Chapter 4 Field Investigation for Selected Sites

This chapter deals with topographic and bathymetric surveys, geotechnical investigation and river morphology for the selected site Mawa-Janjira to provide basic information to the subsequent preliminary designs.

4.1 TOPOGRAPHIC AND BATHYMETRIC SURVEYS

4.1.1 Methodology

The Study Area for the Topographic Survey was selected at Mawa – Janjira area based on the result of screening of alternatives. The study area covers the area for new bridge construction on the Padma River and associated regions. Topographic surveys and bathymetric surveys were carried out along the following location as shown in Figure 4.1.1. Details of the topographic surveys are reported in Appendix-3. Summary of the methodology for the topographic and bathymetric surveys is discussed in this section.

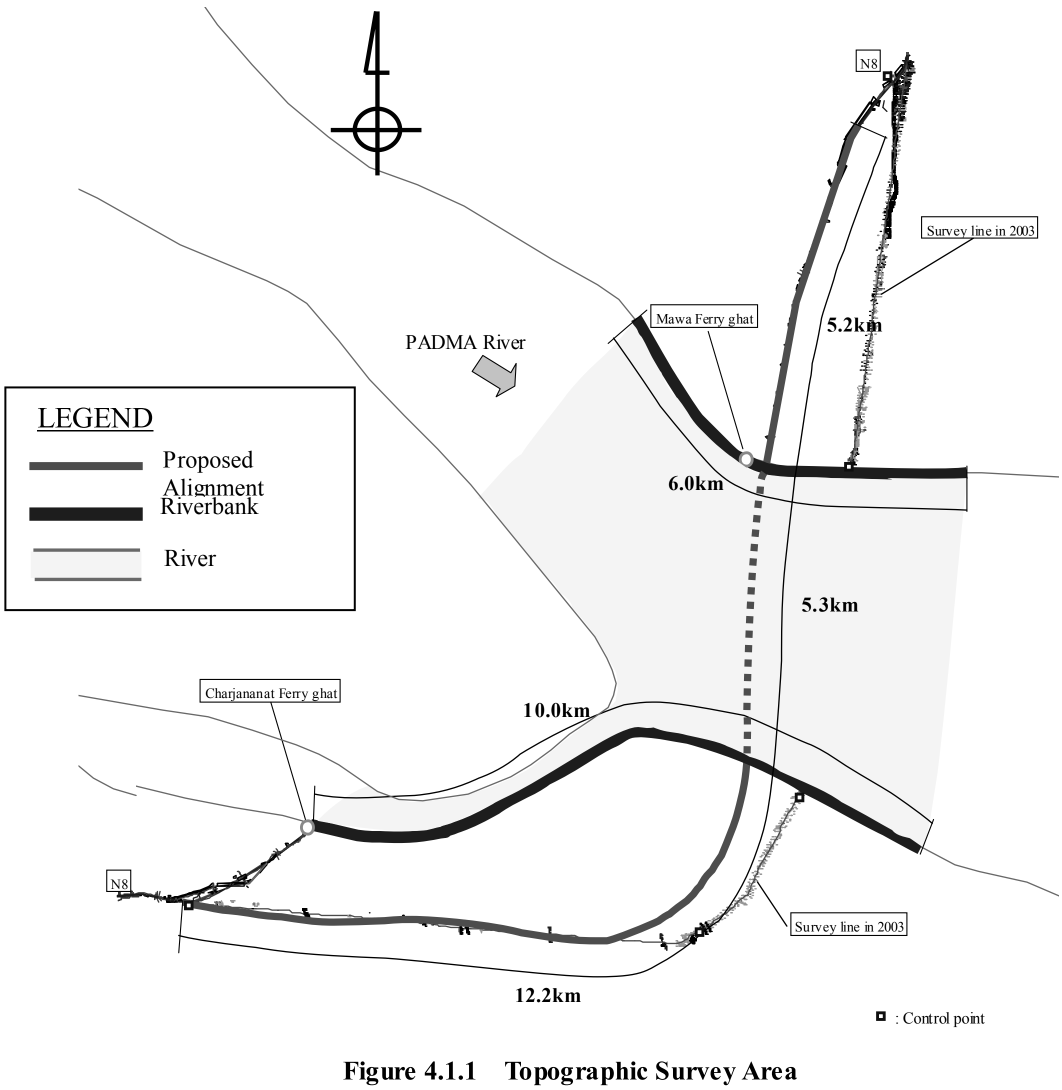


Figure 4.1.1 Topographic Survey Area

4.1.2 Scope of Works

The survey work comprises the checking of the control points installed in the 1st phase of the work, identification and demarcation of centerline in the field longitudinal profile survey, cross section measurement, bathymetric surveys, topographic features surveys, etc. by GPS and Total Station. The scope of works under the project covers the followings activities in the area of the proposed Padma Bridge:

The scope of works covers the followings activities in the study area.

- Check Survey of Installed Control Points
- Centerline Setting
- Connection and Longitudinal Profile Leveling
- Cross Section Survey
- Topographic Survey
- Bathymetric Survey for the Padma River Section

4.1.3 Coordinates System and Datum Level

The local coordinate system was adopted and was related to the coordinate system of the relative civil construction project and existing road.

The datum level also was related to the hydrological data, which is used to analyze the high water level for the bridges of this project.

The horizontal and vertical control data in the vicinity of the proposed bridge crossing was collected from Survey of Bangladesh (SOB).

(1) Horizontal Control:

The Local Grid coordinate system was adopted for both the Topographic and Hydrographical surveys. The Charts were prepared on Bangladesh Transverse Mercator (BTM). The parameter used for conversion of WGS-84 data into BTM coordinates are as follows:

Ellipsoid	:	Everest Modified Bangladesh
Projection	:	Transverse Mercator
Latitude Origin	:	0^{o} 00’ 00”.0
Central Meridian	:	90^{o} 00’ 00”.0
Scale factor	:	0.99960
False Easting	:	500,000.00m
False Northing	:	-2,000,000.00m
Semi-major axis	:	6,377,298.52400
Semi-minor axis	:	6,356,097.52000
Inverse Flattening 1/F	:	300.80170000
Rotation X	:	0
Rotation Y	:	0
Rotation Z	:	0
Translation X	:	-288.000m
Translation Y	:	-735.000m
Translation Z	:	-255.000m

Horizontal control reference to JMC1 established earlier in the project area has been used as the primary control point for survey of this project area.

(2) Vertical control (Datum level)

All elevation references are in meters and reduced to local PWD Datum. Control points and the tide table in PWD datum are reported in the Technical Report.

(3) Bathymetric Survey

To obtain river bed levels, the hydrographic survey has been carried out using a DGPS and Echosounder. Hydrographic survey software PDS1000 was used for online data collection and navigation. The position and its depth have been recorded simultaneously by GPS receiver and Echosounder fitted on the survey boat.

The survey boat ran parallel to the river crossing line at 100 m intervals. The survey covers 2000 m downstream of the proposed alignment and 4000 m upstream. Also at Branch River crossings it follows the same direction and intervals.

(4) Equipment used for Surveys

Digital surveys were carried out for both Topographic and Hydrographic survey works. The equipment used for this project for survey work is as follows:

- Topographic survey equipment
  - GPS Total Station RTK Trimble 5700 with TSI Controller (data Logger)
  - Electronic Total Station
    Model : Sokkia Power Set 2010
    Model : Sokkia Set 3CII
    Model : Sokkia Set 2CII
    Auto Level
    GPS Magellan Tracker
    Walkie-talkie Motorola GP328
- Hydrographic survey equipment
  - GPS Trimble 4000RS(DGPS)
  - Radio link (Pacific Crest)
  - Echosounder Odom Echotrak MIKE-II
- Survey & Processing Software
  - Land survey software
    SDR Mapping & Design
    Eaglepoint, Microsurvey
  - Geodetic software
    Trimble office geometric
  - Hydographic survey software
    PDS1000
    AutoCAD suite

4.1.4 Result of Topographic Survey

The quantities of major activities which have been done in the study area are shown in the following table.

Table 4.1.1 The Quantities of Activities for Study Area

Mawa – Janjira

	Check Survey of Installed Control Points (pts)	Longitudinal Profile Leveling (km)	Topographic Survey (ha)		Bathymetric Survey (km)
			Alignment	Riverbank	
Left Bank Side	2	5.2	117	150	
Right Bank Side	2	12.2	157	250	
In the River					71
Waterways		6 nos			
Total	4	17.4	274	400	71

All topographic data was processed by Eagle Point software, mapping and design have been done by AutoCAD. Bathymetric Survey Data Processed by PDS-1000 and output produced by AutoCAD software.

The Survey Drawing Scale:

Topographic Maps	:	Scale 1:10,000	
Plans and Longitudinal Profiles	:	Scale H = 1:4000	V = 1:400
Cross Sections	:	Scale H = 1:2000	V = 1:2000
Cross Sections for Riverbank and river	:	Scale H = 1:30000	V = 1:3000

4.2 GEOTECHNICAL INVESTIGATION

4.2.1 General

The geotechnical studies and investigations for the feasibility study of Padma Bridge were carried out for the following purposes:

1) Investigation for the foundations for Padma Bridge

To provide the design engineers with information on the engineering properties of the natural soils, which will permit the determination of the foundation type and foundation structure of Padma Bridge.

2) Investigation for foundation of approach roads and minor bridges

To provide the design engineers with information on the foundation of minor bridges over channels and embankments for approach roads.

3) Investigation for embankment materials

To provide the design engineers with information on the embankment materials for approach roads.

The geotechnical investigation consisted of core drilling, standard penetration tests (SPT), pressuremeter tests, laboratory tests of soil and laboratory tests of groundwater for bridges and approach roads at Mawa Janjira site. Details of the geotechnical investigation are reported in Appendix-4.

4.2.2 Outline of Geotechnical Investigation

The geotechnical investigation consisted of core drilling, standard penetration tests (SPT), lateral loading tests, undisturbed soil sampling, groundwater sampling, laboratory tests of soil and groundwater and embankment material tests at the bridge site and approach road route.

The locations of the boreholes and test pits are shown in Figure 4.2.1.

Methodology of the geotechnical investigation is dealt with in Appendix-4.

Laboratory tests for soil consisted of natural water content, specific gravity, unit weight, Atterberg limits, grain size analysis, triaxial compression tests, consolidation tests, mica content, pH, total sulfate content, soluble sulfate content and total chloride content. Laboratory tests for groundwater consist of pH, sulfate content and chloride content. Laboratory tests for embankment material consisted of natural water content, specific gravity, Atterberg limits, grain size analysis, compaction tests and CBR tests.

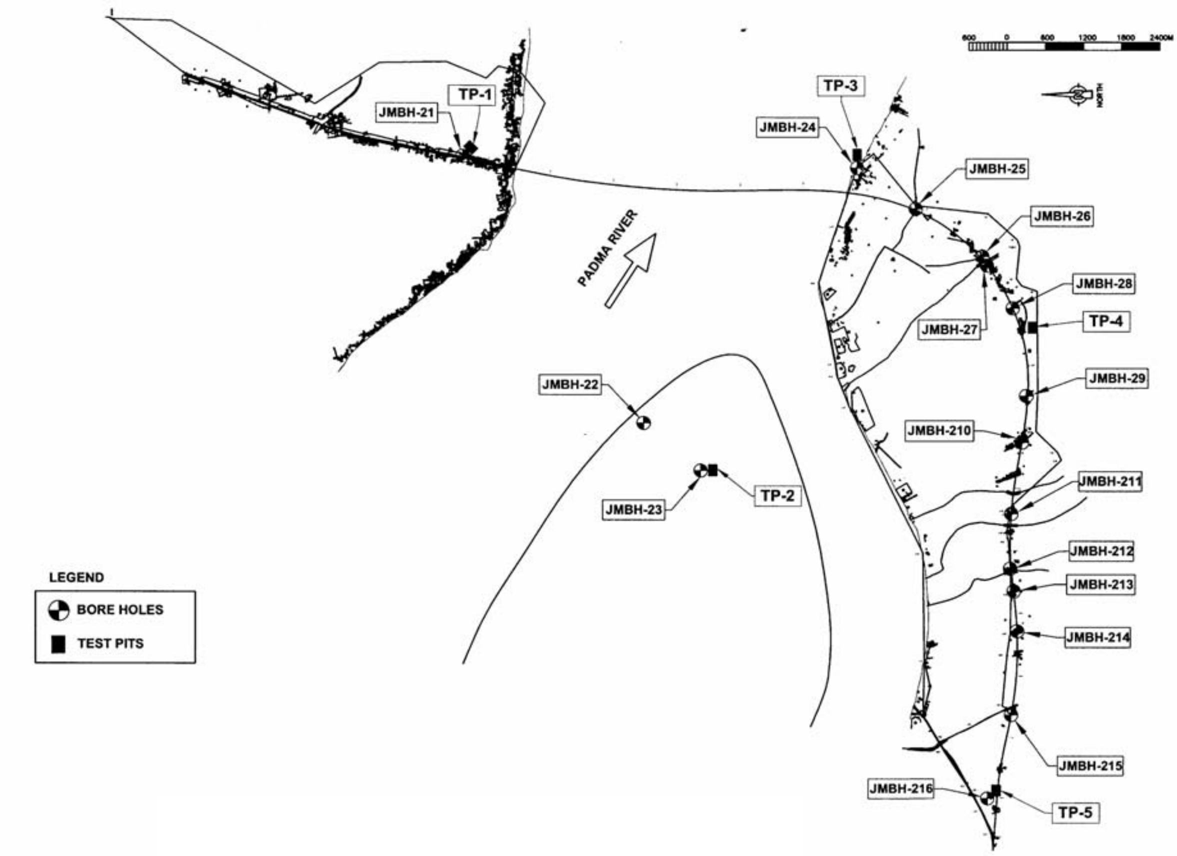


Figure 4.2.1 Location Map of Bore Holes & Test Pits

4.2.3 Foundation of Bridge Site

(1) Geological Profile

The following chronological and stratigraphical classifications were used for drawing the geological profile of the bridge site. Based on the results of grain size analysis, the following classifications are proposed:

Table 4.2.1 Subsoil Classification

Stratum	Description	Criteria
Unit-1a	CLAY or SILT with fine sand	Clay + Silt ≥ 50%
Unit-1b	very silty fine SAND	20% ≤ Clay + Silt < 50%
Unit-2	silty fine SAND	Clay + Silt < 20% and Medium Sand < 10%
Unit-3	slightly silty fine and medium SAND	Clay + Silt < 20% and Medium Sand ≥ 10%

Physical properties of each stratum are summarized in Table 4.2.2, and the geological profile of the bridge site is shown in Figure 4.2.2.

Table 4.2.2 Summary of Physical Properties by Strata at Bridge Site

			Unit - 1a	Unit - 1b	Unit - 2	Unit - 3
Natural Water Content Wn (%)		Min.	29.0	9.2	18.0	
		Max.	37.0	36.0	47.0	
		Ave.	32.8	18.6	24.0	23.0
Specific Gravity Gs		Min.	2.68	2.67	2.69	
		Max.	2.76	2.75	2.77	
		Ave.	2.72	2.71	2.72	2.69
Gradation	Clay + Silt (%)	Min.	63.0	20.0	8.0	
		Max.	100.0	45.0	19.0	
		Ave.	91.2	27.0	14.4	1.0
	Sand (%)	Min.	0.4	55.2	81.8	
		Max.	37.0	80.1	92.0	
		Ave.	8.9	71.8	85.6	99.0
	D_{50} (mm)	Min.	0.003	0.082	0.120	
		Max.	0.047	0.320	0.230	
		Ave.	0.017	0.164	0.170	0.230
	Uniformity Coefficient Uc	Min.	8.6	3.9	2.6	
		Max.	13.8	44.4	9.6	
		Ave.	11.2	16.2	4.7	2.2
Atterberg Limit	Liquid Limit LL (%)	Min.	27.0	-	-	-
		Max.	39.0	-	-	-
		Ave.	34.0	-	-	-
	Plastic Limit PL (%)	Min.	23.0	-	-	-
		Max.	25.0	-	-	-
		Ave.	24.0	-	-	-
	Plasticity Index I_P (%)	Min.	2.0	-	-	-
		Max.	15.0	-	-	-
		Ave.	10.0	-	-	-

(2) N-value

SPT was carried out at an interval of one meter for the full depth of each borehole. The measured N-values have a problem because blow energy has various losses on the way to the bed in the very deep position where the measurements are obtained imperfectly. A number of study results have been provided by Terzaghi - Peck (1948), Ikeda (1959), Thornburn (1963), Uto (1974) and others on the correction of such measured N-values. In this study, considering that the highest is the loss of blow energy caused on its transmission to the bottom of a bore, the loss-error of the measured N-value was corrected by the following Uto (1974) formula.

$N' = N$ $(L < 20m)$
$N' = (1.06 - 0.003 \times L) \times N$ $(L \geq 20m)$
where,
N': corrected N-value
N: measured N-value
L: length of drill rods (m)

The distribution of corrected N-values at the bridge site is shown in Figure 4.2.3. This figure shows that N-value data scatter considerably among the six boreholes. Therefore, the lower line of corrected N-value distribution is proposed for the design of the bridge substructure. In the practical design of the bridge structures in the feasibility study, it is recommended that the design line on which the N-value is limited to 50 as shown in Figure 4.2.3 is used on the basis of Standards adopted by Japan Road Association.

Reduction in overburden pressure at the scoured bed level will induce a reduction in the density of soil. The following equations are applied to obtain the reduced N-values after scour:

$$Ns = N' \times B_1$$

$$B_1 = 0.75 \times \sqrt{\frac{Z + 0.8 \times \sqrt{s \times z + z \times z}}{s + z}}$$

where,

B_1: scour reduction factor — s: scour depth (m)

Ns: N-value at depth after scour — z: depth below GL after scour (m)

N': corrected N-value at depth before scour

The depths of scour in the middle of the river section and adjacent to the riverbank are summarized below:

Location	Maximum scour depth
In the middle of the river section	-23.6 m PWD
Adjacent to the riverbank	-37.6 m PWD

The distributions of N-values after scour up to -23.6m PWD and -37.6 m PWD are shown in Figures 4.2.4 and 4.2.5 respectively. Further, the lower line and design line of N-value distribution after scour are proposed for the design of the bridge substructures in both figures. It is recommended that the bridge design engineer uses the design line for the design of the bridge structure after scour in the feasibility study.

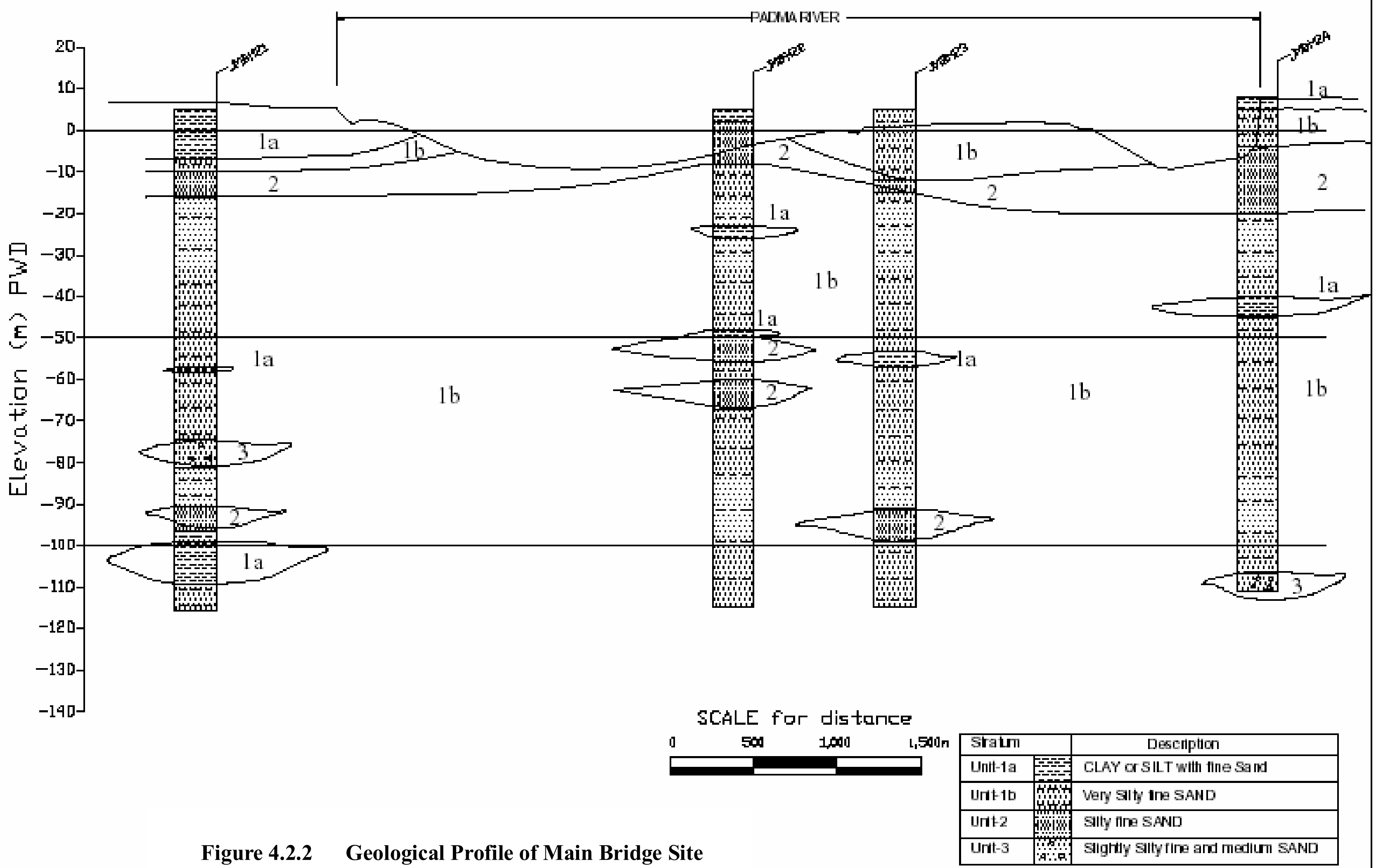


Figure 4.2.2 Geological Profile of Main Bridge Site

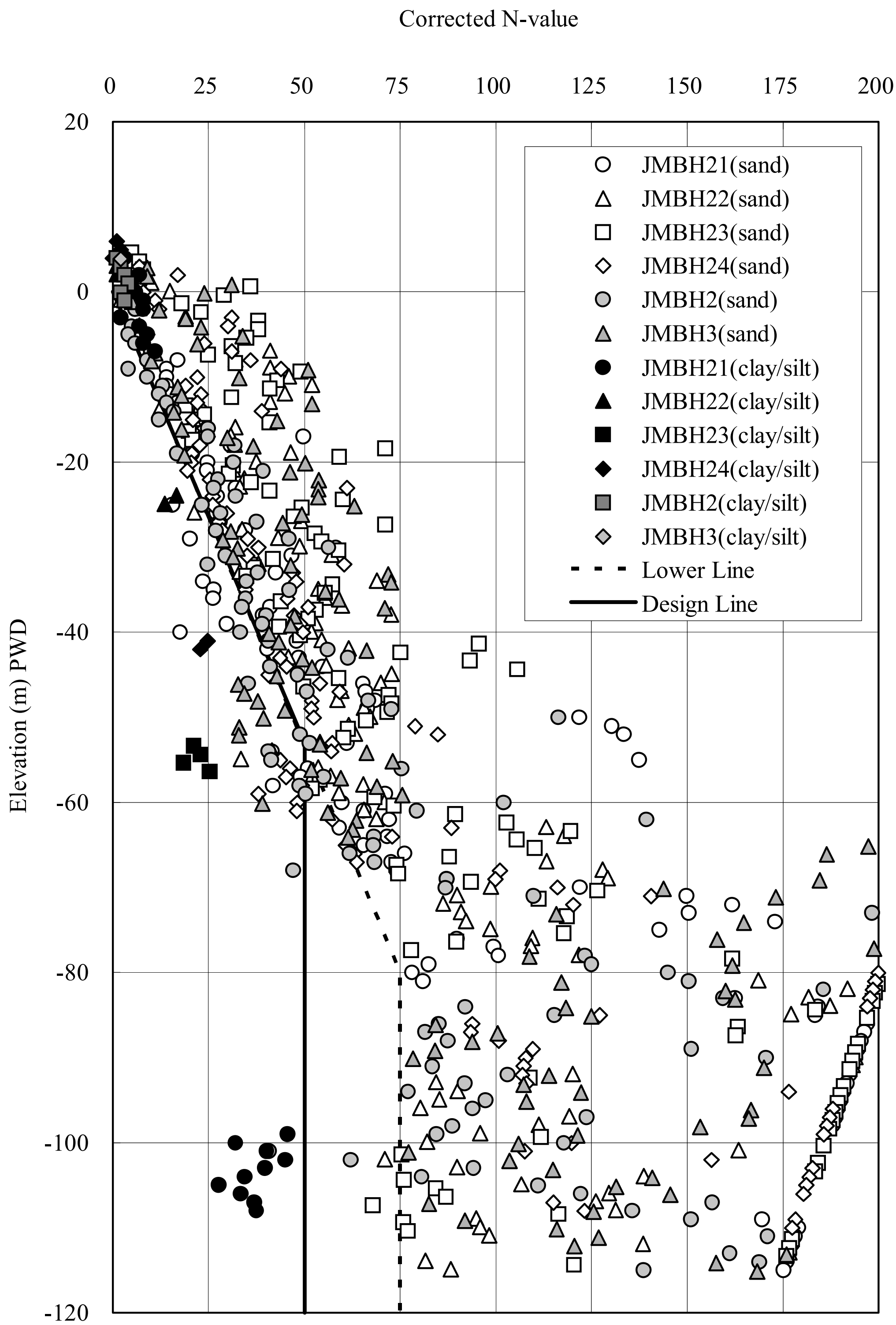


Figure 4.2.3 Distribution of Corrected N-values (Bridge Site)

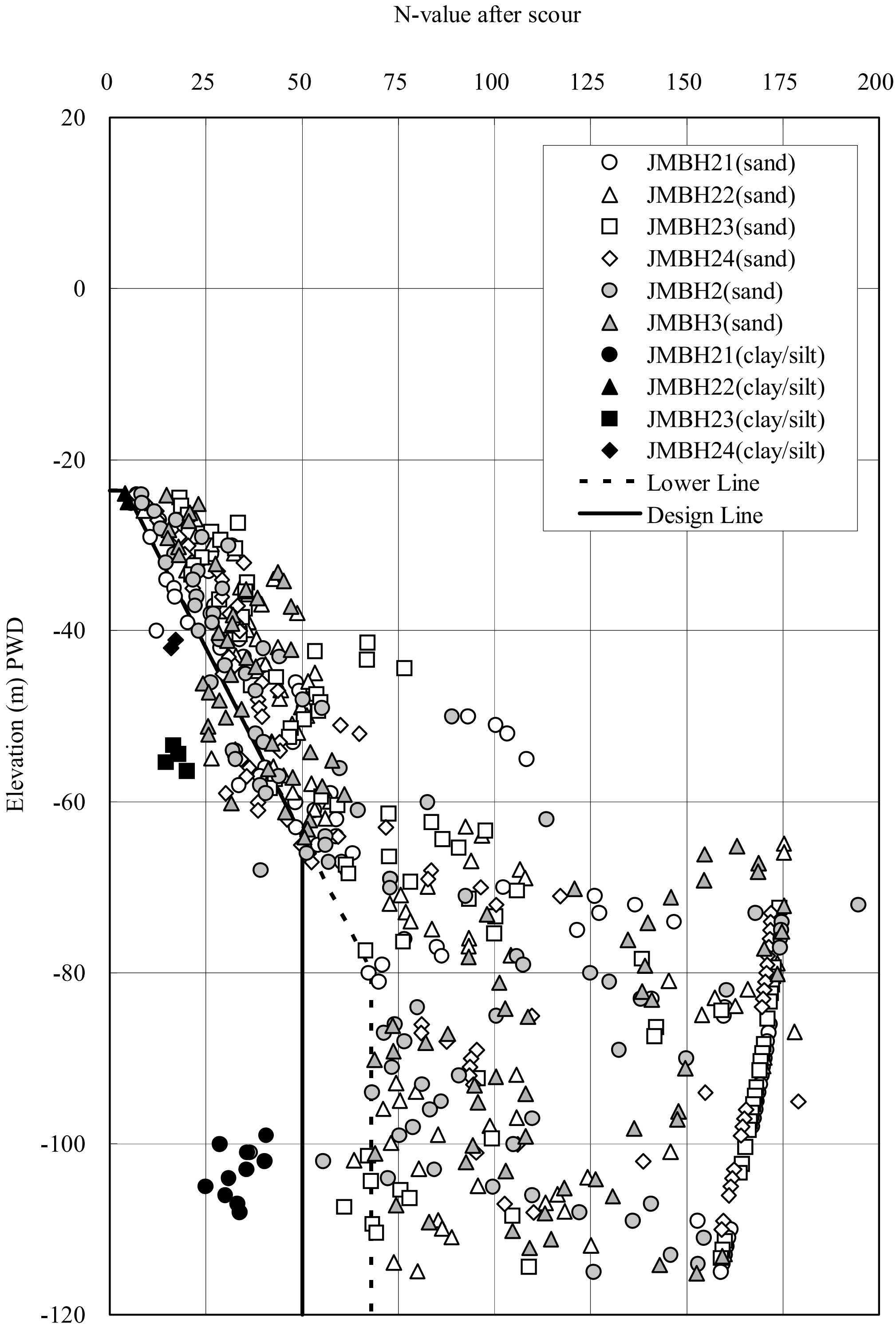


Figure 4.2.4 Distribution of N-values after Scour (River Scour to -23.6m)

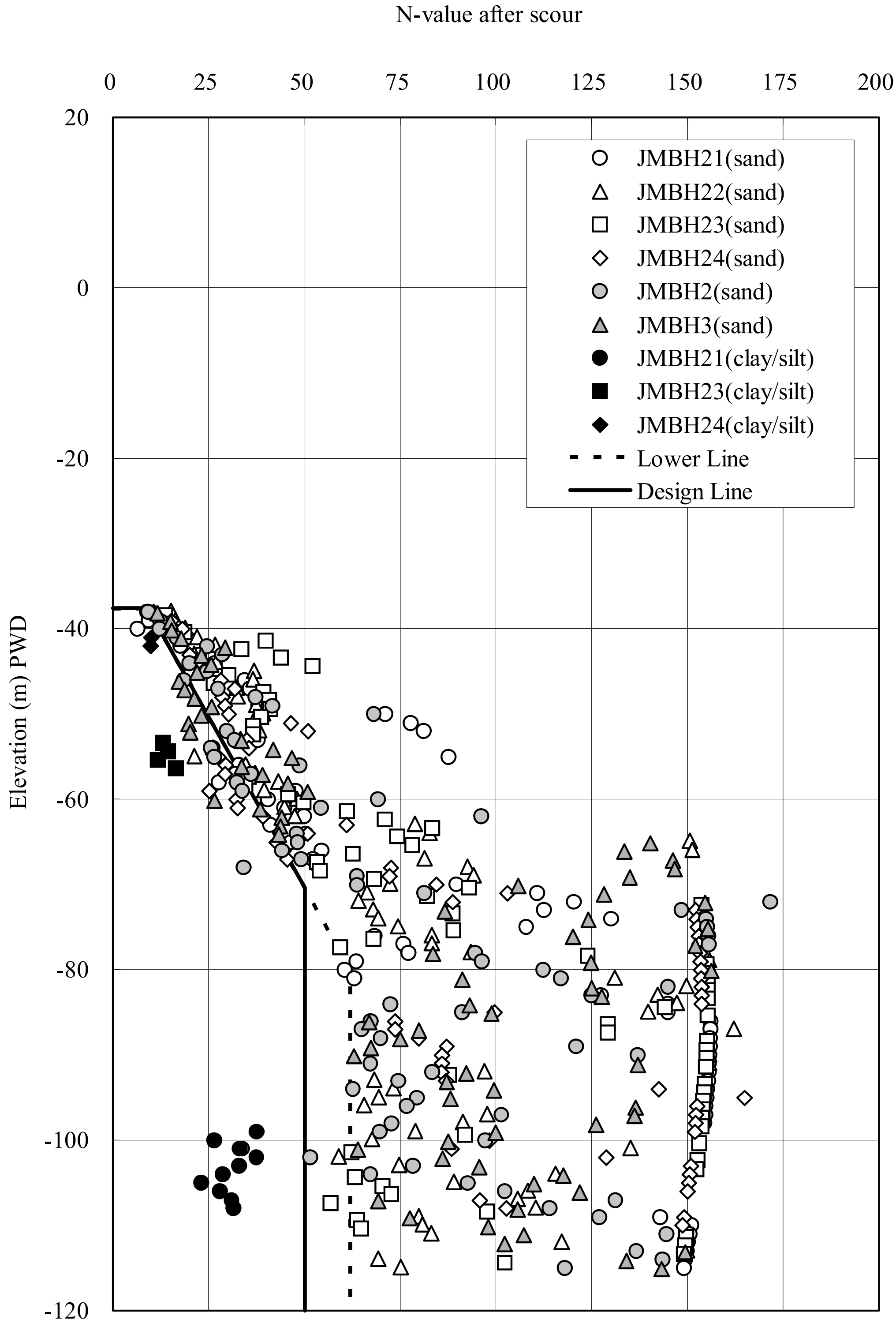


Figure 4.2.5 Distribution of N-values after Scour (River Scour to -37.6m)

(3) Lateral Loading Test

In this study, the lateral loading test was adopted. The correlation between corrected N-values and modulus of elasticity (Em) obtained from lateral loading test is shown in Figure 4.2.6. The formula of N to Em relation, Em = 2.89 N, was obtained for the design of the substructure.

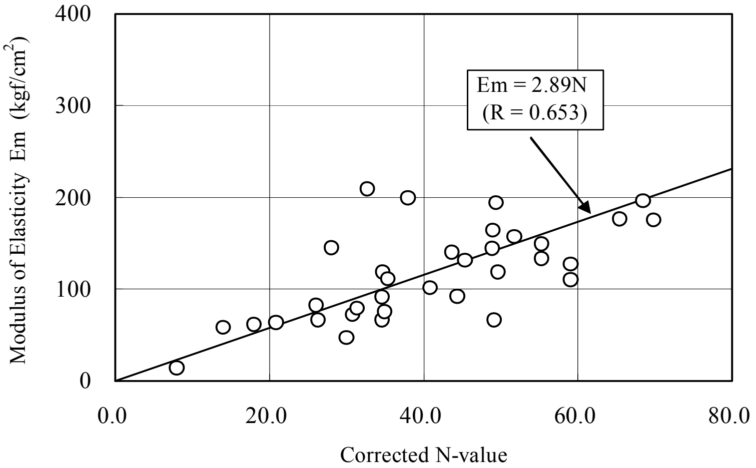


Figure 4.2.6 Correlation between N-values and Modulus of elasticity

(4) Laboratory Test Results

(a) Chemical Properties of Soil Samples

The value of pH ranges from 6.9 to 8.5 and the soil at the bridge site is nearly neutral. Contents of chloride, soluble sulfate and total sulfate range from 0.0272 to 0.0341 %, 0.0026 to 0.0140 % and 0.0309 to 0.1402 % respectively. In this regard, the soil cannot have an adverse effect on the concrete structures such as piles.

At the site of JMBH21, the mica content was found to be very high, 52.7 %. In Jamuna Bridge Project, some flow sliding was reported during excavation works for the guide bund foundations, of which, the soil contains some mica. From these failure accidents it can be surmised that adequate attention needs to be paid to excavation works for guide bund construction in the Padma Bridge Project.

(b) Chemical Properties of Groundwater

The value of pH ranges from 6.5 to 7.8 and the groundwater at the bridge site is nearly neutral. Chloride content and sulfate content range from 10.0 to 27.0 mg/L and from 2.0 to 38.0 mg/L respectively. It is evaluated that the groundwater cannot have an adverse effect on the concrete structures such as piles.

(5) Design Values

The proposed design values for the bridge substructures in case of scour depth of -23.6 m PWD and -37.6 m PWD are shown in Tables 4.2.3 and 4.2.4 respectively.

Table 4.2.3 Design Values for Bridge Substructures (Scour Depth of -23.6m)

Layer	Depth PWD (m)	Thickness (m)	N-value Mean	N-value Design	Density γt (kN/m^3)	Strength Parameters c (kN/m^2)	Strength Parameters ϕ (degree)	Modulus of Elasticity Em (kN/m^2)
(1)*1	1.425 ~ -23.6	25.025	-	-	-	-	-	-
(2)	-23.6 ~ -46.5	22.900	17	17	19.0	0	25	4,913
(3)	-46.5 ~ -64.1	17.600	40	40	19.5	0	33	11,560
(4)	-64.1 ~ -80.0	15.900	59	50	20.0	0	37	14,450
(5)	below -80.0		68	50	20.0	0	37	14,450

Table 4.2.4 Design Values for Bridge Substructures (Scour Depth of -37.6m)

Layer	Depth PWD (m)	Thickness (m)	N-value Mean	N-value Design	Density γt (kN/m^3)	Strength Parameters c (kN/m^2)	Strength Parameters ϕ (degree)	Modulus of Elasticity Em (kN/m^2)
(1)*1	1.425 ~ -37.6	39.025	-	-	-	-	-	-
(2)	-37.6 ~ -54.4	16.800	20	20	19.0	0	26	5,780
(3)	-54.4 ~ -70.4	16.000	40	40	19.5	0	33	11,560
(4)	-70.4 ~ -80.0	9.600	56	50	20.0	0	37	14,450
(5)	below -80.0		62	50	20.0	0	37	14,450

4.2.4 Foundation of Approach Road Route

(1) Geological Profile

Four classifications of Unit-1a, Unit-1b, Unit-2 and Unit-3 were used to determine each stratum based on the results of grain size analysis. Physical properties of each stratum are summarized in Table 4.2.5, and the geological profile is shown in Figure 4.2.7.

Table 4.2.5 Summary of Physical Properties of Each Stratum (Approach Road Route)

			Unit - 1a	Unit - 1b	Unit - 2	Unit - 3
Natural Water Content Wn (%)		Min.	22.4	21.1	19.5	-
		Max.	43.6	37.9	32.6	-
		Ave.	33.5	26.8	24.6	-
Specific Gravity Gs		Min.	2.60	2.60	2.61	-
		Max.	2.66	2.70	2.70	-
		Ave.	2.64	2.66	2.66	-
Gradation	Clay + Silt (%)	Min.	54.0	20.0	4.0	-
		Max.	95.5	46.0	18.0	-
		Ave.	80.7	28.1	12.6	-
	Sand (%)	Min.	4.5	54.0	82.0	-
		Max.	46.0	80.0	96.0	-
		Ave.	19.4	71.9	87.4	-
	D_{50} (mm)	Min.	0.012	0.080	0.100	-
		Max.	0.072	0.130	0.205	-
		Ave.	0.025	0.096	0.141	-
	Uniformity Coefficient Uc	Min.		3.0	1.6	-
		Max.		10.8	5.3	-
		Ave.	5.5	5.5	2.9	-
Atterberg Limit	Liquid Limit LL (%)	Min.	40.0	-	-	-
		Max.	46.0	-	-	-
		Ave.	43.0	-	-	-
	Plastic Limit PL (%)	Min.	20.0	-	-	-
		Max.	24.0	-	-	-
		Ave.	22.0	-	-	-
	Plasticity Index I_P (%)	Min.	20.0	-	-	-
		Max.	22.0	-	-	-
		Ave.	21.0	-	-	-

(2) N-values

As with the bridge foundation, the loss-error of the measured N-values was corrected by the formula of Uto (1974) in the same way as the bridge site foundation. The distribution of corrected N-values at the approach road route is shown in Figure 4.2.8. This figure shows that there is a wide scatter in N-values among the 12 boreholes along the proposed alignment of the approach road. Therefore, the lower design line of corrected N-value distribution is proposed for the design of the approach road.

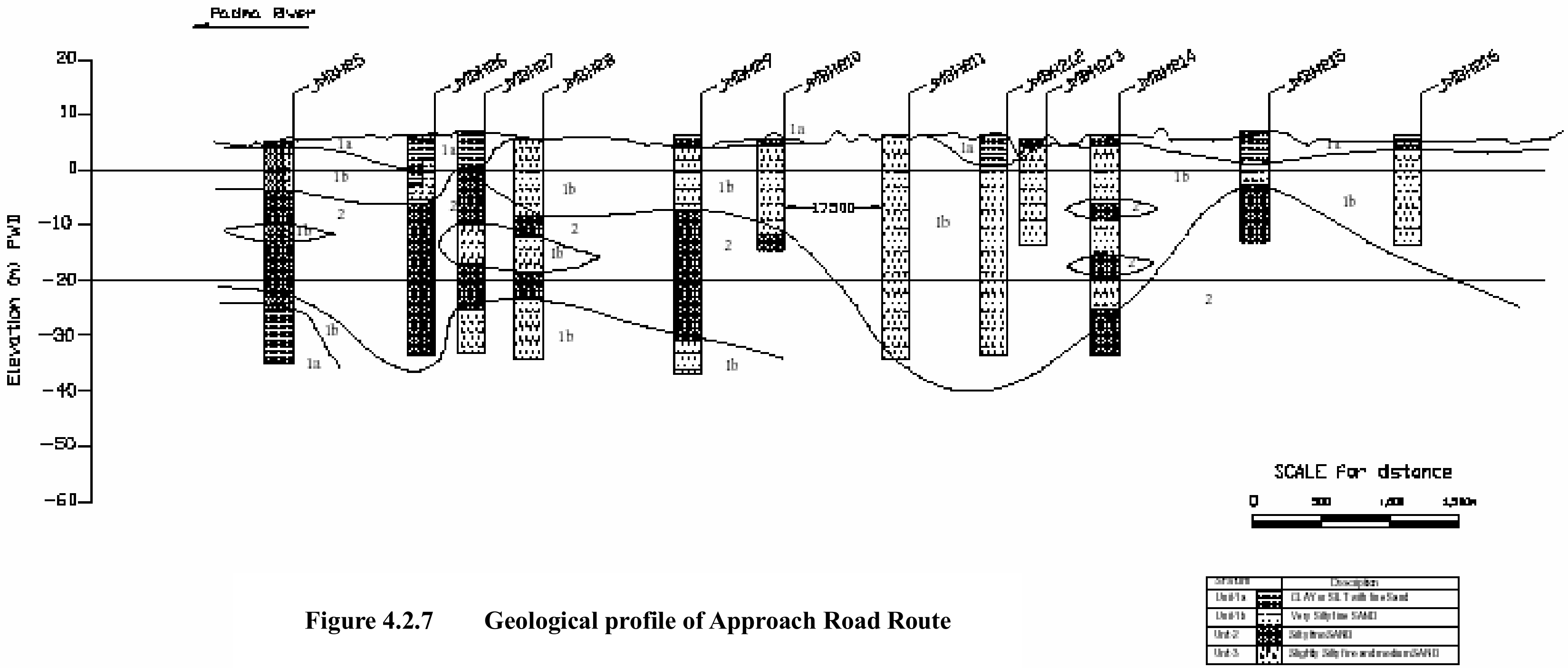


Figure 4.2.7 Geological profile of Approach Road Route

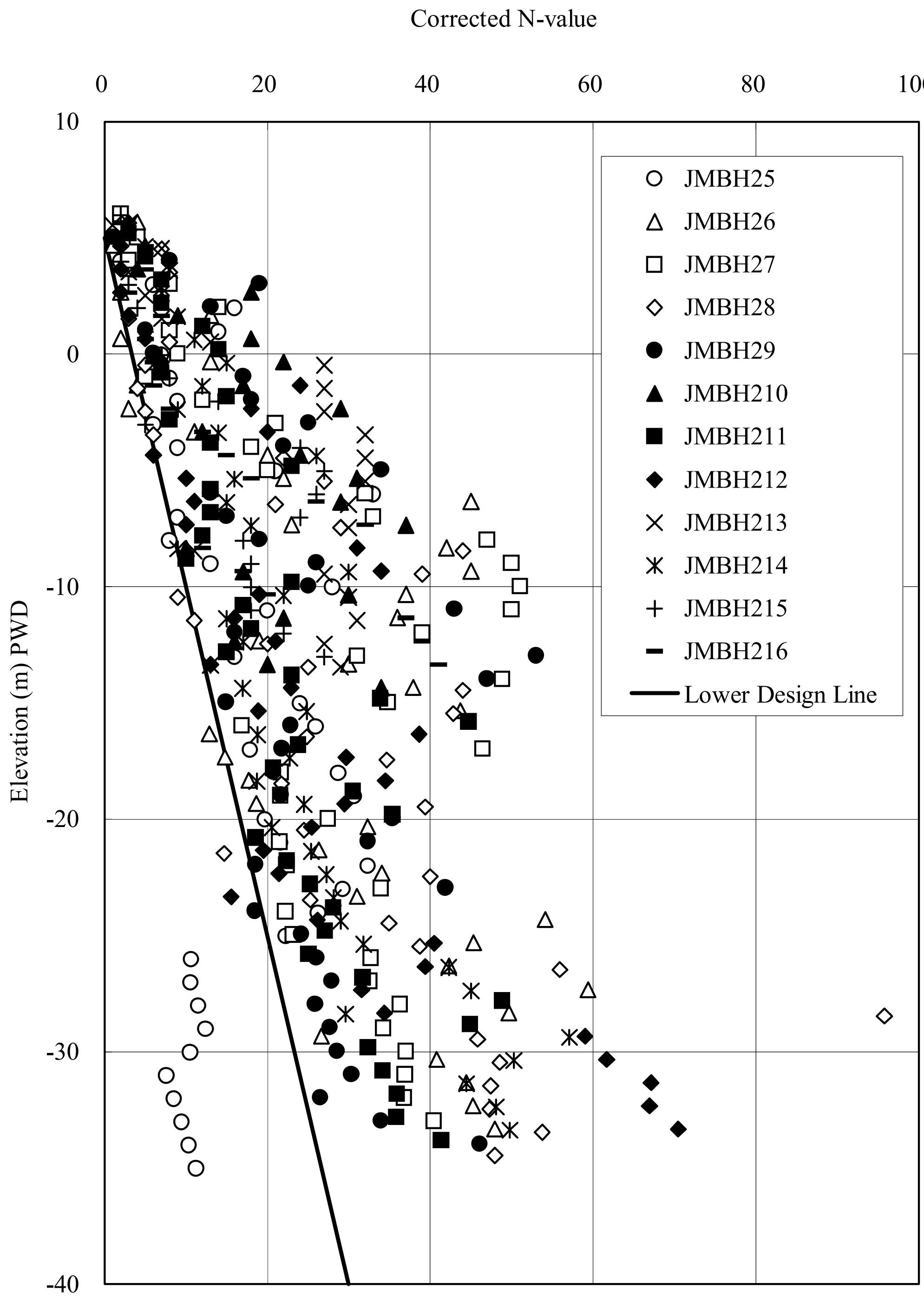


Figure 4.2.8 Distribution of Corrected N-values (Approach Road Route)

(3) Laboratory Test Results

(a) Chemical Properties of Soil Samples

The value of pH ranges from 6.5 to 8.1 and the soil at the approach road route is nearly neutral. Contents of chloride, soluble sulfate and total sulfate range from 0.0293 to 0.0310 %, 0.0030 to 0.0170 % and 0.0380 to 0.1749 % respectively. In this regard, the soil cannot have an adverse effect on the concrete structures such as piles and culverts.

(b) Chemical Properties of Groundwater

The value of pH ranges from 6.8 to 7.6 and the groundwater at the approach road route is nearly neutral. Chloride content and sulfate content range from 11.0 to 165.0 mg/L and from 1.0 to 125.0 mg/L respectively. It is evaluated that the groundwater cannot have an adverse effect on the concrete structures such as piles and culverts.

(4) Design Values

The proposed design values for the approach road are shown in Tables 4.2.6.

Table 4.2.6 Design Values for the Approach Road

Stratum	N-value	Density	Strength Parameters	
		γt (kN/m^3)	c (kN/m^2)	ϕ (degree)
Unit - 1a	Lower Design Line in Figure 4.2.8	18.0	10 x N	0
Unit - 1b	Lower Design Line in Figure 4.2.8	19.0	0	$15 + \sqrt{(15N)} - 6$ *1
Unit - 2	Lower Design Line in Figure 4.2.8	19.0	0	$15 + \sqrt{(15N)} - 6$ *1
Unit - 3	Lower Design Line in Figure 4.2.8	19.0	0	$15 + \sqrt{(15N)} - 6$ *1

Note:
*1) The maximum of estimated ϕ values is assumed to be 37 degrees

4.2.5 Embankment Materials

Laboratory test results of the embankment material are summarized in Table 4.2.7. As a subgrade material, embankment material is required to be more than 8% of CBR. Judging from the CBR test results, compacted density of γd (95%) under 4.5 Ec is required of embankment material for subgrade.

Table 4.2.7 Summary of Embankment Material Tests

TP No.		TP-1	TP-2	TP-3	TP-4	TP-5
Depth (m)		2.4 ~ 3.0	2.0 ~ 2.5	2.4 ~ 2.8	2.4 ~ 3.0	2.4 ~ 3.0
Water Content (%)		30.0	28.0	24.0	32.0	22.4
Specific Gravity Gs		2.74	2.74	2.74	2.66	2.72
Gradation	Clay (%)	17.0	3.0	10.0	0.5	3.0
	Silt (%)	81.0	70.0	77.0	17.0	61.0
	Sand (%)	2.0	27.0	13.0	82.5	36.0
	D_{50} (mm)	0.016	0.050	0.028	0.110	0.058
	Uc	10.3	3.5	7.0	2.1	2.8
Atterberg Limits	Liquid Limit (%)	34	NP	28	NP	NP
	Plastic Limit (%)	23	NP	21	NP	NP
	Plasticity Index (%)	11	NP	7	NP	NP
Soil Classification		CL	ML	ML-CL	SM	ML
Compaction (A)	Wopt (%)	22.0	21.0	20.0	24.0	21.0
· 1 Ec	γdmax (g/cm^3)	1.58	1.51	1.60	1.43	1.51
· 2.5 kg Rammer	γd (90%) (g/cm^3)	1.42	1.36	1.44	1.29	1.36
· 305 mm Drop	γd (95%) (g/cm^3)	1.50	1.43	1.52	1.36	1.43
Compaction (B)	Wopt (%)	17.0	19.0	17.0	18.0	18.0
· 4.5 Ec	γdmax (g/cm^3)	1.73	1.60	1.72	1.52	1.63
· 4.54 kg Rammer	γd (90%) (g/cm^3)	1.56	1.44	1.55	1.37	1.47
· 457 mm Drop	γd (95%) (g/cm^3)	1.64	1.52	1.63	1.44	1.55
CBR corresponding to Compaction (A)	CBR_{90} (%)	-	-	-	-	-
	CBR_{95} (%)	1.6	7.1	4.2	5.1	3.8
CBR corresponding to Compaction (B)	CBR_{90} (%)	4.0	7.8	7.5	5.7	9.5
	CBR_{95} (%)	7.2	13.4	16.2	9.5	20.8

4.2.6 Liquefaction Potential Analysis

Liquefaction potential has been evaluated using the Seed method (ref. Seed and Idriss, 1971) and the Iwasaki method (ref. Iwasaki and Tatsuoka, 1978). The method consists of evaluating the cyclic stress ratio (L) in an element of soil resulting from an earthquake acceleration and comparing it with the cyclic resistance ratio (R). The liquefaction resistance (FL) is defined as FL = R / L. If FL is less than 1.0, liquefaction may occur.

The liquefaction resistance (FL) is conventionally determined from the following equations:

$$F_L = R / L$$
$$R = R_1 + R_2 + R_3$$
$$L = 0.65 \times \alpha_{max} \times \gamma d \times \sigma v / \sigma v'$$
$$R_1 = 0.0882 \times \sqrt{\frac{N}{\sigma v' + 0.7}}$$

$R_2 = 0.19$ $(0.02mm \leq D_{50} \leq 0.05mm)$

$0.225 \times \log_{10}(0.35 / D_{50})$ $(0.05mm < D_{50} \leq 0.6mm)$

-0.05 $(0.6mm < D_{50} \leq 2.0mm)$

R3 = 0.0 $(0\% \leq FC \leq 40\%)$

0.004 x FC – 0.16 $(40\% < FC \leq 100\%)$

$\gamma d = 1.0 - 0.015x$

where,

F_L: liquefaction resistance
R: cyclic resistance ratio
L: cyclic stress ratio
α_{max}: maximum horizontal acceleration coefficient
γd: stress reduction coefficient
σv: total vertical stress (tf/m^2)
$\sigma v'$: effective vertical stress (tf/m^2)
N: N-value
D_{50}: diameter at which 50% of the soil is finer (mm)
FC: fine particle content (%)
x: depth less than 20m (m)

The estimated maximum horizontal acceleration for the bridge design have been assessed at 0.125g at Mawa Janjira site in the pre-feasibility study of Padma Bridge, 2000. In this study, the maximum horizontal acceleration of 0.125g was adopted for the liquefaction potential analysis.

Details of liquefaction potential analysis are presented in Appendix-4. As a result of analysis, the following conclusions were obtained:

1) On borehole sections JMBH21, JMBH23 and JMBH24, there is no potential for liquefaction either before scour or after scour. This result is due to the ground condition of Unit-1b layer containing more than 20 % fine particles, such as clay and silt.
2) On borehole section JMBH22, there is some potential for liquefaction at a depth of about 5 m before scour because the N-value at that depth is very low. However, it is not a serious problem because this liquefaction potential occurs locally.

4.3 RIVER STUDIES

4.3.1 Further Studies on the Erodibility of the Riverbank

Further studies on the erodibility of the riverbank were made specifically for the Mawa-Janjira site (MJ-site), which was finally selected for crossing of Padma Bridge.

(1) Historical River Course Shifting

(a) Data and Methodology

Historical maps and satellite images were collected. The historical maps collected cover the period from 1776 to 1963 and the satellite images from CORONA in 1967 to Landsat in 2003. In addition to these, aerial photos taken in December 1998 and November 1989 were also referred to.

Adjusting the coordinates and clear land marks carefully, maps and satellite images were superimposed over each other and historical plan form changes of the Padma River were studied for the stretch neighboring the crossing location at the MJ-site.

(b) Long-Term River Course Shifting

Long-term river course shifting since 1776 (228 years ago) was studied based on the historical topographic maps in reference to the plan form in 2003 and the major findings are as follows:

1) In the 1776-map, the old Padma River took its river course almost along the present Arialkhan River and it changed to the present river course as seen in the 1860-map.
2) Since 1860, the Padma River has shifted gradually toward the northeast.
3) Since 1914 at latest, the main stream of the Padma River has kept its present river course at the crossing location, not deviating from the less erodible left bank.

(c) Recent River Course Shifting

Changes and evolutions of the riverbanks, main stream and sand bars (char) during recent years since 1960 were studied further based on the historical maps and satellite images of shorter time intervals. As a result, the following behaviors of the Padma River were found:

1) Since 1960, the Padma River at the MJ-site has maintained a single channel section with no char. Though the period of data is limited, the Padma River seems to alternate between forming a meandering river and straight river courses, having a node at the crossing location.
2) The meandering river develops gradually as the main stream upstream from the crossing location moves westward. The development peaks of meandering took place in around 1967 and 1993 when the South Channel got most active.
3) The straight river course appears when the meandering channel becomes inactive or extinct and the straight channel along the less erodible left bank becomes active. The most typical straight river course is found in around 1980.
4) According to the above data, it took about 13 years to change from the peak of meander river (1967) to that of straight river (1980) and took another 13 years to return to the peak of meander river (1993) with a cycle time of 26 years.
5) Along with the alternate process of meandering and straight river courses, riverbanks of the Padma were eroded or sand bars/islands were developed. Judging from the findings mentioned above, the Padma River around the crossing location would be in the transition from the meandering to straight river course.
6) The Padma River experienced big flood events in 1987, 1988 and 1989. However, significant influences were not identified with regard to the plan form change by these flood events.

(2) Rate of Riverbank Change

(a) Data and Methodology

In order to estimate the erosion rate of the riverbanks around the crossing location, periodical river survey data by BWDB were studied. Cross sections selected for the study are CS-P2.0 and P2.1 located downstream from the bridge site and CS-P3.0, CS-P3.1 and CS-P4.0 upstream from the bridge site, for the period of 33 years from 1968/69 to 2000/01.

Locations of survey stakes were examined and the extent of the survey and sectional forms of the river were checked with maps and satellite images to identify the locations of the riverbanks. Based on these data, the rate of riverbank changes and yearly changes of riverbanks were worked out.

(b) Erosion Rate of Left Bank

For the selected sections, the erosion rate of the less erodible riverbank was studied. For this purpose locations of the left bank (natural levee) regardless of attached bars were identified and shown in Figure 4.3.1 setting the reference year in 1972. From the figure, the following are found:

1) Historical change of the left riverbank at section CS-P3.0 which is closest to the crossing location on the left bank indicates an average erosion rate of 5 m/year for the whole data period, though it is 8 m/year for the 13 years until 1985 and 1.5 m/year for the most recent 15 years. Judging from the plan forms of the Padma River, the higher erosion rate seems to be caused by collision of meander flow from the South Channel with protrusions of the riverbank to straight river flow.
2) The bank erosion at section CS-P3.1 located upstream from the crossing location is small. The riverbank at P4.0 did not suffer from erosion during the data period because of the existence of sand bars in front.
3) The bank erosion at section CS-P2.1 downstream from the crossing location was large for the period from 1989 to 1993 probably caused by collision of meander flow from the South Channel, although the natural levees are not always distinct for the sections CS-P2.1 and CS-P2.0.
4) The yearly change of riverbank is small, more or less 20 m at maximum.

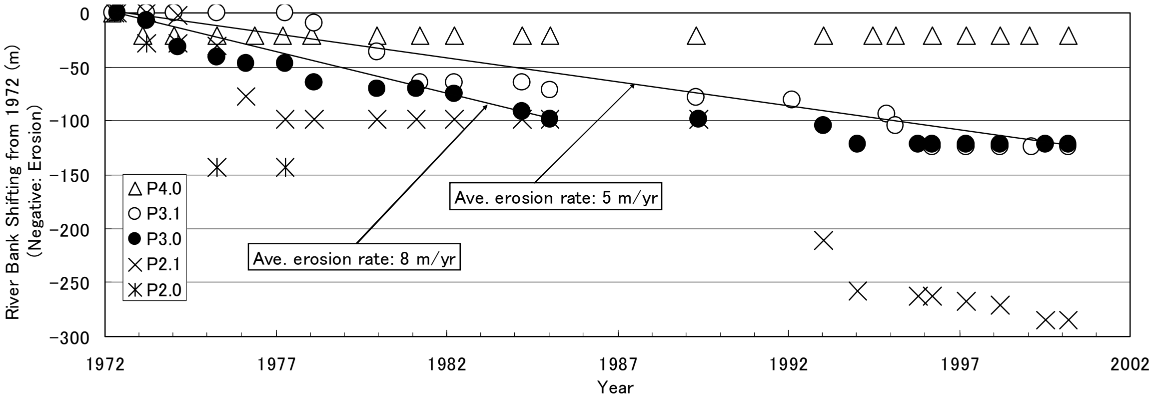


Figure 4.3.1 Average Erosion Rate of Left Bank

(c) Rate of Right Bank Changes

The existing right bank is formed of recent sediment deposits and is highly vulnerable to erosion. Similarly to the left bank, the rate of riverbank changes were studied based on BWDB survey data. Cross section CS-P2.0 and CS-P2.1 were used for this study since other sections upstream from the crossing location are influenced by the evolution of char. Erosion rate of riverbank was markedly high until 1985 showing an average annual rate of about 240 m/yr at CS-P2.1. After 1985, the high erosion rate seems to be terminated and the riverbank location remains the same, though it fluctuates within the range of about 1000 m.

The yearly changes of the right bank are also high. According to the data, yearly erosion amounts to as high as 500 m/yr at maximum at section CS-P2.1 which is the nearest section to the crossing location on the right bank.

(3) Limit of River Course Shifting

The existing left bank composed of geologically older soil layers covered with natural levee is less erodible and can be considered as the left limit of river course shifting.

On the other hand, the existing right bank is formed of young and loose deposits and susceptible to change. However at about 10 km inland from the existing right bank, there exists a boundary of recently active flood plain and high land (old flood plain). The high lands are made up of natural levee eroded by old river movements as shown in Figure 4.3.2.

Examining the historical bank-line changes, the old flood plain on the south-west of the boundary was found not to have experience of erosion due to the main Padma River at least these 90 years since 1914. The boundary can be considered as the right limit of river course shifting.

The Padma River has a long-term tendency to shift toward north-east and is apt to flow along side the less erodible left bank. The Padma River flows between the left and right limits, frequently changing its right bank line probably due to temporal changes of water and sediment flow of the Padma River.

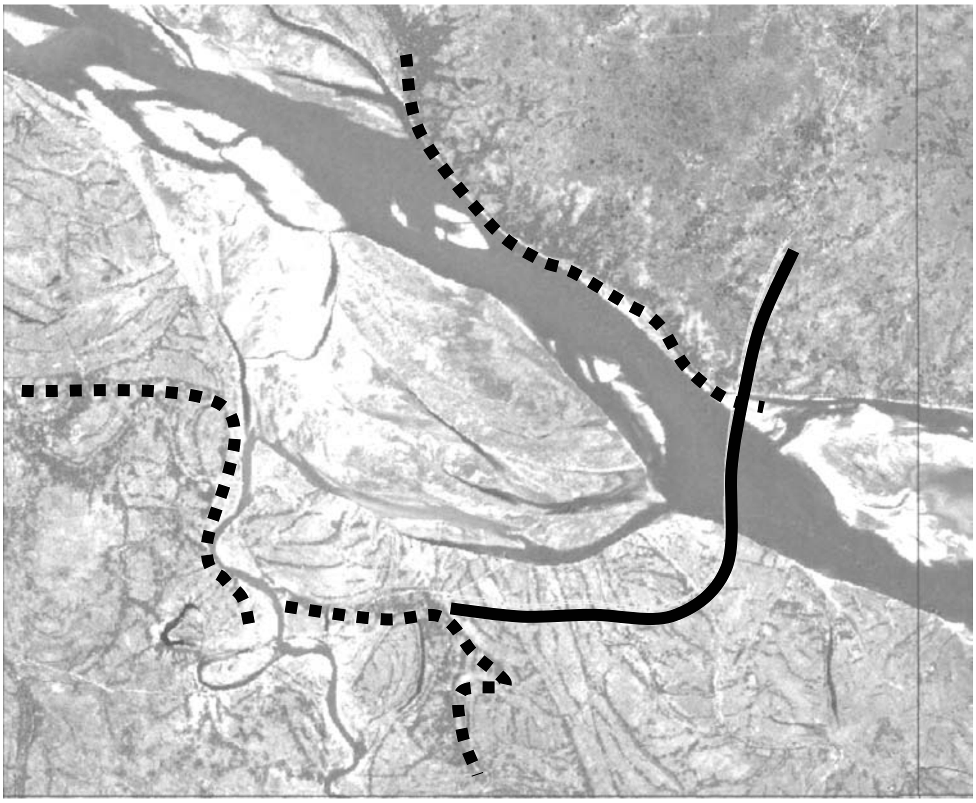


Figure 4.3.2 Limit of River Course Shifting

(4) Geotechnical Structure of Riverbanks

Geotechnical data were collected and analyzed to clarify the cause of the lower erodibility of the left bank and its vertical and spatial distribution. The data collected and analyzed for the study included geotechnical investigations conducted for the Pre-F/S in addition to geotechnical investigations carried out by the JICA Study Team for the 1st and 2nd stage studies.

According to the riverbank investigation made so far it has been found that the silt and clayey soil play an important role to resist erosion due to river flow. The boring data were, therefore, simplified, classifying into two types, (1) silt and clayey soil and (2) other soil materials to show up the characteristics of erodibility. From the study, the following matters are clarified:

1) On the left bank, the thickness of the silt and clayey soil cover is 12 m to 25 m for the most part, while on the right bank the thickness of the soil and clayey soil is as thin as 5

m, more or less. The thicker silt and clayey soil layer would be the main reason f the lower erodibility of the left bank.

2) As for the width of the silt and clayey soil along the left bank, no significant differences were found between the boring log at the river bank and that at about 1 km inland. Judging from the borehole data and distribution of the natural levee, the width of the less erodible layer along the left bank would be more than 1 km, though further investigations are necessary to obtain the spatial distribution definitely.

4.3.2 Estimation of Maximum Scour Depth

The maximum scour depth necessary for the design of river and bridge facilities was estimated specifically for the Mawa-Janjira site (MJ-site) finally selected for the crossing of Padma Bridge.

(1) Methodology

(a) Types of Riverbed Scour

Various types of riverbed scour are conceivable for estimation of scour depth around the structures related to the bridge and river training works. They are shown, in general, classifying depending on the cause and size of acting area as follows:

Natural Scour

1) Long-term degradation
2) Bend scour
3) Confluence scour
4) Scour due to bed-form

Structure-induced Scour

5) Constriction scour
6) Scour around bank protection works
7) Scour around bridge piers

The maximum scour depth was worked out assuming the worst combination of the component riverbed scour types listed above.

(b) Long-Term Degradation

Lowering due to long-term degradation occurs because of a decrease in sediment supply from the upstream reaches or an increase in river discharge. According to the Study Report of FAP 24, the Jamuna and Ganges rivers have a trend of aggradations with an annual sedimentation rate of 0.01 m/year. The riverbed of the Padma, located downstream from the confluence of both rivers, may also have a rising tendency. Therefore, the long-term degradation does not need to be considered as far as the maximum scour depth is concerned.

Just upstream of the MJ-site, the Arialkhan River bifurcates. The Arialkhan River was a main stream of the Padma River until about 150 years ago and now conveys about 10 percent of floodwater of the main Padma River. Considering geomorphologic evolution of the river at the MJ-site, the complete temporary or permanent closure of the Arialkhan River in the future is a definite possibility. The complete closure of the Arialkhan would bring about an increase of river discharge and lowering of the riverbed at the JM-site. In view of this, the maximum scour depth was estimated under assumed conditions of complete closure of the Arialkhan River (Closed Arialkhan R.)

(c) Bend Scour

The riverbed is scoured, in general, at the outer side and raised at the inner side of the channel bend. The bend scour can occur at any part of the river section. The scour depth at the bend was calculated by the empirical formula by Thorne.

(d) Confluence Scour

Confluence scour occurs where two channels meet, namely the main Padma River and the south channel of an anabranch behind Char Kawrakandi. An empirical relation proposed by the FAP-21 Manual in line with the approach by Klaassen and Vermeer was applied to estimate the confluence scour.

(e) Scour due to Bed-form

According to the results of our study on sediment flow near the MJ-site, it was confirmed that the sediment was transported in flat bed conditions during flood season. The scour due to bed-form was not taken into account for the estimation of the maximum scour depth.

(f) Constriction Scour

If the river section at the crossing location is constricted to reduce the bridge length, the concentration of flow at the constriction would bring about an increase in flow depth. The increase in flow depth is defined as constriction scour.

Since river training works proposed for Padma Bridge are to maintain existing river and flow conditions giving less impact to the river, the existing river channel would not be constricted and the flood water in flood plain would not be influenced significantly by the approach roads. Therefore, constriction scour was not considered for the estimation of the maximum scour depth.

(g) Scour around Bank Protection Works

Around the permeable groynes, various scour phenomena take place, i.e., scour downstream from the tip of the groyne, scour around the groyne piles, scour upstream from the groyne due to protrusion. Among these, scour downstream from the tip of groyne is the most important for the design of the structure. The local scour induced by revetment works at the abrupt transition at the tip of impermeable structure in combination with bed protection can be treated as local scour.

An empirical relation based on the formula of Ahmad recommended by the FAP-21 Manual was applied to calculate the scour depth around the bank protection works adjusting the coefficient corresponding to the work.

(h) Scour around Bridge Piers

Local scour around cylindrical bridge piers was estimated using the empirical formula related to the diameter of cylindrical bridge piers. Influence of group piles was also considered applying the empirical diagrams investigated by Hannah

(2) Component Scour Depth

(a) Fundamental Setup

Hydraulic parameters necessary to apply the empirical formula and diagrams were set up based on the probable discharge, probable water level, regime relations of the Padma River section presented in the FAP-21 Manual, etc.

(b) Calculation of Scour Depth

The scour depth due to natural scour (bend scour and confluence scour) and structure-induced scour (scours around groynes, revetments and bridge piers) were calculated according to the methodology mentioned earlier. In estimating the scour depths, the following arrangements were made:

1) Natural scour was estimated as the total of bend and confluence scours that could occur at the same time for the worst.
2) Structure-induced scour was calculated under the flow conditions of natural scour.
3) The scour depth (y_s) is further divided into normal scour (y_n) and incremental scour ($y_{s'}$). The total water depth (h_s) subject to more than one type of scour is estimated as follows:

   $$h_s = h_m + y_n + \sum y_s'$$

4) The normal scour is defined as the difference between the maximum water depth and average water depth of the river section under the straight and normal flow conditions not subject to specific scours mentioned earlier. The normal scour (y_n) was assumed as follows taking the lowest ratio of the maximum water depth and the average water depth based on the periodical survey sections of BWDB:

   $$y_n = 0.25\ h_m$$

(3) Design Maximum Scour Depth

The maximum scour depth was calculated as a total of component scour depths, assuming their worst combination. In order to account for extra scour which may be induced by unforeseen behavior of river channel and compound influence of the structures, the total scour shall be multiplied by a factor of 1.20 to determine design maximum scour depth (DMSD).

Adopting the case of a closed Arialkhan River, DMSD was determined for 100- and 25-year floods as shown below, in which hs is a total scoured water depth below DHWL and Zs is the elevation of the scoured riverbed:

1) DMSD for Permeable Groynes:

			(100-yr.)	(25-yr)
h_s	=	$h_m + y_s$	35.5 m	33.4m
Z_s	=	DHWL - h_s	-28.2 m PWD	-26.5 m PWD

2) DMSD for Revetments/Guide Bunds:

			(100-yr.)	(25-yr)
h_s	=	$h_m + y_s$	44.9 m	42.3 m
Z_s	=	DHWL - h_s	-37.6 m PWD	-35.3 m PWD

3) DMSD for Bridge Structure: Scour around bridge piers is not included

In the middle of the river section

			(100-yr.)
h_s	=	$h_m + y_s$	31.0 m
Z_s	=	DHWL - h_s	-23.6 m PWD

Adjacent to riverbanks (within 300 of riverbanks)

			(100-yr.)
h_s	=	$h_m + y_s$	44.9 m
Z_s	=	DHWL - h_s	-37.6 m PWD

(4) Comparison with Historical Riverbed Records

Figure 4.3.3 shows historical river sections periodically surveyed by BWDB in the river reaches neighboring the crossing location. These river sections are available for a period of 35 years from 1968, however, there is some lack of survey information. The lowest riverbeds surveyed at each section are summarized in Table 4.3.1.

The deep scours took place mostly near the left bank and some in the river center. Judging from the plan form of the river and the stream flow lines on the satellite images, the main causes of scour at the foot of the left bank are due to bend or flow attack to the less erodible bank of the South Channel and protrusion of the left riverbank near Mawa Ghat. Some deep scour in the river center are deemed to be caused by flow turbulence due to confluence of channels downstream from the char.

In comparison with the historical lowest beds mentioned above, the design maximum scour depth (DMSD) determined in the previous sub-section is deemed appropriate.

Table 4.3.1 Lowest Riverbed Surveyed Periodically by BWDB

Year[1)]	CS-P2.0			CS-P2.1			CS-P3.0			CS-P3.1			CS-P4.0		
	Yr.	M.	Lowest bed (m,PWD)	Yr.	M.	Lowest bed (m,PWD)	Yr.	M.	Lowest bed (m,PWD)	Yr.	M.	Lowest bed (m,PWD)	Yr.	M.	Lowest bed (m,PWD)
1968/69	1969	1	−14.11	1968	12	−14.39	1968	12	−17.60	1969	1	−14.13	1968	12	−10.19
1969/70	1970	1	−13.68	1970	1	−10.56	1970	1	−37.01	1970	1	−14.59	1970	1	−7.87
1970/71	1971	2	−9.80	1971	2	−10.89	1971	2	−25.24	1971	2	−10.90	1971	2	−7.23
1971/72	1972	5	−7.62	1972	5	−10.24	1972	5	−23.28	1972	5	−9.49	1972	5	−7.16
1972/73	1973	3	−9.47	1973	3	−12.68	1973	3	−15.50	1973	3	−13.53	1973	3	−9.55
1973/74	1974	2	−13.16	1974	2	−15.09	1974	2	−18.84	1974	2	−10.00	1974	2	−10.82
1974/75	1975	4	−8.52	1975	4	−13.21	1975	4	−17.84	1975	4	−14.47	1975	4	−13.43
1975/76	1976	2	−7.02	1976	2	−9.17	1976	2	−22.07	1976	2	−7.05	1976	2	−8.36
1976/77	1977	4	−6.80	1977	4	−12.54	1977	4	−15.79	1977	4	−8.84	1977	4	−6.55
1977/78	1978	2	−8.31	1978	2	−14.28	1978	2	−12.31	1978	2	−7.82	1978	2	−5.77
1978/79	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−
1979/80	−	−	−	1979	12	−9.83	1979	12	−13.63	−	−	−	1979	12	−7.79
1980/81	1981	2	−9.78	1981	2	−12.05	1981	2	−20.20	1981	2	−11.22	1981	2	−10.36
1981/82	1982	4	−9.33	1982	3	−10.21	1982	3	−12.54	1982	4	−10.60	1982	3	−11.69
1982/83	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−
1983/84	1984	4	−15.63	1984	3	−14.42	1984	3	−24.81	1984	4	−22.91	1984	3	−13.87
1984/85	1985	2	−9.37	1985	1	−10.18	1985	1	−31.73	1985	2	−19.59	1985	1	−12.91
1985/86	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−
1986/87	1987	4	−10.75	−	−	−	−	−	−	1987	4	−6.78	−	−	−
1987/88	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−
1988/89	1989	5	−14.79	1989	5	−5.13	1989	5	−13.76	1989	5	−13.37	1989	5	−7.52
1989/90	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−
1990/91	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−
1991/92	−	−	−	−	−	−	−	−	−	−	−	−	−	−	−
1992/93	1993	3	−16.16	1993	1	−21.74	1993	1	−12.39	1993	3	−16.90	1993	1	−9.19
1993/94	1994	1	−25.00	1994	1	−28.31	1994	1	−9.20	1994	1	−7.47	1994	1	−14.50
1994/95	1995	3	−10.06	−	−	−	−	−	−	1995	3	−11.51	−	−	−
1995/96				1995	10	−4.70	1995	10	−4.94				1995	10	−4.94
	1996	3	−13.57	1996	3	−15.46	1996	3	−11.48	1996	3	−11.89	1996	3	−14.01
1996/97	1997	1	−16.99	1997	3	−21.66	1997	3	−8.62	1997	1	−8.71	1997	3	−10.95
1997/98	1998	3	−14.51	1998	3	−15.09	1998	3	−12.93	1998	3	−3.86	1998	3	−6.43
1998/99	−	−	−	1999	7	−14.10	1999	7	−9.76	−	−	−	1999	7	−6.42
1999/00	1999	11	−15.21	2000	3	−12.50	2000	3	−13.20	1999	11	−7.67	2000	3	−6.60
	2000	3	−12.84												
2000/01	2001	2	−10.35												

(NOTE) 1) Yr: Year starting from September to August next year(e.g., 1968/69: Sep. 1968 to Aug. 1969); Mn: month

2) Riverbed elevation shown in box indicates deep scour lower than −20.0 m,PWD

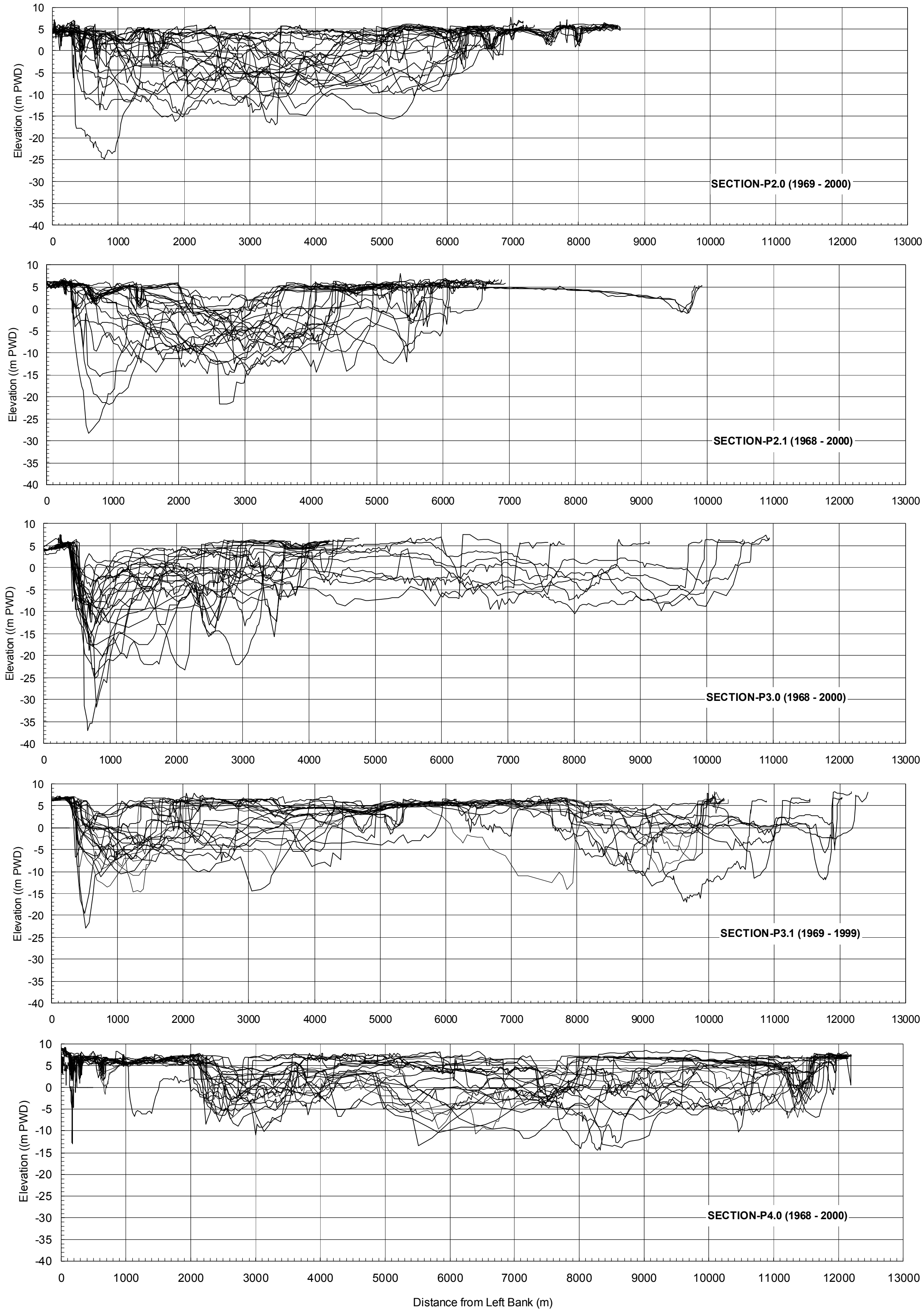


Figure 4.3.3 Historical River-Section Surveyed by BWDB

4.3.3 Mathematical Modeling

(1) Outlines

Mathematical modeling was carried out for analyzing hydraulic and river morphological aspects relevant to the preliminary designs of river works for the Padma Bridge.

The objectives of the mathematical modeling are described hereafter.

(a) Simulation of Present Hydraulic Conditions around Proposed Bridge Site

Mathematical simulation models are prepared to understand the present hydraulic and river morphological conditions around the Padma River at Mawa-Janjira. The models are based on data currently available and obtained through the field investigations carried out in relation to this mathematical modeling.

(b) Verification of Hydraulic Design Parameters and Dimensions for Proposed Structures

Hydraulic design parameters and dimensions required for the proposed structures were prepared with reference to analysis of hydrological records, experience of previous bridge construction projects in India and Bangladesh, available design standards, and other knowledge of river engineering. The preliminary designs for the feasibility study are worked out on the basis of these hydraulic design parameters and dimensions. The mathematical modeling is conducted for verifying the hydraulic design parameters and dimensions of the proposed structures in order to confirm technical appropriateness of the preliminary designs.

(c) Assessment of Impacts caused by Construction of the Project

Impacts to river flow conditions and/or morphological process caused by construction of the project are evaluated respectively in the course of the preliminary designs of the individual structures. Meanwhile, the mathematical modeling aims at evaluating integrated impacts through simulations of flooding in the neighboring areas and the tendency of morphological developments in order to ensure the layout plan of the proposed structures and their dimensions.

The mathematical modeling was carried out by the Institute of Water Modeling (IWM) under sub-contract with the Study Team.

In the beginning of the mathematical modeling, various data relevant to the Padma River, such as hydrology, flood and inundation, topography, river morphology, sediment, chars, and river works were analyzed to develop insight on the hydraulic and morphologic process. The interim results of the river study by the Study Team were also reviewed for conducting preparatory studies of the mathematical modeling.

Field measurements for acquiring necessary data for the mathematical modeling were also conducted before development of mathematical models. Bathymetric surveys were carried out for the Padma River reaches upstream and downstream of the Mawa-Janjira site. Geotechnical investigations for researching erosion resistance of riverbank were also conducted to evaluate the tendency of river morphology around the Mawa-Janjira site.

A quasi two-dimensional model was developed to cover the main stream and branches of the Padma River and flood plains that may be affected by construction of the project. Impacts cause by construction of the project were assessed through comparison of the results of simulations between ‘without project’ and ‘with project’ conditions. Main points for assessment were the extent of the flooding area and distribution of flooding depth in the neighboring flood plains.

A two-dimensional model was developed to cover the main stream of the Padma River and riverbanks that may be affected by construction of the project. Impacts on morphological developments were assessed through comparison of the results of simulations between 'without project' and 'with project' conditions.

In addition, simulations by two-dimensional models also were conducted to assess the middle to long-term (5 to 10 years) impacts on the tendency of the river morphological process through comparison of simulations between 'without project' and 'with project' conditions.

(2) Quasi-two Dimensional Modeling and Simulations

(a) Outlines of Model Development

Development of a quasi two-dimensional model was conducted by using MIKE11 series. A quasi-two dimensional model was developed to cover the major rivers and principal branches for the reaches between the Jamuna-Ganges confluence and Chandpur. The model was developed by re-organizing the existing hydrodynamic models such as the General Model (GM), South West Region Model (SWRM) and North Central Region Model (NCRM), which had been developed and validated by IWM under the funding by BWDB for the hydrological year 2001/02.

Topographic features of a selected extent of flood plain that might be affected by construction of the project were also incorporated into the model in the manner of a digital elevation model (DEM) properly linked with the river network. Minor rivers connecting with the major rivers and principal branches of the Padma were also incorporated within the flood plain area of interest.

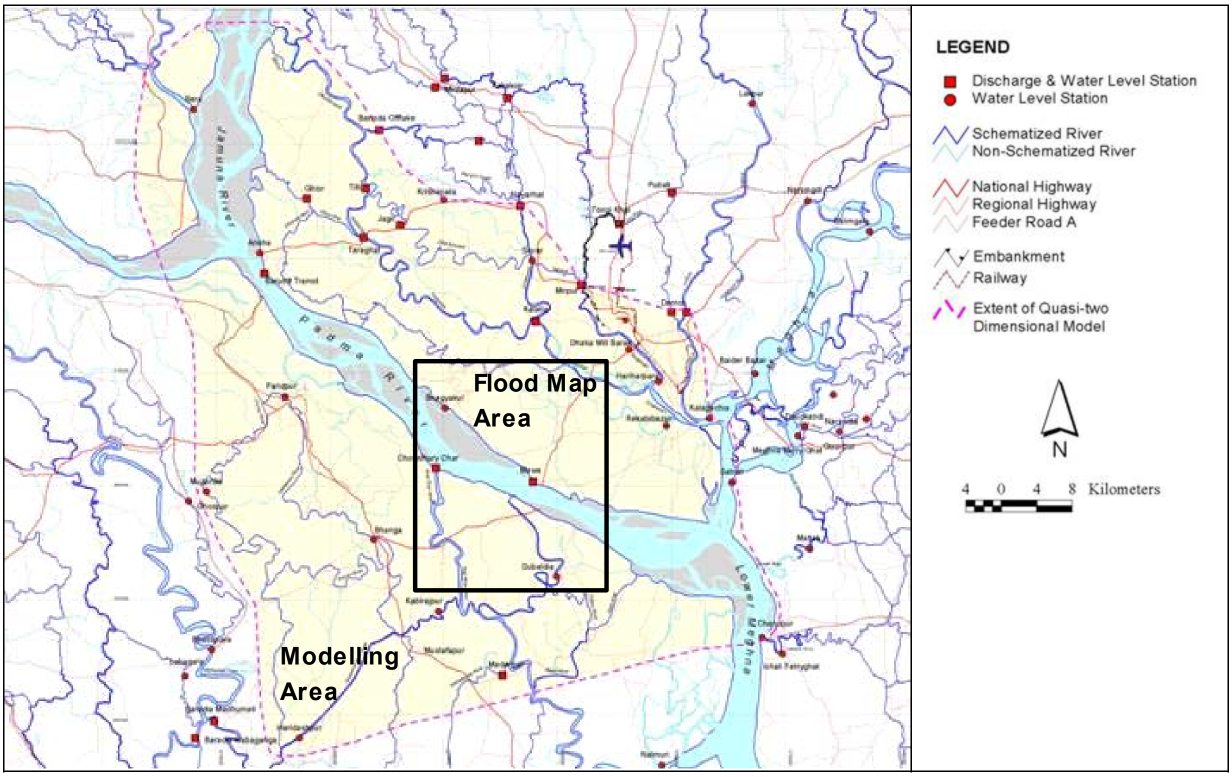


Figure 4.3.4 Extent of Quasi-two Dimensional Model

(b) Simulation by Quasi-two Dimensional Model

The simulation of hydrodynamics requires hydrographs expressed by a time series of hydrological data at the different boundaries. For preparation of the hydrograph at each boundary, the hydrological year 1998/99 was selected as a typical flood year for generating the probable hydrographs at the respective boundaries. Probable hydrographs for the different return periods such as 25- and 100-year were estimated by reduction or enlargement of the 1998/99 hydrograph at each boundary on the basis of probable water levels and discharges at Baruria Transit representing dominantly the hydrological conditions of the Padma.

Simulations for 'without project' and 'with project' conditions were conducted for the design floods of 25- and 100-year return periods, respectively. The results of simulations are presented hereunder together with assessment of impacts by construction of the project.

(c) Summary of Simulation Results and Impact Assessment

Details of the simulation results and impact assessment for hydrodynamics and flooding against the design flood are presented in Appendix-6.

The results of the simulations by the quasi-two dimensional model indicate that some minor impacts would be anticipated in terms of flooding extent and water level after construction of the project, i.e. some +10 cm increment of the highest water levels at Mawa and locations along the upstream side of the proposed approach road on the right bank..

Impacts on flooding were also assessed by the simulations in terms of changes in flooding duration at the selected locations on the upstream sides of the proposed approach road on the right bank. Flooding duration is almost unchanged between 'without project' and 'with project' conditions.

The simulated flood maps for the selected area around the proposed bridge site indicate that the maximum flooding extent by depth becomes slightly wider in the 'with project' condition. An increment of the flooding area with a flooding depth over 2.0 m is only 2% for the 25-year return period and 1% for the 100-year return period, respectively.

Meanwhile, it is also necessary to consider the limitations in the mathematical modeling developed on the basis of the currently available data only. Due to the limitation in topographic data of local channels in the flood plains, the simulations considered only five major openings by local bridges with an opening width ranging from 150 to 310 m for the proposed approach road section on the right bank. Besides the major openings, a number of small openings by minor bridges and crossing culverts are also proposed. Even though the local effects by such small openings could not be evaluated by the present scale of the mathematical modeling, the small increment of flooding extent and water level for 'with project' condition is expected to be further reduced with the local effects of the minor openings.

With due consideration of the above, it is concluded that the adverse impacts by construction of the project will be minor.

(3) Two-dimensional Modeling and Simulation: 2003/04 Padma 2-D Core Model

(a) Outlines of Model Development

Development of the two-dimensional model was conducted by using the MIKE21C software. A two-dimensional model was developed to cover the main stream and branches of the Padma River and riverbanks that may be affected by construction of the project. The model was based on the satellite images of 2003, the available bathymetric data by BIWTA in 2003 and the bathymetric surveys carried out for the mathematical modeling in June and August 2004. This two-dimensional model was therefore named '2003/04 Padma 2-D Core Model' and was calibrated and validated through comparison between simulated and observed hydrodynamics and river morphological processes in the years 2003 and 2004.

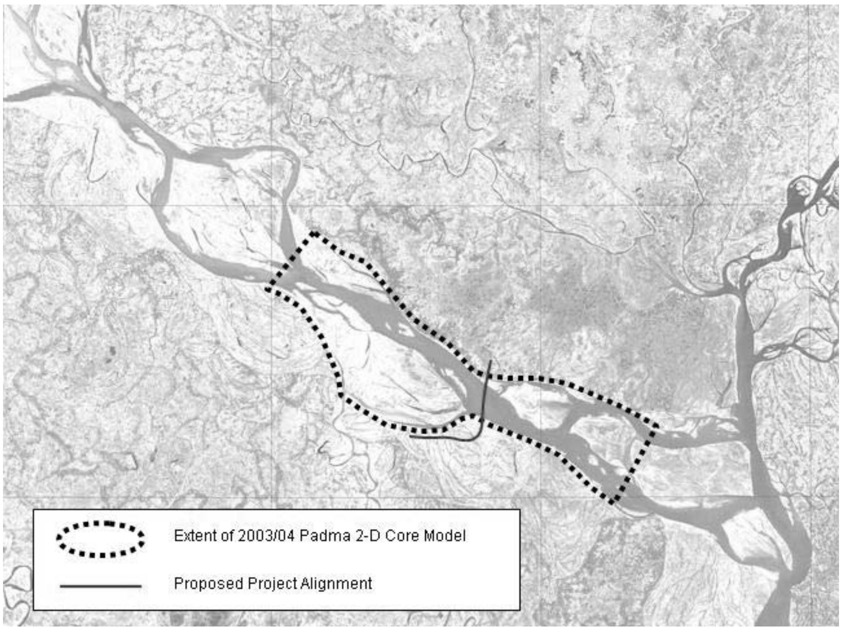


Figure 4.3.5 Extent of 2003/04 Padma 2-D Core Model

(b) Simulations by 2003/04 Padma 2-D Core Model

Hydrological settings for verification of hydraulic design parameters and structural dimensions of the proposed structures were given in the form of a water level hydrograph at the downstream boundary and a discharge hydrograph at the upstream boundary. The probable flood hydrograph at the boundary was generated on the basis of the observed hydrograph in 1998/99.

Based on the analysis of flood magnitude at Baruria Transit, the probable hydrographs for the return periods of 25- and 100-years were estimated by reduction or enlargement of the 1998/99 discharge and water level hydrographs, respectively.

For the purpose of simulations for riverbank erosion, the types of riverbank depending on erosion resistance were classified according to the morphologic study by CEGIS (February 2004), which identified that riverbank materials of the Padma can be classified into three categories, i.e. highly erosive (erosion rate more than hundreds meters per year), moderately erosive (20 to 50 meters per year) and relatively erosion resistant (0 to 15 m per year). According to the CEGIS classifications and the extent of the proposed bank protection works, the riverbank settings for 'without project' and 'with project' conditions were defined as follows.

Highly Erosive Bank represents a river bank showing low erosion resistance with an erosion rate of 100 m/year or more because of natural soil characteristics. This type of river bank is identified mostly along the right bank

Moderately Erosive Bank represents a river bank showing moderate erosion resistance with an erosion rate of 20-50 m/year because of natural soil characteristics. This type of river bank is identified along the left bank, extending 25 km from the proposed bridge site to the downstream.

Relatively Erosion Resistant Bank represents a river bank showing erosion resistance with an erosion rate of 0-15 m/year because of natural soil characteristics. This type of river bank is identified along the left bank upstream of the proposed bridge site with an extent of 15 km approximately.

Non-erosive Bank is defined as a river bank protected by permanent or semi-permanent structures. This type is adopted for the extent of the proposed bank protection works.

(c) Summary of Simulation Results and Impact Assessment

Details of the simulation results and impact assessment for hydrodynamics and morphological processes against the design flood are presented in Appendix-6.

The results of the simulation suggest that the hydraulic design parameters for the preliminary designs are properly given and the adverse impacts caused by construction of the project will be minor in terms of hydrodynamics and river morphology for the design flood under the 2003/04 conditions.

The hydraulic design parameters of flow velocity and lowest scoured level are given in consideration of a critical condition representing that river flow concentrates onto one-side of the riverbank and accordingly naturally and structurally induced scours take place at the same location. Meanwhile, the 2003/04 conditions that are the basis of the present two-dimensional model do not represent such a critical condition and the simulated results indicate lower values for both flow velocity and lowest scoured level. The hydraulic design parameters used for the preliminary design are therefore regarded as conservative but should be considered as a possible critical condition for the design purposes.

Table 4.3.2 Summary of Simulations for Without/With Project Conditions (2003/04 Padma 2-D Core Model)

Hydraulic Parameters at Bridge Site	Without Project	With Project	Design Parameters
Return Period 100-year			
Highest Water Level (m PWD)	7.49	7.49	7.35
Max. Discharge (m^3/sec)	137,800	137,800	134,400
Max. Flow Velocity (m/sec)	2.9	3.0	4.8
Lowest Scoured Level (m PWD)	-12	-16	-37.6
Return Period 25-year			
Highest Water Level (m PWD)	7.04	7.15	6.94
Max. Discharge (m^3/sec)	119,900	119,900	120,100
Max. Flow Velocity (m/sec)	2.8	2.8	4.7
Lowest Scoured Level (m PWD)	-11	-13	-35.3

Note: 'Max. Flow Velocity' means the maximum of depth-averaged velocity.

The observed impacts in the simulated results are attributed to a small extent of channel constriction because of the proposed bank protection works partially designed above Standard High Water Level (SHWL). The extent of such a constriction is some 100 m and that is comparatively smaller than the scale of the crossing width of 5.3 km. The impacts indicated as the differences between 'without project' and 'with project' conditions are minor as a whole. For the design flood of a 100-year return period, an increment of the highest water level is almost negligible and only a +0.1 m increment occurs in the depth-averaged flow velocity. An increment of +4 m takes place in the lowest scoured level for the design flood of a 100-year return period but its elevation is -16 m PWD which is still far above the design scour level of -37.6 m PWD.

On the left bank, noticeable riverbank erosion of some 150 m for a 100-year return period flood is found around 6 km downstream from the proposed bridge site and may correspond with the area of Louhajang. Similarly on the right bank, noticeable riverbank erosion of 450 m for 100-year return period flood is found around 9 km downstream from the proposed bridge site. Comparison of the longitudinal profiles of river bank erosion indicate that effects of the proposed bank protection works to reduce the riverbank erosion are observed along the right bank from the proposed bridge site downstream to where the maximum riverbank erosion takes place. Meanwhile, there is no significant difference in the simulated maximum riverbank erosion between 'without project' and 'with project' conditions. It is therefore concluded that no significant impact in river bank erosion is expected between 'without project' and 'with project' conditions against the design flood.

(4) Two Dimensional Modeling and Simulation: 2003/04 Padma 2-D Long Term Model

(a) Outlines of Model Development

Development of a two-dimensional model for long-term simulation was conducted by using MIKE21C software. A two-dimensional model was developed to cover the main stream and branches of the Padma River and riverbanks between the Jamuna-Ganges confluence to Chandpur. The model was based on the satellite images of 2003, the river cross section surveys by BWDB pre-monsoon in 2003, the available bathymetric data by BIWTA in 2003 and the bathymetric surveys carried out for the mathematical modeling in June and August 2004. This two-dimensional model was therefore named '2003/04 Padma 2-D Long-term Model' and was validated through comparison between simulated and observed hydrodynamics and river morphological process in the years 2003 and 2004.

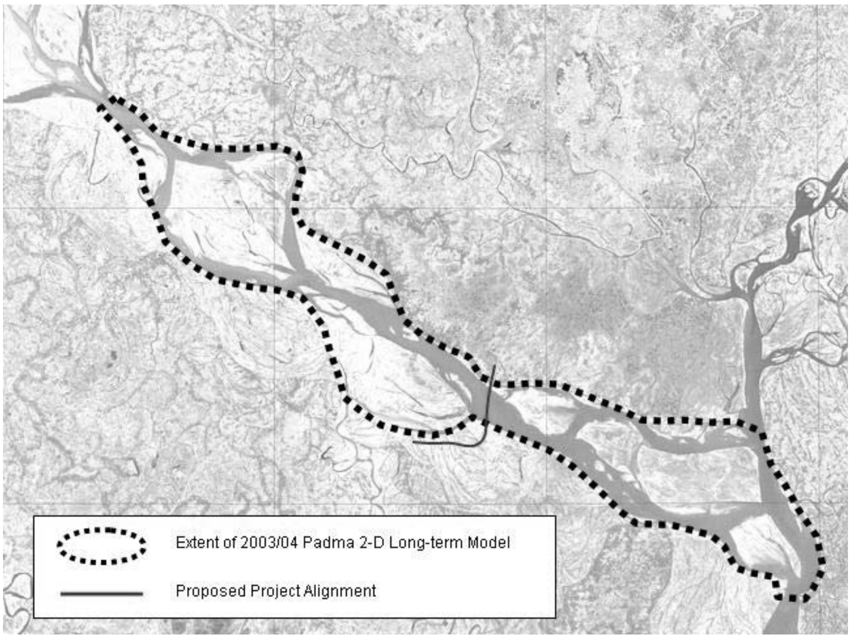


Figure 4.3.6 Extent of 2003/04 Padma 2-D Long-term Model

(b) Simulations by 2003/04 Padma 2-D Long-term Model

Hydrological settings for predictions of the middle- to long-term tendency of river morphology were given in the form of water level and discharge hydrographs in the latest 10 years (from 1993/94 to 2003/04) at the respective boundaries.

The long-term simulations were carried out by using hydrodynamics during the monsoon period in each year and the pre- or post monsoon period were not considered due to difficulties in simulation of morphological developments during the low flow period. This approach was already used in the past for similar two-dimensional simulations for the Jamuna and Gorai rivers. The morphological process was accordingly simulated as an accumulation of morphological developments in the monsoon period year by year. The results do not exactly represent the morphological process for the continuous hydrologic cycle but suggest an indicative morphological process for a long period.

A simulation for ‘without bridge’ condition was conducted at first to simulate the tendencies of the river morphological process for 5 and 10 years. Following the simulation for ‘without bridge’ condition, another simulation for ‘with bridge’ condition was conducted on the condition that the proposed structures were incorporated into the model in the same manner as the simulations by the 2003/04 Padma 2-D Core Model. The results of the simulations are presented hereunder together with assessment of long-term impacts by construction of the project.

(c) Summary of Simulation Results and Impact Assessment

Details of the simulation results and impact assessment for long-term tendencies of morphological processes are presented in Appendix-6.

Comparison of the simulated results between ‘without project’ and ‘with project’ conditions indicate that no significant difference is visible for the main water course in view of macro-basis morphological developments. From the viewpoint of indicative tendencies of riverbed scour or deposition by location, both ‘without project’ and ‘with project’ conditions indicate no large difference with each other in the middle- and long-term tendencies of morphological process.

Comparison of the simulated results between ‘without project’ and ‘with project’ conditions indicate only minor differences in the riverbank erosion. On the left bank, noticeable riverbank erosion of some 300 m after 10 years is found around 7 km downstream from the proposed bridge site. Similar tendencies are also observed on the right bank around 12 km downstream from the proposed bridge site with an erosion of some 700 m after 10 years. Comparison of the longitudinal profiles of river bank erosion show the effects of the proposed bank protection work to reduce the riverbank erosion are observed along the right bank from the proposed bridge site to the downstream where the maximum riverbank erosion takes place. Meanwhile, there is no significant difference in the simulated maximum riverbank erosion between ‘without project’ and ‘with project’ conditions. It is therefore concluded that no significant long-term impact in river bank erosion is expected between ‘without project’ and ‘with project’ conditions.

Chapter 5 Preliminary Facility Design

5.1 PRELIMINARY DESIGN OF RIVER FACILITIES

5.1.1 Establishment of Principles and Basis for Design

(1) Strategy for Planning River Facilities

(a) Objectives of River Works

Main objectives of river works are:

1) To ensure the stability of the river section by maintaining existing river and flow conditions, and
2) To firmly protect the bridge structures.

In order to achieve the objectives, strategies presented in the following paragraphs were established.

(b) Principle for Planning River Facilities: Existing favorable river conditions as bridge site shall be maintained and strengthened by river works, reducing impact to the river and river flow.

Mawa-Janjira site (MJ-site) was selected at the channel forming a narrow and single section without chars. The selected site provides favorable physical conditions for a bridge crossing, and the conditions have been stable for more than 40 years. As to the left bank, it has remained at the present location for more than 90 years. It is very natural to use such favorable conditions for a bridge crossing strengthened by river facilities. As far as the existing river and river flow conditions are maintained, the favorable site conditions shall also be maintained. And the bank protection measures developed through Jamuna Bridge Project, FAP-21 study and other recent efforts enable us to achieve this.

(c) River Width: Existing perennial river width will not be constricted.

According to the latest river survey, the perennial river width at the crossing location is about 5.3 km. River facilities for Padma Bridge are to be constructed on the riverbank to avoid current attack during construction, which in result leaves the existing perennial channel not constricted.

The scheme to narrow the river width further is not acceptable. In order to narrow the perennial river width, intensive river works must be executed in the water current. It is difficult to execute such works in a big river such as the Padma, especially for maintaining embankment slope and placing mattress or launching aprons under the water. Even if it were done, the quality of the work could not be guaranteed.

For the same reason, guide bunds of Jamuna Bridge were constructed on land, i.e., West Guide Bund on the char and East Guide Bund on the bank.

(d) Bank Protection Measures: Conventional measures and state-of-the-art bank protection technologies developed in Bangladesh should be employed in combination, in due consideration of the characteristics of the Padma River:

The Padma River is the so-called “wandering river” which changes its regime between the meandering and straight rivers, while the lower Ganges River is meandering and the Jamuna River is braided.

In Bangladesh, guide bund works have been used conventionally. Recently, much progress in river studies and development of bank protection measures have been made, mainly in relation with the Jamuna Bridge construction project, FAP studies/projects and other bank protection efforts. In order to attain the objectives of the river works for the Padma Bridge, all the applicable measures from the conventional to state-of-the-art technologies should be employed in combination, considering their functions and applicable river regimes. Outlines of these measures and studies are introduced below.

(e) Maintenance Repair: River facilities shall be planned and designed presupposing monitoring and maintenance repair after the construction, considering total cost during the project life.

Unlike bridge structures, the river facilities are, in general, made of earth materials and segments like concrete block or stone, and they are relatively easy to repair part by part. In addition, the construction of permanent facilities with complete durability from the beginning would be a ridiculously high cost. Therefore, it is common practice to design river facilities initially at a certain safety level and maintain the function by maintenance repair in the course of use.

The river facilities are to be designed based on a 25-year return period for ordinary bank protection work following the FAP pilot works and 100-year for those of bridge structures to be firmly protected. Whatever the bank protection works are designed based on 25 year or 100 year floods, the bank would always be protected in a safe manner in association with the monitoring and maintenance repairs. The difference among the two would be the recurrence of repair works.

(2) Basis for Design of River Facilities

(a) Standards and Studies to be Referred to

- Study reports and design documents for Jamuna Bridge, 1989 - 1998
- Standard Design Manual, BWDB, 1994
- Evaluation Report of Bank Protection and River Training Pilot Project and Guidelines and Design Manual for Standardized Bank Protection Structures (FAP-21/22, 2001)
- Reports on Jamuna-Meghna River Erosion Mitigation Project (2002) and other bank protection projects in Bangladesh
- Standards, guidelines and research papers in India, Japan and other countries

(b) Rainfall and Meteorology

1) **Meteorology:** Rainfall is the most distinctive parameter of the meteorological features of the study area. Dhaka, the nearest meteorological station to the proposed bridge site, receives annual rainfall of 2,118 mm on average. The rainy months (> 200 mm) continue for 5 months from May to September.
2) **Rainfall Intensity:** The relationships of rainfall intensity, duration and frequency (IDF) for short duration rainfall developed by FAP-8A (Greater Dhaka Protection Project, JICA, 1992) are to be used.

3) **Number of Rainy Days:** Number of rainy days counted by daily rainfall depth exceeding 5, 10, and 20 mm is shown below for Dhaka and Faridpur. The number of rainy days is related to workable days for construction works at bridge site.

Daily Rainfall > 5 mm

Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Dhaka	0.4	1.5	2.3	6.0	11.2	12.3	13.9	13.0	11.3	6.1	1.2	0.6	79.9
Faridpur	0.3	1.5	2.2	5.5	9.5	11.1	13.8	12.3	10.7	5.4	1.0	0.5	73.7

Daily Rainfall > 10 mm

Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Dhaka	0.2	0.9	1.7	4.6	8.8	9.2	9.7	8.7	7.9	4.4	0.8	0.4	57.2
Faridpur	0.2	0.9	1.5	4.2	7.6	8.7	9.8	8.4	7.4	4.1	0.7	0.3	53.7

Daily Rainfall > 20 mm

Observatory	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Dhaka	0.1	0.3	1.1	2.6	6.2	5.4	5.7	4.6	4.9	2.9	0.5	0.3	34.6
Faridpur	0.0	0.3	0.8	2.5	4.7	5.0	5.0	4.4	4.4	2.6	0.5	0.2	30.4

4) **Past Major Floods:** According to the flood statistics since 1954, the floods that occurred in 1987, 1988 and 1998 are the historic severe events in Bangladesh.

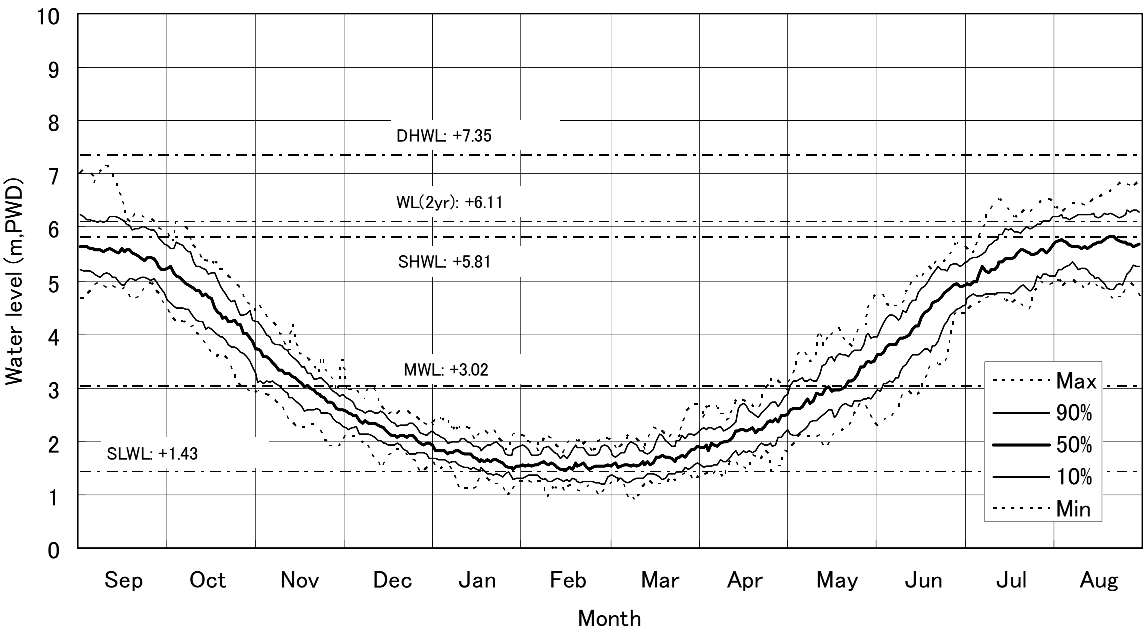
(c) Water Level and Discharge

1) **Design High Water Levels and Design Discharge:** The Design High Water Levels and Design Flood Discharges are estimated based on the BWDB data at Mawa station for various return periods as follows;

Return Period	Design Water Levels (m PWD)	Design Discharges (m^3/sec)
2-year	6.11	90,100
5-year	6.44	102,400
10-year	6.66	110,300
25-year	6.94	120,100
50-year	7.14	127,300
100-year	7.35	134,400

2) **Standard Water Level:** The Standard High and Low Water Levels are defined by BIWTA as average water levels of 5% and 95% exceedance in each year. Based on the BWDB data at Mawa station, the Standard High/Low Water Levels were estimated at the crossing location as follows;
   - Period of data : 1968/69 – 2002/03
   - Standard High water Level (SHWL) : +5.81 m PWD
   - Mean water Level (MWL) : +3.02 m PWD
   - Standard Low water Level (SLWL) : +1.43 m PWD

3) **Annual Water Level Change:** Water level statistics of the Padma River shall be referred to for planning the work program. Water levels at Mawa station are shown below for the maximum, minimum, and dependability of 90%, 50% and 10%.



(d) Other Hydraulic Design Parameters

1) **Design Flow Velocity:** Depth and averaged flow velocity (v_m) shall be calculated by Manning's formula based on coefficient of roughness n = 0.015 for flood flow and n = 0.025 for flows in dry season. Velocity at the riverbed (v_b) is assumed to be $v_b = 0.6$ x $v_{m.}$
2) **Design Wave:** Design wave in the Padma River was determined as follows by SMB-method (Sverdrup-Munk- Bretschneider) for wind speed 25 m/sec and fetch length 5.4 km (river width);
   Design wave height = 1.4 m, with cycle time = 3.5 sec
3) **Design Maximum Scour Depth (DSMD):** The Design Maximum Scour Depth (DSMD) is estimated based on the FAP-21 Manual and other related studies. The riverbed elevations under the design maximum scour conditions are summarized as below;

Description	100-yr (m PWD)	25-yr (m PWD)
For permeable groyne	-28.2	-26.5
For revetment/guide bund	-37.6	-35.3
For bridge structure		
- In the middle of river section	-23.6	-
- Adjacent to riverbank	-37.6	-

(e) Topographic, Geotechnical and Geomorphologic Aspects

1) **Topographic Configurations:** The topographic data surveyed in June to July 2004 shall be utilized for the design.
2) **Seismic Factor:** The seismic factor applied to MJ-site shall be 0.125 according to the "Seismic Zoning Map of Bangladesh" (originally from "Guide to Planning and Design of River Training and Bank Protection Works, BWDB)
3) **Cutting Slope Underwater:** According to the recommendation of FAP-21 study, the cutting slope underwater is to be designed not steeper than 1V: 6H.
4) **Stability of Riverbanks:**
   - Left Bank: Consideration should be given to conserve the existing less erodible left bank as much as possible in designing, execution of works and maintenance. The average annual erosion rate in the past 30 years was about 5 m/year.
   - Right Bank: Higher erodibility of the right bank should be taken into account in

the study of construction methods and plans.

(f) Other Terms to be Considered

1) **Tributaries and Branch Channels:** Tributaries and branch channels of the Padma River shall, in principle, remain open to minimize the impact on the present flow conditions.
2) **Completion of River Works before Flood Season:** Bank protection works shall be planned and designed to complete the construction works to a durable state against flood flow before the flood season begins, considering the workable period based on the statistical annual water level change.
3) **Upstream Termination:** Upstream termination of the bank protection works shall be designed, giving special attention to minimize flow concentration and formation of eddies due to flow separation.
4) **Consideration on Maintenance Repair:** Bank protection works with deep scour at the foot inherently require periodic and emergency maintenance repair. Therefore, the structure shall be planned and designed so as to enable remedial measures with less effort for coping with unpredicted failure after construction. Use of local techniques and resources shall also be considered to the maximum extent possible.

5.1.2 Preliminary Design of River Facilities

(1) Layout Plan and Applicable Works

(a) Crossing Route

Mawa-Janjira site (MJ-site) has been selected as the crossing location for Padma Bridge, and the crossing route at MJ-site was proposed along the route connecting the following points across the stable and single section of the Padma River:

1) Left Bank: About 1 km downstream from Mawa Ghat, just on the extended line of National Highway-N8.
2) Right Bank: About 2 km downstream from the confluence of the South Channel, an anabranch behind Char Kawrakandi.

Topography of MJ-site and the cross section of the Padma River along the crossing route are shown in Figures 5.1.1 and 5.1.2. These were surveyed in July 2004 by the Study Team.

Present main stream of the Padma River approaches to the bridge opening straight along side the left bank, and then it crosses the bridge axis on the skew. The main stream flows further straight and then shifts to the right bank some 10 km downstream from the proposed landing point of the bridge. Along the right bank there is a stream collecting flows from the South Channel and a part of the main Padma flow. The right side stream seems to be confined by the sediment flow due to the main stream of the Padma and right bank forming relatively deep scour at the foot of the right bank.

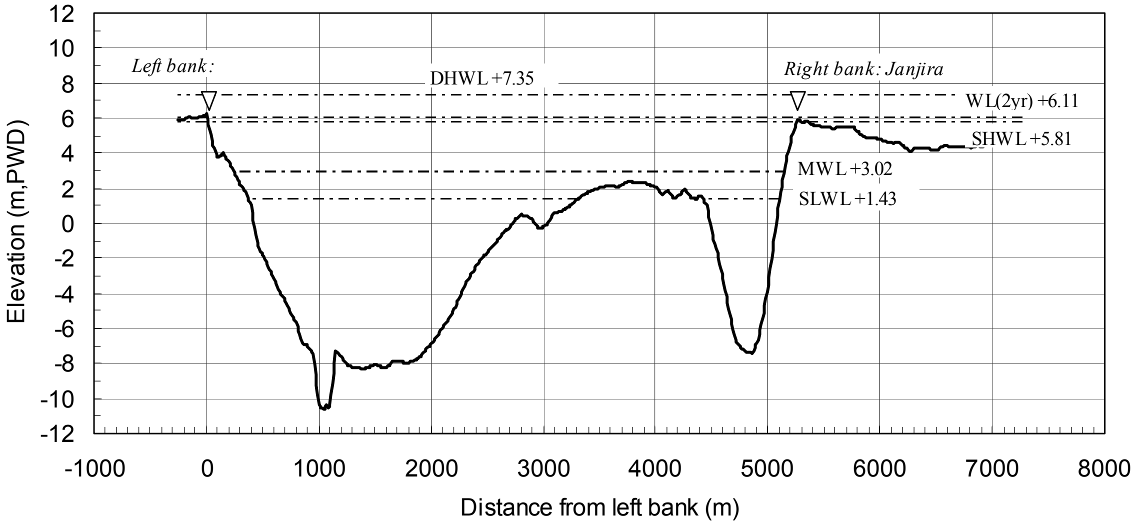


Figure 5.1.1 Cross-Section of Padma River at Crossing Route

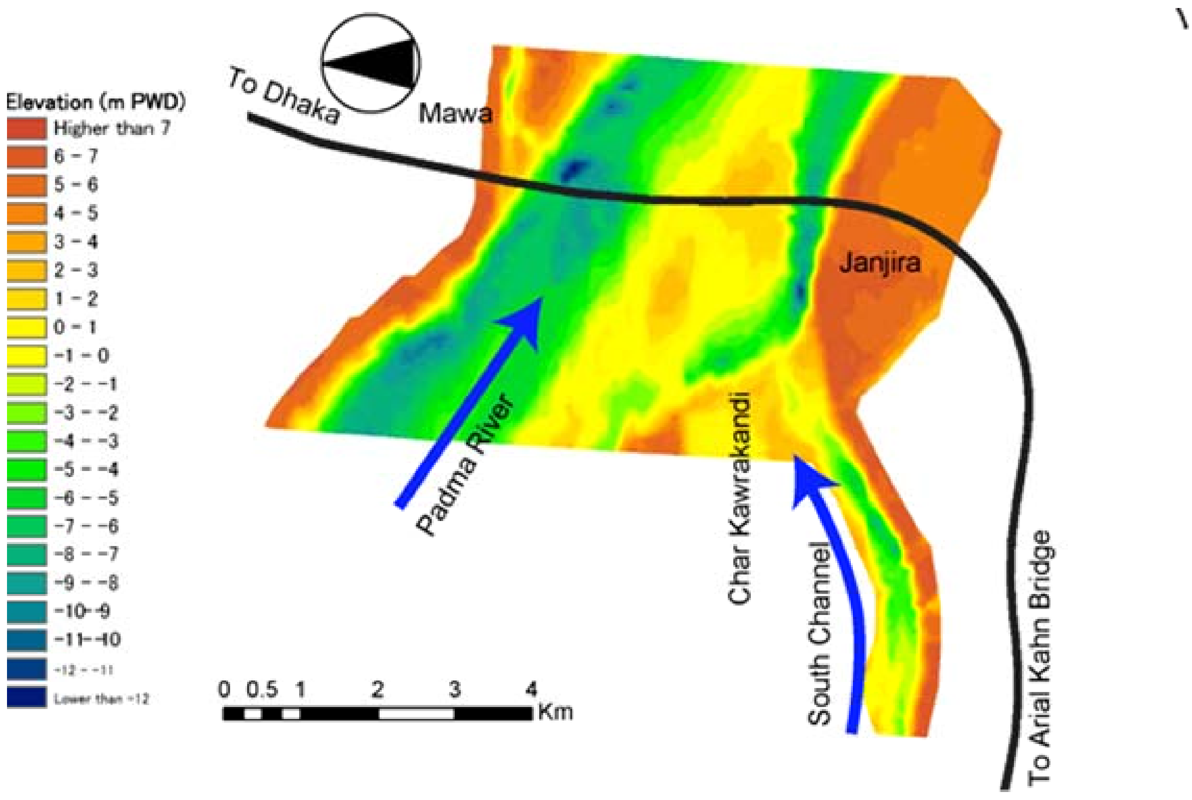


Figure 5.1.2 Topography of Mawa-Janjira Site

(b) Bank Protection Measures Applicable

Revetment and groyne works are the typical measures for the bank protection. Principal features of these works and application to the Padma Bridge are summarized below. After a comparative study on revetment and groyne works, the revetment works were selected for the bank protection of the present study.

Revetment Works: The revetment can be applicable to all locations of the BPW for Padma Bridge. Local materials and manpower could be largely adopted. In addition the revetment works are conventional and construction techniques and equipment available in Bangladesh could be employed. The revetment works, however, induce deep scour at the foot. Since the works are preferable to be executed above water, the workable period for the revetment work is short in general. The work can be executed even under water, but careful design arrangements and construction management are necessary to attain the required quality. Execution of the revetment works in water current is difficult in such a big river as the Padma.

Groyne Works: The groyne works are classified into permeable and impermeable groynes. Impermeable groynes constructed on fine sand beds cause deep scour at the tips and it is difficult to secure the stability of the groyne itself. Impermeable groynes are not recommended for the Padma River. The permeable groyne recommended by FAP-21 is attractive for its reduced scour depth around the works and the workability which enable the work to be carried out even in water current. The groyne works, however, were not adopted for the present study because of uncertainty regarding how well they would function under variable flow conditions and the fact that they would require the use of imported materials mostly.

(c) Protection Line of Left Bank

The protection line of the left bank was first proposed to be set 1 km away from the existing riverbank assuming possible future erosion of the less erodible bank. In this case bank protection work was not taken into account (Scheme-L1).

Later geomorphologic and geotechnical conditions of the less erodible bank were studied further for the preliminary facility design, and a scheme was conceived to shift the protection line to the existing bank-line, strengthening it with 6 km long bank protection works covering neighboring banks of the bridge structures and the protrusive bank upstream of Mawa ghat (Scheme-L2).

In conclusion, Scheme-L2 to set the protection line at the existing riverbank and strengthening the existing less erodible bank was selected, since Scheme-L2 requires less total direct cost and provides the local communities with erosion-free land along the Padma River.

(d) Sites to be Protected

The right and left riverbanks at the MJ-site shall be protected from erosion, so as to maintain the existing bank-lines. Continuous bank protection works are located on both banks so as to fix the riverbanks in the converging shape. The bank protection works on both banks would guide the flow of the Padma River smoothly to the bridge opening no matter where the Padma River may take route between the existing bank-lines. General layout of the bank protection works proposed for Padma Bridge is shown in Figure 5.1.3.

Protection Sites on the Left Bank

Since the left bank is less erodible, extensive bank protection works would not be necessary. The sites that require protection are the bank around the bridge structures and a protrusive bank-line upstream from Mawa ferry ghat. The protrusive bank-line has suffered from severe scouring repeatedly from the latter part of 1960's to the early 1980's. The landing point of the bridge on the left bank is located behind the protrusive bank-line protected from erosion.

In order to secure the stability of the left bank and firmly protect the bridge structure, bank protection works for a continuous length of 6 km were proposed to strengthen the existing bank, covering the banks adjacent to the bridge structures and the protrusive bank. The bank proposed for protection was divided into three (3) work sites depending on their locations and required functions as follows:

Work Site-L1: Site-L1 is located in front of the landing point of the proposed bridge along the main Padma River. This is the core facility to protect bridge structures on the left bank in association with adjoining sites on both ends and the existing less erodible bank. The protection length of Site-L1 is determined to be 1 km considering the width of bridge structures and approach road embankment.

Work Site-L2: Site-L2 is located at the downstream end of Site-L1 with a crest length of 1 km. The bank protection works at this site function as the downstream termination of Site-L1 works.

Work Site-L3: Site-L3 is located next to Site-L1 upstream, extending over a length of 4 km covering the whole protruding bank-line. Strengthening the existing less erodible bank, Site-L3 works protect the protruding bank-line from erosion due to direct current attack of the Padma River.

Protection Sites on Right Bank

The right bank is composed of loose fine sand and is vulnerable to erosion. In order to secure the stability of the right bank and firmly protect the bridge structures, bank protection works for a continuous length of 10.3 km of bank was proposed along the main Padma River and the South Channel.

Considering direct current attack of the Padma River, bank protection from the South Channel confluence to the downstream termination is required for the riverbank in front of the bridge structures and its neighboring areas.

Additionally, study results of historical river changes suggest that the stabilization of the South Channel is a crucial measure to attain the stability of the river channel around the crossing location and to protect the right approach road from outflanking. In view of this, the right bank of the South Channel downstream from Charjanajat Ghat was proposed for protection.

The bank proposed for protection was divided into four (4) work sites depending on their locations and required functions as follows:

Work Site-R1: Site-R1 is located in front of the landing point of the proposed bridge along the main Padma River. This is the core facility to protect bridge structures on the left bank in combination with adjoining sites on both ends. The protection length of Site-L1 is determined to be 1 km considering the width of bridge structures and approach road embankment.

Work Site-R2: Site-R2 is located at the downstream end of Site-R1 with a crest length of 1 km. The works at this site serves as the downstream termination of Site-R1 works.

Work Site-R3: Site-R3, extending over 2.0 km long, including transition to Site-R4, is connected with the upstream end of Site-R1. Site-R3 works function as the upstream termination of Site-R1 works along the main Padma and as a transition to connect Site-R4 works.

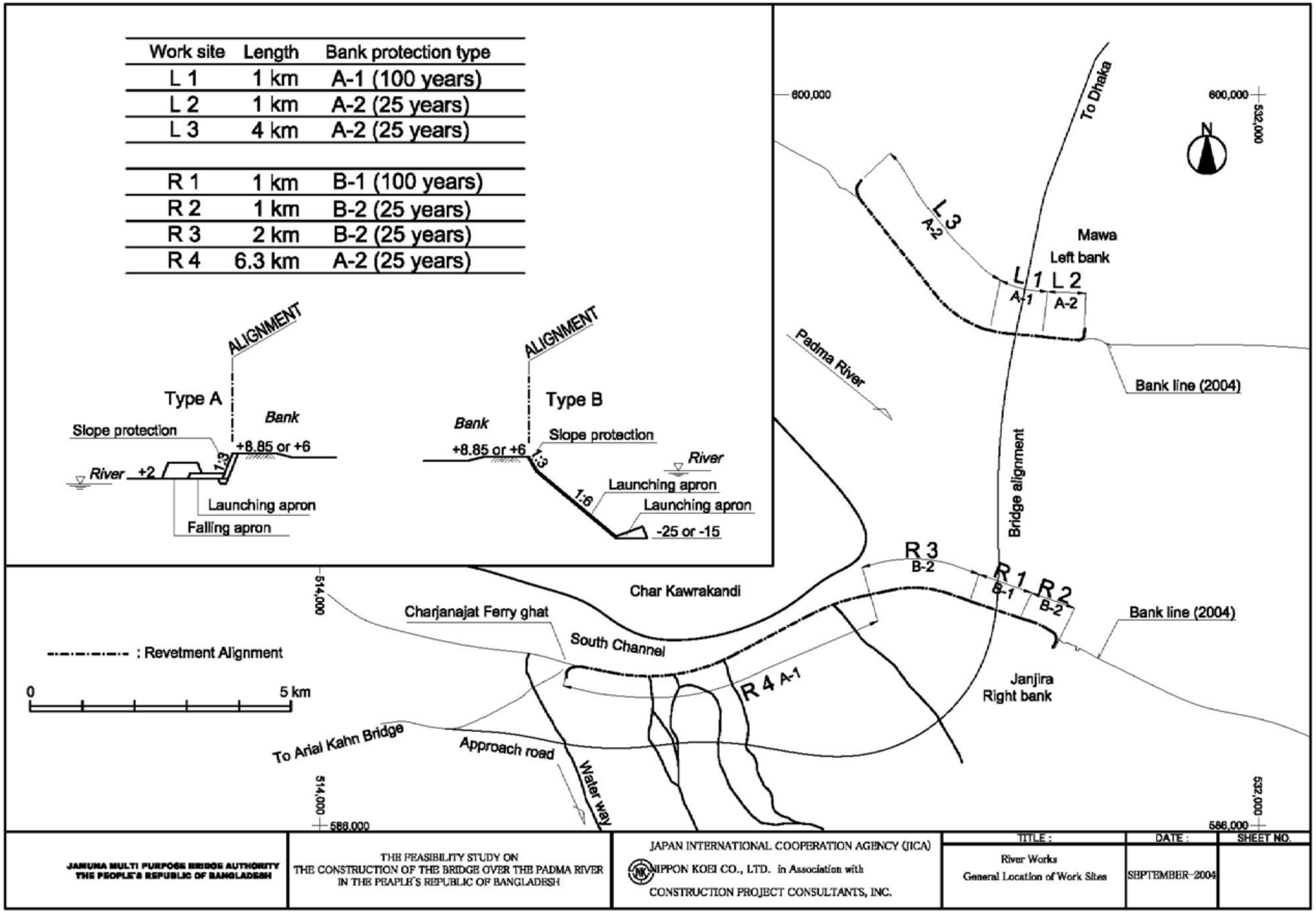


Figure 5.1.3 General Location of Work Site

Work Site-R4: Connected with Site-R3, Work Site-R4 extends over a total length of 6.3 km from the Padma river confluence to Charjanajat Ghat along the right bank of the South Channel. Though the erosion of the present South Channel is not active, the South Channel has been the main stream of the Padma River twice in the past 30 years and it may happen repeatedly in the future. When the South Channel becomes a main stream of the Padma in the future, Site-4 works will play a vital role to check south-word shifting of the South Channel and prevent outflanking of the right approach road. The extent of protection was proposed, considering the existing bank length exposed to possible current attack, historical movements of the right bank, and the location of the right limit of the active flood plain.

(e) Revetment Works Applicable to Respective Sites

Structure of Revetment Works: Standardized revetment structures recommended by the FAP-21 Manual mainly consist of slope protection, launching aprons and falling aprons. The slope protection is a main part of the revetment, and the launching and falling aprons are the toe protection works to protect the main part jointly from deep scour at the toe. FAP-21 Manual defines the falling and launching aprons as follows:

1) Falling apron: Toe protection of material, such as concrete blocks or boulders, placed directly on the existing subsoil or riverbed without filter.
2) Launching apron: Integrated and articulating toe protection, i.e., placed on prepared slopes and a filter layer above and below water or in a horizontal excavation above Standard Low Water Level.

Type-A and Type-B Revetments: Two types of revetment works are proposed depending on the difference of construction method as follows:

1) Type-A Revetment: Launching apron and falling apron are constructed on land above Standard Low Water Level (SLWL).
2) Type-B Revetment: Launching apron and falling apron are constructed under the water. The launching apron is placed on the design slope formed beforehand by dredging.

Principal features of both types of revetment works are shown in Table 5.1.1. The Type-A works ensure high quality construction, but the method is not always dependable since the construct is frequently attacked by the natural flow and scour. In order to compensate for the risk, excess protection materials should be stockpiled for the successive maintenance that would be required more for this type. By utilizing the Type-B works, definite function can be attained immediately after the construction. Since the works are executed under the water, uncertainty in construction cannot be avoided to some extent. Careful managements of works are required for the execution of Type-B revetment works.

The Type-A revetment is recommended for ordinary bank protection works by the FAP-21 Manual but there is no case yet to be applied to protect important structures as Padma Bridge. On the other hand, Type-B revetments were applied to the guide bunds for Jamuna Bridge and have functioned well.

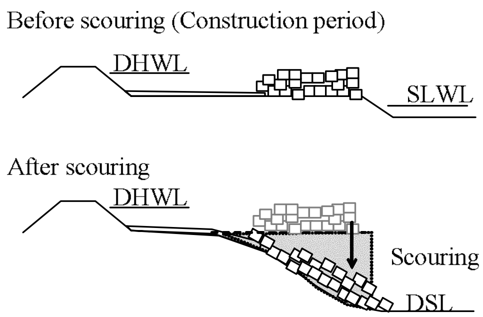
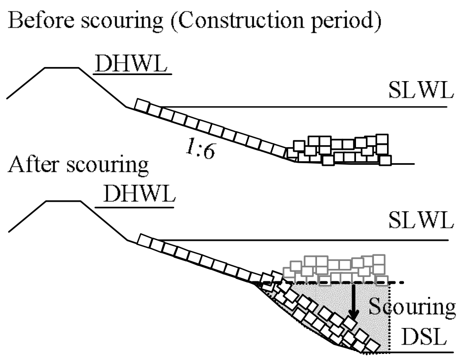
Giving conservative design considerations on the achievement of required functions, Type-B works were proposed in the present study for the right bank protection directly related to the bridge structures and Type-A works for other portions of bank protection as follows:

1) Work sites on the left bank of the Padma River (Sites-L1, L2 and L3): Type-A revetment was proposed, since the main objective is strengthening the existing less erodible bank.
2) Work sites on the right bank of the Padma River (Sites-R1, R2 and R3): Type-B

revetment was proposed, since the right bank is suffering from the current attack of the Padma and the river bank is susceptible to erosion, definite protection effects are required immediately.

3) Work site along South Channel (Site-R4): Type-A revetment was proposed, since the main objective is to prepare for attacks of the main Padma River in the future.

Table 5.1.1 Types of Proposed Revetment Works

Types	Type-A	Type-B
Description of works	 *DHWL:Design High Water Level* *SLWL:Standard Low Water Level* *DSL:Design Scour Level* Launching apron and falling apron are constructed on land above SLWL. The falling apron and a part of the launching apron fall down and protect the slope coping with the scour at the slope toe.	 Launching apron is placed on the design bank slope formed beforehand by dredging. Coping with the local scour at the slope toe the falling apron falls down and protects the slope.
Construction work	Since the works are executed on dry land, work is easier and sure. Man-power can be employed intensively.	Since the works are executed underwater use of heavy equipment is inevitable. Employment of man-power would be limited.
Cost	Medium	High
Peculiarity	Construction is sure, but the function is not always guaranteed since it is affected by the natural flow and scour. In order to compensate for the risk, excess protection materials should be stockpiled and successive maintenance would be required.	Uncertainty in construction cannot be avoided to some extent; however, once works are constructed thoroughly, definite function can be expected immediately after the construction.
Application	Recommended as ordinary bank protection works for big rivers such as the Jamuna, Ganges and Padma rivers by FAP-21 manual.	Applicable to bank protection work for protection of important structures such as bridges and weirs. The guide bund falls under this category.

Safety Levels of Revetment Structures: The bank protection works are to be designed based on a 25 year return period as recommended by the FAP-21 Manual for the standardized bank protection structures, except for the banks in front of the bridge structures, which are to be designed based on 100 years return period on the same safety level adopted for bridge design.

Whether the bank protection works are designed based on 25 year or 100 year floods, the bank would be kept stable, in association with the monitoring and maintenance repairs. The difference between the two would be the recurrence of repair works. The banks in front of the bridge structures should be designed with longer recurrence period of repairs considering the importance of the structures to be protected.

Proposed Revetment Works: As a summary, revetment works proposed for the respective work sites are shown in Table 5.1.2.

Table 5.1.2 Proposed Revetment Works for Respective Work Sites

Work sites	Site descriptions	Revetment type	Design flood
Site-L1	Left bank along the Padma R.(less erodible); protection of bridge/important structure	Type-A	100-yr.
Site-L2	Left bank along the Padma R.(less erodible); downstream termination of Site-L1	Type-A	25-yr.
Site-L3	Left bank along the Padma R.(less erodible); protection of protrusive bank-line	Type-A	25-yr.
Site-R1	Right bank along the Padma R.(highly erodible); protection of bridge/important structures; definite and immediate effects required	Type-B	100-yr.
Site-R2	Right bank along the Padma R.(highly erodible); downstream termination of Site-R1; definite and immediate effects required	Type-B	25-yr.
Site-R3	Right bank along the Padma R.(highly erodible); upstream termination of Site-R1; definite and immediate effects required	Type-B	25-yr.
Site-R4	Right bank along the South Ch.; preparation for future attack of Padma flow	Type-A	25-yr.

(2) Design Considerations

(a) General

Standards and References: Revetment Type-A and Type-B were designed for bank protection, mainly referring to the designs of river works for Jamuna Bridge and the FAP-21 Manual. The FAP-21 Manual was prepared in 2001 based on the results of studies and pilot works for about 10 years.

Structural Components of the Revetments: The proposed revetments mainly consist of slope protection and toe protection. Along the bank to be protected earth embankment is constructed, and the embankment slope and the existing bank slope are to be protected. In order to ensure the stability of the slope protection, toe protection is provided. The toe protection is composed of two structural elements, i.e., launching aprons and falling aprons.

Revetment Types A and B: Two types of revetments were designed depending on the site conditions and functions required as follows:

1) Revetment Type-A:
    - Sites L1, L2 and L3 along the left bank of the Padma River
    - Sites R4 along the right bank of the South Channel
2) Revetment Type-B:
    - Sites R1, R2 and R3 along the right bank of the Padma River

Considerations given to the design of revetments Type-A and Type-B are basically the same except for the structural design of the toe protection. Unless otherwise mentioned, the considerations given in the following subsections were commonly applied to the revetments Type-A and Type-B.

(b) Alignment of Revetments

Alignment of revetments is defined for the present study as the riverside shoulder of the embankment crest. The alignment which governs the alignment of the slope pavement was set considering smooth curvature, enough work space for construction and fewer adverse impacts to the social and natural environment.

(c) Embankment and Slope Protection

Function of Embankment: Embankments provided along the banks to be protected were planned mainly for stabilization, slope protection and use as a road for inspection and maintenance activities. Embankments at sites L1 and R1 located in front of the bridge structure are used for inspection activities and emergency repair throughout the year, whereas other embankments are used in all periods except for high flood.

Crest Elevation: Except for sites L1 and R1 in front of the bridge structure, crest elevation +6.00 m PWD was proposed considering the existing bank level (approx. +5.00 m PWD for left and right banks), so as not change the over-land flow conditions significantly during flood season and standard High Water Level (SHWL: +5.81m PWD for the convenience of use as an inspection road). Since the crest elevation is not high, the embankment would be submerged for several days a year during high flood. The dike should be armored for protection from the overtopping flows. As to sites L1 and R1, the crest elevation was proposed at +8.85 m PWD based on Design High Water Level (+7.35 m PWD for 100 year flood) considering a freeboard of 1.50m. Since this part of the embankment is designed to securely protect the bridge structure, the embankment crest should be free from flood water for emergency repair and stockpiling even in case of high floods. Furthermore the embankment should be accessible all the year round connected with the approach road.

Cross Section: Crest width was proposed at 10m for use as a road for inspection and maintenance and as stockpile yards. Slopes of the embankment were designed at 1V:3H for both the river-side and land-side slopes. The crest and both side slopes are to be paved for traffic and overtopping flows. In order to protect the land behind the embankment, toe protection works and plantation strips were also proposed for land-side slope.

Protection of River-Side Slope: Toe elevation of the river-side slope protection was proposed at +2.00 m PWD, considering Standard Low Water Level (+1.43 m PWD) and some allowance, and the work is planned to be executed in dry conditions. The slope protection generally consists of a cover-layer and filter layer placed on the embankment slope. For the present study, cubical blocks (C-blocks) were adopted for the cover layer and geo-textiles for the filter layer.

Treatment of Branch Channels: Along the right banks of the South Channel (Site-R4), several branch channels cross the embankment. These branch channels are not perennial and have water flow only in flood season. The embankment and slope protection at the channel crossing are designed as follows:

1) Embankment is cut at the crossing of branch channel to leave the existing waterway section unclosed, while the toe protection works of the revetment are constructed continuously to protect the banks.
2) In order to connect the inspection roads at the channel crossing, the waterway section is paved with bed consolidation works providing appropriate slope for traffic. The consolidation works also serve for prevention of enlargement of channel section due to scour.
3) Although the inspection road on the embankment is cut at the channel during flood season, the traffic can be maintained by use of existing rural roads.

(d) Size of Cover Layer Materials

The cover layer must provide protection against current and wave attacks. Characteristic sizes of cover layer materials that can resist these attacks were calculated separately and a larger size was adopted for the design. The sizes of cover-layer materials were applied to the slope protections and toe protections. As to C-blocks (C-blocks: cubical concrete blocks or equivalent), a minimum block size Dn = 0.3m is recommended by the FAP-21 Manual. The crushing strength of the C-blocks is specified to be equal to or more than 25 N/mm^2 for the launching apron and 15 N/mm^2 for other use.

(e) Toe Protection

Toe protection is required to protect the foundation of the slope protection works from scouring and undermining due to river flow. In case the scour depth is not much, simple riprap works or consolidation works could cope with this. However, where deep scour takes place like the Padma River, special considerations are necessary.

Applicable Measures: Falling aprons used for Hardinge Bridge (constructed in 1915) are a conventional measure for toe protection. This measure requires a large stockpile of apron materials to maintain continuous cover of the bank slope as it is naturally changed by scouring. Jamuna Bridge (constructed in 1998) adopted toe protection consisting of launching aprons placed on dredged permanent bank slopes up to a certain depth and falling aprons below the launching apron. Recently, the Flood Action Plan-21 (FAP-21, 2001) recommended a toe protection consisting of launching and falling aprons constructed on land in dry conditions as standardized revetment structures. The revetment structure was developed after the studies and field pilot works over a period of 10 years.

Proposed Toe Protection: The Jamuna-Bridge type revetment was adopted for the present Study as revetment Type-B. Revetment Type-B was applied to the erosion susceptible riverbank along the right bank of the Padma River (Sites R1, R2 and R3), expecting definite protection effects immediately after the work. Riprap damping on bamboo mattress is proposed for the launching aprons and riprap damping for falling aprons for this type considering the higher flexibility of the riprap. The revetment structures standardized by the FAP-21 Study were adopted as revetment Type-A. Revetment Type-A has been applied to other riverbanks to be protected. Cable connected concrete block mattresses were adopted for the launching apron for Type-A revetment and the mixture of several sizes of cubical blocks (cubical concrete blocks or equivalent of which crushing strength : 15 N/mm2) were used for the falling aprons.

Dimensions of Launching Aprons: The width of the launching aprons for revetment Type-A was chosen 20m according to the recommendation by the FAP-21 Manual. The launching apron units should be placed on a heavy geo-textile. The launching apron was further extended for 10m under the falling apron. Both the geo-textile mats and the interconnected cover units need sufficient anchoring at the toe of the upper revetment. The launching apron for revetment Type-B is described separately in the item (f).

Dimensions of Falling Aprons: Following the assumptions that the elements of falling aprons would cover the scour holes at the slope of 1V:2H with a thickness of 1.5 x (characteristic material size: Dn), the required volume of a scour protection blanket per meter of bank was estimated geometrically.

Falling Aprons Designed: The required volume of the falling aprons was first estimated as mentioned above based on the site conditions and revetment types, according to the formulas discussed above, and then the width and thickness of the apron was determined. As to the thickness of the apron, the river-side 2/3 of apron was thickened reducing the thickness of land-side apron so that the total volume should not less than the required volume.

(f) Lower Slope of Revetment Type-B

Lower Slope of Type-B Revetment: Type-B revetment is applied to the riverbank along the right bank of the Padma River (sites R1, R2 and R3). For the construction of Type-B revetments, a permanent bank slope is formed under the water (lower slope) prior to placing the launching apron. Determination of the lower slope gradient is an important issue, since the gradient definitely governs the dredging volume. For the present study, the lower slope was set at 1V : 6H, referring to the experience of the Jamuna Bridge works, recent examples of other bridge projects, and suggestion by the FAP-21 study as introduced in the following paragraphs.

Experience of Jamuna Bridge: Channel dredging for the West Guide Bund started at the end of December 1995, and five failures had occurred in the permanent slope during the work period of 1.5 months. In order to cope with these failures two measures which markedly reduced the failures in the permanent slope were taken, i.e., (1) modification of slope gradient to be more gentle as follows and (2) improvement of dredging methods.

Recent Examples: The permanent slopes of the guide bank works of the bridges constructed recently are listed below.

Bridge	Upper slope (above water)	Lower slope (under water)
Jamuna Br.	1 : 3.0	1 : 5.0 & 1 : 6.0
Bhairah Br.	1 : 4.0	1 : 6.0
Paksey Br.	1 : 2.5	1 : 50

Suggestion by the FAP-21/22: With regard to the underwater slope, Evaluation Report of the FAP-21/22, Dec. 2001 suggests as follows:

Significant slope slides occurred during dredging of the 1V : 3.5H underwater slopes for the Jamuna Bridge guide bunds and Sirajganj Town Protection. A likely reason for this is the fast excavation of the slope which did not allow the soil to consolidate sufficiently. Therefore, underwater slopes should not be built steeper than about 1V : 6H.

(g) Construction Method of Revetment Type-B

As to the construction methods for Type-B revetments on the right bank, two alternative methods were considered, i.e., construction on the bank (Scheme-R1) and in the river (Scheme-R2). For Scheme-R1, design alignment of bank should be moved back by about 500 m to provide enough space for revetment work. Owing to this, bridge length becomes longer and a wider land area has to be acquired and many village houses to be relocated, while for Scheme-R2, bridge length can be shortened and land and house compensation is minimized, though difficult works in river must be executed during the limited low-water period.

Direct costs were preliminarily estimated and compared to each other. In conclusion Scheme-R2 was selected considering the lower total cost of river and bridge works and fewer social issues. For the construction of Scheme-R2, careful arrangement and work time control would be necessary.

(h) Dredging Level of Revetment Type-B

Sites R1, R2 and R3 are located on the right bank facing directly the main Padma River. For these sites Type-B revetment is proposed, for the riverbank is highly susceptible to erosion and immediate bank protection effects are expected. For a Type-B revetment, the launching apron is placed on the design slope prepared beforehand by dredging and the falling apron is at the foot of the launching apron. These works are executed under the water. According to the earlier study, the slope of the launching apron was determined at 1V : 6H.

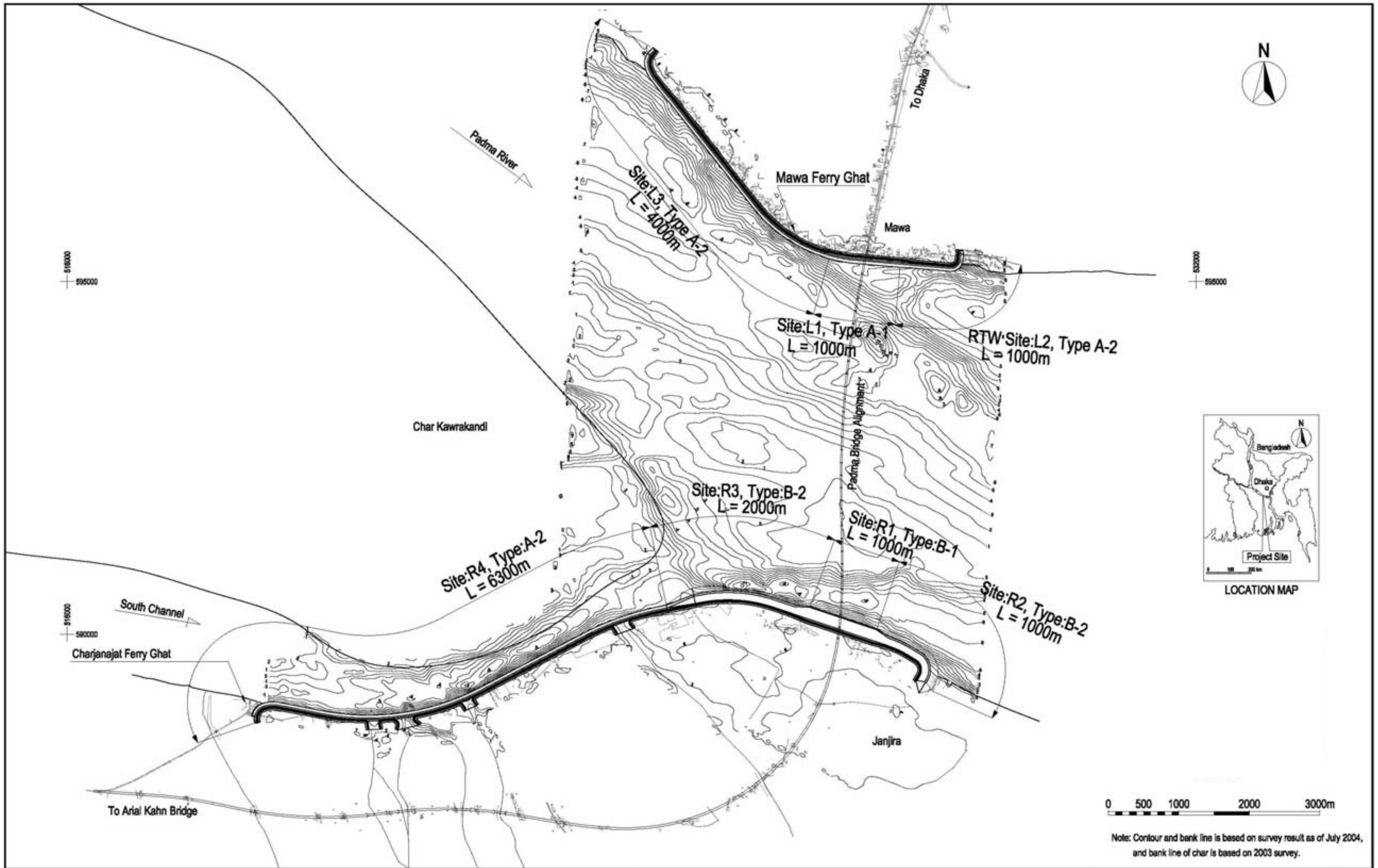
Dredging Level for Site-R1: In order to protect the riverbank adequately from erosion, it is desirable to place the launching apron up to the depth of design maximum scour, by dredging the earth in front of the revetment. However, the deep dredging costs a lot and requires much time. For the Type-B revetment at Site-R1 (named TypeB-1), dredging level was decided at -25 m PWD considering the maximum dredging depth of a cutter suction dredger (assumed at 30 m) and the estimated natural scour level. This bed elevation corresponds to about 1.5 m below the riverbed under natural scour conditions. With this revetment, the natural scouring which would take place in relatively longer stretch is coped with by the launching apron placed on the permanent slope and the structure induced scours which would occur locally are protected against by the falling apron.

Dredging Level for Sites R2 and R3: As for the revetment at sites R2 and R3 (named Type B-2), dredging level was set at -15.0 m PWD considering the capacity of dredgers and the workable period in a year. The revetment works at sites R1, R2 and R3 must be completed within one dry season.

(3) Design Drawings

Based on the results of design considerations, river facilities were preliminarily designed. In order to demonstrate the general design features, some representative drawings of revetment works are shown as follows:

Figure No.	Title
5.1.4	General Plan of River Works
5.1.5	Standard Design of Revetment: Type A-1
5.1.6	Standard Design of Revetment: Type B-1
5.1.7	Miscellaneous Details



JAMUNA MULTI PURPOSE BRIDGE AUTHORITY THE PEOPLE'S REPUBLIC OF BANGLADESH	THE FEASIBILITY STUDY ON THE CONSTRUCTION OF THE BRIDGE OVER THE PADMA RIVER IN THE PEOPLE'S REPUBLIC OF BANGLADESH	JAPAN INTERNATIONAL COOPERATION AGENCY (JICA) NIPPON KOEI CO., LTD. in Association with CONSTRUCTION PROJECT CONSULTANTS, INC.	TITLE : River Works General Plan of River Works	DATE : September-2004	SHEET NO.

Figure 5.1.4 General Plan of River Works

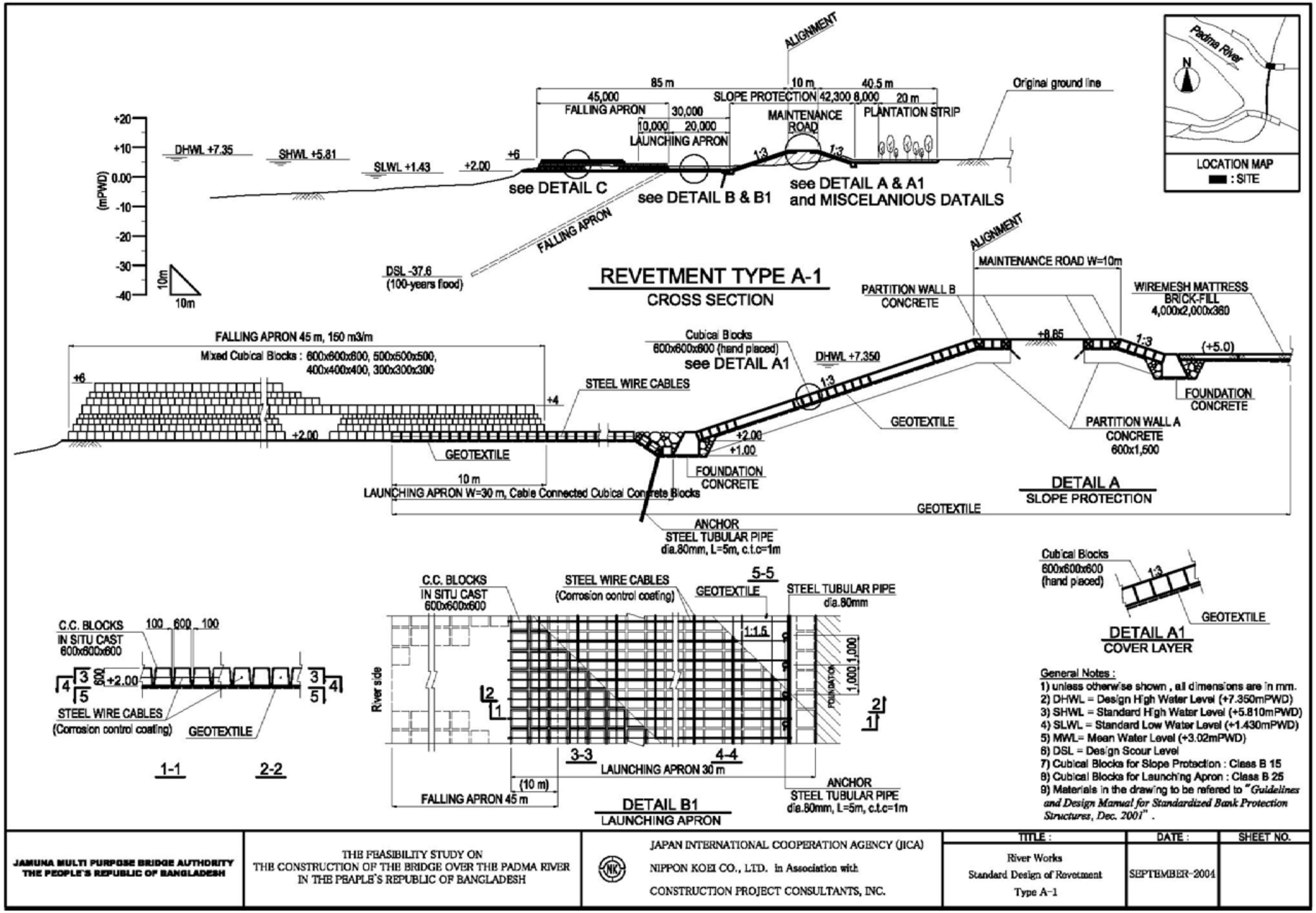


Figure 5.1.5 Standard Design of Revetment: Type A-1

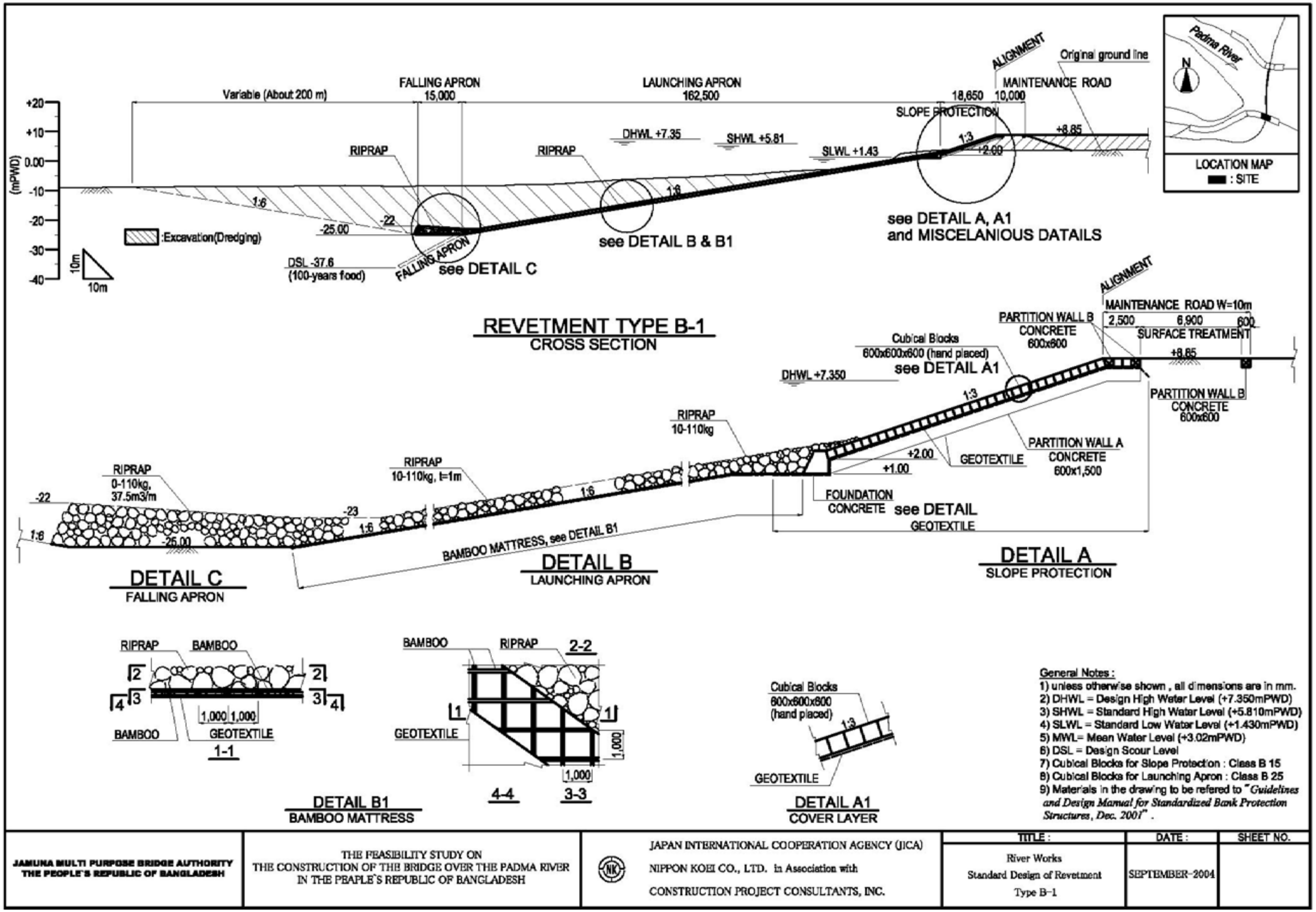


Figure 5.1.6 Standard Design of Revetment: Type B-1

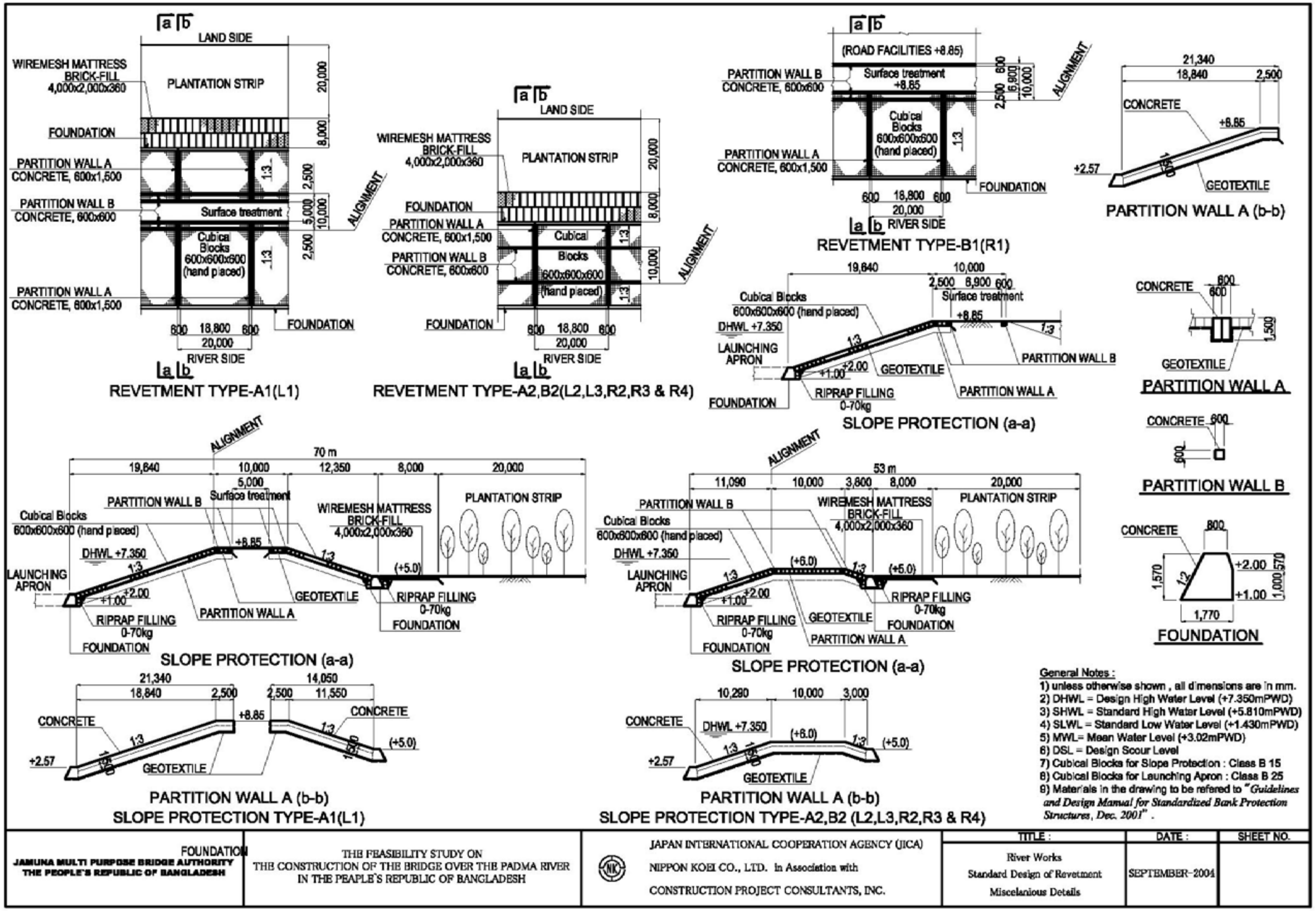


Figure 5.1.7 Miscellaneous Details

5.1.3 Construction of River Works

(1) Execution Method of River Works

Revetments are adopted as the bank protection works for Padma Bridge. The revetments mainly consist of slope protection, launching aprons and falling aprons, and is classified into Type-A and Type-B depending on the construction method of the launching and falling aprons.

For the Type-A revetments the launching apron and falling apron are constructed on land above Standard Low Water Level (SLWL), while for the Type-B revetments the launching apron is placed on the design bank slope shaped beforehand by dredging. Major work items and execution methods for these works are presented below.

(a) Execution Methods for Revetment Type-A

1) Site clearing and excavation
2) Embankment
3) Concrete foundation and partition
4) Slope protection: Placing geo-textile filter and concrete blocks on it
5) Launching apron: Placing geo-textile filter and connected concrete blocks on it
6) Falling apron: Placing concrete blocks
7) Surface treatment: For embankment crest as maintenance road and stock pile yards
8) Land side protection: Slope protection, toe protection, etc.

(b) Execution Methods for Revetment Type-B

1) Temporary works: Driving steel piles to create a still water zone for dredging and revetment works, and extracting steel piles after the work.
2) Dredging and Excavation
3) Site clearing
4) Embankment
5) Concrete foundation and partition
6) Slope protection: Placing geo-textile filter and concrete blocks on it
7) Launching apron: Furnishing and placing bamboo mattresses, and riprap work on the mattresses
8) Falling apron: Riprap
9) Surface treatment: For embankment crest as maintenance road and stock pile yards
10) Land side protection: Protection of land side slope, toe protection, etc.

(2) Preliminary Construction Schedule

(a) Review of Jamuna Bridge Works

The experience of the construction of Jamuna Bridge provides us invaluable information. In view of this, monthly reports during the construction period of Jamuna Bridge were reviewed. There are mainly four work groups of river works for Jamuna Bridge, and plan and actual progress of each work group is summarized below.

No.	Work group	Plan	Actual progress
1	Bhuapur Hard Point Works	Dec. 1994 – Jun. 1995	Jan. 1995 – May 1995
2	Work Harbor on East Bank	Oct. 1994 – May 1995	Oct. 1994 – May 1995
3	West Guide Bund	Oct. 1995 – Apr. 1996	Oct. 1995 – Oct 1996
4	East Guide Bund	Oct. 1996 – Apr. 1997	Oct. 1996 – May 1997

(b) Workable Period

Water level is the most dominant factor to determine the workable period for river works at a site. According to the water level records at Mawa, the workable period above several water levels were estimated as follows:

Water level (m PWD)	Workable months by dependability (months)		
	90 %	50 %	10 %
1) +5.00 (Ground L.)	8.0	9.0	10.0
2) +4.00	6.7	7.5	8.2
3) +3.02 (Mean W.L.)	5.2	5.9	6.8
4) +2.00	2.2	3.4	4.5

(c) Work Groups

The river works for Padma Bridge can be grouped as follows:

1) **Preparatory Works:** Prior to the bank protection works, several preparatory works are necessary to be executed.
2) **Left Bank Works:** Revetment Type-A is proposed for left bank works of the Padma River. Since the revetment Type-A is executed on land in principle and the bank is less erodible, the work schedule would not be so tight because of relatively longer workable period on the left bank.
3) **Right Bank Works:** Revetment Type-B is proposed for right bank works of the Padma River. The revetment Type-B includes temporary works and dredging works prior to the underwater works for the launching aprons and falling aprons. These dredging and underwater works must be executed under still water conditions in the dry season. In addition all the bank protection works at sites R1 through R3 must be completed in one dry season, since the right bank is erodible. The right bank work schedule would be the most tight and critical for construction.
4) **South Channel Works:** Revetment Type-A is proposed for right bank works of the South Channel covering work site R4. The revetment Type-A is executed on land in principle. The work schedule would not be so tight considering the workable period on the right bank.

(d) Overall Construction Schedule

Taking into account work groups and their features, sequence of works is proposed as follows:

- 1st year: Preparatory works
- 2nd year: Left bank works
- 3rd year: Right bank works
- 4th year: South channel works

With regard to the construction schedule of the river works, there would be issues to be discussed further from the overall viewpoint of the construction schedule including bridge and approach road works. The issues are (1) the execution order of right and left bank works of the Padma River and (2) timing of execution of South Channel works.

As to the execution order of the right and left bank works of the Padma River, there is an argument which works should be constructed first. From the river work viewpoint, the right bank is preferable to be executed after the experience of the left bank works, since the right bank work schedule would be the most tight and critical. On the other hand, from the viewpoint of stabilization of site conditions, it is preferable to execute the right bank as early as possible, since the banks are susceptible to change.

As to the timing for execution of the South Channel works, Site-R4 works were proposed to be executed as a component of the present bridge project, since there is no guarantee for financing to implement Site-R4 works in the future separately from the present project. The Site-R4 works are duly necessary to check south-word shifting of the South Channel and ensure the stability of the bridge structures and the right approach road when the South Channel becomes a main stream of the Padma in the future. However, erosion of the South Channel is not active now and the Site-R4 works constructed at the same time of bridge construction would be left for a long period without full use. It is reasonable to execute the works when they become necessary based on the monitoring results of the channel behavior, provided there is a definite financial guarantee for future execution. By so doing, the latest developments in bank protection technology could also be incorporated.

(3) Further Studies

In the present study, designs of river facilities were made preliminarily at the Feasibility Study level. Through the studies made so far, the following studies were recognized to be necessary in the succeeding stages before construction in order to make the project more economical and to provide a more definite technical basis:

1) **Implementation of Model Studies:** Mathematical models were prepared within the scope of the present study. The models should be upgraded to meet with the requirements to be expected in the following stages, mainly for river morphologic prediction. In addition to the mathematical modeling, physical model tests would also be required to provide more definite design data for layout of river facilities and structural details.
2) **Risk Analysis:** Since the right riverbank is highly mobile, the design layout of bridge structures as well as river facilities may need to be revised and their construction program to be adjusted accordingly, considering the latest bank location and river features at the time of bridge construction. Therefore, possible risks that could be encountered should be analyzed and measures to reduce and cope with the risks should be studied in the definite design stage.
3) **Development of low cost bank protection works:** For the protection of the bridge structure, Type-B revetment works were proposed. The Type-B works were applied to the Jamuna Bridge, for which launching and falling aprons were constructed under water. However, the Type-B works have considerably higher cost compared to the Type-A works. If the low cost works are developed for the bank protection around the bridge structure, it would contribute much to the reduction of the project cost. Applicability of Type-A works instead of Type-B works should be examined, monitoring the existing Type-A works in the field and enhancing its function for the protection of important structures.

5.2 PRELIMINARY DESIGN OF PADMA BRIDGE

This Section deals with preliminary designs of highway bridge alternatives for the Padma Bridge. The bridge alternative for the railway provision is dealt with in the subsequent Chapter 10 of this report.

5.2.1 Design Criteria and Standards adopted in Preliminary Design

(1) Bridge Width for Preliminary Design alternatives

Preliminary design alternatives are discussed in the subsequent Subsection 5.2.4. These alternatives were prepared according to the following bridge width conforming to the Asian Highway Standard or Roads and Highways Department (RHD) of Bangladesh.

(a) Bridge Width based on Asian Highway Standard

The Padma Bridge would be built on the Asian Highway (AH) Route No. A-1 that is planned under UNESCAP. The Study Team has examined the standard bridge widths with that stipulated in AH standard by UNESCAP and with Bangladesh highway standard by RHD. Figure 5.2.1 shows the standard section of a 2 lane dual carriageway with reduced median width as stipulated in the AH standard.

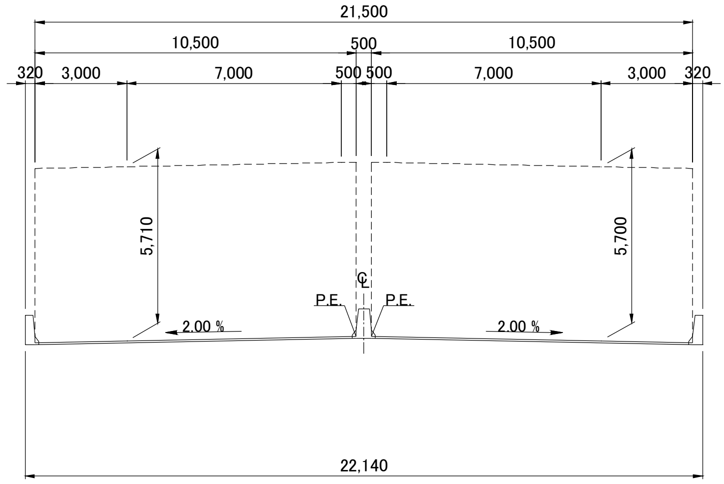


Figure 5.2.1 Bridge Cross Section based on AH Standard without Median Strip with Reduced Median Width (Base Case)

The bridge cross section in the above is considered as a "Base Case" for examining the standard bridge type and span length, which is discussed in the subsequent Subsection 5.2.2 and 5.2.3.

(b) Bridge Width based on RHD Standards

In addition to the above bridge widths on the basis of AH standard, one more option will be examined to determine the minimum investment case as shown Figure 5.2.2. This option has a 2 lane dual carriageway width of 7.3 m specified in RHD Standard and minimum side belt of 0.5 m on each edge side.

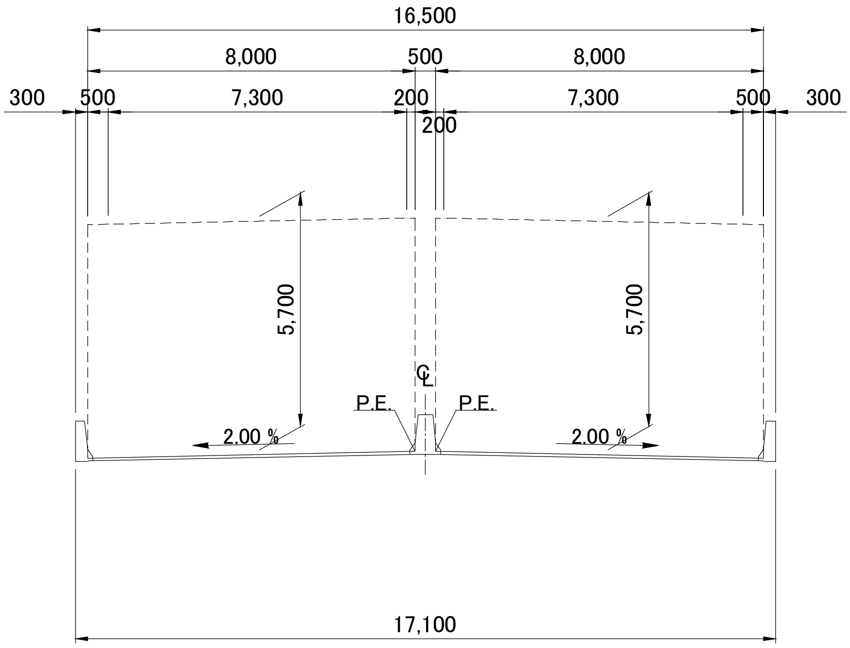


Figure 5.2.2 Bridge Cross Section based on RHD Standards

(2) Design Criteria and Standards

The preliminary design in the Study would be conducted on the basis of AH standard and RHD standards, the latter is similar to AASHTO standards. Standards adopted by Japan Road Association (JRA), BSI and Indian Road Congress (IRC) would be used as supplementary ones.

Design Loads

i) Dead Load

The following unit weights are to be used for the preliminary design.

Table 5.2.1 Unit Weight by Structural Items

Items	Unit Weight
Steel, cast steel	77.0 kN/m^3
Aluminum	27.5 kN/m^3
Reinforced concrete	24.5 kN/m^3
Plain concrete	23.0 kN/m^3
Asphalt pavement	22.5 kN/m^3
Electric power transmission line and supports	13.0 kN/m^3
Gas pipeline and supports	7.0 kN/m^3
Telecom fiber optics	Not significant

ii) Live Load

According to AH standard and RHD standards, HS 20-44 stipulated by AASHTO standards are to be used.

iii) Impact

Impact effect is calculated based on the following:

Impact fraction: I = $\frac{15}{L+38}$

In which,

I = impact fraction (Max. 30 %)
L = Length in meters of the portion of the span that is loaded to produce the maximum stress in the member.

iv) Longitudinal Forces

Not significant.

v) Centrifugal Forces

Not significant.

vi) Thermal Effect

Referring to AASHTO sixteenth edition, 1996, the following temperatures are considered.

Temperature Rise: 17ºC
Temperature Fall: 22ºC

vii) Earthquake Effect

The following statically equivalent seismic force will be considered based on the report "Preliminary Study of Seismic Design Parameters for Padma Bridge Corridor (Aricha-Goalundo, Mawa)" by the Department of Civil Engineering, Bureau of Research, Testing and Consultation, BUET as per Appendix B of the Prefeasibility Report, Padma Bridge Study Phase I, February 2000, RPT, Nedeco & BCL.

Horizontal Accelleration Coefficient = 0.125 g

viii) Wind Loads

The Study Team collected the wind records from 1964 to 2003 from Bangladesh Meteorological Survey Department and produced Figures 5.2.3 and 5.2.4 for Faridpur observatory and Dhaka observatory, in which a wind velocity of 95 knots (48.9 m/sec) was recorded in October 1964 and May 1972 at Faridpur observatory, which is higher than AASHTO based wind velocity of 100 miles/hour (44.7 m/sec).

The Power Grid Company of Bangladesh has been adopting a design wind velocity of 44.4 m/sec to 70 m/sec. For the design of electric power transmission lines for the Jamuna Bridge, wind velocity of 70 m/sec was used for electric pole design while 62.6 m/sec was for overhead wires.

In Bangladesh National Building Code 1993, the basic wind speed of 200 km/h (55 m/sec) is stipulated.

As for the bridge structure design, the basic wind velocity of PGCB is too high. Accordingly the basic wind speed of 55 m/sec, which is higher than the maximum wind record in Faridpur, is used for this preliminary design.

Basic Wind Velocity for bridge structural design = 55 m/sec

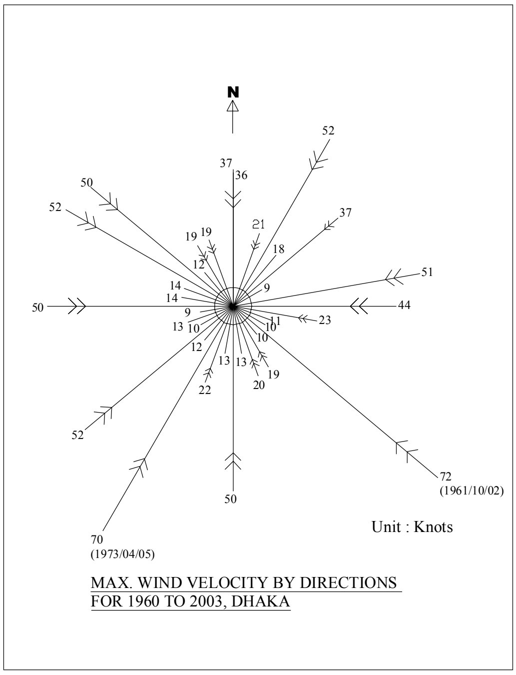


Figure 5.2.3 Maximum Wind Velocity by Directions for the Period of 1960 to 2003, Dhaka

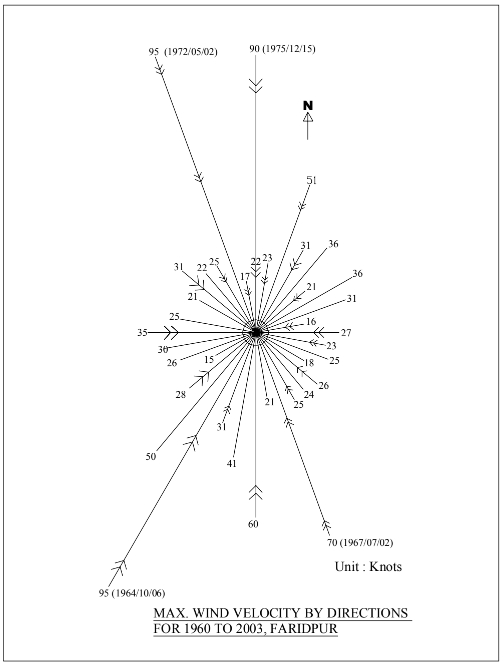


Figure 5.2.4 Maximum Wind Velocity by Directions for the Period of 1960 to 2003, Faridpur

ix) Significant Wave Height

From the S-M-B monogram, the significant wave height is obtained in the following when the wind velocity is 95 knots and fetch is 25 km.

Significant Wave Height: $H_{1/3} = 5.5$ m

x) Ship Collision Force

As a result of discussions with BIWTA, the following tonnage and speed are to be considered for the calculation of ship collision force against piers located in the navigable course.

Dead Weight Tonnage = maximum 1,500 ton

Vessel Speed = maximum 10.0 knots = 16.9 fps

Impact Level of Ship collision Force = SHWL + 9.0 m

The following formula is to be applied based on the Guide Specification and Commentary for Vessel Collision Design of Highway Bridges Volume I: Final Report, February 1991 by AASHTO.

$$Ps = 220(DWT)^{1/2}\left[\frac{V}{27}\right]$$

$$= 5,333 \text{ kips} = 23.7 \text{ MN}$$

Where,
Ps = equivalent static ship impact force (kips)
DWT = deadweight tonnage of ship (tones)
V = ship impact speed (fps)

xi) Parameters related to River Engineering

Water Levels

Design High Water Level (100 year return period): DHWL = +7.35 m PWD
Standard High Water Level: SHWL = +5.81 m PWD
Mean Water Level: MWL = +3.02 m PWD
Standard Low Water Level: SLWL = +1.43 m PWD

Water Discharge and Velocity

Design Discharge (100 year return period): Qd = 134,400 m^3/sec
Design Water Velocity (100 year return period): Vd = 4.6 m/sec

Scour Levels

Adjacent to Riverbank (100 year return period)
300m from Riverbank: Zs = -37.56 m PWD
In Middle of River (100 year return period): Zs = -23.63 m PWD

5.2.2 Examination of Preferable Bridge Type and Span Length

(1) Procedures for Selection of Preferable Span Length

Comparison of steel type bridges and concrete type bridges are made in the appendices of Volume I of this report, in which concrete type bridges are concluded as favorable bridge types mainly from the view points of cost performance and material availability in Bangladesh.

Construction costs per one meter of span length were estimated and compared for the combinations of span and type below in order to find out the most preferable span length.

Table 5.2.2 Concrete Type Bridge by Span Length to be Considered

SPAN LENGTH	Continuous Box	Extradosed	Cable Stayed
100m	O		
120m	O		
140m	O	O	
160m	O	O	
180m		O	O
200m		O	O
220m			O
240m			O

(2) Considerations on the Results

(a) Base Case

The Span-Cost Graph in the Figure 5.2.5 shows the span lengths and the total unit costs per meter of super- and substructure of the Base Case.

The longer the span is, then the higher the unit cost for superstructure becomes almost linearly for each superstructure type

On the other hand, unit substructure costs generally decrease as the span becomes longer but the rate of decrease in substructure costs diminishes, or costs even increases in some cases, with increasing span.

The minimum of the total unit cost, which is a summation of the super- and substructure unit costs, comes out at 180 meter span for an extradosed girder bridge type.

(b) Conclusion from the comparison

The preliminary design will be conducted for the span of 180 meters, and an extradosed girder bridge is to be regarded as the standard superstructure type for the main bridge over the Padma.

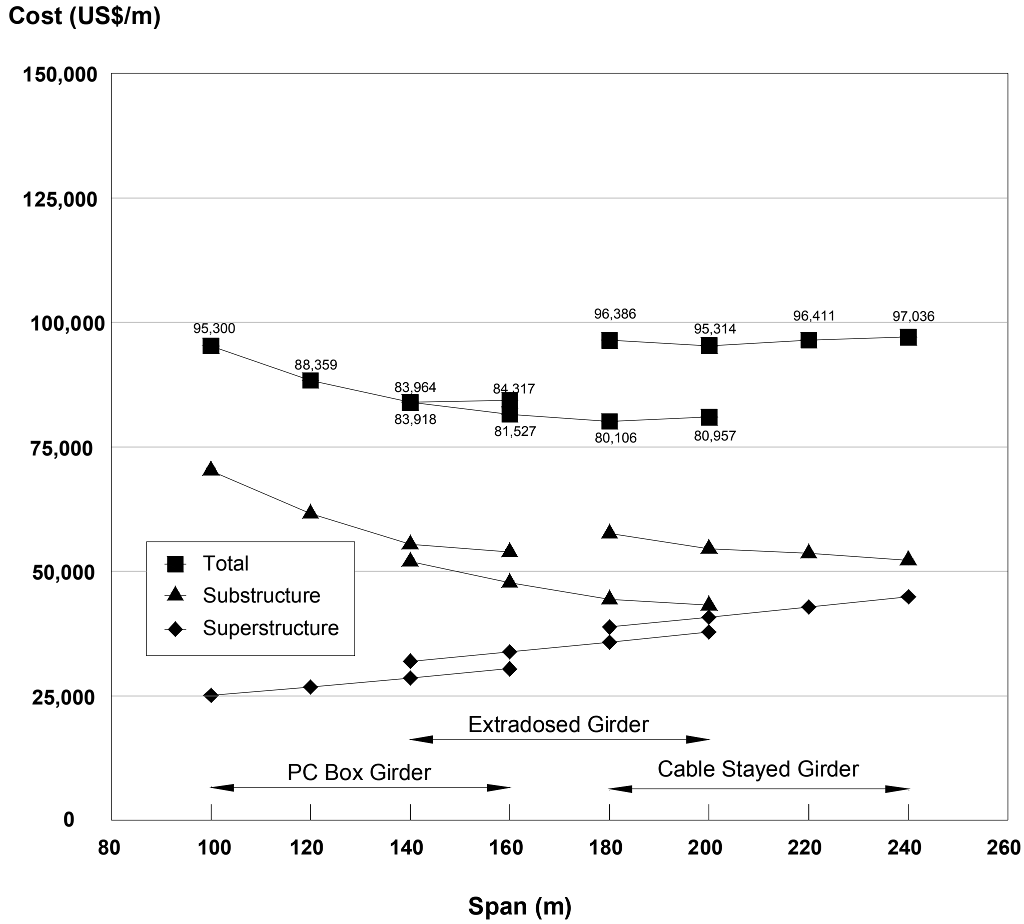


Figure 5.2.5 Cost-Span Graph for Padma Highway Bridge, Width = 21.5 m

Table 5.2.3 Cost-Span Relation for Padma Highway Bridge, Width = 21.5 m

(Unit: US$ /m)

Span (m)	Bridge Type	Superstructure	Substructure	Total
100	Box Girder	25,089	70,211	95,300
120		26,790	61,569	88,359
140		28,566	55,398	83,964
160		30,415	53,902	84,317
140	Ex. Girder	31,958	51,960	83,918
160		33,850	47,677	81,527
180		35,752	44,354	80,106
200		37,792	43,165	80,957
180	CS. Girder	38,785	57,601	96,386
200		40,783	54,531	95,314
220		42,801	53,610	96,411
240		44,862	52,173	97,036

5.2.3 Standard Bridge for Preliminary Design

Based on the results from the preferable span length and type, preliminary designs of the extradosed bridge have been made for the highway bridge. The main reasons to decide on the extradosed girder as the standard structure for the Padma Bridge are as follows:

- Extradosed girder shows less cost than other bridge types as shown in Figure 5.2.5 and Table 5.2.3.
- Concrete strength required for the extradosed girder is just same as the PC box girder, which is a common bridge type for long span bridges in Bangladesh. No issue will take place in the quality of concrete in construction period.
- Exterior cables are used for the extradosed girder and therefore future maintenance is relatively easier than a PC box girder of which cables are generally encased in cable conduits followed by cement mortar grouting as in the Jamuna Bridge and other PC box girder bridges in Bangladesh.

(1) Superstructure

The extradosed bridge with a width of 21.5 m and 180m long span is shown in Figure 5.2.6.

(2) Substructure

The preliminary design of the substructure is based on voided reinforced concrete piers, reinforced concrete pile caps and driven 3,000 mm nominal diameter tubular steel piling. The various general arrangements for each of the bridge alternatives are summarized in Section 5.2.4

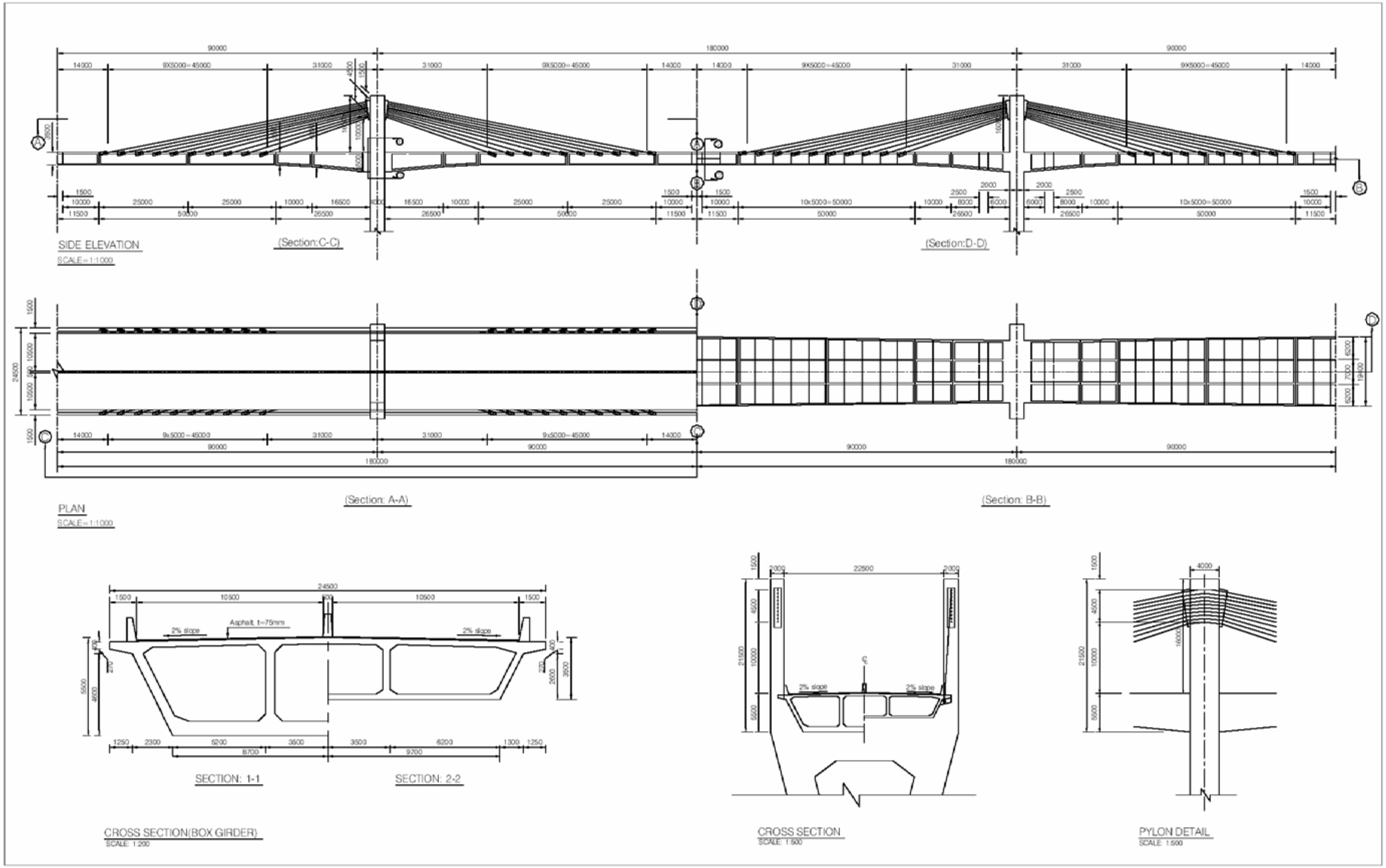


Figure 5.2.6 General View of Extradosed Bridge for the Base Case

5.2.4 Bridge Design Options

As stated in 5.1 the main bridge portion is 5400 m long. This is a multiple of 180 m, which is the most preferable span length recommended in 5.2.2.

Taking into account the thermal expansion, one continuous superstructure as a whole cannot be longer than approximately 750 meters, thus (4x180m=720m) constitutes a standard module of superstructure, and expansion joints must be placed at a spacing of 720 meters or less between the neighboring structures.

In case of adopting a cable stayed girder bridge, as requested by the Bangladesh side, in a part of the main bridge, one module of 720 meters will be substituted by a composite cable stayed girder, of which the span arrangement is (48+132+360+132+48=720m).

Regarding the requirements for vertical alignment, major portions of the bridge have to keep the maximum slope of 3%, but where the slope is not long enough such as the approach viaducts 4% is allowed for the roadway.

At least one, but preferably three navigational routes have to have a vertical clearance of 60 feet, and 40 feet must be maintained for the remainder of the whole navigational course of 4800 meters wide.

As bridge costs are affected by the width of a bridge, an additional alternative for the minimum width was prepared in order to examine the minimum investment cost.

Consequently the following three combinations, all of which satisfy the above-mentioned requirements, are to be examined;

- Alternative-H1: PC Extradosed Girder Bridge based on Bridge Width of Asian Highway Standard with Reduced Median Width
- Alternative-H2: PC Extradosed Girder Bridge and PC Cable Stayed Girder Bridge based on Bridge Width of Asian Highway Standard with Reduced Median Width
- Alternative-H3: PC Extradosed Girder Bridge based on Bridge Width of RHD Standards

(1) Alternative –H1: PC Extradosed Girder Bridge based on Bridge Width of Asian Highway Standard with Reduced Median Width

(a) Superstructure

Figure 5.2.7 shows a general arrangement of spans for Alternative-H1. A general view of the conjunction portion between the viaduct and the standard bridge is shown in Figure 5.2.8.

This is the base case and also the most economical where all the spans at the main bridge portion are composed of PC Extradosed Girders, and the viaducts are of PC T-section girder bridges.

Five modules of 720 meters long and two modules of 540 meters long constitute the main bridge portion of 5400 meters to be preferably spanned by the 180 m extradosed bridges.

The viaducts on both banks are of PC T-girder type, whose span is 30 meters. The left bank viaduct is 60 meters long, and the right 120 meters.

The total bridge length is consequently 5580 meters.

The cross-section of the deck is 21.5 meters, which can accommodate 2-lanes for vehicles in each direction with margins for the cable anchors.

(b) Substructure

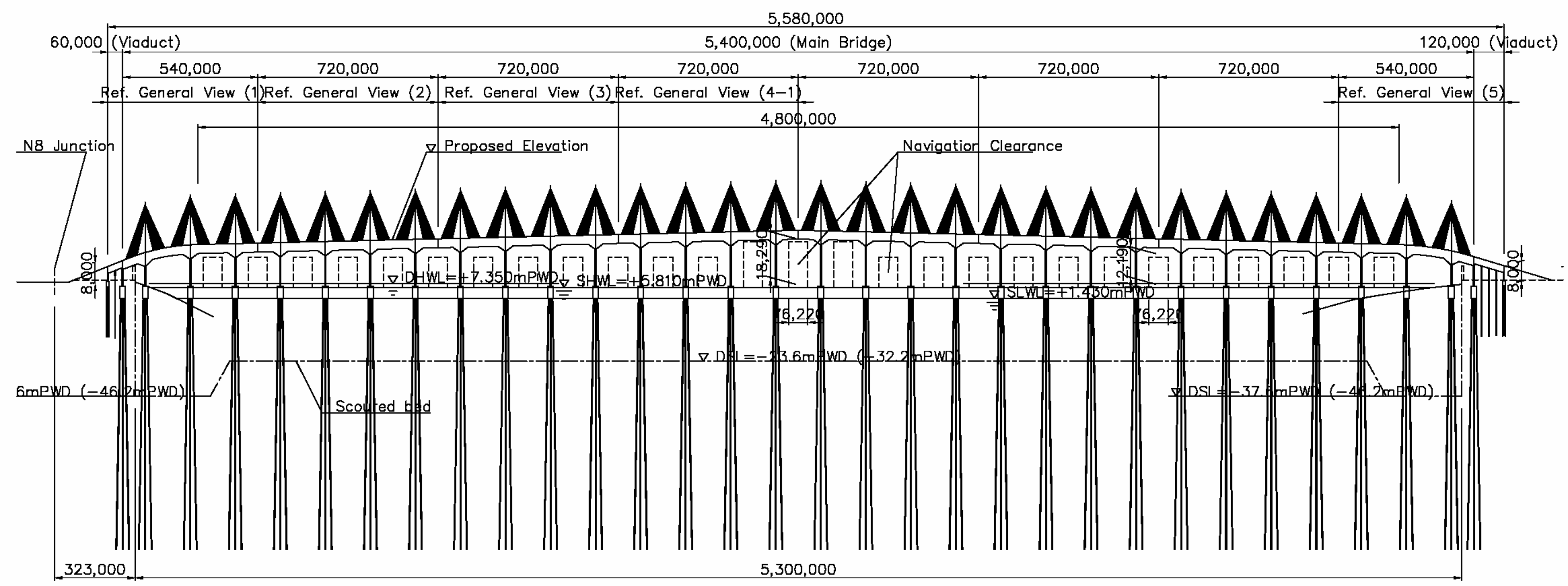
The Figures 5.2.9 and 5.2.10 show the general arrangements of the river edge and mid river substructures respectively. A more extensive foundation is required for the piers at the edge of the river due to the greater scour resulting in a lower design river bed level.

In order to minimize the effect of river flow the pile caps are provided with rounded corners. The 3150mm diameter hollow steel tubular piles have a maximum rake of 1 in 6 to the vertical, and are to be in-filled with a mass concrete toe plug to generate additional end bearing.

The four edge river piers comprise a 17.5m x 37.0m x 7.1m deep pile cap supported on 12 number piles with average maximum wall thickness of 58mm. Average toe elevation is -87.5m PWD providing an approximate depth of embedment of 43m below the design river bed level.

The twenty-six mid river piers comprise a 17.5m x 31.5m x 7.1m deep pile cap supported on 8 number piles with average maximum wall thickness of 63mm. Average toe elevation is -83.5m PWD providing an approximate depth of embedment of 53m below design river bed level.

The twin pier columns forming the base of the extradosed cable towers are each 5.5m x 6.0m in overall cross sectional area with minimum wall thickness of 1.25m, with similar connecting cross-beam 5.5m deep. This cross beam may be alternatively hidden within the superstructure.



(All dimensions are in millimeters)

Figure 5.2.7 General View (Alternative-H1)

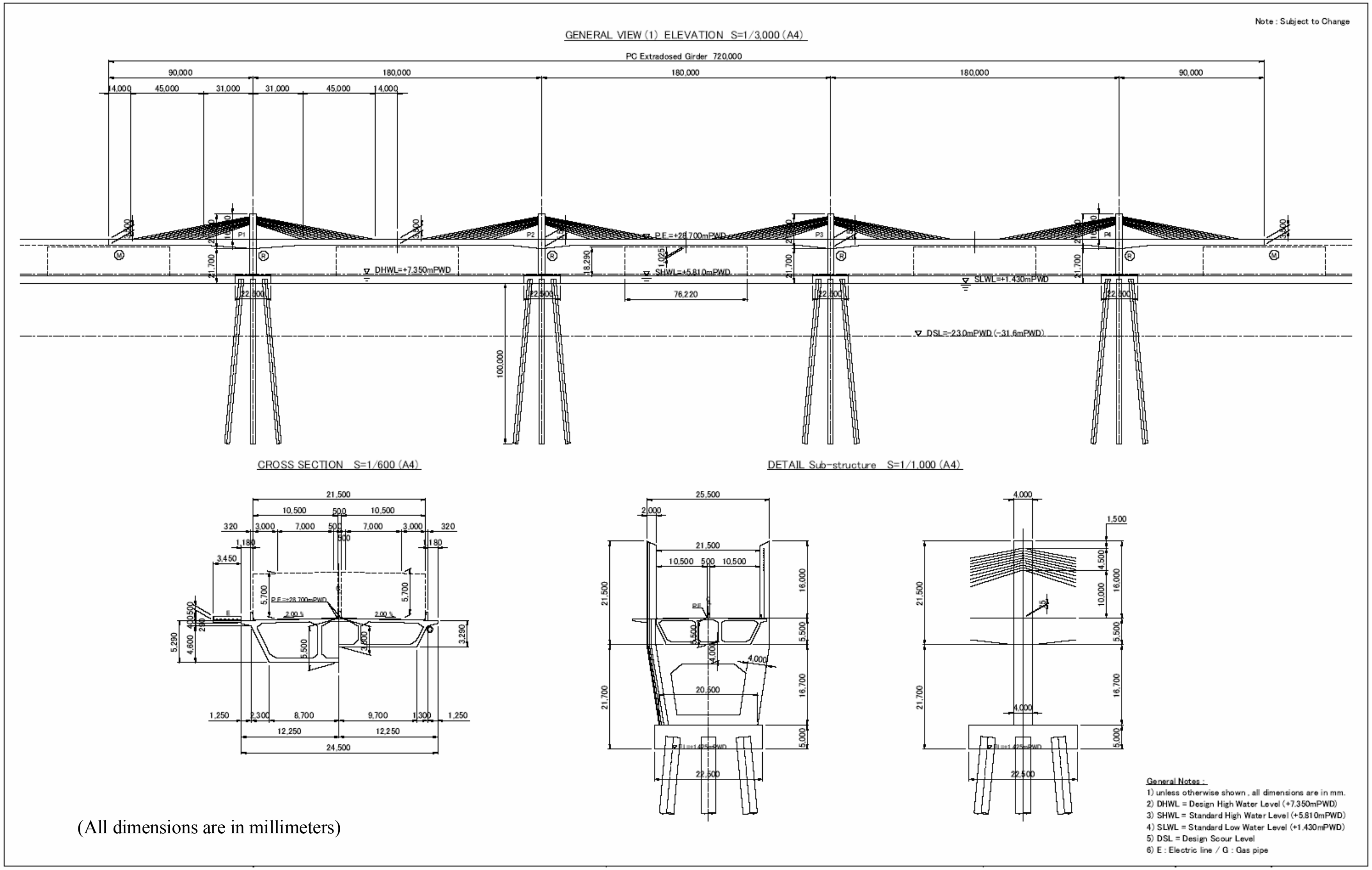


Figure 5.2.8 General View of Viaduct and Standard Bridge Portion (Alternative-H1)

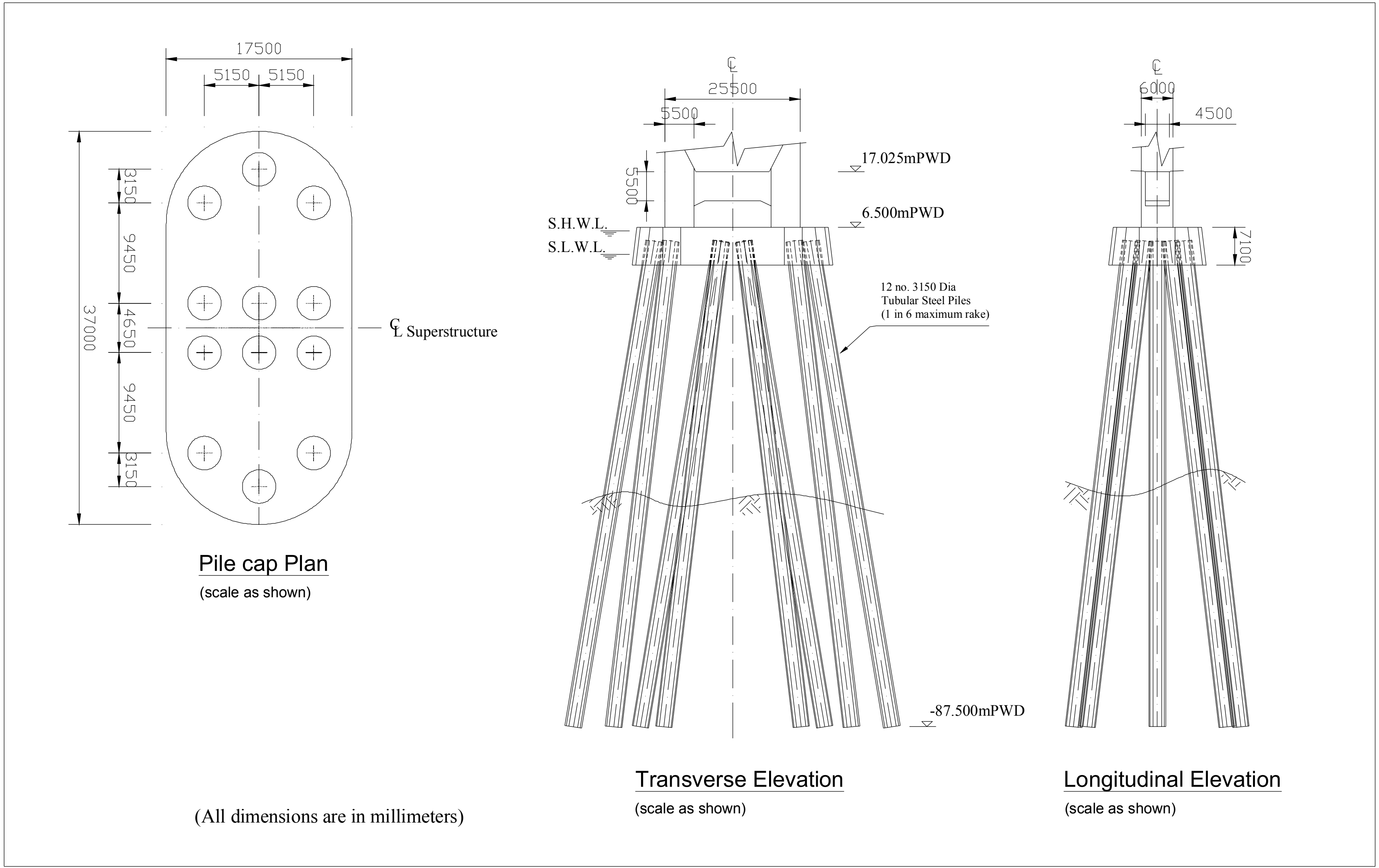


Figure 5.2.9 Edge River Substructure General Arrangement (Alternative-H1)

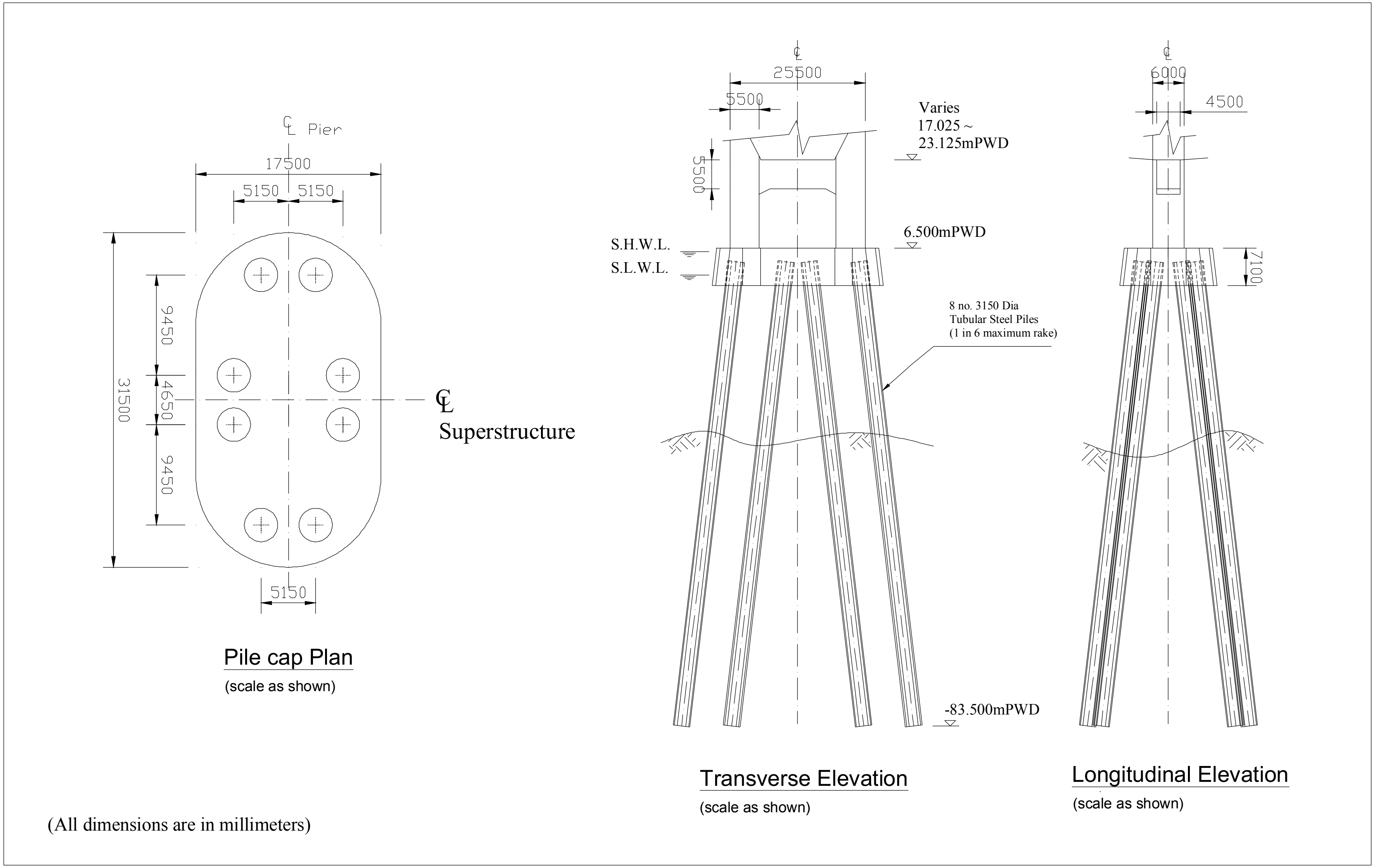


Figure 5.2.10 Mid-River Substructure General Arrangement (Alternative-H1)

(2) Alternative –H2: PC Extradosed Girder Bridge and PC Cable Stayed Girder

(a) Superstructure

The Figure 5.2.11 shows a general arrangement of spans for Alternative-H2. A general view of the conjunction portion between the viaduct and the standard bridge is the same as Alternative-H1 shown in Figure 5.2.8.

The only difference from Alternative-1 is that one module of 720 meters long is substituted by a composite cable stayed girder bridge, and all other elements are exactly same as in Alternative-1.

The cable stay bridge spans 360 meters which is unnecessarily wide for the required navigation clearance, and on the contrary several piers must be added in the back spans in order to pull down the cable tension forces coming from the main span. This substitution will surely raise the construction cost.

Other disadvantages of the substitution by the composite cable stay bridge are repaint work of the steel deck girder and aerodynamic instability. A steel deck girder is adopted for the main span to reduce the self-weight and this element usually has to be repaired every twenty years. As the cable stay bridge is much longer than the extradosed bridge, it is comparatively less stable aerodynamically. Further studies must be made not only against vortex shedding but flutter oscillation, which is divergent amplitude response to the wind.

Three main navigation routes 60 feet high are secured below the cable stay bridge, but the total number of the routes decreases from 27 to 26 by the substitution.

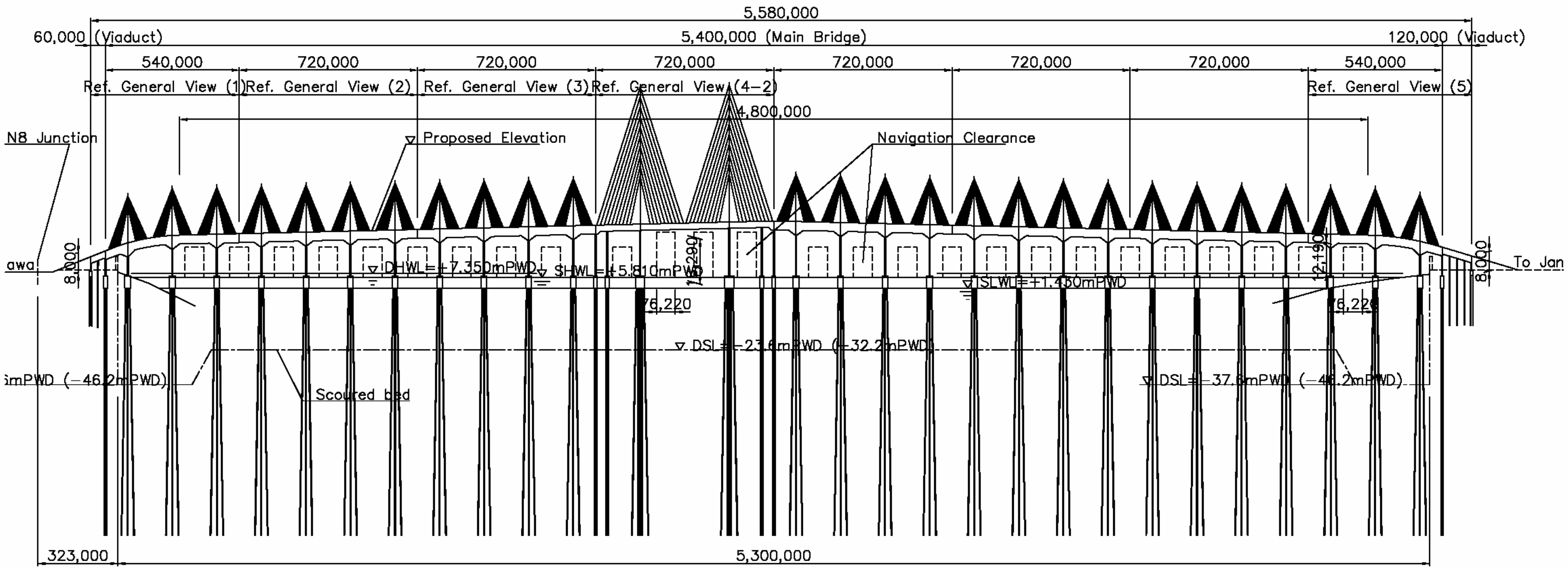
(b) Substructure

The form of substructure supporting the extradosed bridge arrangement is identical to Alternative-H1.

Two forms of piers and substructures are required to support the cable stayed bridge module. The Pylon Piers will flank the main span and carry the majority of the superstructure load via the cable stays. The Back Span Piers will carry the tensile cable forces together with direct loads from river influences. All pile caps are provided with rounded corners to minimize the effect of river flow, and are supported by 3150mm diameter steel tubular piles with a maximum rake of 1 in 6 to the vertical.

The two pylon piers each consist of a 35.0m x 54.0m x 7.1m deep pile cap supported on 24 number composite steel piles with an average wall thickness of 54mm. Pile toe elevation is -79.0m PWD providing an approximate depth of embedment of 49m below the design river bed level. The twin pier columns forming the base of the cable stay pylons are each 9.0m x 9.0m in overall cross sectional area at their base with a wall thickness of 3m. A cross beam of approximate dimensions 2.0 x 3.0m is located below the superstructure level to provide transverse stiffness. The general arrangement of the pylon substructure is shown in Figure 5.2.12.

The four back span piers each consist of a 17.5m x 31.5m x 7.1m deep pile cap supported on 8 number hollow steel tubular piles, maximum wall thickness of 56mm, with mass concrete toe plugs. Pile toe elevation is -55.0m PWD providing an approximate depth of embedment of 25m below the design river bed level. The twin pier columns are each 4.0m x 4.0m in overall cross sectional area with a wall thickness of 1.25m. These support the 2.0 x 3.0m bearing cross beam. The general arrangement of the back span substructure is shown in Figure 5.2.13.



(All dimensions are in millimeters)

Figure 5.2.11 General View (Alternative-H2)

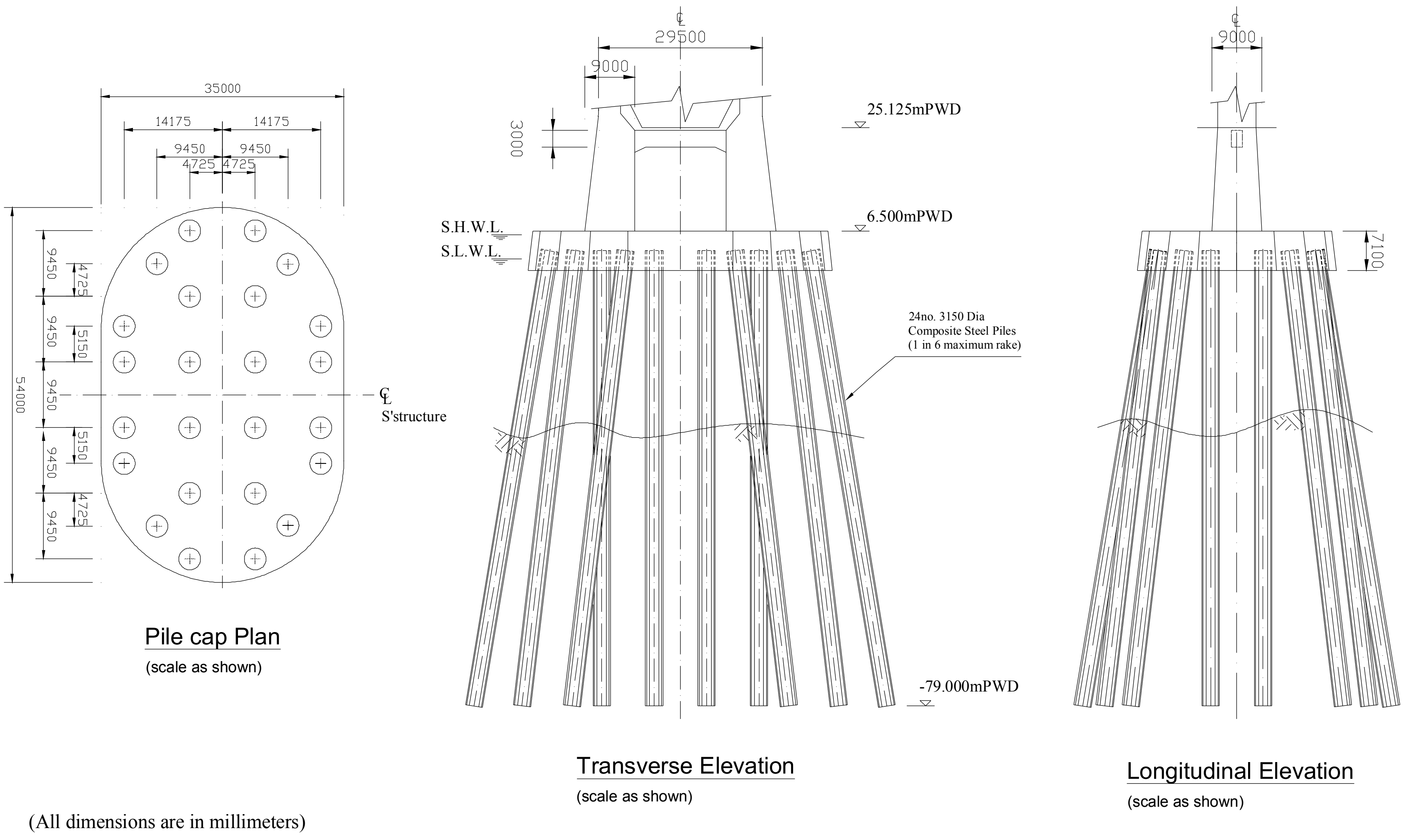


Figure 5.2.12 Mid-River Pylon Substructure General Arrangement (Alternative-H2)

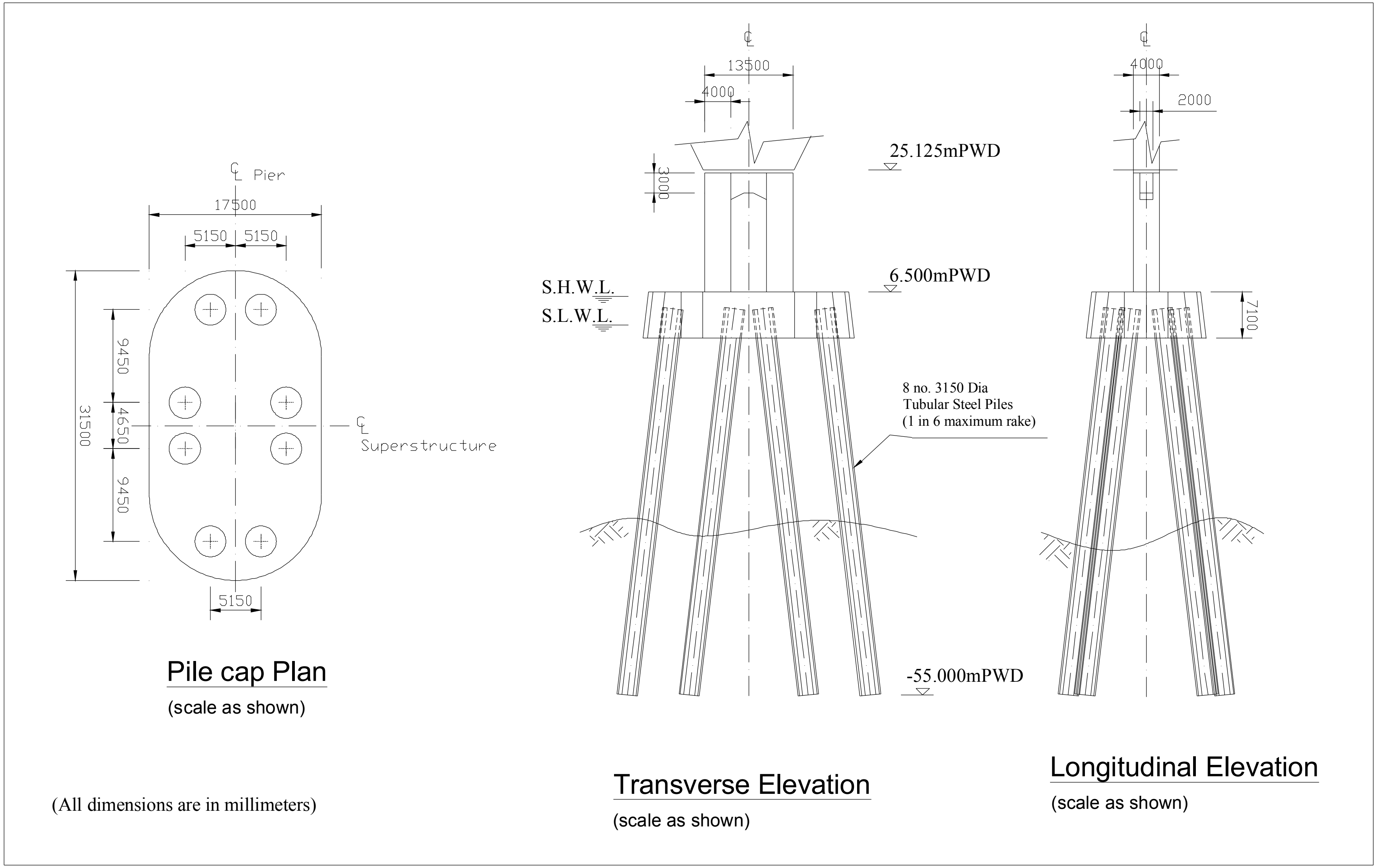


Figure 5.2.13 Mid-River Back Span Substructure General Arrangement (Alternative-H2)

(3) Alternative –H3: PC Extradosed Girder Bridge based on Bridge Width of RHD Standards

(a) Superstructure

A general view of Alternative H3 is the same as Alternative H1 shown in Figure 5.2.7. Figure 5.2.14 gives the conjunction portion of the viaduct and the standard bridge for the Alternative-H3.

This is an alternative having almost the same features as those of Alterntative-1except the cross-section.

The alternative was added to the aforesaid four to show how much it would reduce the cost if the width would be decreased to 17.1 meters as specified by the RHD standard.

(b) Substructure

The Figures 5.2.15 and 5.2.16 show the general arrangements of the river edge and mid river substructures respectively. The reduced substructure is largely due to the reduction in superstructure self weight when compared to Alternative-H1.

In order to minimize the effect of river flow the pile caps are provided with rounded corners. The 3150mm diameter hollow steel tubular piles have a maximum rake of 1 in 6 to the vertical, and are to be in-filled with a mass concrete toe plug to generate additional end bearing.

The four river edge piers comprise a 17.5m x 31.5m x 7.1m deep pile cap supported on 10 number piles with an average maximum wall thickness of 62mm. Average toe elevation is -87.5m PWD providing an approximate depth of embedment of 43m below the design river bed level.

The twenty-six mid river piers comprise a 17.5m x 31.5m x 7.1m deep pile cap supported on 8 number piles with an average maximum wall thickness of 63mm. Average toe elevation is -78.0m PWD providing an approximate depth of embedment of 47m below design river bed level.

The twin pier columns forming the base of the extradosed cable towers are each 5.0 m x 6.0m in overall cross sectional area with a minimum wall thickness of 1.25m, with similar connecting cross-beam 5.5m deep. This cross beam may be alternatively hidden within the superstructure.

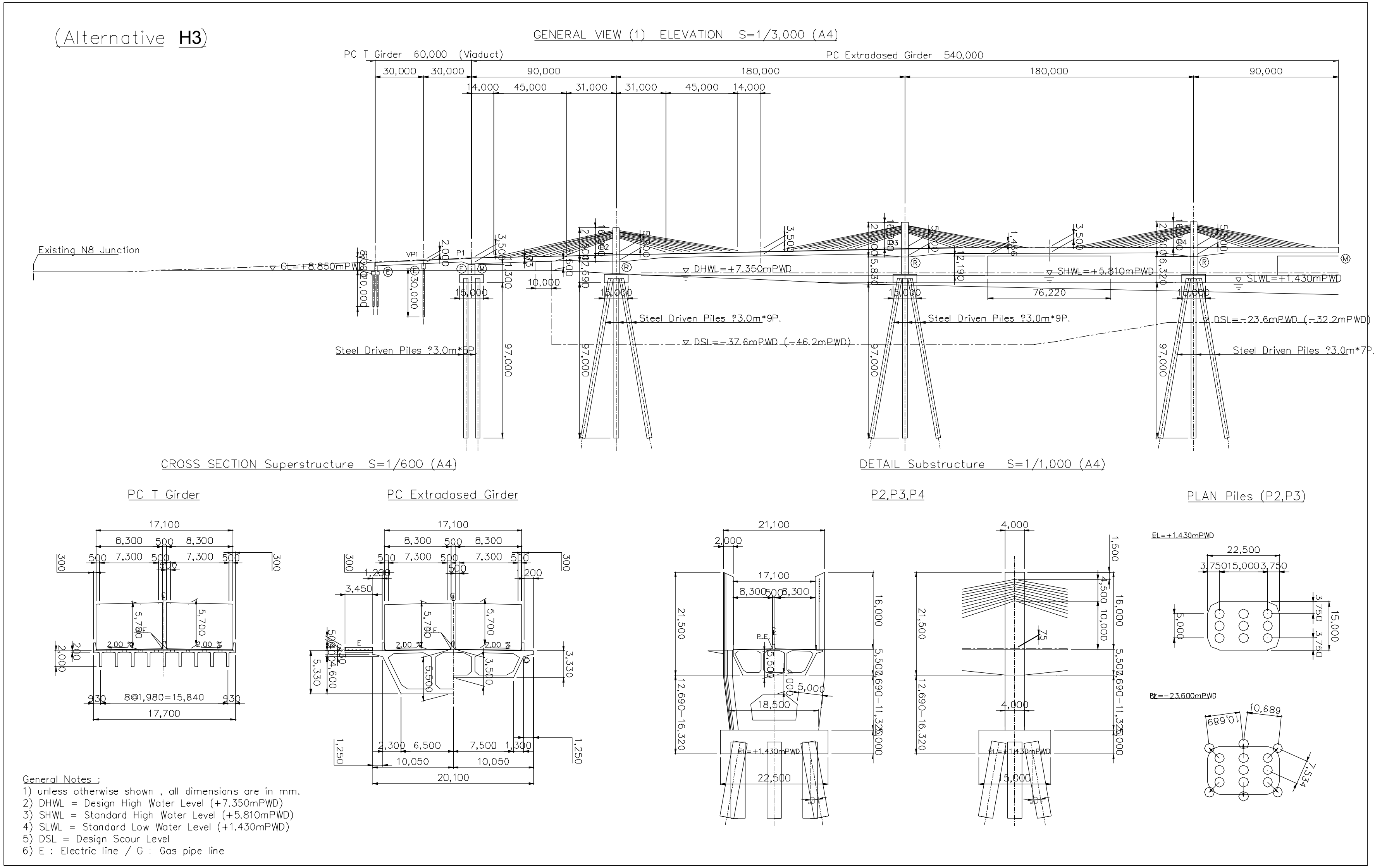


Figure 5.2.14 General View of Viaduct and Standard Bridge Portion (Alternative-H3)

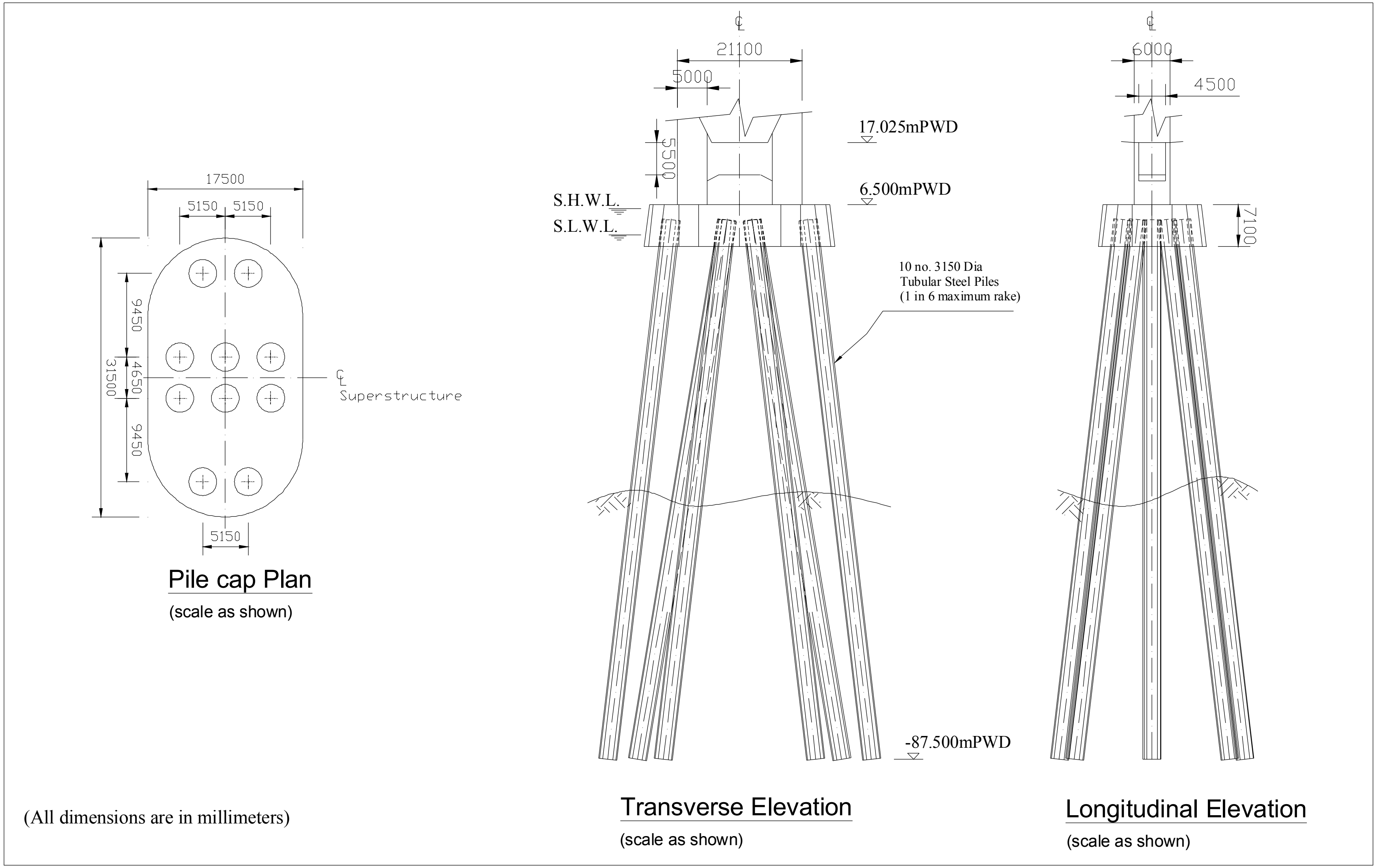


Figure 5.2.15 Edge River Substructure General Arrangement (Alternative-H3)

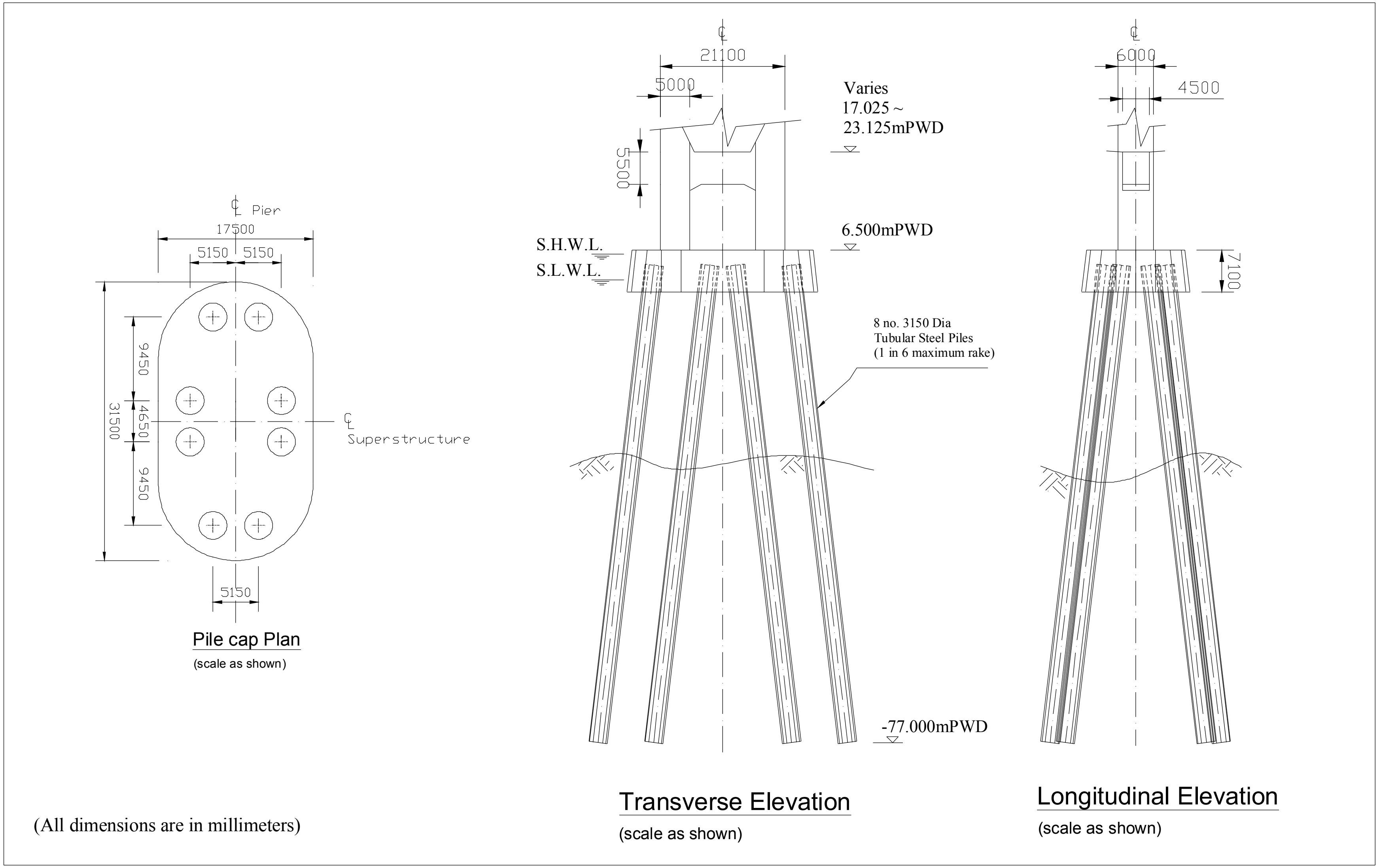


Figure 5.2.16 Mid-River Substructure General Arrangement (Alternative-H3)

5.2.5 Summary of Quantities

Summary of quantities required for construction of the superstructures and substructures are shown in the Table 5.2.4 and 5.2.5 respectively.

Table 5.2.4 Summary of Quantities of Superstructure

Alternative –H 1

Bridge Element	Item	Unit	Extradosed	Cable Stayed	Continuous Box	PC T-Girder	Total
Deck Girder	Concrete Volume	m	137,490			1,912	139,402
	Tendon Weight	ton	5,754			68	5,822
	Rebar	ton	21,999			179	22,178
Pylon	Concrete Volume	m	12,000				12,000
	Rebar	ton	2,640				2,640
Stay Cable	Cable & Fixture Weight	ton	4,695				4,695

Alternative –H 2

Bridge Element	Item	Unit	Extradosed	Cable Stayed	Continuous Box	PC T-Girder	Total
Deck Girder	Concrete Volume	m	119,158	7,488		1,912	128,558
	Tendon Weight	ton	4,987	337		68	5,392
	Rebar	ton	19,066	1,213		179	20,458
Pylon	Concrete Volume	m	10,400	3,325			13,725
	Rebar	ton	2,288	632			2,920
Stay Cable	Cable & Fixture Weight	ton	4,069	802			4,871
Steel Girder	Steel Weight	ton		4,851			4,851

Alternative –H 3

Bridge Element	Item	Unit	Extradosed	Cable Stayed	Continuous Box	PC T-Girder	Total
Deck Girder	Concrete Volume	m	109,305			1,520	110,825
	Tendon Weight	ton	4,574			54	4,628
	Rebar	ton	17,489			142	17,632
Pylon	Concrete Volume	m	9,540				9,540
	Rebar	ton	2,099				2,099
Stay Cable	Cable & Fixture Weight	ton	3,733				3,733

Table 5.2.5 Summary of Quantities of Substructure

			Alternative H 1		Alternative H 2		Alternative H 3	
		Unit	Main Bridge	Viaduct	Main Bridge	Viaduct	Main Bridge	Viaduct
Concrete Vl.	Pier	m3	29015	873	32935	873	25507	683
	Pilecap	m3	134879	487	189235	487	131680	381
	Total	m3	163894	1360	222170	1360	157187	1064
	Σ	m3	165254		223530		158251	
			100%		135%		96%	
Form	Pier	m2	45248	1315	46220	1315	36192	1047
	Pilecap	m2	32936	221	38292	221	32238	176
	Total	m2	78184	1536	84512	1536	68430	1223
	Σ	m2	79720		86048		69653	
			100%		108%		87%	
Rebar	Pier	t	6757	162	7387	162	4843	125
	Pilecap	t	6546	22	8147	22	6400	17
	Total	t	13303	184	15534	184	11243	142
	Σ	t	13487		15718		11385	
			100%		117%		84%	
Pile	Type		Steel Driven	Cast in Palce	Steel Driven	Cast in Palce	Steel Driven	Cast in Palce
	Dia	m	3.15	1.2	3.15	1.2	3.15	1.2
Vertical Pile	No.		0	56	16	56	0	47
	Length	m	0	1379	1280	1379	0	1140
Racked Pile	No.		256	0	288	0	248	0
	Length	m	21856	0	23488	0	19914	0
	Σ of No.		256	56	304	56	248	47
	Σ of Len.	m	21856	1379	24768	1379	19914	1140
			100%		113%		91%	