependent on national income levels.[23] Note that goods vehicles pay at least twice this rate, which is equivalent to about 3 to 9 Rs. per km based on current exchange rates (July 2007). On the other hand, in an optimization study for the CKE, a base toll of Rs.7.8 per km for light vehicles and Rs.26 per km for trucks is applied.[24] In the case of the SH, a much lower base toll of Rs.1 per km for light vehicles and Rs.3 per km for heavy vehicles is considered.[25] Based on the preceding, a relatively low toll equal to Rs.2 per km for light vehicles and Rs.4 per km for trucks and buses is examined for the OCH.

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[22] Based on data contained in materials for the ADB Workshop on the Road Sector Master Plan (TA No. 4315-SRI) held at the Villa Ocean View in Wadduwa on 19th February 2005.

[23] http://www.worldbank.org/transport/roads/toll_rds.htm

[24] Colombo Katunayake Expressway Project, Project Optimization Study, Final Transport Planning Report, SMEC International Pty Ltd., March 2004.

[25] Economic Analysis, Consultancy Service for the Preliminary Engineering Design and Detailed Engineering Design for the Southern Transport Development Project JBIC Funded Section from Kottawa to Kurundugahahetekma, October 2000.

Second, depending on the tolling scheme that is to be applied, there will be differences in revenue flow. Here, it is assumed that a variable rate would be adopted, as the GOSL has stated its wish to introduce a closed tolling system. Assuming that road users on average use at least half of the OCH (or 10km), light vehicles would on average spend Rs. 20 and heavy vehicles Rs.40 when using the OCH.

Third, the traffic volume using the OCH is the final factor that is needed to calculate the revenue that could be expected from tolling. Using the STRADA traffic model, it is assumed that a minimum of 50,000 vpd would use the OCH. Based on this and the preceding, the toll revenue that could be collected in the sample year of 2020 is calculated (see Table 11.7.7).

Table11.7.7 Indicative Forecast of Toll Revenue for 2020 for OCH (Rt. A1 to Rt. A4)

Type of Vehicle	Share of Total Traffic (Approx.)	Daily No. of Vehicles	Toll Rate (Rs./km)	Average Dist. Traveled on OCH (km)	Annual Expansion Factor*	Annual Toll Revenue (Rs. million)
Light Vehicle	70%	35,000	2	10	300	210.0
Trucks & Buses	30%	15,000	4	10	300	180.0
Total						390.0

*Norte that this expansion factor has been used in other studies in Sri Lanka as well.

As the above table indicates, the annual toll revenue that could be expected in 2020 would be about Rs.390.0 million and is much large than annual maintenance costs. Given this, it is possible to have a minimal toll that would be affordable to all and still keep the expressway in good condition.

11.7.5.Recommendations for Realizing O&M Scheme for OCH

The above analysis provides important hints on how the OCH could be operated and maintained. However, it is recommended that the following be considered to ensure that the investment in the OCH is optimized:

- Timely enactment of the Expressway Authority Act.
- Hiring of a consultant to execute the detailed design for tolling facilities.
- Determination of the location of maintenance facilities for the OCH, together with facilities for police and emergency services, taking into account the facility location plans for the SH.
- Establishment of a performance-based contract that would be used for the execution of maintenance for the OCH.
- Determination in a timely manner by the GOSL whether tolls are to be set taking into account O&M costs only or both O&M and construction costs.
- Determination on how tolls are to be collected and utilized by the Expressway Authority.

CHAPTER 12

TENDER DOCUMENT PREPARATION

CHAPTER 12 TENDER DOCUMENTS PREPARATION

12.1. General

Bidding documents for tender of OCH Northern Section-1 have been prepared referring to following documents:

1) OCH Southern Section Tender documents (September 2007)
2) Sample Bidding Documents, JBIC, November 1999 (Amended in March 2005)
3) Sample Prequalification Documents, JBIC, November 1999 (Amended in April 2005)
4) Handbook for Procurement under JBIC ODA Loans, JBIC, January 2005
5) Check List for One Sided Contracts, JBIC, December 2006
6) Evaluation Guide for Prequalification and Bidding, JBIC, June 2000 (Amended in January 2007)
7) Various comments on OCH Northern Section-1 tender documents from RDA (OCH) and JICA.

Regarding General Conditions of Contract, OCH's tender documents are based on FIDIC's 4th edition (1987), reprinted in 1988 and 1992 with minor amendments because FIDIC's new version of 1999 has not been authorized in JBIC's sample documents. However, the FIDIC 1999 is referred to finalise the OCH Northern Section-1 tender documents.

12.2. Bidding Method

For Northern Section-1, Two-envelope method was recommended and the Draft Tender documents were prepared based on the method.

The evaluation of Bidders' technical proposals is important because due to lack of construction experience of the contractors, some road works fall much behind the construction schedule in Sri Lanka.

12.3. STEP Loan

For Northern Section-1, Special Terms for Economic Partnership (STEP) of JBIC loan will be applied to raise the visibility of Japanese ODA loan among citizens in both Sri Lanka and Japan through the best use of advanced technologies and know-how of Japanese firms.

In the STEP loan, not less than 30 % of the total amount of contract (excluding consulting services) must be accounted for by goods from Japan and services provided by Japanese firms.

12.4. Alternative Bid

In Northern Section-1, an alternative bid is allowed for the following works:

1) Soft ground treatment,
2) Bridge structure (including selection of steel or concrete bridges).

If any alternative bid is proposed by the Contractor and accepted by the Employer, the Contractor shall take full responsibility on his design of his alternative design parts.

12.5. Detailed Designs Required

Generally the tender drawings are Detailed designs, however some of the bridges are only Basic designs (including all steel bridges). The Contractor shall complete the detailed design of those bridges.

12.6. Amount of Liquidated Damages

FIDIC General Conditions, Sub-Clause 47.1 says:
"If the Contractor fails to comply with the Time for Completion ……, the Contractor shall pay to the Employer the relevant sum ….. as liquidated damages for such default and not as a penalty ….."

If Liquidated damages are included, Bonus clause also should be considered to make a fair contract. In fact, early completion rarely took place in Sri Lanka highway projects. Therefore, Liquidated damages only included in the Particular conditions.

Next step is how to value the damages. They should have good reasons. If toll roads, the revenue income loss is apparent but not for OCH. At least RDA's administration costs including the consulting services should be covered. There are three major components considered for the valuation.

1) Project benefit loss calculated from Economic analysis
2) Supervision Consultant's cost per day
3) Total construction cost divided by total days

Item 3) is simple but has no appropriate reasoning, so discarded.

1) Project benefit for A-4 to A-1 (Southern+Northern Sections) is calculated in Detailed Design, Main Text (July 2005). In year 2012 (completion target year):

Total benefit for (N+S):	Rs.3,209 million/year
Benefit for Southern only:	Rs.1,316 million/year
Benefit for Northern-1 only:	Rs.1,893 million/year
Rs.1,893 million/year/365 days =	Rs.5,186,000/day.
Assumed damages as 50 %=	Rs.2,593,000/day

2) Consulting service cost for Northern Section-1 is estimated as Rs. 700 million.

Thus,

Rs.700 million/3 years/365 days = Rs.639,000/day.

Total estimated Liquidated damages (1)+2))= Rs.3,232,000/day, say Rs.3 million/day.
The amount accounts for approx. 0.03 %/day of the total construction cost and the maximum amount is fixed as around 10 % (approx. Rs. 900 million).

CHAPTER 13

IMPLEMENTATION PROGRAM

CHAPTER 13 IMPLEMENTATION PROGRAM

13.1. General

The implementation program for the OCH project had been considered as consisting of three contract packages in the Basic Design of 2001. Package 1 was to consist of an 8.75km length of section from the Kelawarapitiya land reclamation area to Rt. A1 via CKE and Rt. A3. Package 2 was to consist of an 8.25km length of section from Rt. A1 to Rt. AB10 and include the construction of the Kelani Bridge. Finally, Package 3 was to consist of a 12.10km length of section from Rt. AB10 to Rt. A4 and link up with the STDP. Detailed Design for Southern Section as Package 3 was completed in 2005 and now, contractor procurement is on going. On the other hand, Northern Section 2 as Package 1 was omitted from the Scope of this Study.

In this Study, the implementation program for the OCH Northern Section 1 as Package 2 has been re-considered as follows.

13.2. Construction Phasing

In the Study, the detailed design work of the OCH was conducted assuming that in the initial stages there would be a dual carriageway with four lanes that would ultimately be widened to six lanes. In this chapter, only the initial stage is considered.

13.3. Implementation Schedule

The Project implementation schedule should be consistent with the technical realities of a project and should ensure the proper sequencing of activities, taking into account institutional capabilities and the availability of resources for construction. The suggested implementation schedule for the Northern Section 1 of OCH is shown in the bar chart in **Fig. 13.3.1**. Note that the schedule is composed of a number of events and includes the following:

■ Procurement of Supervision Consultant
Period for procurement of Supervision Consultant will be omitted because the Consultant for OCH Southern Section will supervise Northern Section 1 also.

■ Acquisition and Resettlement
The legal process for acquisition and resettlement should be undertaken in parallel with the detailed design. All land should be acquired and cleared prior to the award of the construction contracts.

■ Pre-construction
According to the JBIC ODA loan process, two and half months is required for the pre-qualification of contractors prior to the tender procedure.

■ Contractor Selection
Eight and half months may be required for the two-envelope method described in **Chapter 12**.

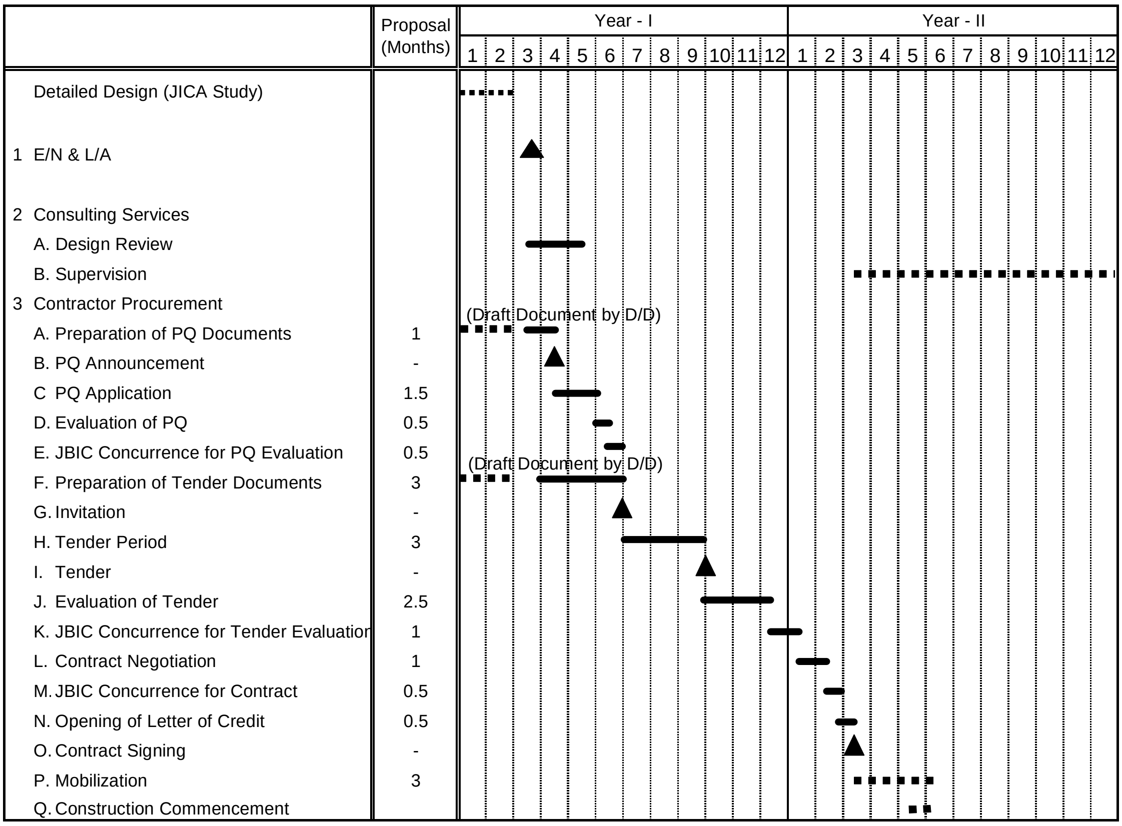

Fig 13.3.1 Implementation Schedule for OCH

13.4. Cost and Disbursement Schedule

The disbursement schedule for the Northern Section 1 of the OCH Project is given in **Fig. 13.4.1** and has been established in accordance with the implementation schedule described above. The disbursement schedule shall be based on the following:

1) Disbursement shall start after the signing of the contract and end with the final payment after the issuing of the Completion Certificate.

2) The figures given in **Fig. 13.4.1** indicate the percentage of the total contract amount.

3) The contract amount is disbursed in equivalent monthly amounts in accordance with the work and timing of construction, excluding the advance payment (which is 20% of the total contract price).

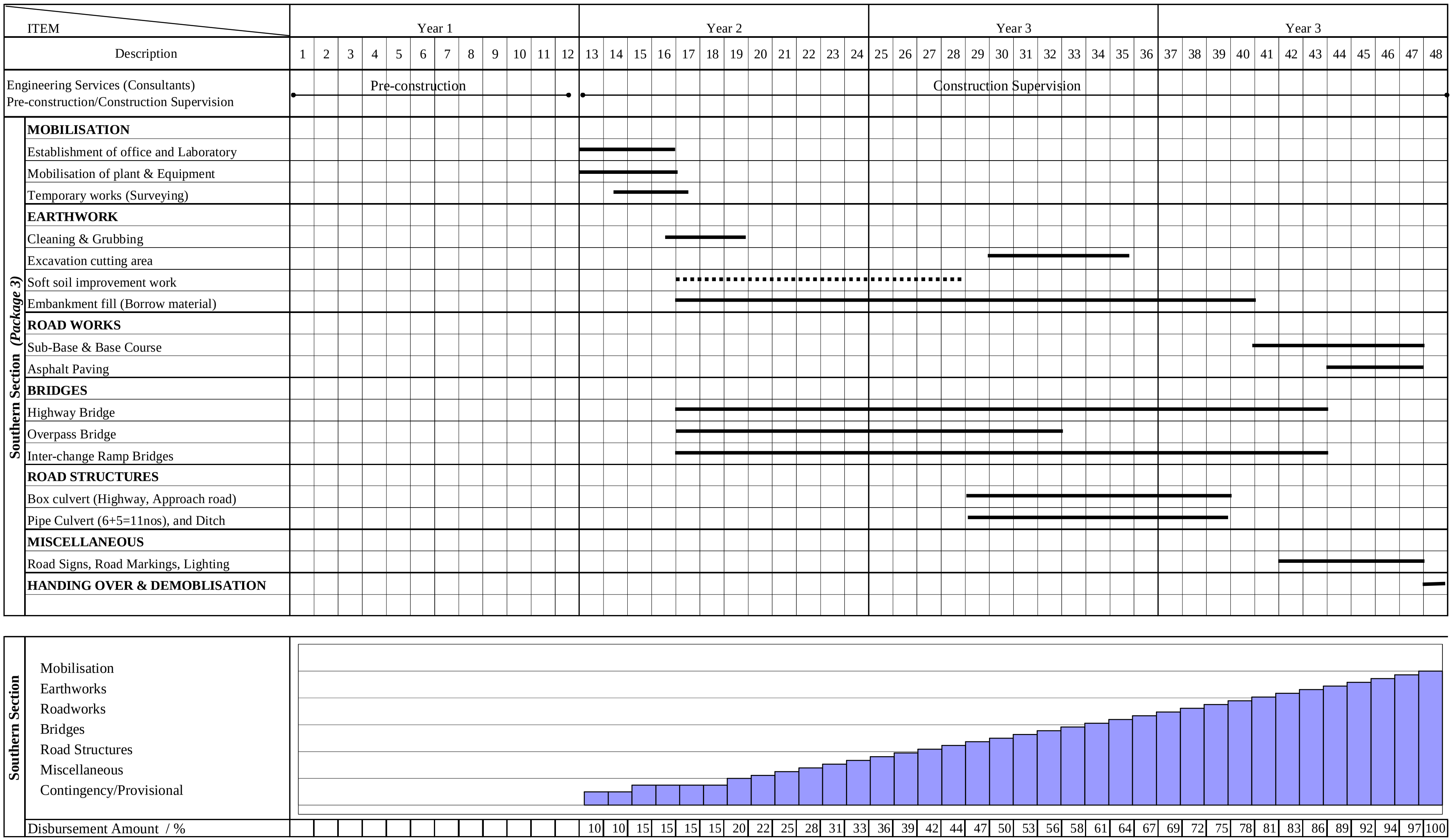

Fig. 13.4.1 Disbursement Schedule for OCH