3.4 Preliminary Design

3.4.1 Highway Design

(1) Design Criteria and Standards

The expected main role of the Matarbari Port Access Road is to carry long-distance freight traffic smoothly from the proposed Matarbari Port to National Highway No.1 (N1). In order to provide a smooth traffic condition for such long-distance traffic, the project road should be planned as a semi-access controlled highway with accessible points provided at certain intervals.

As several development plans exist in Matarbari and Maheshkhali area, the project road is also expected to function as a vital access to the development areas in future. Considering such circumstances, the expected function of the road, road length and relationship with the other interconnected roads such as N1 and Regional Highway No.172, the design speed of 60 km/h was adopted for the project road.

In reference to the following design standards, the geometric design conditions for the Matarbari Port Access Road were determined as shown in Table 3.4-1.

- RHD, Geometric Design Standards Manual (Revised) 2005
- AASHTO, A Policy on Geometric Design of Highway and Streets
- Japan, Road Structure Ordinance

Table 3.4-1 Geometric Design Conditions

	Unit	Applied Value	Remarks
General Design Considerations			
Design Traffic Volume	PCU/hour	1,975	in year 2035
Design Speed	km/h	60	Japan
Number of Through-Traffic Lanes	lane	4	RHD
Design Vehicle	-	WB-15	AASHTO
Minimum Stopping Sight Distance	m	90	RHD
Cross Section Elements			
Normal Cross Slope	-	3%	RHD
Traveled Way Width	m	3.65	RHD
Median Width (inc. inner shoulder)	m	4.20	0.6 + 3.0 + 0.6
Inner Shoulder Width	m	0.60	RHD
Outer Shoulder Width	m	1.50	RHD
Horizontal Alignment			
Minimum Radius	m	250	RHD
Maximum Radius for use of a Transition Curve	m	999	$R=0.29\times V^2$ (Japanese Standard)
Radius for Normal Crown	m	2,930	e=-3%, f=0.0397 (AASHTO)
Minimum Curve Length	m	180	L=3V (AASHTO)
Minimum Transition Curve Length	m	50	L=V/3600×3 (Japanese Standard)
Maximum Superelevation Rate	-	6%	Japanese Standard
Superelevation Runoff	-	1/167	AASHTO
Vertical Alignment			
Maximum Grade	-	3%	RHD
Minimum Grade	-	0.3%	
Minimum Rate of Crest Vertical Curvature (K)	K	18	RHD
Minimum Rate of Sag Vertical Curvature (K)	K	18	RHD, AASHTO
Minimum Vertical Curve Length	m	50	L=V/1.2 (Japanese Standard)
Vertical Clearance	m	5.5	4.9 + 0.3 overlay

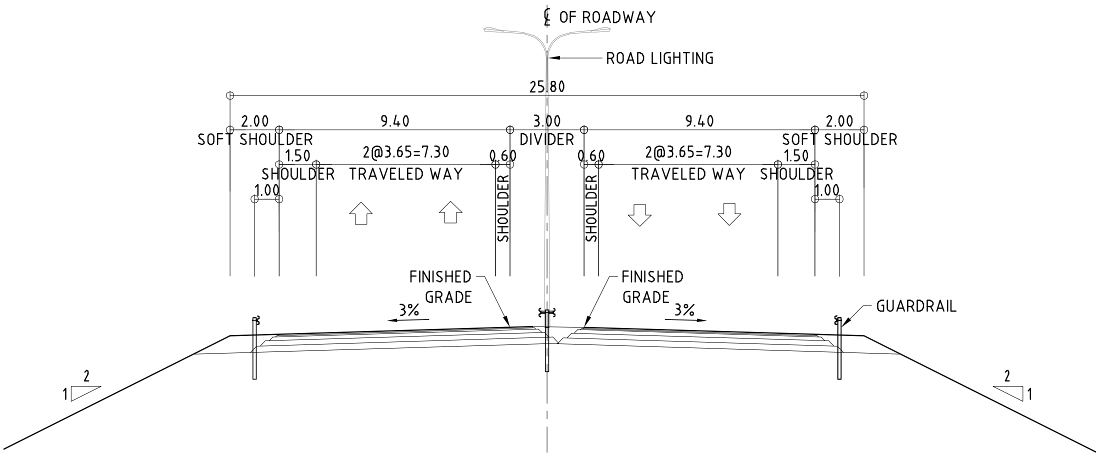
Source: JICA Survey Team

(2) Typical Cross Sections

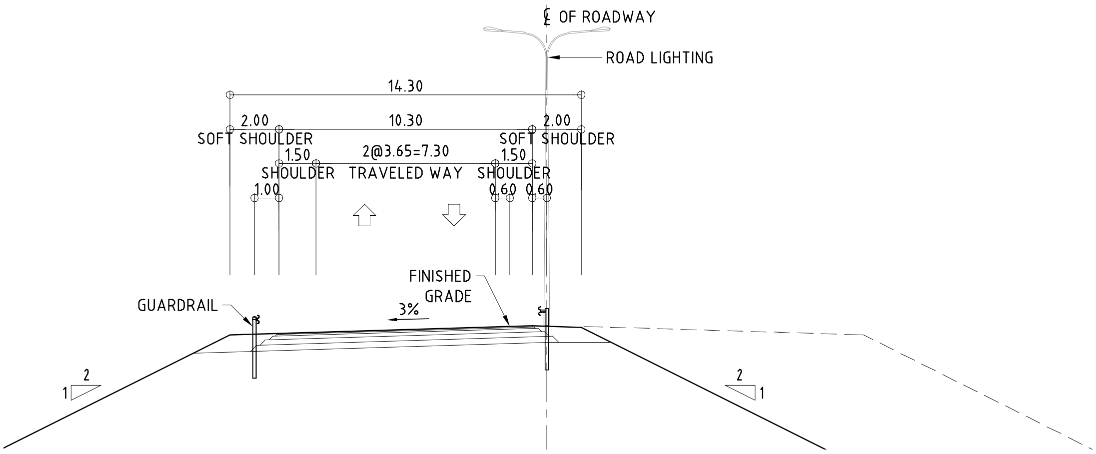
The projected future traffic volume in 2035 (10-year after opening of the project road) is 5,655 vehicle/day (or 12,103 PCU/day) and 2-lane (1-lane for each direction) can accommodate such traffic volume as described in the section 3.2.3. However, it is expected that the traffic volume may increase more depending on the progress of the development plans in Matarbari and Maheshkhali. Therefore, expandability from 2-lane to 4-lane in future should be taken into consideration from the beginning of the project and thus phased construction is recommended.

The width of each cross section element is decided in compliance with the RHD standard. For phased construction, the shoulder width at the interim stage should be 1.5 m on both sides of the traveled way for safety reasons; however only 0.6 m for the inner shoulder width would be adequate at the final stage.

Figure 3.4-1 and Figure 3.4-2 represent the typical cross sections of the project road at the embankment sections and the bridge sections respectively.



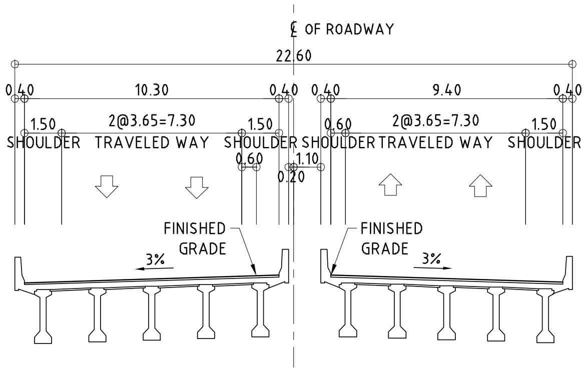
Embankment Section at Final Stage



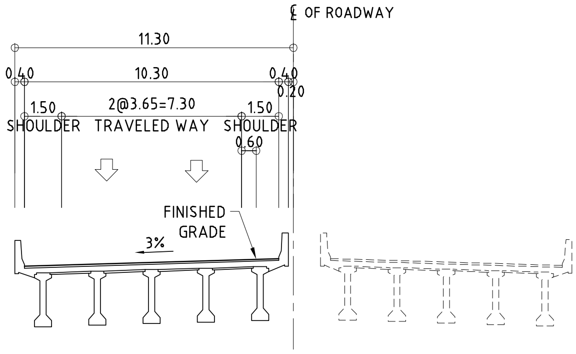
Embankment Section at Interim Stage

Source: JICA Survey Team

Figure 3.4-1 Typical Cross Sections of Matarbari Port Access Road (Embankment Sections)



Bridge Section at Final Stage



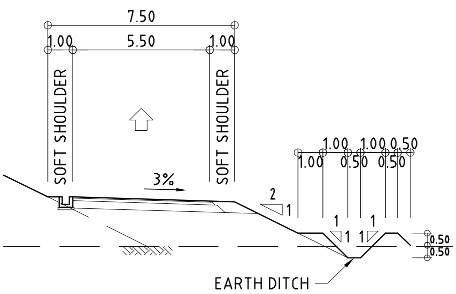
Bridge Section at Interim Stage

Source: JICA Survey Team

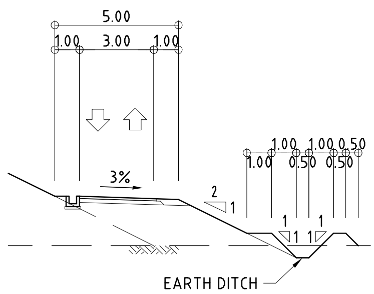
Figure 3.4-2 Typical Cross Sections of Matarbari Port Access Road (Bridge Sections)

At the embankment sections of the project road, service roads and underpass box culverts will be provided along the project road in order to avoid community severance. The service road will have the following two (2) types depending on the traffic conditions.

- The service road type-A aims to accommodate local vehicular traffic. In reference to the RHD Design Standard, minimum 2-lane width of 5.5 m was adopted because the expected traffic volume wouldn't be significant. Although the optimum minimum width for shoulders of RHD jurisdictional roads is 1.5 m, 1.0 m width of unpaved soft shoulder for both sides of the service road was adopted in consideration of the expected vehicle type of the road and traffic safety. Within the space of the soft shoulder, drainage ditch will be installed.
- The service road type-B aims to accommodate local non-motorized traffic (NMT). In reference to the RHD Design Standard, minimum lane width of 3.0 m was adopted. Same as the service road type-A, 1.0 m width of unpaved soft shoulder for both sides of the service road was adopted.



Type A (for Vehicular Traffic)



Type B (for Pedestrian Traffic)

Source: JICA Survey Team

Figure 3.4-3 Typical Cross Sections of Service Road

(3) Alignment Design

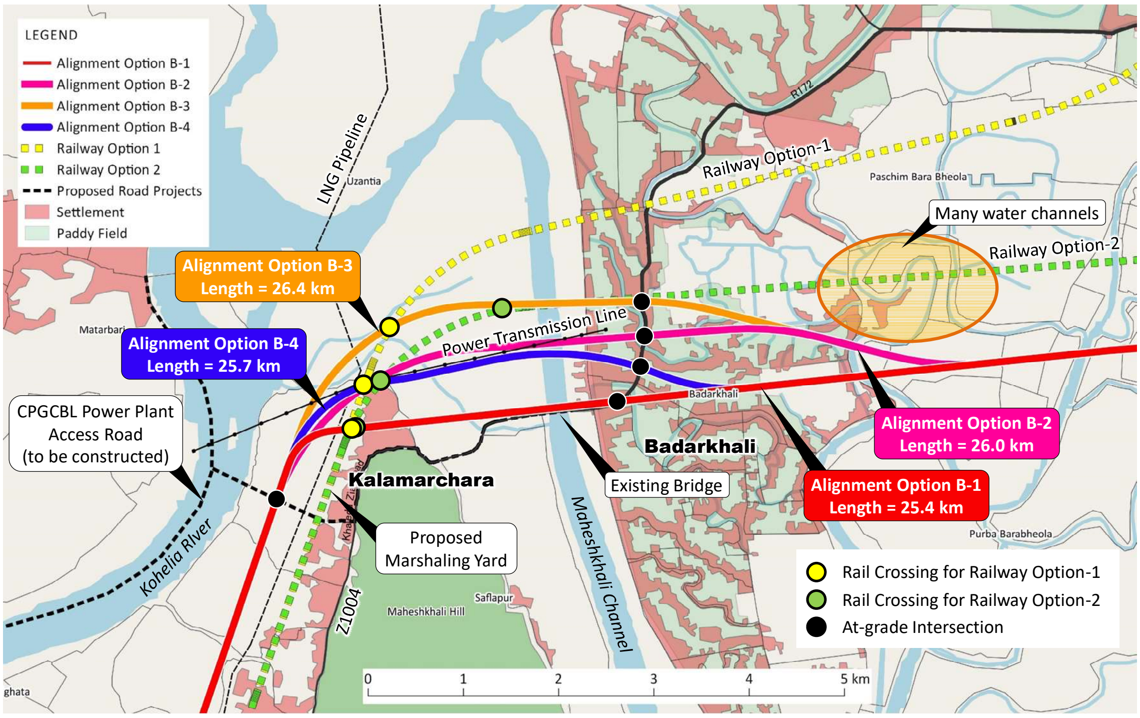
Comparative Analysis

As described in section 3.2.2, Route Option B was selected as the optimum route of the project road. The available space for the selected route (Route Option B) is limited due to the settlements in Chakaria especially at the area near N1 as well as the existence of many water channels in between Badarkhali and Fashiakhali which were previously used as sites for the fishing project. Basically the alignment can be designed by avoiding settled areas. However, it would need to pass through the high-densly settled areas of Badarkhali and the north of Maheshkhali. Therefore, a more detailed comparative study on the road alignment was done especially at the areas in Kalamarchara (Maheshkhali) and Badarkhali (Chakaria).

Initially, three (3) alternative alignment options (Alignment Options B-1 to B-3) were analyzed and the Alignment Option B-1 was evaluated as the optimal alignment for the Route Option B because of the following reasons:

- Alignment Option B-1 can cross the proposed railway to Matarbari Port at the same elevation but the other options need grade separation over the railway, which would require at least 1 km-long viaduct costing more than BDT 2.6 billion;
- Alignment Option B-1 does not require relocation of CPGCBL's power transmission line; and
- Alignment Option B-1 has a certain level of social impact but with the lowest construction cost.

However, following the stakeholders' meeting held at the project sites, further social environmental issues against Alignment Option B-1 were identified. Therefore, another alternative option, namely Alignment Option B-4, was also considered and decided as the final alignment for the Matarbari Port Access Road.

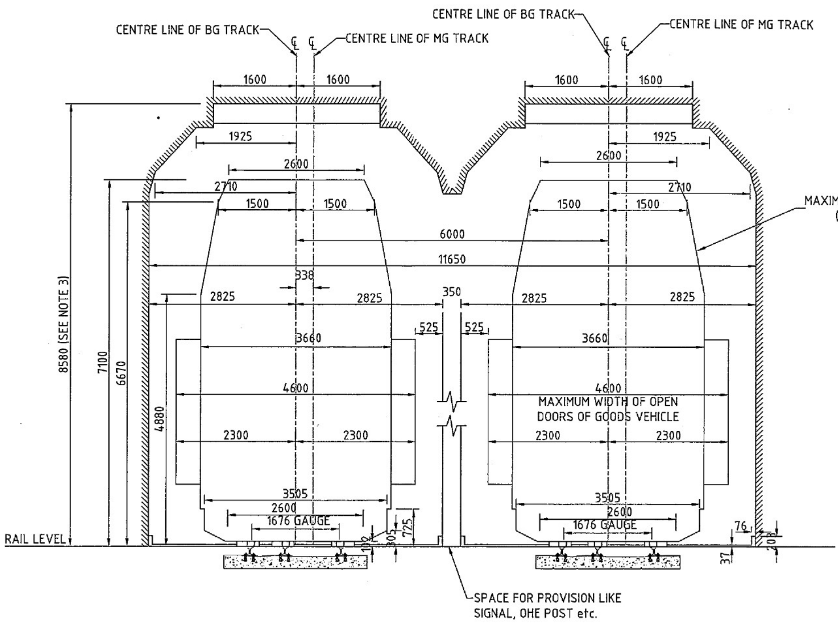


Source: JICA Survey Team

Figure 3.4-4 Alternative Alignments for Route Option B

Construction of a railway connection to the proposed Matarbari Port from the proposed Dohazari-Cox's Bazar Railway is also under feasibility study and the most provable plan includes construction of a marshaling yard in Maheshkhali in between the LNG pipeline and Zila Road No.1004. Therefore, the Matarbari Port Access Road should avoid such an area and be aligned to the river side along the LNG pipeline.

Currently two alternative railway route options are studied by the ADB consultant. Considering that the embankment of the railway is high (planned height: 10 m +MSL) and the railway will require 8.58 m vertical clearance for accommodating a double-stack container train, as well as the expected frequency of the train operation, the railway crossing should be at-grade instead of grade separation with approximately 1 km-long viaduct over the railway.. The Alignment Option B-1 can cross the railway at-grade because of its cross angle (60 degree) distance from the bridge section. But the Alignment Options B-2 and B-3 will cross the railway at acute angles henceforth, at-grade crossing would be difficult.



Source: ADB Consultant for Study for Dhaka-Chittagong-Cox's Bazar Railway Project Preparatory Facility

Figure 3.4-5 Clearance for Railway

Also, a power transmission line has been constructed by the CPGCBL's power plant project at the north of Maheshkhali. The Alignment Options B-2 and B-3 will interfere with the power transmission line and its relocation will be required. Considering that the proposed railway alignment will also interfere with the power transmission line and will require its relocation, these two (2) options would require the relocation of power transmission line twice at different timings for access road construction and railway construction.

The project implementation schedules of the Matarbari Port Access Road and the railway project are different and the Access Road Project is more urgent. Also, the railway project has uncertainty due to the status of the project implementation (it is still at the pre-feasibility study stage). The project road should be planned without such complexity and uncertainty, and thus the Alignment Option B-1 has more advantages over B-2 or B-3.

However, as a result of the stakeholders' meeting held at the project sites, the local people requested JICA Survey Team to reconsider the road alignment because of the following reasons:

- There are many academic institutions, market area, and religious buildings in Badarkhali (Chakaria Upazila) and the Alignment Option B-1 passes through such areas. The local residents prefer to shift the alignment of the project road away from such areas.
- Utternalvila Baruapara Village under Kalamarchara Union of Maheshkhali is a high-densely populated village. The people of the village strongly requested to change the project road alignment to the north to avoid the village.

Therefore, another alternative option, namely Alignment Option B-4, was considered as a minor modification to the Alignment Option B-1. The Alignment Option B-4 passes through the narrow area in between the CPGCBL's power transmission line and Utternalvila Baruapara Village (to avoid relocation of the power transmission line), and the less-populated area in Badarkhali. Although the initial request from Badarkhali was to divert whole area of the town from the south, the requested alignment would require extra construction costs due to its longer road length and similar social environmental impacts at the Maheshkhali side. In this regard, it is inevitable to pass through the Badarkhali area but the Alignment Option B-4 can minimize the adverse social environmental impact. The people of Badarkhali finally accepted the project implementation with the Alignment Option B-4. The people of Utternalvila Baruapara Village also accepted the Alignment Option B-4 because the number of affected buildings is minimal.

Table 3.4-2 summarizes the result of comparative study of the alignment options. The Alignment Option B-4 is decided as the final alignment having advantage to mitigate adverse social environmental impact with acceptance by the local people. However, it should be noted that the alignment does not have compatibility with the railway project because the project road should be constructed as viaduct at the possible rail crossing point in order to avoid the relocation of power transmission line and resettlement of houses of Utternalvila Baruapara Village in Maheshkhali. Depending on the plan of the railway project, the project road may need to be renovated for accommodating at-grade rail crossing together with relocation of power transmission line, or the railway can be constructed over the project road with railway viaduct.

Table 3.4-2 Comparison of Alternative Alignments for Route Option B

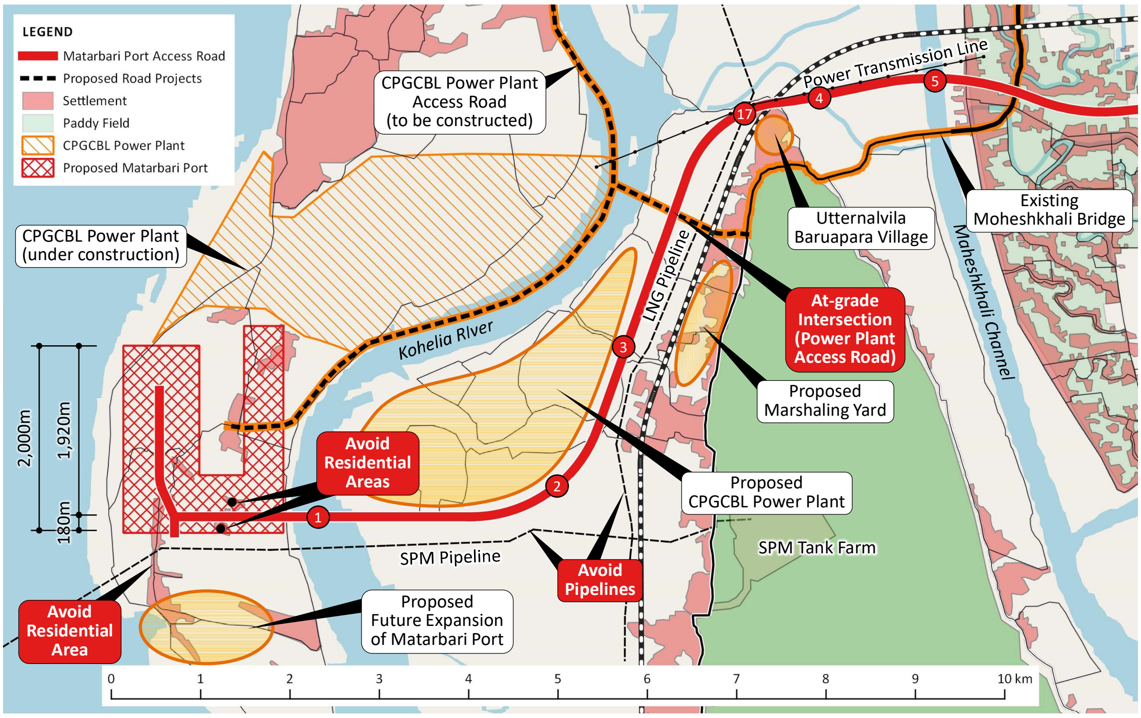
	Alignment Option B-1	Alignment Option B-2	Alignment Option B-3	Alignment Option B-4
Road Length	25.4 km	26.0 km	26.4 km	25.7 km
Bridge Length	6.4 km	7.6 km	7.0 km	7.0 km
Affected Buildings	212	173	137	132
Affected Major Utilities	None	CPGCBL's power transmission line needs to be relocated	CPGCBL's power transmission line needs to be relocated	None
Compatibility with Railway Project	At-grade railway crossing (10 m MSL)	Grade separated railway crossing with 1 km-long viaduct (22 m MSL)	Grade separated railway crossing with 1 km-long viaduct (22 m MSL)	Railway shall be grade separated over the access road
Construction Cost	Base Case	+ BDT 2.15 billion	+ BDT 2.24 billion	+ BDT 0.93 billion
Evaluation				**Recommended**

Source: JICA Survey Team

<u>Designed Alignment</u>

The control points for the alignment design in Matarbari and Maheshkhali areas are the following:

- To secure north-south directional connectivity for future expansion of Matarbari Port at the southern side of Dhalghata area;
- The beginning point of the access road should be within the proposed Matarbari Port area and at about 180 m offset from the southern boundary of the port area in order to avoid settled areas;
- The alignment in Maheshkhali area is away from the LNG pipelines at minimum 100 m offset distance;
- To avoid proposed CPGCBL's another power plant in Mahashkhali area at the opposite side of Kohelia River;
- To avoid the CPGCBL's power transmission line and Utternalvila Baruapara Village.

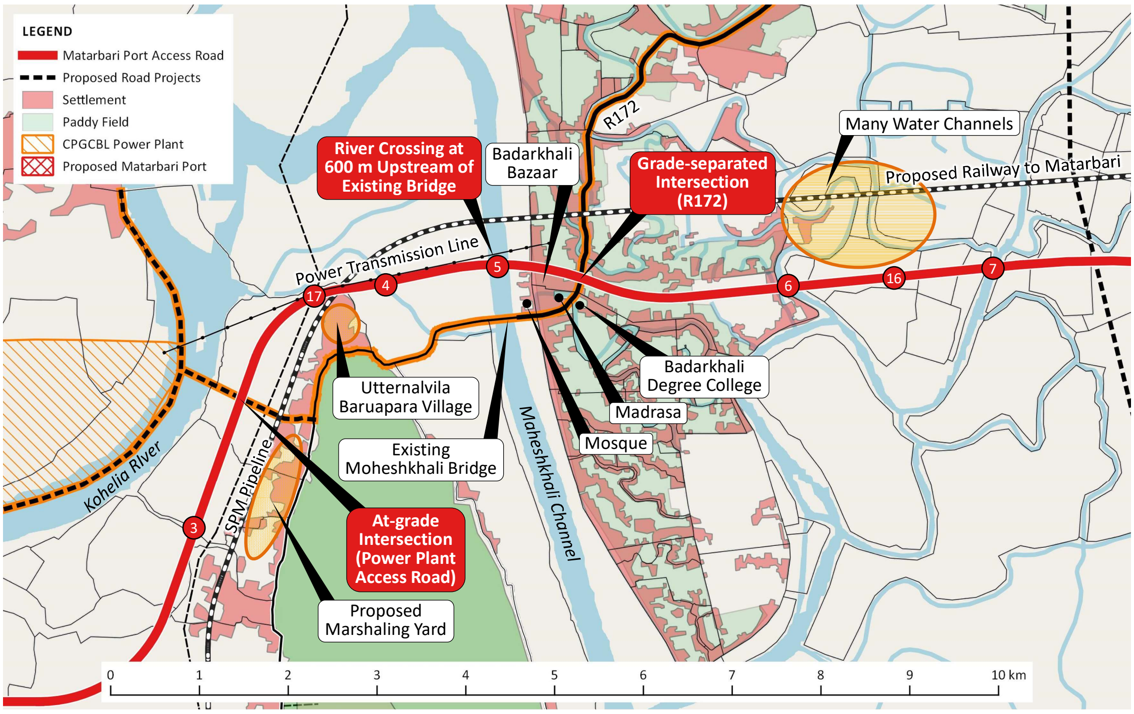


Source: JICA Survey Team

Figure 3.4-6 Designed Road Alignment (1/3)

For the section around the Maheshkhali Channel crossing, the alignment at about 600 m upstream of the existing bridge was selected. The following are the control points for the alignment design in Kalamarchara and Badarkhali:

- In order to avoid large scale social environmental impact, the sections through settled area in Badarkhali should be bridge structure instead of embankment;
- To avoid Badarkhali Bazaar, which is the busiest location in the area;
- To avoid academic institutions (such as Badarkhali Degree College, Badarkhali High School, Madrasa, Little Jewel Kindergarten, Iqra Academy, and Badar Sha Academy School) and religious facilities in Badarkhali.

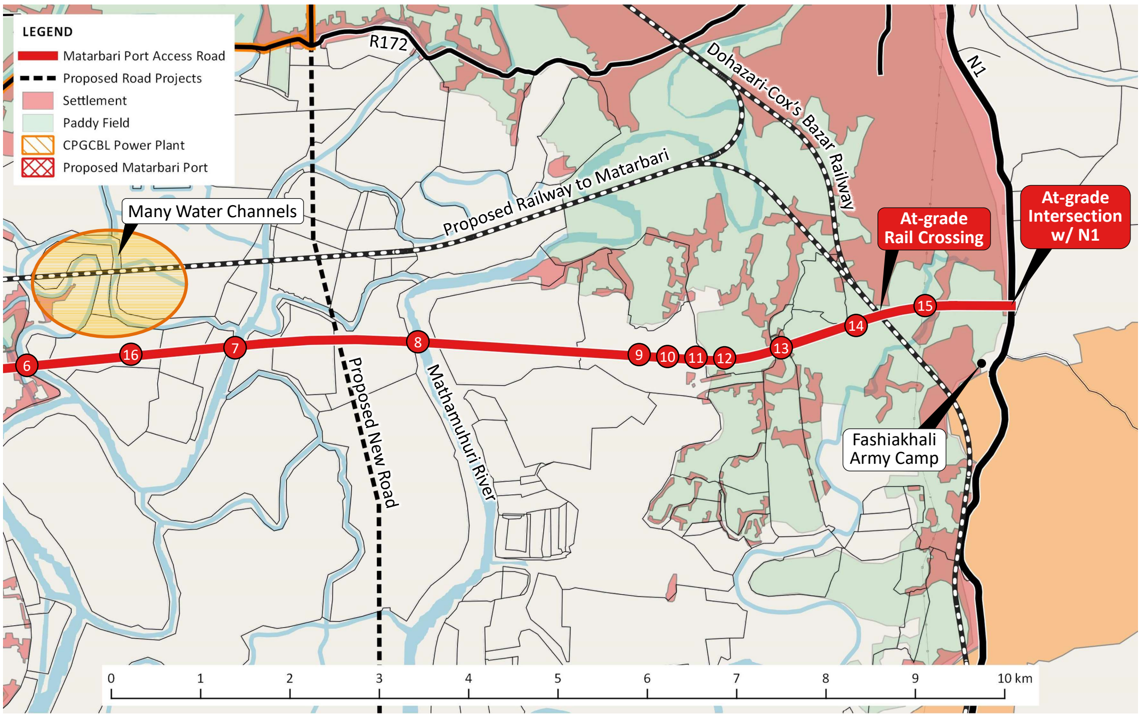


Source: JICA Survey Team

Figure 3.4-7 Designed Road Alignment (2/3)

The control points at the ending section are the following:

- To avoid the settled areas in Chakaria;
- To avoid the national park in Fashiakhali;
- To intersect with the water channel crossings at angles more than 60 degrees.



Source: JICA Survey Team

Figure 3.4-8 Designed Road Alignment (3/3)

(4) Intersection Design

The Matarbari Port Access Road will intersect with the following four (4) roads:

- Matarbari Port North-South Connector Road
- CPGCBL Power Plant Access Road
- Regional Highway No.172 (R172)
- National Highway No.1 (N1)

Design philosophy of each intersection is as follows:

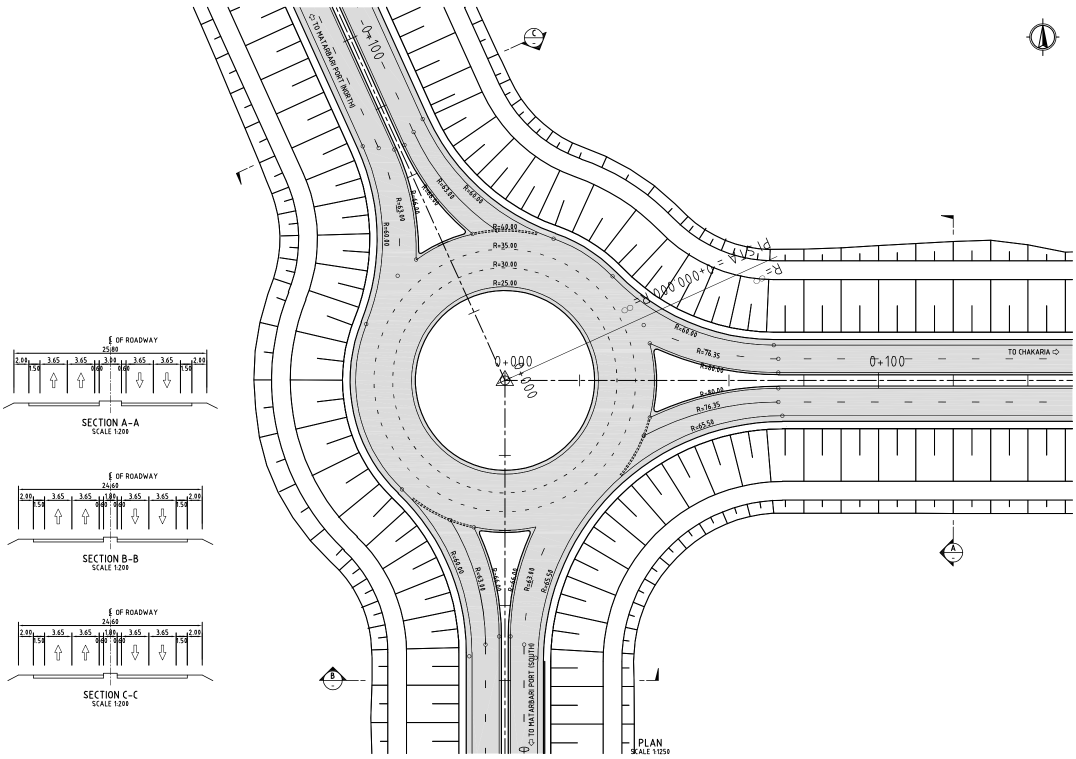
<u>Matarbari Port North-South Connector Road</u>

The beginning point of the project road was determined based on the following control points:

- SPM pipeline will be constructed about 100 m south of the proposed Matarbari Port area and the access road should be located at the same side as the port area;
- There are some residential houses at the coastal area of Dhalghata and the project road should avoid such houses;

The proposed Matarbari Port is planned to expand its function into the southern part of Dhalghata area across the SPM pipeline so that the north-south directional linkage in the island should be secured. Therefore, the north-south linkage in Matarbari and Dhalghata areas and east-west linkage between the port area and N1 should be separately considered in the port area to meet with the future expansion plan of the Matarbari Port.

For providing smooth traffic flow in the port area where internal and external traffic flow may be mixed, roundabout type intersection is applied for the intersection. The roundabout can also be used as the U-turn space for the project road, which will have access control with median barrier.



Source: JICA Survey Team

Figure 3.4-9 Intersection with Matarbari Port North-South Connector Road

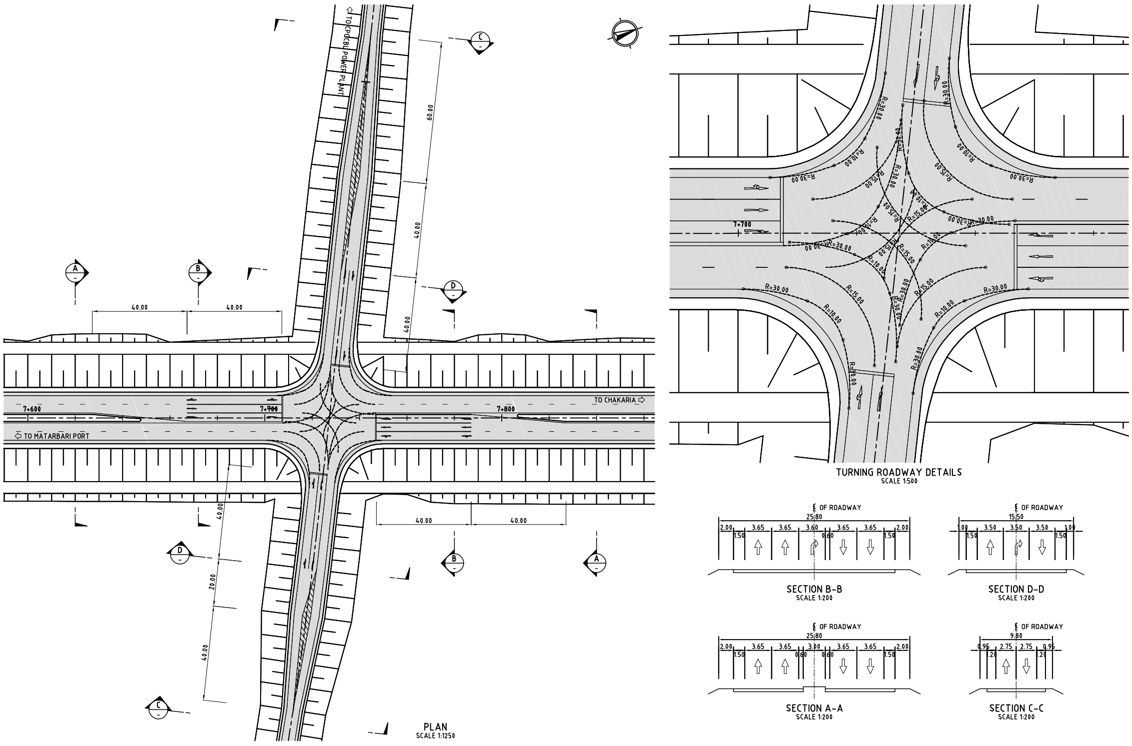
<u>CPGCBL Power Plant Access Road</u>

The intersection with CPGCBL's Power Plant Access Road is located at STA 7+727. As the project road is expected to have good accessibility to the power plant as well and the proposed power plant access road is designed as embankment structure at the intersecting point (about 6 m MSL), provision of at-grade intersection would be preferable. It is expected that the traffic volume from power plant to the port access road would not be so high and non-signalized intersection would be good enough to serve the traffic at the intersection.

Table 3.4-3 Minimum Length at Intersection Area

		Adopted Value	Minimum Requirement
Matarbari Port Access Road	Deceleration Lane (m)	40	> $V \times \Delta W/6 = 60 \times 3.65/6 = 36.5$
	Right-Turn Storage (m)	40	30
Power Plant Access Road	Lateral Shift (m)	60	> $V \times \Delta W/2 = 60 \times 1.75/2 = 52.5$
	Deceleration Lane (m)	40	> $V \times \Delta W/6 = 60 \times 3.50/6 = 35.0$
	Right-Turn Storage (m)	40	30

Source: JICA Survey Team



Source: JICA Survey Team

Figure 3.4-10 Intersection with CPGCBL's Power Plant Access Road

<u>Regional Highway No.172</u>

R172 is the solo RHD's highway intersect with the project road in between Matarbari Port and N1. The distance between R172 and N1 is approximately 13 km. therefore, interconnectivity between R172 and the project road should be provided.

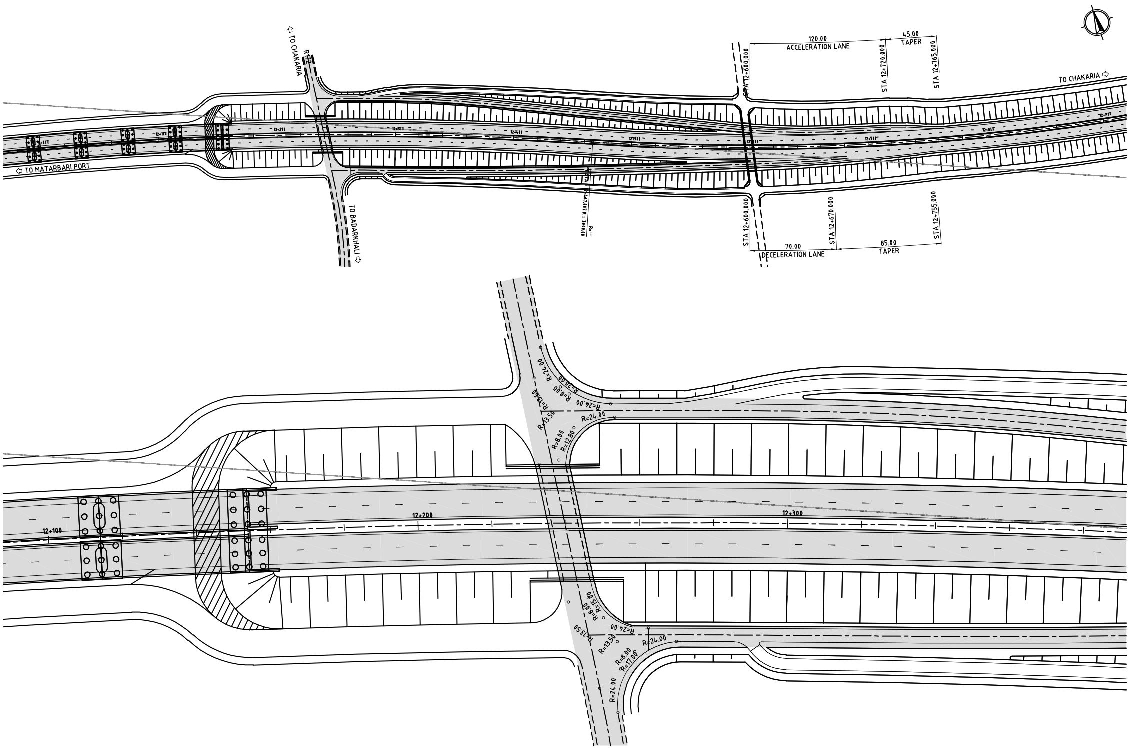
Considering that R172 caters many local traffic not only vehicular traffic but also non-motorized traffic, grade separation would be preferable for the intersection for ensuring smooth traffic and traffic safety. Based on the analysis of the traffic pattern in the Badarkhali area, it can be assumed that the traffic from Badarkhali to Matarbari and Maheshkhali side would not be diverted to the project road but the traffic from Badarkhali to N1 will be diverted to the project road. Therefore, accessibility between R172 and the project road was considered only for the direction of N1 and diamond interchange would be preferable for the interchange configuration.

For the ramp terminal design, parallel type acceleration lane would be preferable for merging section and tapered type deceleration lane would be preferable for the diverging section based on the experience in Japan. RHD Design Standard does not clearly described the design criteria for merging and diverging section of grade separated interchange. Therefore, the length of the merging and diverging sections was designed in accordance with the design standards in Japan.

Table 3.4-4 Minimum Length of Interchange Ramp Terminal (60 km/h)

	Speed Change Lane Length	Ramp Terminal Type	Taper Length
Merging Section	Acceleration Lane: Min. 120 m	Parallel Type	Min. 45 m
Diverging Section	Deceleration Lane: Min. 70 m	Tapered Type (1/15)	Min. 45 m

Source: Road Structure Ordinance, Japan



Source: JICA Survey Team

Figure 3.4-11 Intersection with R172

<u>National Highway No.1</u>

According to the estimated future traffic volume, the intersection should be controlled by traffic signal. Using the future traffic volume in 2026 and 2035, traffic capacity of the intersection was calculated. From the calculation result, it was identified that the following measures should be provided:

- The right-turn traffic volume from N1 to the port access road is high. 1-lane right-turn lane can accommodate the traffic volume in 2026 but 2-lane is necessary for the traffic volume in 2035;
- The left-turn traffic volume from the port access road to N1 is also high. If this flow is signal controlled, traffic capacity of the intersection would be saturated. Therefore, free flow lane should be provided for this traffic flow;
- Comparing the necessary lane arrangement of the intersection in 2026 and 2035, the difference is only

the number of right-turn lane from N1 to the port access road. N1 is currently 2-lane but it will be widened to 4-lane in future. Considering the easy expandability of N1 to 4-lane, the intersection design should be made based on the traffic volume in 2035.

Table 3.4-5 Comparison of Traffic Saturation Rate at N1 Intersection

		To Chittagong			
		1-lane	2-lane	3-lane	1-lane w/ right-turn Free
To Matarbari Port	1-lane	Year 2026: **2.09** (NG) Year 2035: **3.03** (NG)	* * *	* * *	* * *
	2-lane	* * *	Year 2026: **0.84** (NG) Year 2035: **1.25** (NG)	* * *	* * *
	2-lane Free	* * *	Year 2026: **0.68** Year 2035: **0.93** (NG)	Year 2026: **0.66** Year 2035: **0.88**	Year 2026: **0.66** Year 2035: **0.88**

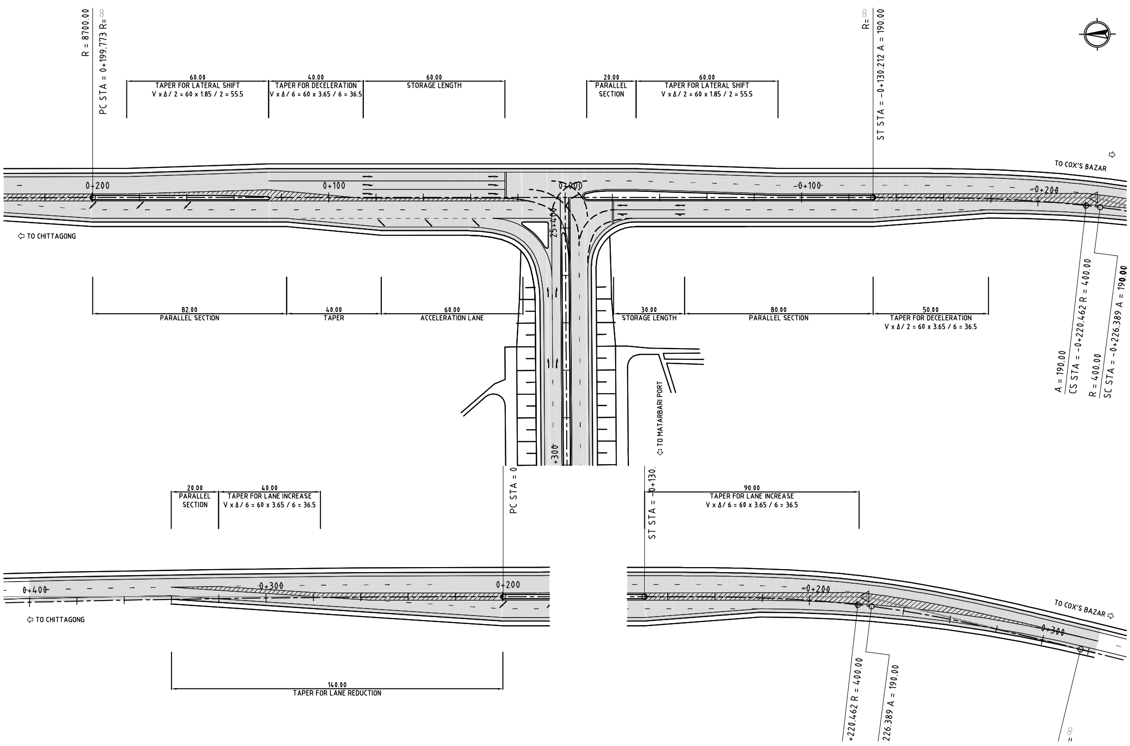
Source: JICA Survey Team

In these regards, approximately 300 m on both sides of N1 from the intersection with the port access road should be widened to 4-lane. Also, widening of N1 should be undertaken at the one side of the road so that the present traffic will not suffer from the traffic restriction during the construction works.

Table 3.4-6 Minimum Length at Intersection Area

		Adopted Value	Minimum Requirement
National Highway No.1 (From Chittagong Side)	Lateral Shift (m)	60	Min. $V \times \Delta W/3 = 60 \times 1.85/2 = 55.5$
	Deceleration Lane (m)	40	Min. $V \times \Delta W/6 = 60 \times 3.65/6 = 36.5$
	Right-Turn Storage (m)	60	Min. 30
National Highway No.1 (To Chittagong Side)	Acceleration Lane (m)	60	Min. 60
	Taper	40	Min. $V \times \Delta W/6 = 60 \times 3.65/6 = 36.5$

Source: JICA Survey Team



Source: JICA Survey Team

Figure 3.4-12 Intersection with N1

(5) Pavement Design

Flexible pavement (or called asphalt concrete pavement) is selected for the Matarbari Port Access Road instead of applying ridged pavement (or called concrete pavement) because the project road will retain residual settlement of the embankment and the flexible pavement has the advantage to keep smoothness of the pavement surface on such condition.

Considering that the project road will serve a lot of heavy vehicle traffic, polymer modified asphalt concrete pavement should be applied for both wearing course and binder course in order to minimize rutting of pavement. Also, semi-flexible pavement should be applied for the pavement at intersections, even though this type of pavement has not been applied in Bangladesh.

Pavement design was undertaken based on "AASHTO Guide for Design of Pavement Structure 1993" using 10-year accumulated future traffic volume on the road from 2026 to 2035 and the parameters used in Bangladesh. The calculated pavement thickness of each layer is the following:

- Wearing course (polymer modified asphalt concrete): 50 mm
- Binder course (polymer modified asphalt concrete): 50 mm
- Base course -1 (asphalt treated): 150 mm
- Base course -2 (crushed aggregate): 200 mm
- Subbase course (granular aggregate): 250 mm

1) Selection of Pavement Type

Flexible pavement would be suitable pavement type on embankment which would have uneven settlement at different subsoil conditions and box culvert installation. The estimated future traffic volume of heavy vehicles on the project road in 2035 is over three thousand and polymer modified asphalt concrete should be applied for both wearing course and binder course in order to minimize rutting of pavement. On the other hand, semi-flexible pavement would be suitable pavement type at intersections where rutting is likely to occur.

Table 3.4-7 Comparison of Pavement Types

	Flexible Pavement (Asphalt Pavement)	Semi-Flexible Pavement	Rigid Pavement (Concrete Pavement)
Characteristics	Asphalt pavement is the most common pavement type but has inferior in oil resistance and heat resistance.	Semi-flexible pavement is a combination of flexible and rigid pavements. Special cement milk is filled in the air voids of asphalt pavement.	Concrete pavement is the most durable pavement type.
Traveling performance	Surface of asphalt pavement is smooth and traveling performance is high. 5	Surface of semi-flexible pavement is smooth and traveling performance is high. 5	Surface of concrete pavement is not smooth and it has horizontal joint. Thus, traveling performance is low. 3
Durability	Ruts and potholes are most likely to occur due to heavy traffic. 3	Intermediate between flexible pavement and rigid pavement. 4	Concrete pavement is the most durable pavement type against heavy traffic. 5
Flexibility against embankment settlement	Asphalt pavement flexibly deforms together with uneven settlement of embankment. 5	Intermediate between flexible pavement and rigid pavement. 3	Cracks are most likely to occur due to uneven settlement of embankment. 1
Constructability	2,300 m^2/day 5	1,050 m^2/day (curing of concrete is needed) 4	140 m^2/day (curing of concrete is needed) 3
Construction cost	1.00 5	1.02 4	1.10 3
Evaluation	**Recommended** for normal section 23	**Recommended** for intersections 20	15

Source: JICA Survey Team

2) Design Formula

Pavement design for the Matarbari Port Access Road is based on the "Pavement Design Guide for Roads & Highways Department" and "AASHTO Guide for Design of Pavement Structures (1993)". Flexible pavement design is based on identifying a flexible pavements structural number (SN) to withstand the projected level of axle load traffic over the design period of the facility. The SN is obtained from a nomograph that relates the component of the pavement structure that can withstand the project ESALs. SN can also be obtained from the following equation by iteration:

$$log_{10}(W_{18}) = Z_R \times S_0 + 9.36 \times log_{10}(SN+1) - 0.20 + \frac{log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times log_{10}(M_R) - 8.07 \quad \text{..............(3.4.1)}$$

where,

W_{18}: predicted number of 18-kip equivalent single axle load applications,

Z_R: Standard normal deviate,

S_0: Combined standard error of the traffic prediction and performance prediction,

ΔPSI: Difference between the initial design serviceability index, p_0, and the design terminal serviceability index P_t, and

M_R: Resilient modulus (psi).

Drainage factors in flexible pavement design are generally taken into account through the use of modified structural layer coefficients. The factor for modifying the structural layer coefficient is called a m_i value. This drainage coefficient is integrated into the structural number (SN) equation shown below and is used to calculate the thickness of the various layers of the pavement structure.

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \quad \text{..............(3.4.2)}$$

where,

a_i: i^{th} layer coefficient

D_i: i^{th} layer thickness (inches), and

m_i: i^{th} layer drainage coefficient.

3) Design Condition

The design conditions of flexible pavement for the project road is summarized in the table below and the details are described in the following sections.

Table 3.4-8 Pavement Design Conditions

Criteria		Adoption	Remarks
Design Life		10 years	
Lane Distribution Factor	2-lane 4-lane	100% 80%	Guide for Design of Pavement Structures, AASHTO, 1993
Equivalence Factors of Vehicles	Passenger Car, Utility Bus Truck	0.0008 1.00 4.80	Pavement Design Guide for Roads & Highways Department
Design ESAL (W_{18})	Matarbari Port Access Road National Highway No.1 Other Intersecting Roads	20.82×10^6 40.01×10^6 5.70×10^6	
Level of Reliability		85%	P. II-9, AASHTO
Standard Normal Deviate (Z_R)		-1.037	P. I-62, AASHTO
Standard Deviation (S_0)		0.45	P. II-10, AASHTO
Initial Design Serviceability Index (P_0)		4.2	P. II-10, AASHTO
Design Terminal Serviceability Index (P_t)		2.5	P. II-10, AASHTO
Design serviceability index (ΔPSI)		1.7	$= P_0 - P_t$
Structural Layer Coefficient	Asphalt Concrete Asphalt Treated Base Course Crushed Aggregate Base Course Granular Aggregate Subbase Course	0.40 0.30 0.13 0.10	Fig 2.5, AASHTO 1993 1962 Interim AASHTO Coefficients Fig 2.6, AASHTO 1993, CBR=80 Fig 2.7, AASHTO 1993, CBR=25
Resilient modulus (MR) of subgrade	Asphalt Treated Base Course Crushed Aggregate Base Course Granular Aggregate Subbase Course Subgrade	200,000 28,000 15,000 7,500	Structural Layer Coefficient = 0.30 CBR = 80 CBR = 30 = 1,500 x CBR (soaked CBR = 5)

<u>Time Constraints</u>

Analysis period refers to the interval of time between reconstruction or major pavement rehabilitation efforts. The analysis period includes provision for periodic surface renewal or rehabilitation strategies which will extend the overall service life of a pavement structure before complete reconstruction is required. The AASHTO (1993) guide defines the performance period, often referred to as the design life, as the time from original construction to a terminal condition (see Serviceability below). The flexible pavements are designed for specified design life of 10 years.

<u>Traffic</u>

Traffic on a pavement structure includes numerous types of vehicles with varying weights and axle configurations (mixed traffic). To simplify procedures, equivalent single axle load (ESAL) is used to quantify various types of axle loadings into a single design number for pavement design. It is defined as one 80-kN, four-tire dual-axle load. Based on the traffic demand forecast, the traffic W_{18} of for the design period of 10 years (base year of 2026) is calculated based on the following formula.

$$w_{18} = D_D \times D_L \times \hat{w}_{18} \quad (3.4.3)$$

where

D_D: a directional distribution factor

D_L: a lane distribution factor, and

$\hat{w}_{18}$: the cumulative two-directional 18-kip ESAL units

Projected traffic loads on the project road, N1 and the other intersecting roads based on the future traffic demand forecast described in the section 3.2.2 are shown in Table 3.4-9, Table 3.4-10 and Table 3.4-11. For calculating the traffic loads on the other intersecting roads, the estimated traffic volume on R172 was used.

Table 3.4-9 Future Traffic Demand and ESAL 18-kip (Port Access Road)

		Future Traffic Demand (vehicle/day)				Cumulative Equivalent Standard Axle Load (ESAL) 18-kip				
		Car	Utility	Bus	Truck	Car	Utility	Bus	Truck	Total
						0.0008	0.0008	1.00	4.80	
1	2026	1,398	26	47	1,672	1.12	0.02	46.83	8,026.54	8,074.51
2	2027	1,462	28	47	1,802	1.17	0.02	46.83	8,648.24	8,696.26
3	2028	1,531	29	47	1,942	1.22	0.02	46.83	9,322.66	9,370.73
4	2029	1,606	30	47	2,095	1.28	0.02	46.83	10,054.23	10,102.37
5	2030	1,687	31	47	2,260	1.35	0.02	46.83	10,847.77	10,895.98
6	2031	2,029	32	66	2,416	1.62	0.03	65.81	11,598.99	11,666.44
7	2032	2,112	33	66	2,584	1.69	0.03	65.81	12,402.02	12,469.54
8	2033	2,200	34	66	2,763	1.76	0.03	65.81	13,260.35	13,327.94
9	2034	2,294	36	66	2,954	1.84	0.03	65.81	14,177.68	14,245.35
10	2035	2,395	37	66	3,158	1.92	0.03	65.81	15,157.95	15,225.71
Total		18,715	316	563	23,645	14.97	0.25	563.16	113,496.43	**114,074.82**

Source: JICA Survey Team

Table 3.4-10 Future Traffic Demand and ESAL 18-kip (N1)

		Future Traffic Demand (vehicle/day)				Cumulative Equivalent Standard Axle Load (ESAL) 18-kip				
		Car	Utility	Bus	Truck	Car	Utility	Bus	Truck	Total
						0.0008	0.0008	1.00	4.80	
1	2026	2,999	2,631	1,073	4,375	2.40	2.11	1,072.92	20,998.31	22,075.74
2	2027	3,106	2,739	1,093	4,589	2.48	2.19	1,093.35	22,029.33	23,127.35
3	2028	3,219	2,851	1,114	4,818	2.58	2.28	1,114.17	23,125.97	24,245.00
4	2029	3,340	2,968	1,135	5,061	2.67	2.37	1,135.41	24,293.10	25,433.56
5	2030	3,468	3,089	1,157	5,320	2.77	2.47	1,157.08	25,535.94	26,698.27
6	2031	3,852	3,194	1,196	5,560	3.08	2.56	1,196.15	26,687.38	27,889.17
7	2032	3,978	3,302	1,217	5,813	3.18	2.64	1,216.61	27,901.54	29,123.97
8	2033	4,111	3,414	1,237	6,080	3.29	2.73	1,237.44	29,182.20	30,425.66
9	2034	4,251	3,530	1,259	6,361	3.40	2.82	1,258.64	30,533.37	31,798.24
10	2035	4,399	3,650	1,280	6,658	3.52	2.92	1,280.23	31,959.31	33,245.98
Total		36,724	31,369	11,762	54,635	29.38	25.10	11,762.01	262,246.47	**274,062.96**

Source: JICA Survey Team

Table 3.4-11 Future Traffic Demand and ESAL 18-kip (Other Intersecting Roads)

		Future Traffic Demand (vehicle/day)				Cumulative Equivalent Standard Axle Load (ESAL) 18-kip				
		Car	Utility	Bus	Truck	Car	Utility	Bus	Truck	Total
						0.0008	0.0008	1.00	4.80	
1	2026	98	651	18	548	0.08	0.52	18.00	2,630.40	2,649.00
2	2027	101	688	18	570	0.08	0.55	18.00	2,736.00	2,754.63
3	2028	104	726	19	594	0.08	0.58	19.00	2,851.20	2,870.86
4	2029	108	767	19	618	0.09	0.61	19.00	2,966.40	2,986.10
5	2030	112	810	19	643	0.09	0.65	19.00	3,086.40	3,106.14
6	2031	114	838	20	660	0.09	0.67	20.00	3,168.00	3,188.76
7	2032	117	866	20	678	0.09	0.69	20.00	3,254.40	3,275.19
8	2033	120	895	20	697	0.10	0.72	20.00	3,345.60	3,366.41
9	2034	123	926	21	716	0.10	0.74	21.00	3,436.80	3,458.64
10	2035	126	957	21	735	0.10	0.77	21.00	3,528.00	3,549.87
Total		1,123	8,124	195	6,459	0.90	6.50	195.00	31,003.20	**31,205.60**

Source: JICA Survey Team

Based on the above formula and cumulative ESAL, accumulative axle loads of heavy vehicles (W_{18}) is calculated as follows:

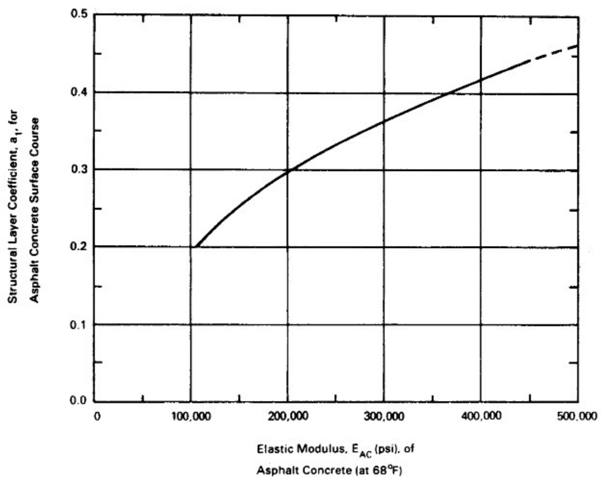
Table 3.4-12 Calculated Design ESAL (W_{18})

	Port Access Road	National Highway No.1	Other Intersecting Roads
Cumulative ESAL	114,074.82	274,062.96	31,205.60
Conversion to Yearly Traffic Volume	365 days	365 days	365 days
Directional Distribution Factor	50%	50%	50%
Lane Distribution Factor	100%	80%	100%
Design ESAL (W_{18})	20.82×10^6	40.01×10^6	5.70×10^6

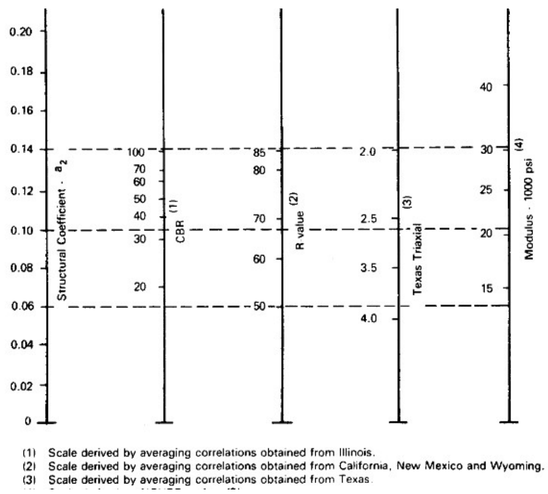
Source: JICA Survey Team

<u>Material Properties</u>

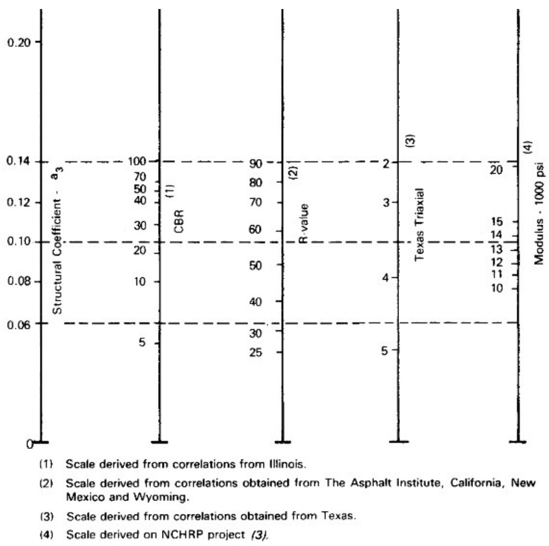
The resilient modulus (M_R) of each pavement layer was determined by the expected strength of it, such as structural layer coefficient or CBR, and the following nomograms.



Asphalt Treated Base Course



Crushed Aggregate Base Course



Granular Aggregate Subbase Course

Source: Guide for Design of Pavement Structures, AASHTO, 1993

Figure 3.4-13 Resilient Modulus (M_R)

4) Determination of Structural Layer Thicknesses

Based on the equation (3.4.1) and above mentioned design conditions, Structural Number (SN) of access road, N1 and other intersecting roads are obtained as 5.31, 5.80 and 4.40 respectively. Using the same equation, the required thickness of each pavement layer is calculated.

Surface Layer (Wearing and Binder Courses)

SN_1 is calculated from the equation (3.4.1) using the M_R of asphalt treated base course. This determines the amount of support the surface layer needs to provide in order for the asphalt treated base course layer to perform adequately.

Table 3.4-13 Calculated Thickness of Surface Layer

	Port Access Road	National Highway No.1	Other Intersecting Roads
M_R of asphalt treated base course	200,000 psi (Figure 2.5 of AASHTO 1993, Structural Layer Coefficient = 0.3)		
SN_1	1.55	1.73	1.20
Layer depth (D^*)	SN_1 / a_1 = 1.55 / 0.40 = 3.875 inch = 98.4 mm	SN_1 / a_1 = 1.73 / 0.40 = 4.33 inch = 109.9 mm	SN_1 / a_1 = 1.20 / 0.40 = 3.00 inch = 76.2 mm
Rounded value of D^*	100 mm (3.937 inch)	110 mm (4.331 inch)	80 mm (3.150 inch)
SN contribution of D_{surf}	$D_{surf} \times a_1$ = 3.937 × 0.40 = 1.575	$D_{surf} \times a_1$ = 4.331 × 0.40 = 1.73	$D_{surf} \times a_1$ = 3.150 × 0.40 = 1.26

Source: JICA Survey Team

Asphalt Treated Base Course Layer

SN_2 is calculated from the equation (3.4.1) using the M_R of crushed aggregate base course. This determines the amount of support the surface and base 1 layers needs to provide in order for the crushed aggregate base course layer to perform adequately.

Table 3.4-14 Calculated Thickness of Asphalt Treated Base Course Layer

	Port Access Road	National Highway No.1	Other Intersecting Roads
M_R of crushed aggregate base course	28,000 psi (Figure 2.6 of AASHTO 1993, CBR = 80)		
SN_2	3.32	3.70	2.68
SN_{surf}	1.57	1.73	1.26
SN_{base1}^* = SN_2 - SN_{surf}	1.75	1.97	1.42
Layer depth (D^*)	SN_{base1}^* / a_2 = 1.75 / 0.30 = 5.833 inch = 148.2 mm	SN_{base1}^* / a_2 = 1.97 / 0.30 = 6.567 inch = 166.8 mm	SN_{base1}^* / a_2 = 1.42 / 0.30 = 4.734 inch = 120.2 mm
Rounded value of D^*	150 mm (5.906 inch)	180 mm (7.087 inch)	120 mm (4.724 inch)
SN contribution of D_{base1}	$D_{base1} \times a_2$ = 5.906 × 0.30 = 1.77	$D_{base1} \times a_2$ = 7.087 × 0.30 = 2.12	$D_{base1} \times a_2$ = 4.724 × 0.30 = 1.41

Source: JICA Survey Team

Crushed Aggregate Base Course Layer

SN_3 is calculated from the equation (3.4.1) using the M_R of granular aggregate subbase course. This determines the amount of support the surface, base 1 and base 2 layers needs to provide in order for the granular aggregate subbase course layer to perform adequately.

Table 3.4-15 Calculated Thickness of Crushed Aggregate Base Course Layer

	Port Access Road	National Highway No.1	Other Intersecting Roads
M_R of crushed aggregate base course	15,000 psi (Figure 2.7 of AASHTO 1993, CBR = 30)		
SN_3	3.32	4.75	3.50
$SN_{surf} + SN_{base1}$	3.34	3.85	2.67
$SN_{base2} = SN_3 - SN_{surf} - SN_{base1}$	0.96	0.90	0.83
Layer depth (D^*)	$SN_{base2}^* / a_3 = 0.96 / 0.13$ = 7.385 inch = 187.6 mm	$SN_{base2}^* / a_3 = 0.90 / 0.13$ = 6.923 inch = 175.8 mm	$SN_{base2}^* / a_3 = 0.83 / 0.13$ = 6.386 inch = 162.2 mm
Rounded value of D^*	200 mm (7.874 inch)	200 mm (7.874 inch)	180 mm (7.087 inch)
SN contribution of D_{base2}	$D_{base2} \times a_3 = 7.874 \times 0.13$ = 1.02	$D_{base2} \times a_3 = 7.874 \times 0.13$ = 1.02	$D_{base2} \times a_3 = 7.087 \times 0.13$ = 0.92

Source: JICA Survey Team

Granular Aggregate Subbase Course Layer

SN_3 is calculated from the equation (3.4.1) using the M_R of the subgrade. This determines the amount of support the surface, base 1, base 2 and subbase layers need to provide in order for the pavement perform adequately.

Table 3.4-16 Calculated Thickness of Granular Aggregate Subbase Course Layer

	Port Access Road	National Highway No.1	Other Intersecting Roads
M_R of subgrade	7,500 psi (1,500 × CBR = 1,500 × 5)		
SN	5.31	5.80	4.40
$SN_{surf} + SN_{base1} + SN_{base2}$	4.36	4.87	3.59
$SN_{sub} = SN_4 - SN_{surf} - SN_{base1} - SN_{base2}$	0.95	0.93	0.81
Layer depth (D^*)	$SN_{sub}^* / a_4 = 0.95 / 0.10 =$ 9.500 inch = 241.3 mm	$SN_{sub}^* / a_4 = 0.93 / 0.10 =$ 9.300 inch = 236.2 mm	$SN_{sub}^* / a_4 = 0.81 / 0.10 =$ 8.102 inch = 205.8 mm
The greater of D^* and D_{min}	250 mm (9.843 inch)	250 mm (9.843 inch)	220 mm (8.661 inch)
SN contribution of D_{sub}	$D_{sub} \times a_4 = 9.843 \times 0.10 =$ 0.98	$D_{sub} \times a_4 = 9.843 \times 0.10 =$ 0.98	$D_{sub} \times a_4 = 8.661 \times 0.10 =$ 0.86

Source: JICA Survey Team

Table 3.4-17 to Table 3.4-19 and Figure 3.4-14 represent the summary of the above calculations.

Table 3.4-17 Calculated Thickness of Pavement Layers for Access Road

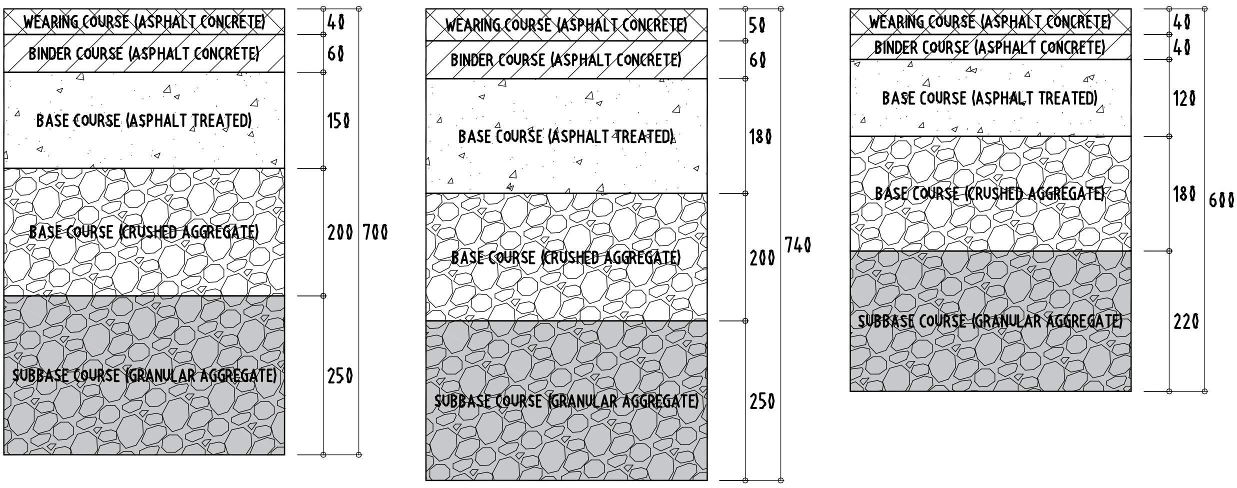
Pavement layer	Strength coefficient (a_i)	Drainage coefficient (D_i)	Thickness of layer		SN
			(mm)	(inch)	
AC wearing course	0.40		40	1.969	0.78
AC binder course	0.40		60	1.969	0.78
Asphalt treated base	0.30	1.00	150	5.906	1.77
Crushed aggregate base course	0.13	1.00	200	7.874	1.02
Granular aggregate subbase course	0.11	1.00	250	9.843	0.98
Total	-	-	700		5.33
Source: JICA Survey Team					5.33 > 5.31

Table 3.4-18 Calculated Thickness of Pavement Layers for N1

Pavement layer	Strength coefficient (a_i)	Drainage coefficient (D_i)	Thickness of layer		SN
			(mm)	(inch)	
AC wearing course	0.40		50	1.969	0.78
AC binder course	0.40		60	2.362	0.94
Asphalt treated base	0.30	1.00	180	7.087	2.12
Crushed aggregate base course	0.13	1.00	200	7.874	1.02
Granular aggregate subbase course	0.11	1.00	250	9.843	0.98
Total	-	-	740		5.84
Source: JICA Survey Team					5.84 > 5.80

Table 3.4-19 Calculated Thickness of Pavement Layers for Other Intersecting Roads

Pavement layer	Strength coefficient (a_i)	Drainage coefficient (D_i)	Thickness of layer		SN
			(mm)	(inch)	
AC wearing course	0.40		40	1.575	0.62
AC binder course	0.40		40	1.575	0.62
Asphalt treated base	0.30	1.00	120	4.724	1.41
Crushed aggregate base course	0.13	1.00	180	7.087	0.92
Granular aggregate subbase course	0.11	1.00	220	8.661	0.86
Total	-	-	600		4.43
Source: JICA Survey Team					4.43 > 4.40



Port Access Road **National Highway No.1** **Other Intersecting Roads**

Source: JICA Survey Team

Figure 3.4-14 Designed Pavement Layers

(6) Roadside Facilities

The following roadside facilities should be provided for ensuring road safety:

<u>Guard Rail</u>

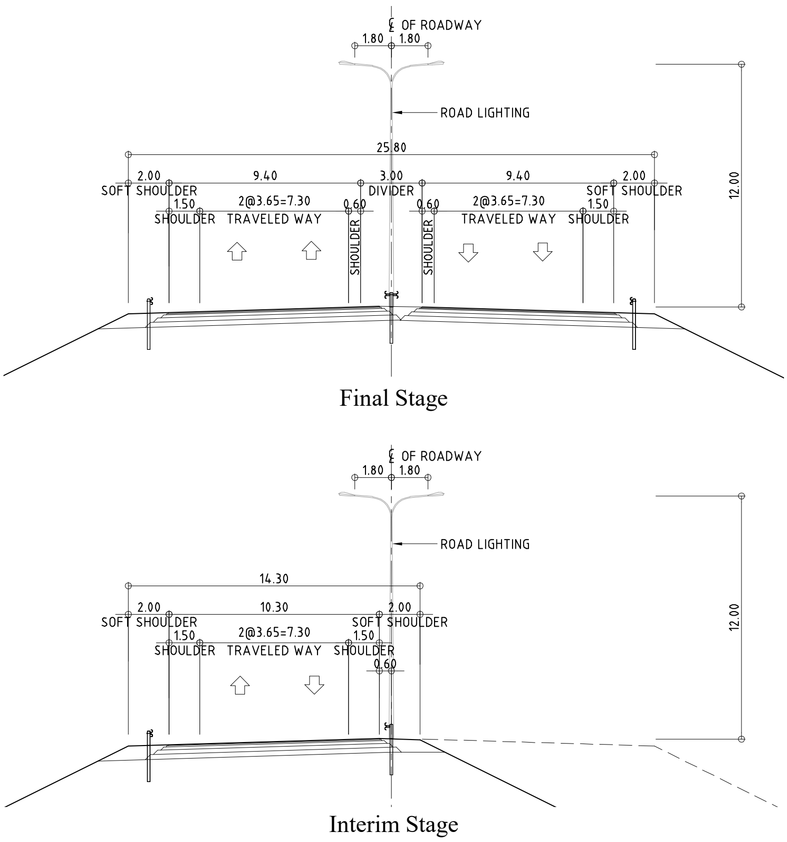
- Outer shoulder: Single-beam guardrail
- Median: Double-beam guardrail
- Height of guardrail: 0.6 ~ 1.0 m
- Interval of poles: 4.0 m
- Resistance to impact: 230 kJ (N･m, 25 ton, collision speed: 60 km/h)

<u>Delineators</u>

- Size: φ100 mm
- Installation intervals: 20 m

<u>Road Lightings</u>

- Height of lighting poles: 12 m
- Spacing of lighting poles: 35 m
- Type and lamp: LED (152 VA, 13,600 lm)
- Color temperature: 4,000 ~ 6,000 K



Source: JICA Survey Team

Figure 3.4-15 Road Lighting Poles

3.4.2 Bridge Design

(1) Design Criteria and Standard

1) Bridge Design Standards in Bangladesh

In accordance with the design standards in Bangladesh and in reference to the previous road and bridge projects in Bangladesh, the following design standards are applied for the bridge design of the project:

- Bridge Design Standards, Roads & Highways Department (2004)
- Bangladesh National Building Codes (BNBC)-1993 (Gadget 2006)
- Geometric Design Standards Manual (Revised) 2005, Roads and Highway Division
- Standard Tender Documents – Section-7: Technical Specifications, RHD, 2011
- AASHTO-LRFD Bridge Design Specifications (2010, 5th edition)
- AASHTO-Guide Specifications for LRFD Seismic Bridge Design (2011, 2nd edition)
- Standard Specifications and Code of Practice for Road Bridges Section :II (Indian Road Congress (IRC), 2010)
- Specifications for Highway Bridges-Japan Road Association (JRA) (2012)

2) Navigation Clearance and Design High Water Level

Bangladesh Inland Water Transport Authority (BIWTA) has specified the minimum vertical and horizontal clearance for free navigation considering the type of navigational routes which are classified as Class I to IV as shown in the table below. The water level for the basis of the vertical clearance shall be the Standard High Water Level (SHWL) to be given by BIWTA, which is the fortnightly mean water level with 5% of exceedance. BIWTA also requires prior consultation for approval of bridge design conditions for all bridge having a length of 100 m or longer.

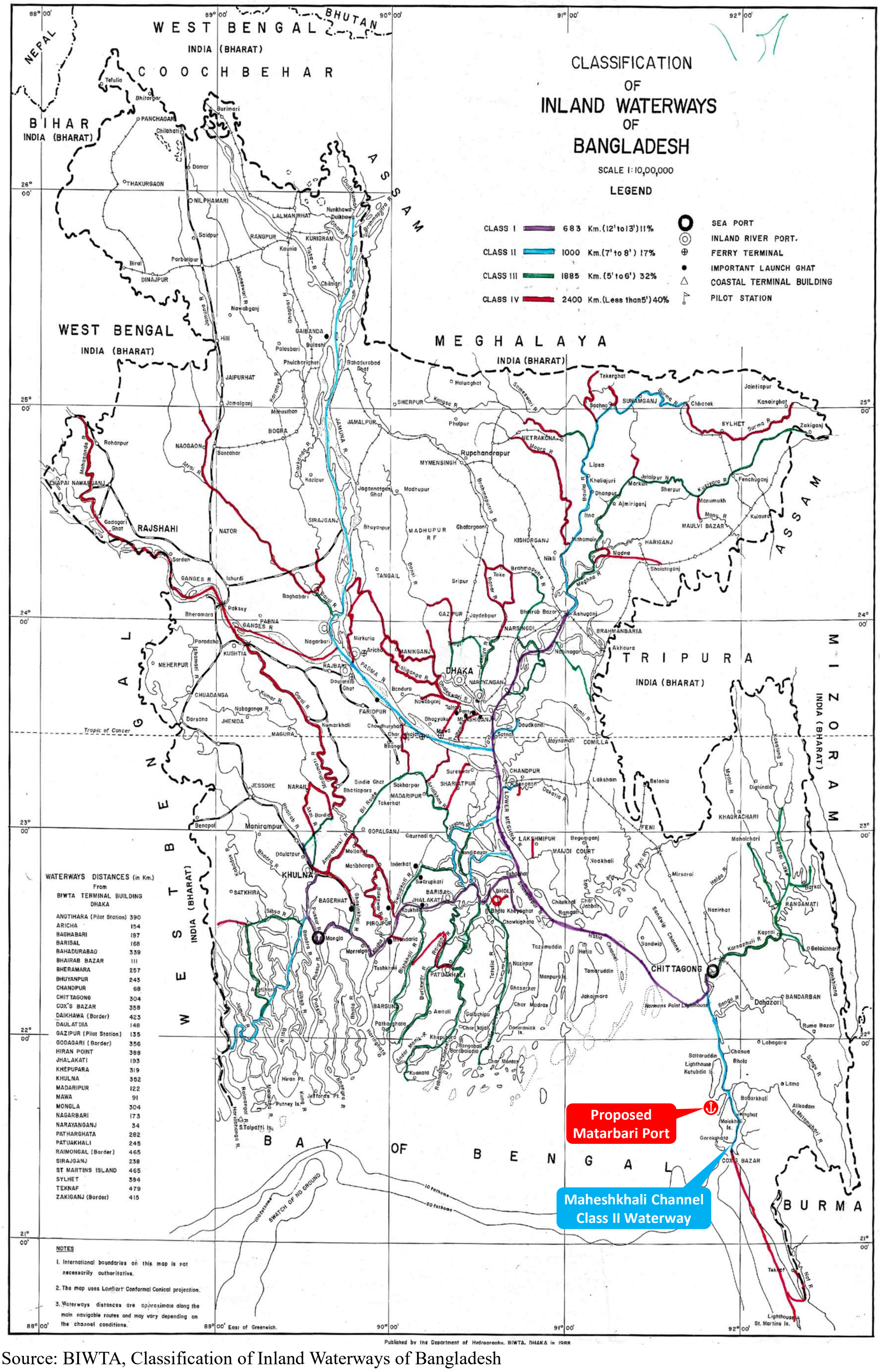
Whereas Maheshkhali Channel has been recognized as the Class II waterway, BIWTA suggested that Kohelia River should also be newly classified into Class II waterway as the result of joint site inspection with the officials of BIWTA and JICA Survey Team. Although, this study follows the BIWTA's recommendation for the purpose of securing budget of the project, the navigation clearance of Kohelia River should be re-examined through consultation with RHD and BIWTA during the detailed design stage.

In addition to the above, this study secured the vertical clearance under the girder bottom (soffit level) of the existing bridges across the rivers, which the Project Road will pass through, as one of the minimum requirements for bridge design. The design high water levels at targeted bridges are calculated as a100 year flood level.

Table 3.4-20 Navigation Waterways Limitation

Classification of Waterways	Minimum Vertical Clearance	Minimum Horizontal Clearance	Remarks
Class- I	18.30 m (60 ft)	76.22 m (250 ft)	
Class- II	12.20 m (40 ft)	76.22 m (250 ft)	Kohelia River Bridge Maheshkhali Channel Bridge
Class- III	7.62 m (25 ft)	30.48 m (100 ft)	
Class- IV	5.00 m (16.5 ft)	20.00 m (66 ft)	

Source: BIWTA



Source: BIWTA, Classification of Inland Waterways of Bangladesh

Figure 3.4-16 Classification of Inland Waterways

3) Design Loads

For bridge design, three (3) design loads are generally taken into account, namely, (i) dead load, (ii) live loads and (iii) earthquake.

Dead Load

For design dead load, the unit weights prescribed by AASHTO can be used to calculate the dead load of the structure.

Table 3.4-21 Unit Weight of Bridge Materials for Dead Load Calculation

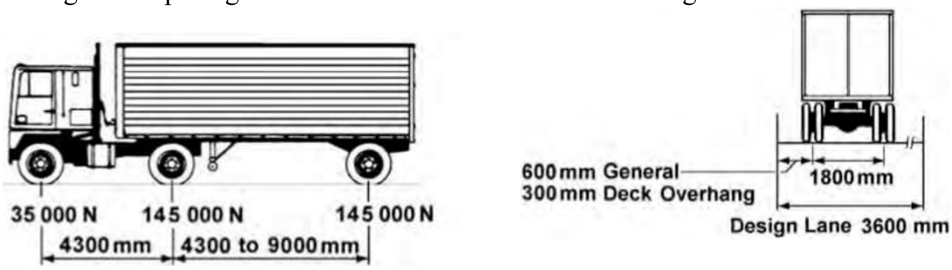
Material	Unit weight (kN/m^3)
Steel	77.0
Plain Concrete	23.0
Reinforced Concrete	24.5
Pre-stressed Concrete	24.5
Asphalt mix	22.5

Source: AASHTO-LRFD

Live Loads

According to AASHTO-LRFD, design live loads of the bridges shall consist of the following:

- Design Truck Load: In accordance with AASHTO (HS20-44), the total weight of truck load is 325 kN and the weight and spacing of each axle and wheel are shown in Figure 3.4-17.



Source: AASHTO-LRFD

Figure 3.4-17 Design Truck Load

- Design Lane Load: The lane load for girder and substructure design is summarized in Table 3.3. A uniform load of 9.3 kN/m is distributed in the longitudinal direction and spreads over a lane of 3 m width. The lane load should not be subjected to dynamic load allowance. A lane load should not be interrupted to provide space for the design truck or tandem (concentrated load), except where interruption in a patch loading pattern produces an extreme value for certain force effects.

Table 3.4-22 Lane Load Specifications for Girder and Substructure Design

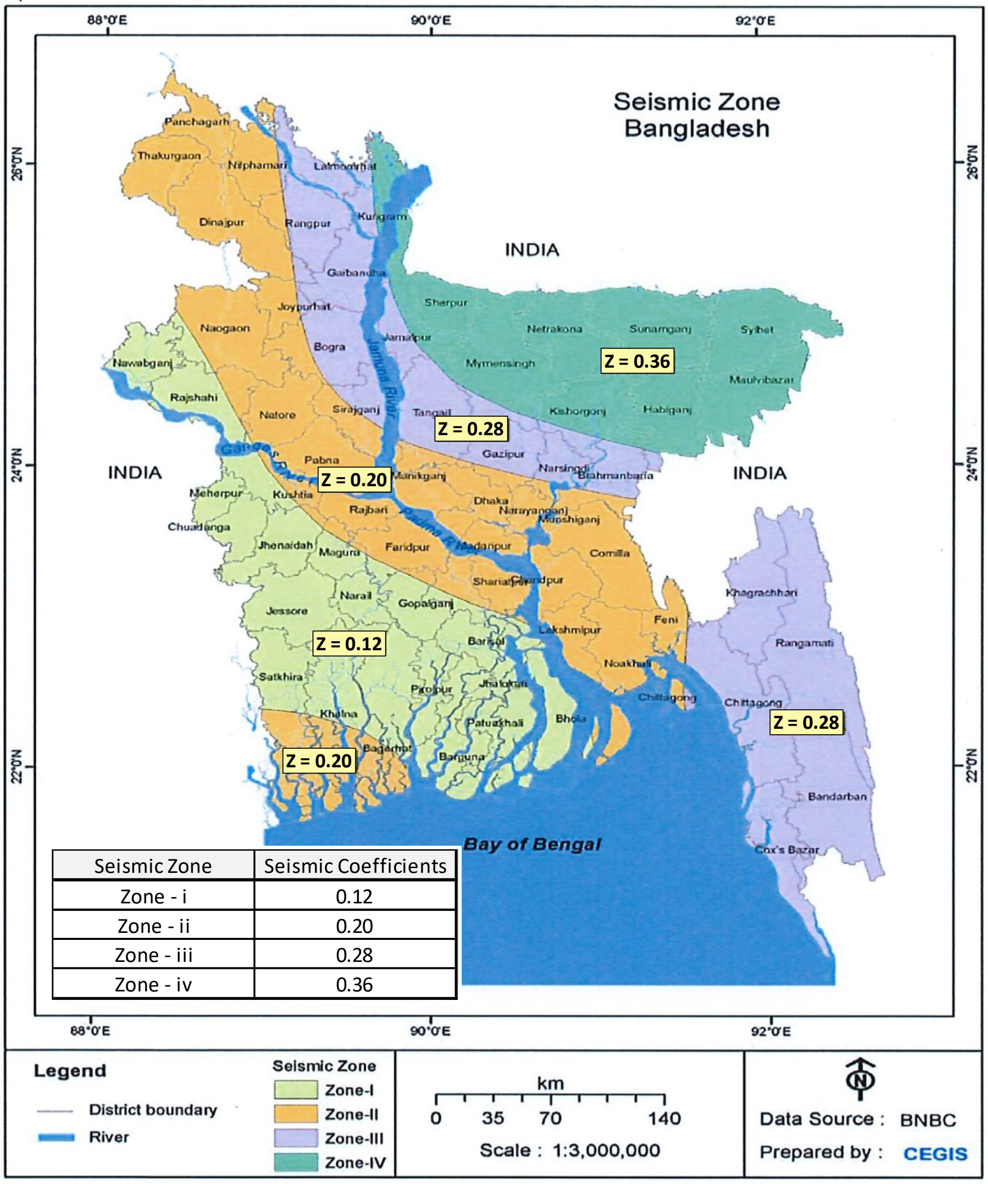
Specification	Truck load per lane (concentrated load)	Lane load over 3m lane width (uniformly distributed)	Multiple presence factor for 4-lane bridge	Impact (IM)
AASHTO (HS20-44)	325 kN	9.3 kN/m	65 %	33 % for truck load only

Source: AASHTO-LRFD

Earthquake (EQ)

To calculate the earthquake load, several input parameters including the zone coefficient, site soil coefficient and acceleration response spectrum are necessary to be considered. Therefore, BNBC (2013) was used as the standard to derive the design spectral acceleration in Bangladesh.

- Zone coefficient: In order to compute the earthquake load, firstly it is necessary to select the seismic zone under which area will be selected for bridges construction. The seismic zones are defined in the Bangladesh seismic zone map (see Figure 3.4-18) which is stipulated according to BNBC-2013 and with a return period of 2475 years. Based on the severity of the probable intensity of seismic ground motion and damages, Bangladesh is divided into four seismic zones which are shown with their zone coefficient in Figure 3.2. The seismic zoning map is upgraded from BNBC-2006 version where only three seismic zones were coded.
- Site classification: Site will be classified as type SA, SB, SC, SD, SE, S1 and S2. Classification will be made according to soil properties of upper 30 meters of the site profile as shown in **Table 3.4-23**.



Source: BNBC-2013

Figure 3.4-18 Bangladesh Seismic Zone Map

Table 3.4-23 Site Class for Soil Profile

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity $\overline{V}_s$ (m/s)	Standard Penetration Value, $\overline{N}$ (blows/30cm)	Undrained shear strength, $\overline{S}_u$ (kPa)
SA	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	--	--
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
SD	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
SE	A soil profile consisting of a surface alluvium layer with V_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with V_s > 800 m/s.	--	--	--
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	--	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types SA to SE or S_1	--	--	--

Source: BNBC-2013

- Design Spectral acceleration, Sa: The spectral acceleration for the design earthquake is given by the following equation:

$$S_a = \frac{2}{3} Z \frac{I}{R} C_s \geq \frac{0.4}{3} ZI$$

Where,

Sa: Design spectral acceleration (g),

Z: Seismic zone coefficient,

R: Response reduction factor which depends on the type of structural system (refer to AASHTO)

I: Structure importance factor; 1.25 for National Highway and Regional, *Cs*: Normalized acceleration response spectrum, which is a function of structure period and soil type (site class) as defined by following equations:

$$C_s = S\left(1+\frac{T}{T_B}(2.5\eta-1)\right) \quad \text{for} \quad 0 \le T \le T_B$$

$$C_s = 2.5S\eta \quad \text{for} \quad T_B \le T \le T_C$$

$$C_s = 2.5S\eta\left(\frac{T_C}{T}\right) \quad \text{for} \quad T_C \le T \le T_D$$

$$C_s = 2.5S\eta\left(\frac{T_C T_D}{T^2}\right) \quad \text{for} \quad T_D \le T \le 4\,\text{sec}$$

Where,

S = Soil factor which depends on site class and is given in Table 3.5,

T = Structure period which can be determined by applying the concept of structural dynamics and structural mechanics,

T= $2.01\times\sqrt{\delta}$

and

$$\delta = \frac{W_s h^3}{3EI} + \frac{0.8 W_p h_p^3}{8EI}$$

Ws: Weight of superstructure

Wp: Weight of substructure (pier)

h: Deck height from pile cap

hp: Pier height from pile cap

EI: Flexural rigidity of pier

TB: Lower limit of the period of the constant spectral acceleration branch given in Table 3.5 as a function of site class,

TC: Upper limit of the period of the constant spectral acceleration branch given in Table 3.5 as a function of site class,

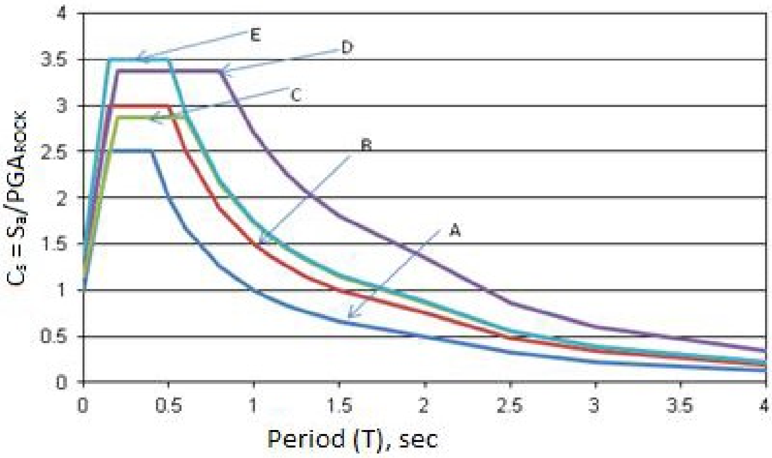
TD: Lower limit of the period of the constant spectral displacement branch given in Table 3.5 as function of site class,

η: Damping correction factor as a function of damping with a reference value of η=1 for 5% viscous damping.

Table 3.4-24 Site Dependent Soil Factor and Other Parameters

Soil type	S	T_B(s)	T_C (s)	T_D (s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

Source: BNBC-2013



Source: BNBC-2013

Figure 3.4-19 Normalized Design Acceleration Response Spectrum

- Response Modifications Factor (R): Seismic design force effects for substructures and the connections between parts of structures, listed in Table 3.6, shall be determined by dividing the force effects resulting from elastic analysis by the appropriate response modification factor R. Since BNBC recommends R values for building structures only, those values are determined from AASHTO Standard Specifications for Highway Bridges, which are as specified in **Table 3.4-25**.

Table 3.4-25 Response Modifications Factor R (AASHTO LRFD)

Substructure	R	Connection	R
Wall-type piers	2	Superstructure to abutment	0.8
Reinforced concrete pile bents		Expansion joint within a span of the superstructure	0.8
a. Vertical piles only	3		
b. One or more batter piles	2	Columns, piers or pile bents to cap beam or superstructure	1.0
Single columns	3		
Multiple column bent	5	Column or piers to foundations	1.0

Source: AASHTO-LRFD

4) Technical Specifications for Construction Materials

Concrete

In accordance with RHD practice, the values for 28-day-compressive strength of concrete cylinders for substructure components (RC pile caps, abutments, piers) shall be 30 MPa, whereas the concrete strength of deck slabs shall be 35 MPa, and pre-stressed concrete girders shall be 40 MPa. The concrete strength values according to bridge component are listed in Table 3.7.

Table 3.4-26 Strength Requirements of Concrete for Bridges

Bridge Components	**28-day compressive strength of concrete cylinder, σ_{ck} (MPa)**
RCC pile caps, abutments, piers, other structural components	30
Concrete deck slab	35
Prestressed concrete girder	40

Reinforcing steel bars

Reinforcing steel bars shall be deformed, except that plain bars or plain wire may be used for spirals, hoops, and wire fabric. Grade-500W is available in the Bangladesh market and their strengths are specified by the American Society for Testing Materials (ASTM). The ASTM specifications for the said grades are shown in Table 3.8.

Table 3.4-27 Nominal Stress of Reinforcing Steel Bars for Bridges

Steel grade	Yield stress σ_y (MPa)	Tensile strength σ_u (MPa)
Grade-500W	500	575

Source: JICA Survey Team

Pre-stressing Steel

Uncoated low relaxation seven-wire strands shall be used as prestressing steel in PC girder bridges. Prestressing steel shall conform to the ASTM specifications shown in Table 3.9.

Table 3.4-28 Nominal Stress of Pre-stressing Steel

Prestressing steel	Grade	Yield stress σ_y (MPa)	Tensile strength σ_u (MPa)
Strand (7-wire)	SWPR7BL	1,670	1,860

Source: JICA Survey Team

Steel Material

As per JRA/JIS specification, the steel material used in this project is summarized in Table 3.10 with tensile strength and yield stress.

Table 3.4-29 Nominal Stress of Steel

Steel grade ($16 < t \leq 40$ mm)	Yield stress σ_y (MPa)	Tensile strength σ_u (MPa)
SM400	235	400
SM490Y	355	490
SM520	355	520
SM570	450	570

Source: JICA Survey Team

(2) Bridge Type Selection

1) Long-Span Bridges

Superstructure

Kohelia River and Maheshkhali Channel are classified into Class II waterway of BIWTA and the navigation clearance of it is 76.22 m in horizontal direction and 12.20 m in vertical direction. The other rivers do not have such navigational requirement and that only the two bridges across Kohelia River and Maheshkhali Channel need to be considered as long-span bridges. The condition of the river and the required design conditions are as follows.

- The river and canal width is about 300 m;
- Navigational clearance is 76.22 m in horizontal direction and 12.20 m in vertical direction (BIWTA Class II);
- The bridge across the Maheshkhali Channel will be bridged to about 600 m upstream of the existing bridge (span length: 43.5 m @ 8).
- The ground conditions at the river crossings are viscous soil with the N-value about 10 on the upper layer, and the support layer (sand layer) at the position of GL - 25 m to 30 m is a soft ground.

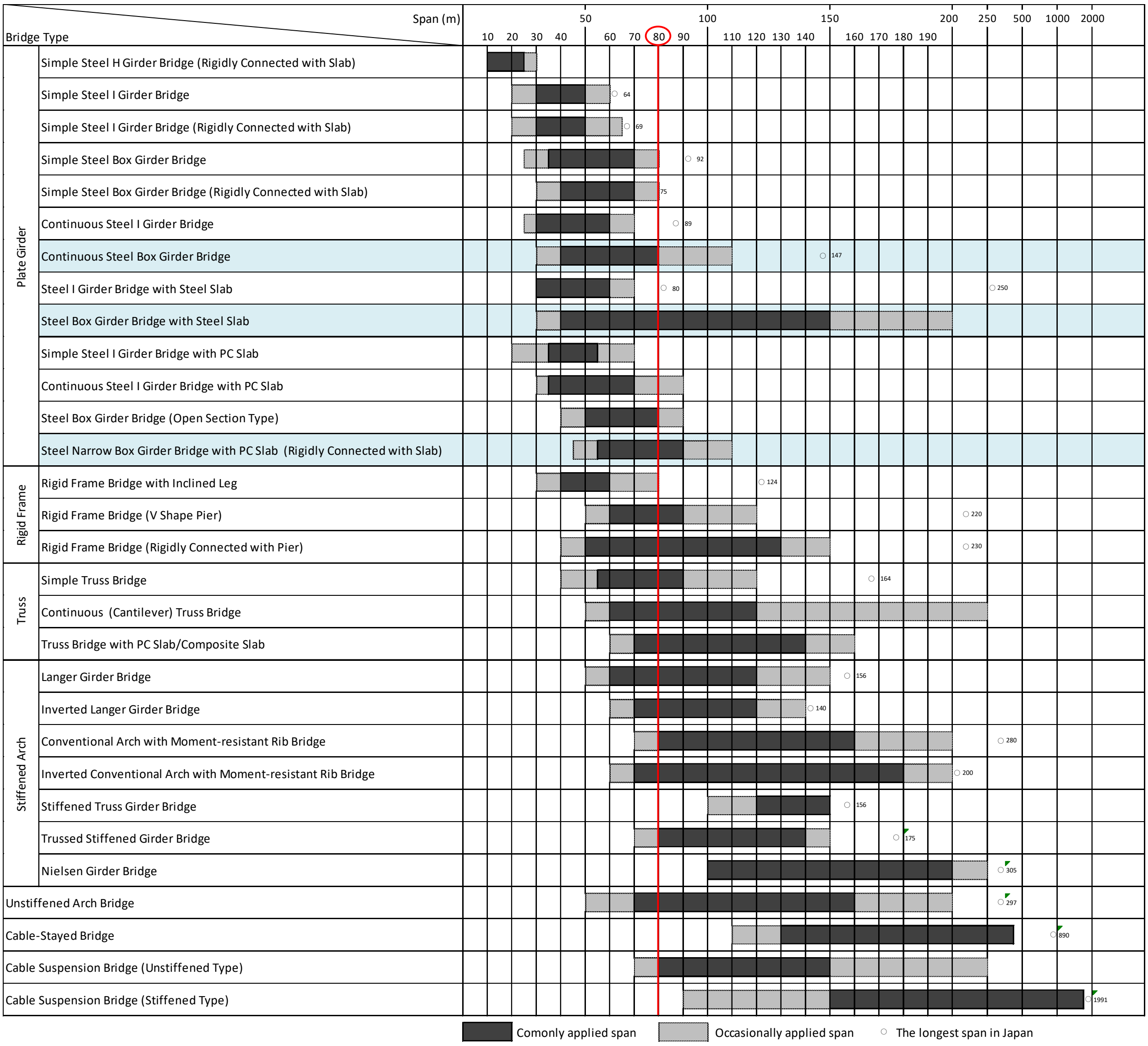
As for the river crossing part of these two bridges, a span length of about 80 m is required to secure the BIWTA's clearance. In reference to the "Applicable Bridge Span for Bridge Types" (Design Data Book 2016 Japan Bridge Construction Association, see Table 3.4-30), the "steel box girder type" and the "steel narrow box girder type" (with composite deck slab) were selected from the plate girder types as the candidates of the bridge type options (steel slab box girder bridge is obviously costly and therefore it was excluded from the candidates). Also, from the Prestressed Concrete bridge type, PC box girder bridge, which is an economical bridge style with a span length of 80 meters, were selected as the alternative option. The following three alternative options were examined.

- PC box girder bridge
- Conventional steel box girder bridge
- Steel narrow box girder bridge (composite deck)

The result of comparative analysis of the three options is shown in Table 3.4-31, and the steel narrow box girder bridge was selected as the optimum option for long-span bridges because of the following reasons:

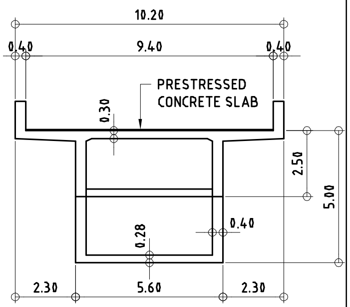
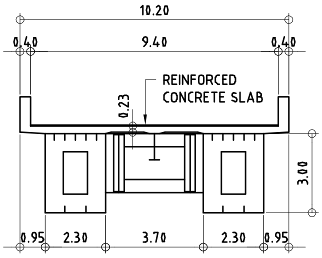
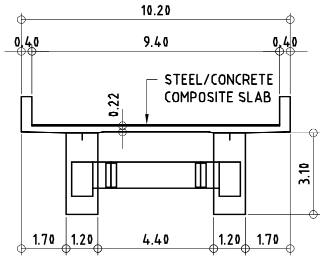
- The weight of steel narrow box girder is relatively light (55% of the weight of PC box girder) and it has advantage against the soft ground conditions;
- The steel/concrete composite deck is more durable than conventional deck slab;
- The number of parts and the painted area can be minimized and that cost for construction and maintenance would be smaller than conventional steel box girder bridge; and
- Required time for construction is less because less number of parts and the formwork for the deck slab as well as the unnecessity of scaffold.

Table 3.4-30 Applicable Bridge Span for Bridge Types (Experience in Japan)



Source: Japan Bridge Association, Design Data Book (2016)

Table 3.4-31 Comparison of Bridge Type for BIWTA's Class II Waterway

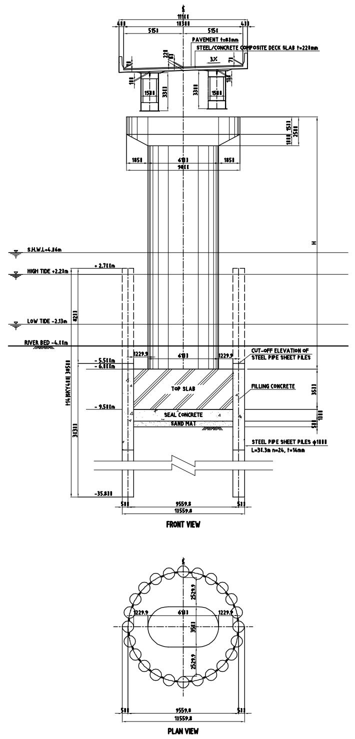
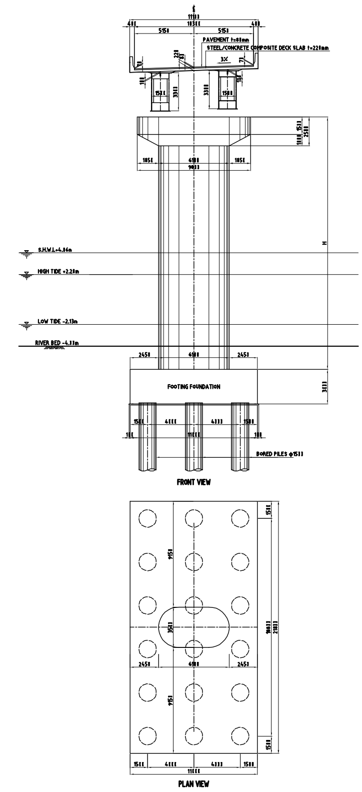
	PC Box Girder	**Conventional Steel Box Girder**	**Steel Narrow Box Girder**
Cross Section			
Structural Stability	**Fair** • Heavier than steel bridge and substructure is bigger than steel bridge. • Higher girder is required than steel bridge and elevation of roadway is higher than steel bridge.	**Good** • Lighter than PC box girder (approximately 60% of the weight of PC box girder) and has advantage in seismic durability against soft soil condition.	**Excellent** • Steel and concrete composite slab has more durability than RC slab. • Lighter than PC box girder (approximately 55% of the weight of PC box girder) and has advantage in seismic durability against soft soil condition.
Construction Cost (Rate)	**1.05**	**1.03** • Weight of Steel: 100% • No. of Major Parts: 100% • No. of Minor Parts: 100% • Welding Length: 100% • Painting Area: 100%	**1.00** • Weight of Steel: 90% • No. of Major Parts: 45% • No. of Minor Parts: 55% • Welding Length: 70% • Painting Area: 65% (71%, including steel deck slab) • Weight of steel, number of elements and painting area can be reduced and more economical than conventional steel box girder bridge.
Constructability	**Good** • Cantilever method • Construction period for superstructure: **21.5** months	**Good** • Launching erection method from the deck slab of approach bridge. • Duration of fabrication of steel parts is longer because of the complexed structure. • Construction period for superstructure: **17.5** months	**Excellent** • Launching erection method from the deck slab of approach bridge • Duration of fabrication of steel parts is shorter because of the simple structure. • Construction period for superstructure: **17** months
Easiness of Maintenance	**Excellent** • Concrete structure is prone to be damaged by airborne salinity and painting would be required for such case. However the need of maintenance is less than steel bridge.	**Fair** • Repainting on steel elements and rehabilitation of RC slab is required.	**Good** • Repainting is required but the area is fewer than conventional steel box girder bridge. • Composite slab has more durability than RC slab.
Aesthetic Aspect	**Fair** • Elevation of roadway is higher than steel bridge and the height difference with the existing bridge is bigger.	**Good** • Girder height is lower than PC box girder.	**Good** • Girder height is lower than PC box girder.
Evaluation			**Recommended**

Source: JICA Survey Team

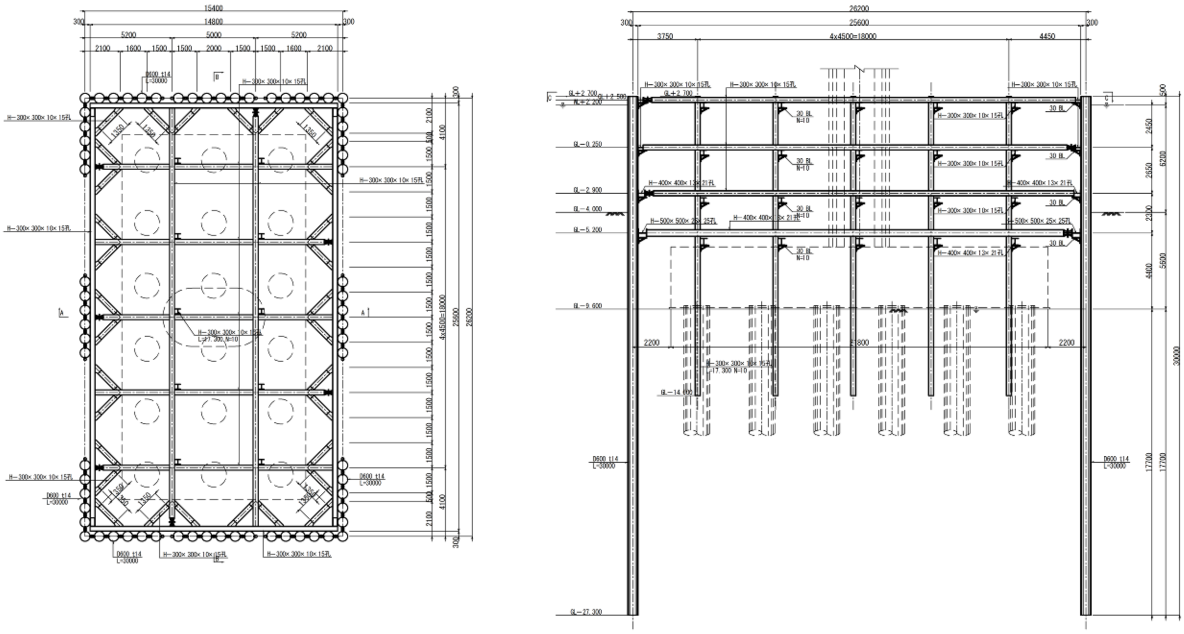
Foundation

The substructure of long-span bridges, which would receive large reaction forces from the superstructure, would require large-scale cofferdam work within river for construction. Therefore, comparison of foundation type selection for long-span bridges was made with the two options, namely the steel pipe sheet pile foundation and the bored pile foundation. The result of the comparative analysis is summarized in Table 3.4-32.

Table 3.4-32 Comparison of Foundation Type for Long-Span Bridges

	Steel Pipe Sheet Pile Wall (SPSP)	**Bored Pile**
		
Structural Stability	**Fair** • The steel pipe sheet pile behaves as a unit, and has many achievements on the basis of large bridges.	**Fair** • Bored pile is a general foundation structure and has many achievements.
Constructability	**Good** • Because the foundation structure also serves as a coffering work, no coffering work is required. • Duration of fabrication：6.2months	**Bad** • The coffering work by a steel pipe sheet pile is required at the time of footing and pier structure construction. • Duration of fabrication：11.6months
Impact on Water Environment	**Good** • The foundation shape is smaller than the bored pile plan and the construction period is short, so the influence on the water environment is small.	**Fair** • The foundation shape is bigger than the steel pipe sheet pile plan and the construction period is long, so the influence on the water environment is great.
Riverbed Scouring	**Good** • Because the basic shape is small, river scouring is small.	**Fair** • Because of its large base shape, river scouring is large.
Construction Cost (BDT) • Pile & Pier • Coffering • Total	 421,066,000 - 421,066,000 (1.00)	 194,105,000 613,043,000 807,148,000 (1.92)
Evaluation	**Recommended**	

Although a caisson foundation can also be considered as an alternative option, it was excluded because it would require construction of temporary islands within the river, which is a BIWTA's Class II waterway. For the bored pile foundation option, cofferdams made with the steel pipe sheet pile shown in Figure 3.4-20 are necessary for construction of the pier.



Plan View Section View

Source: JICA Survey Team

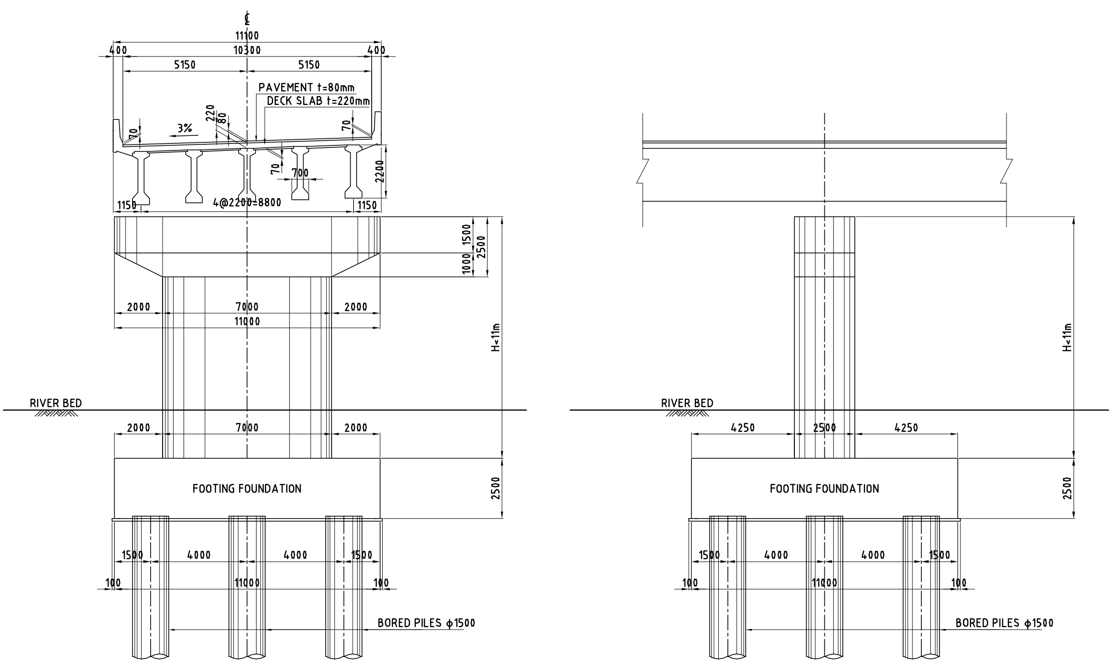
Figure 3.4-20 Cofferdam of the Bored Pile Plan

As the result of comparative analysis for the foundation structure type of long-span bridges, steel pipe sheet pile foundation was selected as the optimal option because of the following reasons.

- The steel pipe sheet pile can be used for both permanent foundation and cofferdam work and it is not necessary to construct cofferdam and foundation separately unlike the bored pile option;
- More economical than the bored pile option (including the cost for cofferdam work);
- The construction period is shorter than that of the bored pile plan option; and
- The size of steel pipe sheet pile option is smaller than that of the bored pile option, and the impact on rivers such as scouring would be minimized.

2) Short-Span Bridges

As a short span bridge, it is generally recognized that the PC-I girder type is an optimal and economical bridge type at most field conditions and many RHD's bridge projects in Bangladesh applied this particular bridge type. Therefore, the PC-I girder type bridge is adopted as a short-span bridge without special comparison. Figure 3.4-21 represents the general view of the PC-I girder type bridge.



Front View Side View

Source: JICA Survey Team

Figure 3.4-21 PC-I Girder Bridge for Short-Span Bridges

3) Selected bridge Types

Based on the hydrological study and the site investigation, a total of 15 locations were evaluated as probable candidates for bridge construction.. Kohelia River and Maheshkhali Channel require long-span bridges for providing the navigation clearance of BIWTA's Class II waterways. These bridges require long bridge lengths due tothe high-vertical clearance (12.20 m above Standard High Water Level). However, the necessity for long-span bridges is only within the waterways and it would be more reasonable to adopt PC-I girder type bridges for the approach sections.. Therefore, the steel narrow box girder bridge type was adopted only at the river crossing section of BIWTA's Class II waterways and PC-I girder type bridge was adopted for the remaining sections.

According to the hydrological study, 9 out of the 15 locations may not necessarily be of bridge construction because these watercourses do not serve for draining watershed and currently have no water flow because the downstream of these watercourses are dammed. However, this study recommends considering these locations as bridge construction for budget securing purposes. Further detailed engineering study should be undertaken during the detailed design of the project for finalization of the scope of the works.

In addition to the above, existence of alternate layers of sand and clay were observed at the section from 14+640 to 15+900. Consolidation settlement at the section would not be achieved by surcharge with PVD during the target construction period because PVD cannot be penetrated into sand layer. Installation of sand compaction piles would be an alternative option for embankment stability but it is too costly if compared with bridge construction. Therefore, an additional bridge was determined to be constructed.

The bridge types of the 16 bridges are summarized Table 3.4-33.

Table 3.4-33 List of Bridge

No.	Station		Bridge Type	No of Spans	Span Arrangement (m)	Total Length (m)	River Name	Remarks
	From	To						
1	0+806	2+040	PC-I	11	11@40m=440m	1234	Kohelia River	BIWTA Class II
			Steel Narrow Box Girder	4	70m+2@87m+70m =314m			
			PC-I	12	12@40m=480m			
2	4+430	4+670	PC-I	6	6@40m	240	Nonaichnari Khal	*
3	6+080	6+215	PC-I	3	3@45m	135		*
4	9+890	10+115	PC-I	4	4@40m	160		*
5	10+680	12+154	PC-I	11	11@40=440m	1474	Maheshkhali Channel	BIWTA Class II
			Steel Narrow Box Girder	4	70m+2@87m+70m =314m			
			PC-I	18	18@40m=720m			
6	14+090	14+450	PC-I	9	9@40m	360	Bura Matamuhuri Khal	
7	16+490	16+760	PC-I	6	6@45m	270	ditto	
8	18+550	18+910	PC-I	9	9@40m	360	Matamuhuri	
9	20+460	20+580	PC-I	3	3@40m	120	Batamani Khal	*
10	21+340	21+430	PC-I	2	2@45m	90	ditto	*
11	21+530	21+690	PC-I	4	4@40m	160	ditto	*
12	21+785	21+920	PC-I	3	3@45m	135	ditto	*
13	22+680	22+840	PC-I	4	4@40m	160	Fasiakhali Chara	*
14	23+390	23+550	PC-I	4	4@40m	160	ditto	*
15	24+455	24+495	PC-I	1	40m	40		
16	14+640	15+900	Steel I Girder	23	50m+4@60m+50m x 3 +45m+3@50m+45m	1,260		Soft Ground
17	9+012	9+683	PC-I	3	3@45m=135m	671		LNG Pipeline
			Steel Narrow Box Girder	1	70m			
			PC-I	11	35m+5@45m+26m +4@45m=466m			

Note: * represents that bridge construction may not be necessary in view of drainage purpose. Further engineering study should be made during the detailed design stage in order to justify the necessity of bridges.

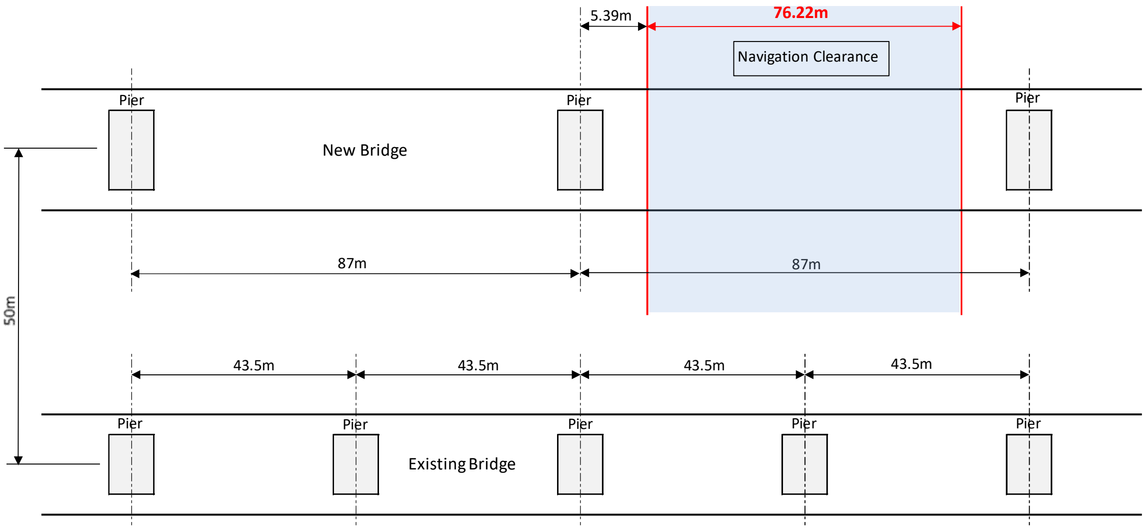
Source: JICA Survey Team

(3) Steel Narrow Box Girder Bridges

1) Superstructure Design

<u>Span</u>

Because the bridge across Maheshkhali Channel will be constructed about 50m upstream of the existing Maheshkhali-Badarkhali Bridge, it is necessary to consider the pier arrangement of the existing bridge. In order to avoid as much as possible local scouring due to disturbance of the streamlines, there is need to secure a minimum 76.22 m of horizontal clearance, also new bridge piers should be placed beyond two spans of the existing bridge (43.5 m intervals) which is equivalent to 87.0 m intervals.

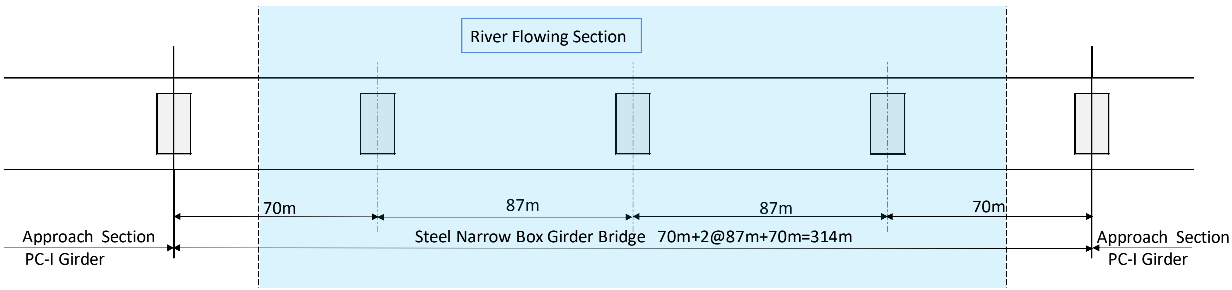


Source: JICA Survey Team

Figure 3.4-22 Pier Arrangement of Steel Narrow Box Girder Bridges

Considering that the width of Maheshkhali Channel is approximately 250 m and that the BIWTA's navigation clearance should be secured only at the middle of the watercourses. Therefore, only the two spans at the middle of the bridge should have the navigation clearance and the length of the span next to the center spans should be 70 m as the most preferable span from the moment balance, which should be approximately 0.8 times of the center span.

There is no physical restriction for Kohelia River but the width of the watercourse is almost same as Maheshkhali Channel. Therefore, same dimensions of bridge spans as Maheshkhali Channel Bridge are applied for Kohelia River Bridge. In this regard, the span arrangement of the steel narrow box girder bridge was decided as 70 m + 2 @ 87 m + 70 m = 314 m continuous 4 spans.



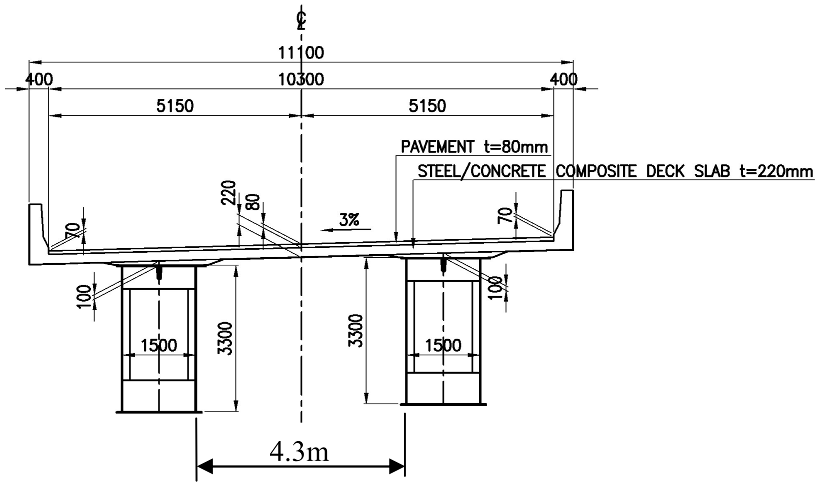
Source: JICA Survey Team

Figure 3.4-23 Pier Span of Steel Narrow Box Girder Bridges

Cross Section

The box width of the steel narrow box girder should be 1.5 m width (one vertical rib arrangement is possible) which makes it possible to simplify the structure inside the box. Also, the span length of the deck slab is 4.3 m, and the thickness of the deck of the composite slab at this time is 220 mm based on the following formula (Specifications for Highway Bridges, Japan Road Association).

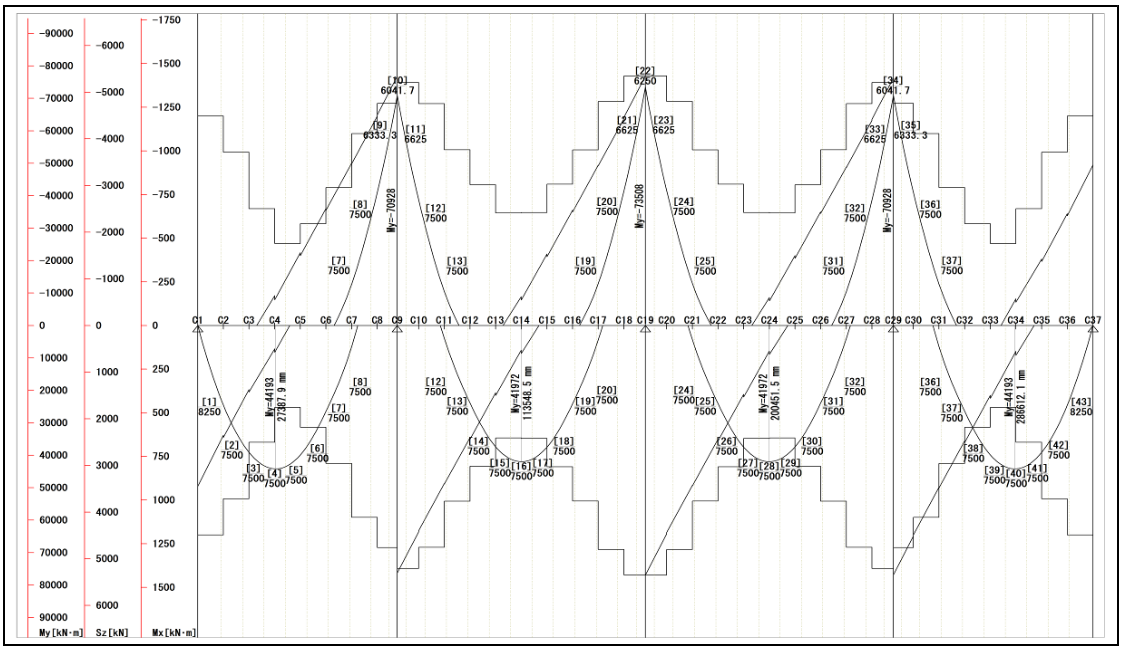
$25\times L+110=25\times 4.3+110=217.5 \quad \rightarrow 220$ mm



Source: JICA Survey Team

Figure 3.4-24 Cross Section of Steel Narrow Box Girder Bridges

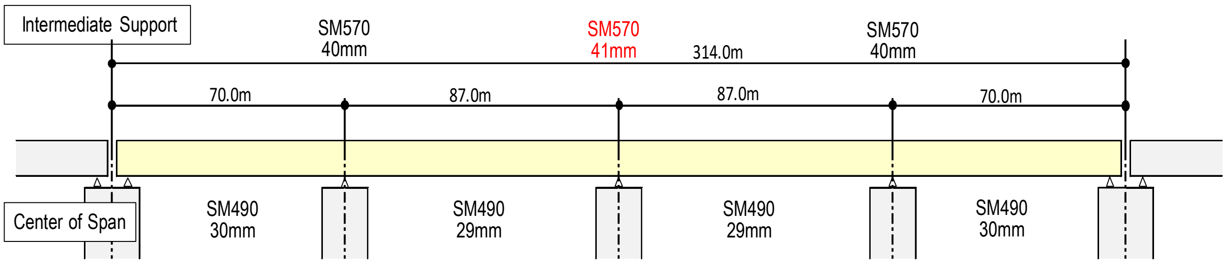
The girder height was set to 3.3 m, and it was confirmed from the preliminary calculation result that the cross section configuration can be achieved with a plate thickness allowing bolt attachment. The figure below shows the bending moment and the shear force diagram of the main girder, Figure 3-10 shows the steel type of the flange and the maximum plate thickness.



Source: JICA Survey Team

Figure 3.4-25 Bending Moment and Shear Force Diagram of Main Girder

The maximum thickness is 41 mm (SM 570) at the intermediate fulcrum.



Source: JICA Survey Team

Figure 3.4-26 Steel Type and Maximum Thickness of Main Girder Flange

Approximate quantity

The table below shows the approximate quantities of steel weight and painted area of steel narrow box girder bridge used for cost calculation.

Table 3.4-34 Approximate Quantity of Steel Narrow Girder Bridge

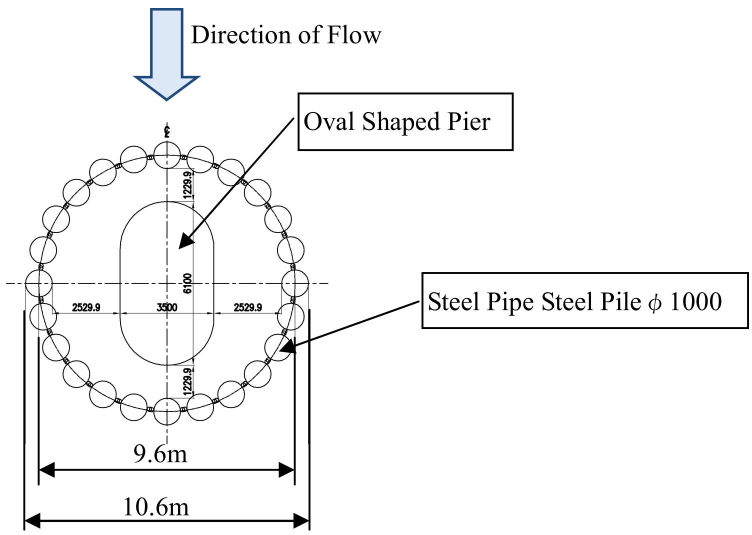
		Unit	Steel Narrow Box Girder L=314m
Steel Weight	Girder	ton	1,251.4
	Cross Beam	ton	75.7
	H.T. Bolt	ton	39.8
	Total	ton	1,366.9
Composite Slab		cu.m	876.3
Painted Area		sq.m	27,253.1

Source: JICA Survey Team

2) Substructure Design

Pier Shape

The bridge piers to be constructed should be of oval shape as shown in the figure below so as to minimize as much as possible obstruction to water flow within the river.



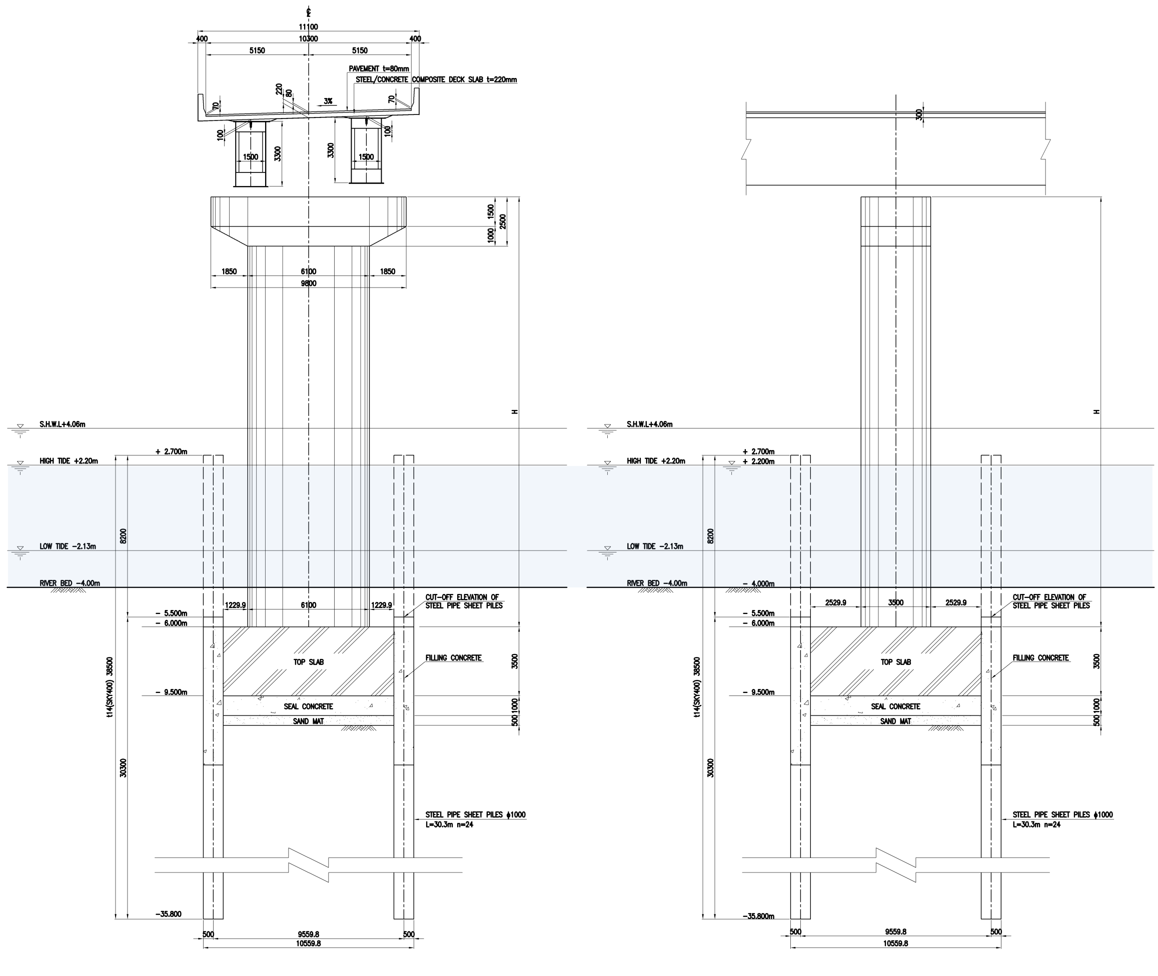
Source: JICA Survey Team

Figure 3.4-27 Cross Sectional Shape of Pier and Foundation

Because the shape of the steel pipe steel pile foundation can be compact, the pier type is designed as column type with overhanging beams as shown in the front view of Figure 3.4-28.

Foundation Shape

The shape of steel pipe sheet pile foundation should be a circular shape as shown in Figure 3.4-27 and Figure 3.4-28, and the foundation shape can be fitted within the width of the superstructure (11.1 m) in order to secure the space for the construction works of the bridge which will be constructed at the final stage.



Front View Side View

Source: JICA Survey Team

Figure 3.4-28 Pier and Foundation Shape of Steel Narrow Girder Brodge

(4) Prestressed Concrete I Girder Bridges

1) Superstructure Design

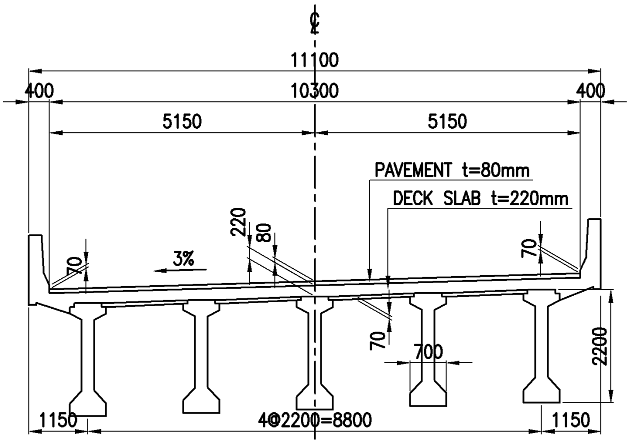
Standard Span

The spans of the PC - I girder bridge are standardized to several types in order to optimize with each site and shorten the construction period. Also, since the ground conditions at the location of bridge construction are dominant with a soft ground, the less number of substructures would result in the less construction cost. Therefore, two types of girder length, namely 40 m and 45 m, which are the maximum length applicable for PC-I girder, are set as the optimum standard span.

Typical Cross Section

Figure 3.4-29 shows the typical cross section of the PC - I girder bridge (cross section on the provisional side, total width W = 11.1 m). Secure the overhang length of the deck slab so that the drainage pipe of the bridge surface drainage will not interfere with the main girder, to be 1.15 m. Also, the thickness of the deck slab is 220 mm as shown in the following formula from the main girder spacing 2.2 m based on the equation of Specification for Highway Bridges (Japan Road Association).

$(30 \times L + 110) \times 1.25 = (30 \times 2.2 + 110) \times 1.25 = 220.0 \rightarrow 220$ mm



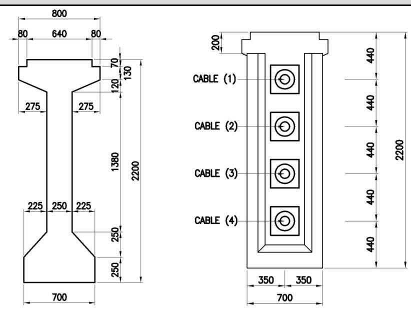
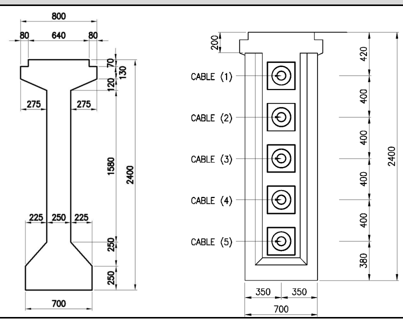
Source: JICA Survey Team

Figure 3.4-29 Typical Cross Section of PC-I Girder Bridge

Cross Section of PC-I Girder

The PC - I girder section with standard span lengths of 40 m, 45 m and the approximate number per girder for cost calculation are shown in the table below.

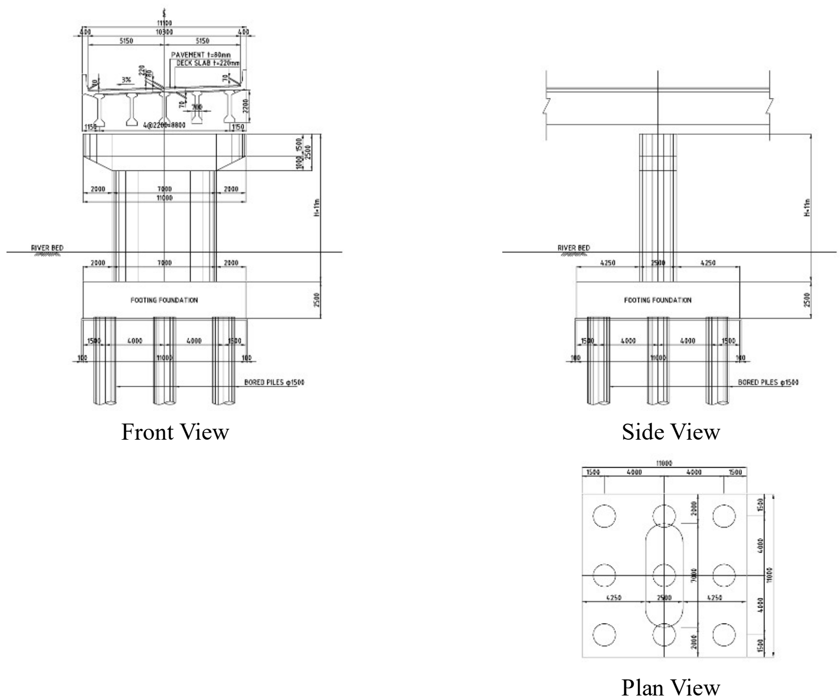
Table 3.4-35 Approximate Quantity of Steel Narrow Girder Bridge

Girder Length (m)	L=40m	L=45m
Girder Shape		
Girder Height (m)	2.2 m	2.4 m
Concrete (cu.m)	38.50 cu.m per girder	45.59 cu.m per girder
Cable Strand (7-wire)	12T15 × 4	12T15 × 5
Cable Weight (ton)	2.10 ton per girder	2.96 ton per girder

2) Substructure Design

Pier Shape

The shape of the bridge piers within the rivers should be an oval shape same as the bridge piers of the steel narrow box girder bridge. Also, the footing width in the direction perpendicular to the bridge axis should be within the width of superstructure (11.1 m) in order to secure the space for the construction works of the bridge which will be constructed at the final stage.



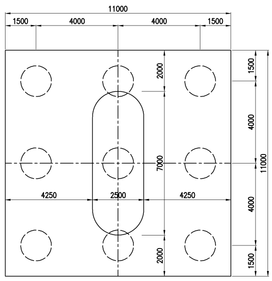
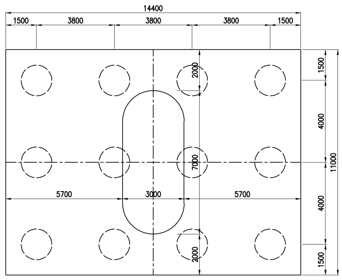
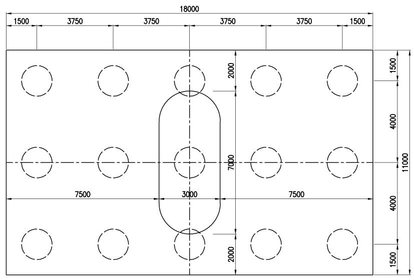
Source: JICA Survey Team

Figure 3.4-30 Pier Shape of PC-I Girder Bridge

<u>Foundation Pile</u>

Similar to the other bridges in Bangladesh, bored pile foundation was adopted because of the material availability and easiness of construction. The pile diameter adopted in this preliminary design was 1.5 m where the pile support force become large, and the length of each pile is determined from the result of the ground survey at the bridge construction site. In consideration of the magnitude of the bending moment at the lower end of the piers, the required numbers of piles were determined as three types depending on the height of piers.

Table 3.4-36 Number of Piles of PC-I Girder Bridge

	H＜11.0m	11.0m≦H≦15.0m	15.0m＜H
Pile Arrangement			
Number of Piles	3×3=9	3×4=12	3×5=15

3.4.3 Soft Ground Analysis and Design

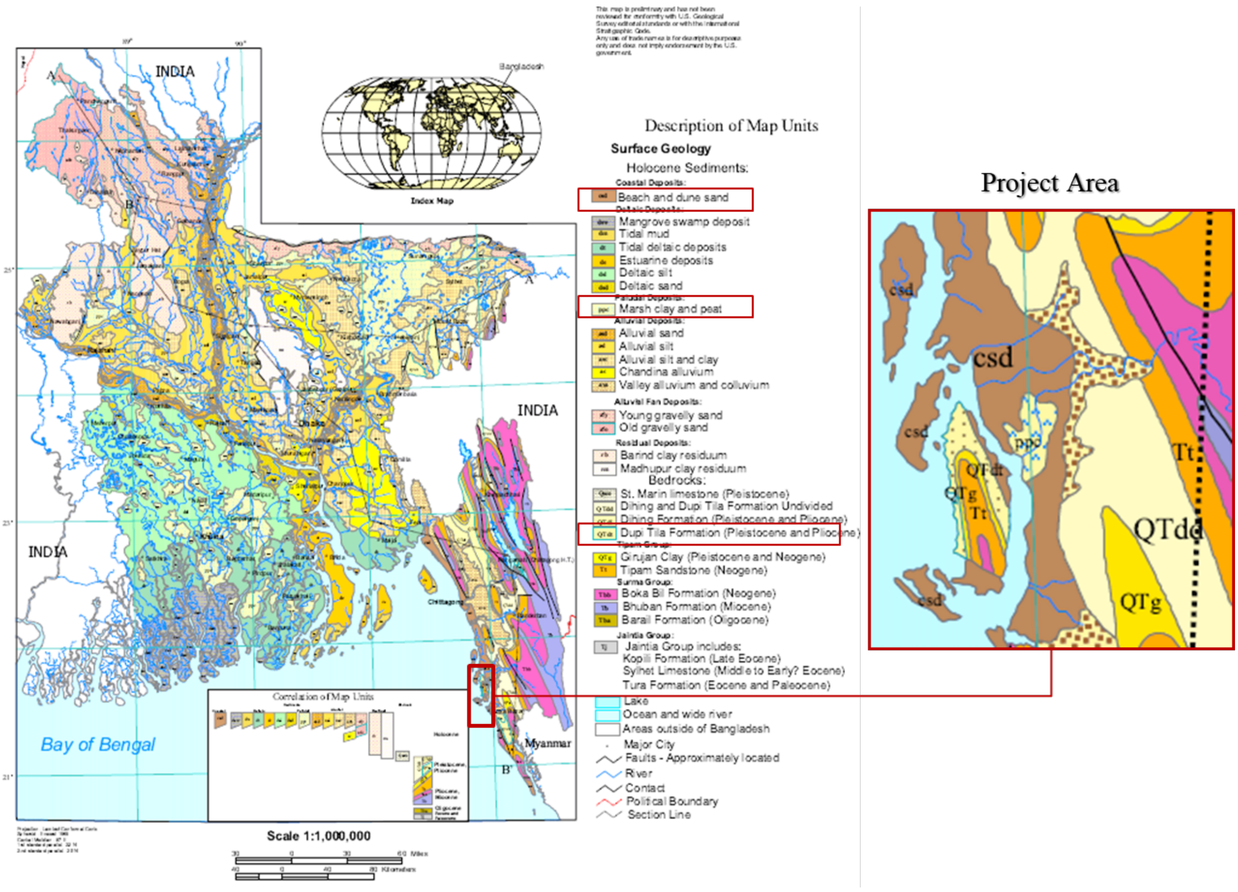
The proposed alignment of the Matarbari Port Access Road overlies on weak compressible soils. The results of the boreholes and laboratory tests indicate that the presence of a soft soil layer along the alignment with thickness ranges from 1.0 m to 15.5 m. At some sections, the general ground profile comprises soft clay sandwiched between loose to dense sand stiff clay.

The filling height for embankments along the Project alignment ranges from 6.5 m to 12 m. However, the soft layer beneath typically exhibits low shear strength which tends to the instability of an embankment during the construction phase. Considering such conditions and the limited construction time frame necessary complete before the opening of the Matarbari Port, the following three methods have been selected as measures for the soft soil improvement for the project road.

- Excavation and replacement with suitable materials
- Consolidation and dewatering by applying a surcharge with Prefabricated Vertical Drains (PVDs)
- Compaction using Sand Compaction Piles (SCPs)

(1) Geological Condition

Figure 3.4-31 represents the geological features of the project site. Soil strata are mainly formed in Holocene Epoch consisted of coastal and paludal deposits. Coastal deposit which is Beach and dune sand (csd) largely distributed in Project area, while paludal sediment consisted of marsh clay and peat (ppc) presents in mainland area. Bedrock in the project site is Dupi Tila Formation (QTdt) formed in Pleistocene and Pliocene which is particularly in Maheshkhali Hill.

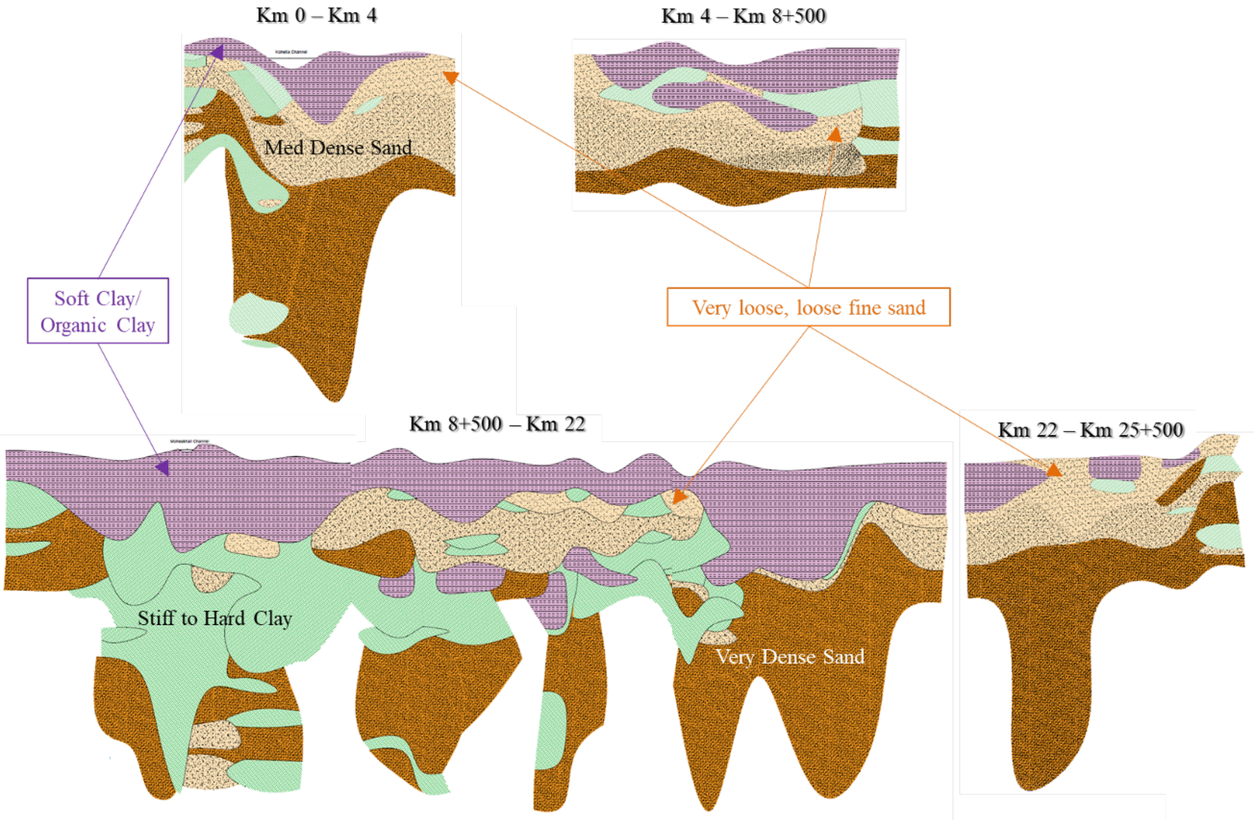


Source: JICA Survey Team

Figure 3.4-31 Geological Features at Project Site

(2) Site Condition

The proposed alignment of Matarbari Port Access Road overlies weak compressible soil. The results of the boreholes and laboratory test indicate that the presence of soft soil layer along the alignment with thickness ranges from 1.0 m to 15.5 m. At some sections, existence of sandwiched layers of stiff clay, loose to dense sand is confirmed. Project soil profile is shown in Figure 3.4-32.



Source: JICA Survey Team

Figure 3.4-32 Soil Profile of the Matarbari Port Access Road

Filling height of embankment under the Project ranges from 6.5 m to 12 m. However, the soft layer beneath typically shows the low shear strength which leads to the instability of embankment during the construction. Especially, lateral force acting on bridge abutments and excessive settlement caused by consolidation in long-term are critical reason to consider about the necessity of soft soil improvement for Project.

(3) Design Criteria and Methodology of Analysis

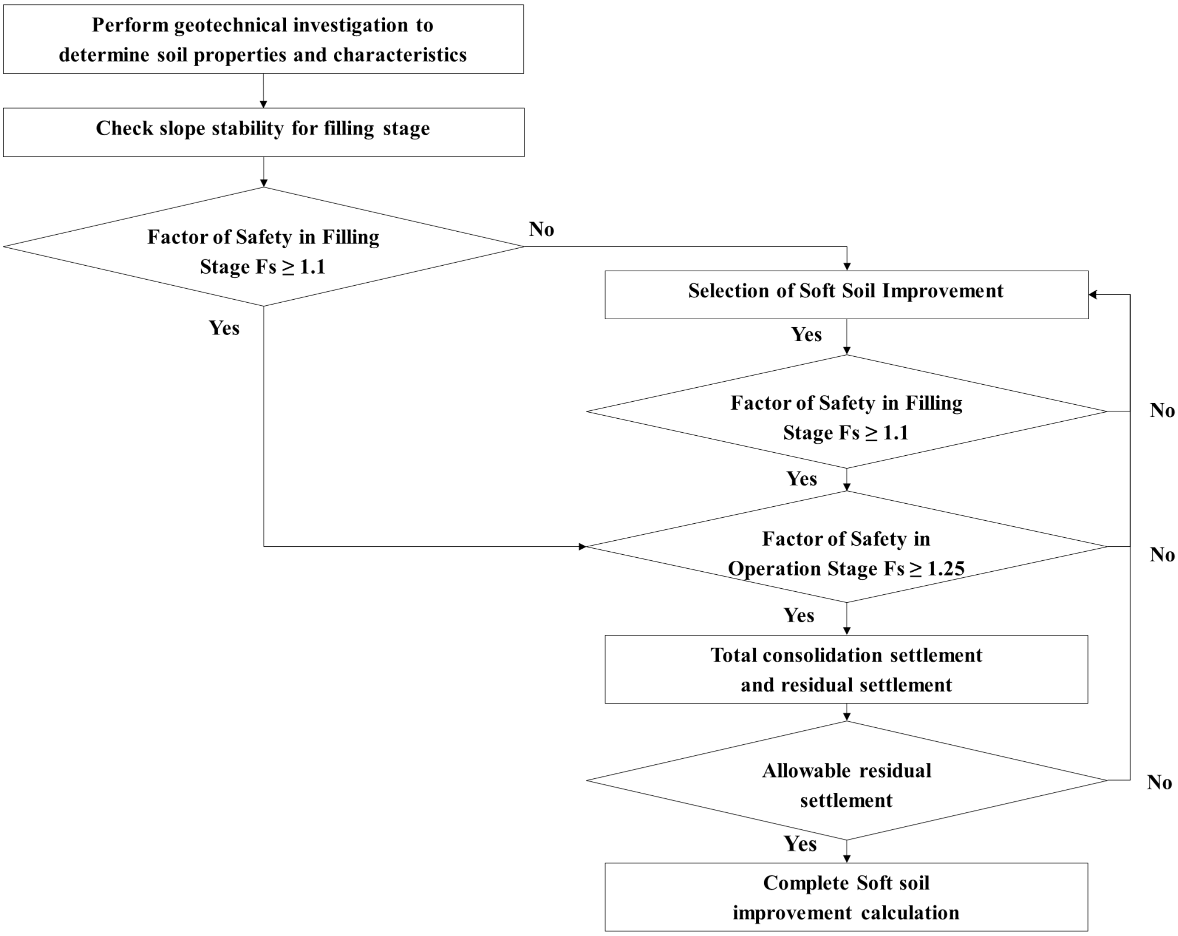
To ensure the safety in construction and smooth travelling during the Project operation in long-term, improvement methods to be selected must meet the requirements for slope stability and residual settlement. Furthermore, the Project is located in the medium seismic activity; therefore, consideration for stability under earthquakes also needs attention. Table 3.4-37 shows the minimum requirements for slope stability factor of safety and allowable residual settlement at the center of embankment.

Table 3.4-37 Design Criteria for Soft Soil Improvement

Requirements		Filling Stage	Seismic	Operation Stage
Slope Stability – Factor of Safety (Fs)		1.1	1.0	1.25
Residual Settlement (cm)	Normal Embankment	-	-	20
	Approach Road	-	-	10

Source: JICA Survey Team

Calculation steps for soft soil improvement selection are shown in Figure 3.4-33.



Source: JICA Survey Team

Figure 3.4-33 Calculation Flowchart

Due to the limited timeframe for the implementation of the Matarbari Port Development Project, it was decided that the allowable construction time including preparation works and necessary treatment time should be within one (1) year.

Besides, based on embankment filling height, soft layer thickness and available investigation data, Project area is divided into sections as tabulated in Table 3.4-38 for easier selection of improvement method.

Table 3.4-38 Project Section

Location		Average Soft soil thickness (m)	Embankment Height (m)	Location		Average Soft soil thickness (m)	Embankment Height (m)
From	To						
1/ Embankment				2/ Approach Road for Bridges			
Km 0+000	Km 0+700	2.0	9.0 - 11.0	Kohelia Bridge	Left side	-	9.8
Km 2+200	Km 3+200	4.5	8.8 - 10.6		Right side	11.5	10.0
Km 3+300	Km 4+300	1.0	7.4 - 10.1	Bridge 2	Right side	4.5	11.0
Km 5+000	Km 5+900	9.5	6.7 - 10.0	Bridge 3		4.0	9.3
Km 6+500	Km 8+500	11.5	7.0 - 8.6	Bridge 4		11.5	9.0
Km 8+500	Km 9+500	10.5	6.6 - 9.9	Moheskhali Bridge		11.0	8.4
Km 12+300	Km 13+700	12.5	7.2 - 8.8	Bridge 6		6.5	9.7
Km 14+700	Km 15+900	Sandwiched	7.6 - 9.2	Bridge 7		4.5	8.8
Km17	Km18	6.5	7.8 - 9.3	Bridge 8	Left side	2.0	9.2
Km 19	Km20	14.5	6.7 - 8.1		Right Side	Sandwiched	8.1
Km20	Km21	15.5	7.7 - 9.0	Bridge 9		15.5	9.7
Km21	Km23	7.5	7.3 - 8.2	Bridge 10/11/12 LeftSide		5.5	8.3
Km23	Finish	-		Bridge 12 Right side		7.5	8.9
				Bridge 13		3.5	8.2

Source: JICA Survey Team

(4) Selection for Improvement Methods

Taking into account the features of Project that soft soil thickness is largely different and construction time is limited, the following four methods are considered as the candidates of the soft soil improvement for the project road.

- Replacement
- Slow Banking
- Preloading with/without PVD
- Sand Compaction Pile

In case the soft layer is thin, the most economical and least time consuming method is replacement. When the thickness increases, Preloading with/without PVD combined with Slow banking method is applied to secure the slope stability and residual settlement. However, if the required construction time for Preloading method and feasibility for construction at site i.e. soft soil layer is interbedded with stiff clay or dense sand, Sand Compaction shows the advantage compared to other methods.

Table 3.4-39 Comparison of Soft Soil Improvement method

Method		Application for thick soft layer	Settlement		Stability		Deformation
			Accelerate Settlement	Reduce total settlement	Increase soil shear strength	Increase resistance force	Reduction of stress
Replacement							
Slow Banking		✓			✓		
Preloading	without PVD	✓			✓		
	with PVD	✓	✓		✓		
Sand Compaction Pile		✓	✓	✓	✓	✓	✓

Table 3.4-39 shows the comparison of soft soil improvement methods in general; specifically for thick layer, the comparison in construction time and cost is presented in Table 3.4-40.

Table 3.4-40 Comparison of Construction Time and Cost

Improvement Method		Construction Time	Cost (Material + Construction Cost)
Slow Banking		Most	Least
Preloading	without PVD		
	with PVD		
Sand Compaction Pile		Least	Most

Principles for each method are explained as follows.

<u>Replacement method</u>

When soft soil is located near the surface with thickness ranging from 1.0 m to 3.0 m, this shall be replaced with embankment materials to ensure slope stability and reduce settlement. This method also helps reduce construction time.

<u>Slow banking</u>

When the soft soil layer thickness is larger than 3.0 m and the filling rate is quick, soil strength is not sufficient enough to bear the embankment load; stability for embankment is not secured. Therefore, it is usually constructed at a slower speed so that soil layer can gain strength from consolidation process. The recommended filling rate is listed in Table 3.4-41 from various project experiences. However, this rate can be speed up if the Factor of Safety Fs after being checked is secured. This method is usually combined with other methods such as Preloading at the beginning of construction time, then increase the filling rate after soil gains strength.

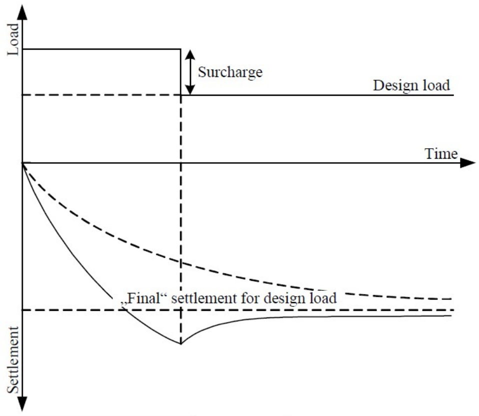
Table 3.4-41 Recommended Filling Rate

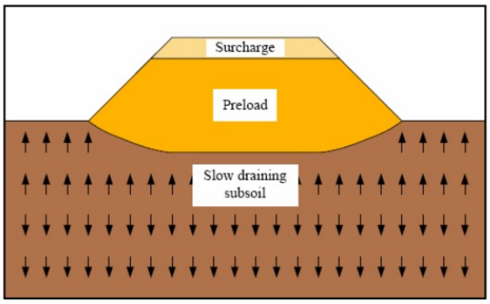
Ground Conditions	Filling rate (cm/day)
Thick cohesive soil ground and muck, or peaty ground with thick deposit of organic soil	3
Ordinary cohesive ground	5
Thin cohesive soil ground and muck, or thin peaty ground with almost no organic soil inter-bedded	10

Source: JICA Survey Team

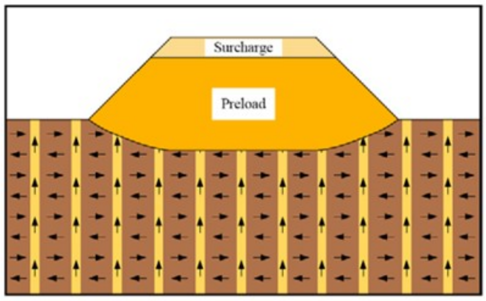
<u>Surcharge with or without Prefabricated Vertical Drain (PVD)</u>

Surcharge generally refers to the process of compressing the soil under applied vertical stress prior to construction and placement of the final construction load. This method is usually applied for thick soft soil layer that needs controlling residual settlement. The two common preloading techniques are conventional surcharge (i.e. without PVD) and with PVD. Figure 4 explains the principles for this method. The consolidation settlement of soft clays takes a long time to complete. In case, the residual settlement does not meet the requirement within allowable construction time, vertical drains (PVD) are installed together with preloading by an embankment. Vertical drains are artificially-created drainage paths which are inserted into the soft clay subsoil. Thus, the pore water squeezed out during consolidation of the clay due to the hydraulic gradients created by the preloading can flow faster in the horizontal direction towards the vertical drains.





Preloading without PVD



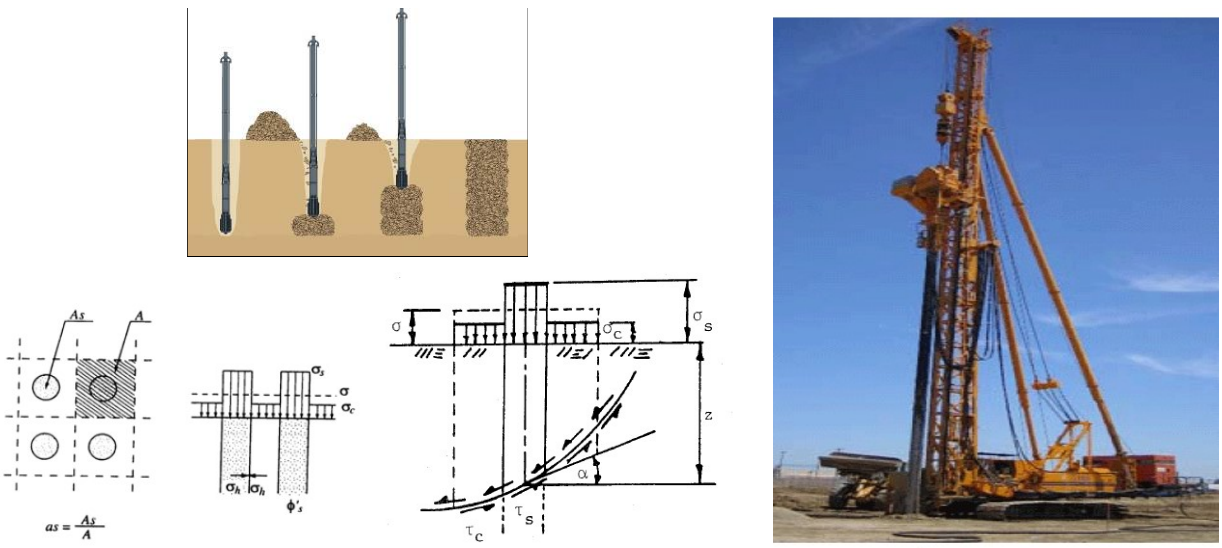
Preloading with PVD

Source: JICA Survey Team

Figure 3.4-34 Preloading Method Conventional with/without PVD

<u>Sand Compaction Pile</u>

In this method, sand is pressure-fed into the ground by means of impact loading or vibration loading so as to form sand piles in the ground. One of the best advantages of this method is that it is feasible to improve the soft clay layer interbedded with other good soil layers. Moreover, in sandy soil ground by means of this method prevents the occurrence of liquefaction, and for cohesive soil ground it ensures ground strength enhancement and settlement reduction due to arching effect.



Source: JICA Survey Team

Figure 3.4-35 Sand Compaction Pile Method

Reinforced geotextile

To ensure slope stability, high strength woven geotextile should be utilized combined with other methods. From the above comparison, it is recommended to mainly improve by Preloading method. For sections that Preloading is not feasible, Sand Compaction Pile will be used.

(5) Results for each section

Summary of selected method for each section is presented in Table 3.4-42 and the detailed calculation is shown in Appendix 3-1.

Table 3.4-42 Summary of Soft Soil Improvement method

Section		**Average Soft Soil Thickness (m)**	**Improvement Method**	**Installation Pattern**	**SCP**		**PVD**
From	**To**				**Diameter (m)**	**Spacing (m)**	**Spacing (m)**
1. Embankment							
0+000	0+700	2.00	Replacement	-	-	-	-
2+200	3+200	4.50	Surcharge	-	-	-	-
3+300	4+300	1.00	Replacement	-	-	-	-
5+000	5+900	9.50	Surcharge + PVD	square	-	-	1.65
6+500	8+500	11.50	Surcharge + PVD	square	-	-	1.65
8+500	9+500	10.50	Surcharge + PVD	square	-	-	1.65
12+300	13+700	12.50	Surcharge + PVD	square	-	-	1.65
17+000	18+000	6.50	Surcharge + PVD	square	-	-	1.65
19+000	20+000	14.50	Surcharge + PVD	square	-	-	1.65
20+000	21+000	15.50	Surcharge + PVD	square	-	-	1.65
21+000	23+000	7.50	Surcharge + PVD	square	-	-	1.65
23+000	25+412	-	Replacement	-	-	-	-
2. Approach Road for Bridges							
Bridge 1 Kohelia Bridge	Left side		Replacement	-	-	-	-
	Right side	11.50	Sand Compaction Pile	square	0.7	1.1	-
Bridge 2	Right side	4.50	Surcharge + PVD	square	-	-	1.65
Bridge 3		4.00	Surcharge Method	-	-	-	-
Bridge 4		11.50	Surcharge + PVD	square	-	-	1.65
Bridge 5 Maheshkhali Bridge		11.00	Surcharge + PVD	square	-	-	1.65
Bridge 6		6.50	Surcharge + PVD	square	-	-	1.65
Bridge 7		4.50	Surcharge + PVD	square	-	-	1.65
Bridge 8	Left side	2.00	Replacement	-	-	-	-
	Right Side	Sandwiched	Sand Compaction Pile	square	0.7	1.1	-
Bridge 9		15.50	Sand Compaction Pile	square	0.7	1.1	-
Bridge 10/11/12 Left Side		5.50	Surcharge + PVD	square	-	-	1.65
Bridge 12 Right side		7.50	Surcharge + PVD	square	-	-	1.65
Bridge 13		3.50	Surcharge	-	-	-	-
Bridge 14		-	No improvement needed	-	-	-	-
Bridge 15		-	No improvement needed	-	-	-	-
Bridge 16		Sandwiched	Sand Compaction Pile	square	0.7	1.1	-

Source: JICA Survey Team

(6) Others

Liquefaction possibility

Some sections in Project Area exhibit presence of very loose sand with an average thickness of about 5m. Even though no much emphasis is given to the possibility of liquefaction during the Feasibility Study Stage, it is recommended that further caution is taken to investigate the possibility of liquefaction in Detailed Design Stage.

Lateral movement at bridge abutments

Sub-soil near abutments after being improved as shown in Table 5 with minimum 90% degree of consolidation is considered to have no effect to bridge abutments.

3.4.4 Hydraulic and Hydrological Study

The destructive power of storm surge is enormous and the calculation result under this study shows that the influence of storm surge is to be extended up to National Highway No.1. Whereas, the external forces of the storm surge at the coastal area gradually decrease as the flow run up to the inland area, the damage caused by flash-floods alone at the inland area become greater than coastal area. Therefore, protection works for the embankment and bridges against the run up of storm surge and/or the flash flood should be provided properly.

As the results of 2-dimensional flood analyses, it was confirmed that i) slope protection should be provided at the bridge openings of six (6) bridges where permissible shear-stress may exceed the standard, ii) riverbed protection should be provided for the whole riverbeds around 3 bridges across Kohelia River, Maheshkhali Channel and Mangla River and iii) partial riverbed protection should be provided at around bridge piers inside of the water bodies of rivers against local scouring.

(1) General

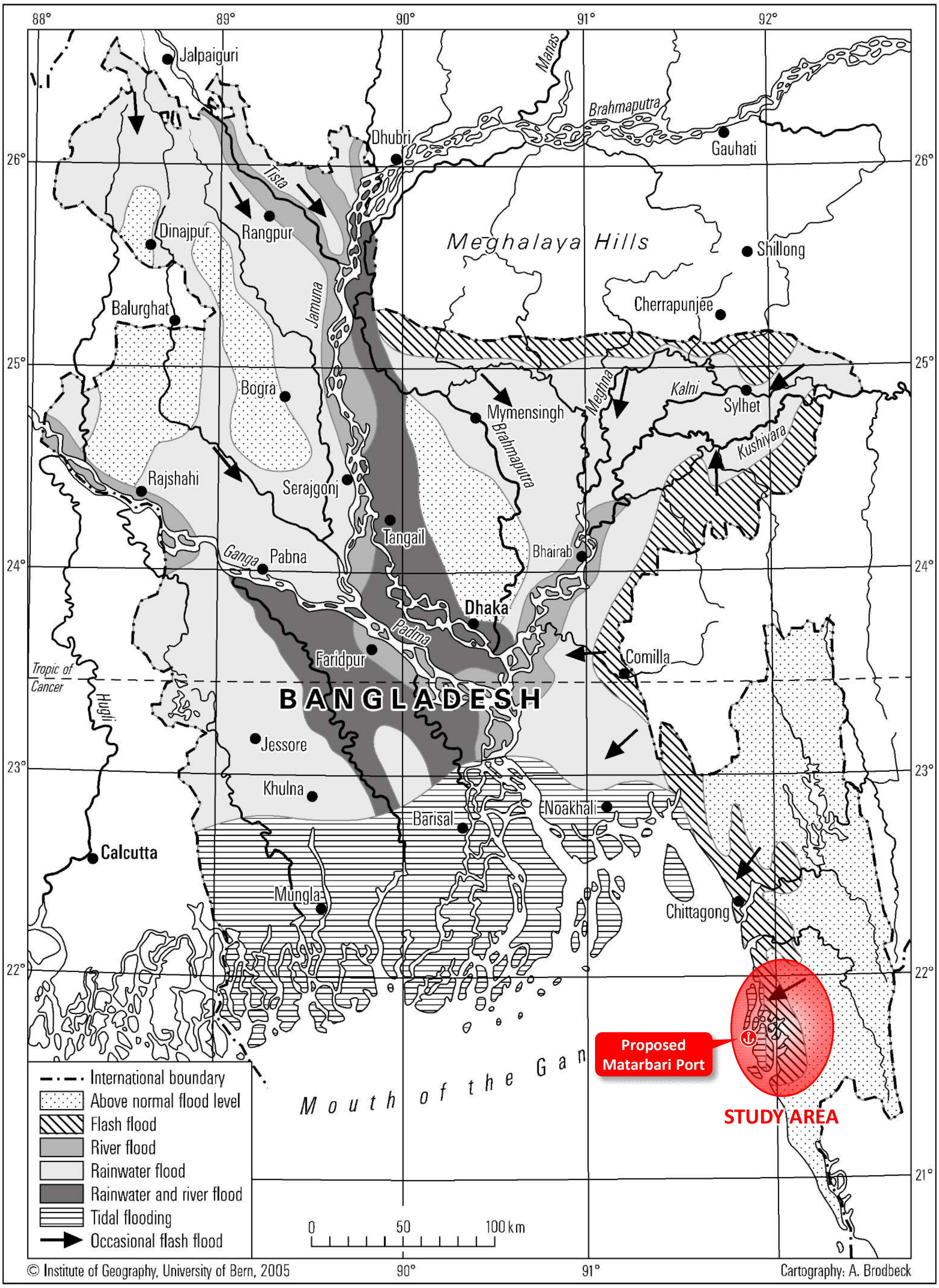
1) Area Characteristics

Bangladesh is located in tropical monsoonal region, and the climate in the area is characterized by high temperature, heavy rainfall, often excessive humidity, and fairly marked seasonal variations. The most prominent feature of its climate is the reversal of the wind circulation between summer and winter, which is an integral part of the circulation system of the South Asian subcontinent. From the climatic point of view, three distinct seasons can be recognized in Bangladesh: i) the cool dry season from November through February, ii) the pre-monsoon hot season from March through May, and iii) the rainy monsoon season which lasts from June through October.

Bangladesh is well-known as a low-lying riverine country (water country) located in southern Asia. A flat and low-lying topography is the most characteristic geomorphological feature: 60% of the country lies less than 6m above sea-level. Therefore, flooding occurs in Bangladesh frequently, and on average 20% of Bangladesh is flooded annually. The study area is located in south-eastern coastal area of the country and also in low lying area.

The catchment area for the study area can be defined as a "flood plain" or "hills", and since it is close to the coast, also have the characteristics of the coastal plains. The coastal plains in the Chittagong and Cox's Bazar areas occupy a narrow strip of land between the Chittagong Hills and the Bay of Bengal. The area is often subjected to shallow flooding from the sea and flash floods from the hills. It is also exposed to the tropical cyclones and the associated storm surges. In April 1991, one of the Bangladesh history's most severe cyclones hit the coastal areas, and the study area suffered catastrophic damage by the storm surge. Also, the ingress of saline water at high tides has been a major handicap for agriculture and a lot of polder dikes and gates have been constructed for controlling the saltwater intrusion. The rivers related to the project area are comprised the Matamuhuri, Kohelia Rivers, Maheshkhali Channel and other small rivers. The Matamuhuri River is a transboundary river, and it is generated from the Arakan hills borders with Myanmar.

Considering the above geographical situation, it is evident that large parts of the project area have potential to be affected by both floods and cyclones, and the flooding characteristics need to be regionally differentiated by the causes. The flooding characteristics in Bangladesh are shown in Figure 3.4-36.



Source: Floods in Bangladesh: History, dynamics and rethinking the role of the Himalayas (Thomas Hofer and Bruno Messerli, United Nations University, 2006)

Figure 3.4-36 Flood Types in Bangladesh

2) Design Policy and Criteria

The design standards and criteria necessary to be considered under this study is summarized as follows:

<u>Design policy against flood and storm surge for the project road</u>

- The elevation of the project road shall be higher than the maximum expected disaster events (embankment: 50-year, bridge: 100-year return period);
- The maximum possible storm surge is referred the study result of "Data Collection Survey on the Matarbari Port Development (hereinafter called as, "Data collection survey")";
- Since the occurrence of inundation, run-up to the inland and river upstream by the storm surge can be assumed, the flood simulation (2-dimensional flood analysis) are carried in order to evaluate the possible events;
- The design parameters and hydraulic assessment for each hydraulic structure are decided based on the result of above 2-dimensional analysis, 1-dimensional analysis and hydrological statistics.

<u>Design Return Period</u>

According to the RHD's Design Manual (2005), roads and bridges on inter-regional highways in Bangladesh should be designed in consideration of the maximum flood level for 50-year return period. The manual also recommends that bridges on international corridor such as Asian Highways are designed against 100-year flood event. Therefore, it was decided that road embankment and bridges under this project will be constructed based on the flood levels of 50-year return period and 100-year return period respectively. For culvert design, 20-year return period was adopted. According to the Standard Design Manual of BWDB, the standard design return period for river training works is 50-yeras return period for the major rivers, except big rivers of the Jamuna, Padma and Meghna Rivers. Therefore, the design scale of 50-years return period is also applied against the scouring countermeasure.

<u>Design Freeboard and Clearance</u>

The vertical clearance (or called freeboard) under the bridge girders should be provided over the high water level based on the magnitude of the designed river discharge in order to allow safe passage of flowing debris during flooding. The freeboard allowances for bridges considered in Japan are shown in Table 3.4-43. The freeboard allowance under this project is decided with reference to the Japanese standard, since Bangladesh standard applies uniform freeboard value not depending on the design discharge. Also, at least 1.0 m clearance should be secured as minimum freeboard in consideration of the anticipated sea level rise caused by the climate change. For designing culverts, the design water depth sets lower than 80% of the inner height of culverts.

Table 3.4-43 Freeboard Allowances for Bridge

Design Flood Discharge (m^3/s)	Freeboard above Design High Water Level (m)	Applied Value (m)
Less 200	0.6	1.0
200 – 500	0.8	1.0
500 – 2,000	1.0	–
2,000 – 5,000	1.2	–
5,000 – 10,000	1.5	–
Over 10,000	2.0	–

Source: Government ordinance for structural standards for river administration facilities, Japan

Design Criteria for the Bridges

Lateral road drainage flows mainly through culverts and bridges. The size of the flood opening should be determined by the magnitude of design discharge, and the culverts should be classified by the allowable discharge for assumed maximum culvert size (assumed maximum culvert: B=6.0m×H=5.0m, Qa=72.47 m3/s).

In order to determine the required waterway openings at the bridge locations, the following design policy for hydraulics were considered:

- To mitigate increase of flood damage by the backwater to the properties upstream of the bridges;
- To mitigate high velocity through the bridge openings not to damage the road facility or the downstream properties;
- To ensure the flow of the maximum discharge within the space of the existing waterways;
- To minimize the flow disruption at the piers and abutments;
- To mitigate the occurrence of local scours within acceptable extent;
- To secure vertical clearance under the structures for safely passage of anticipated debris.

In this study, the preliminary hydraulic and hydrological designs are based on the standard of HEC series of Federal Highway Administration (FHWA, USA), which is widely-used worldwide.

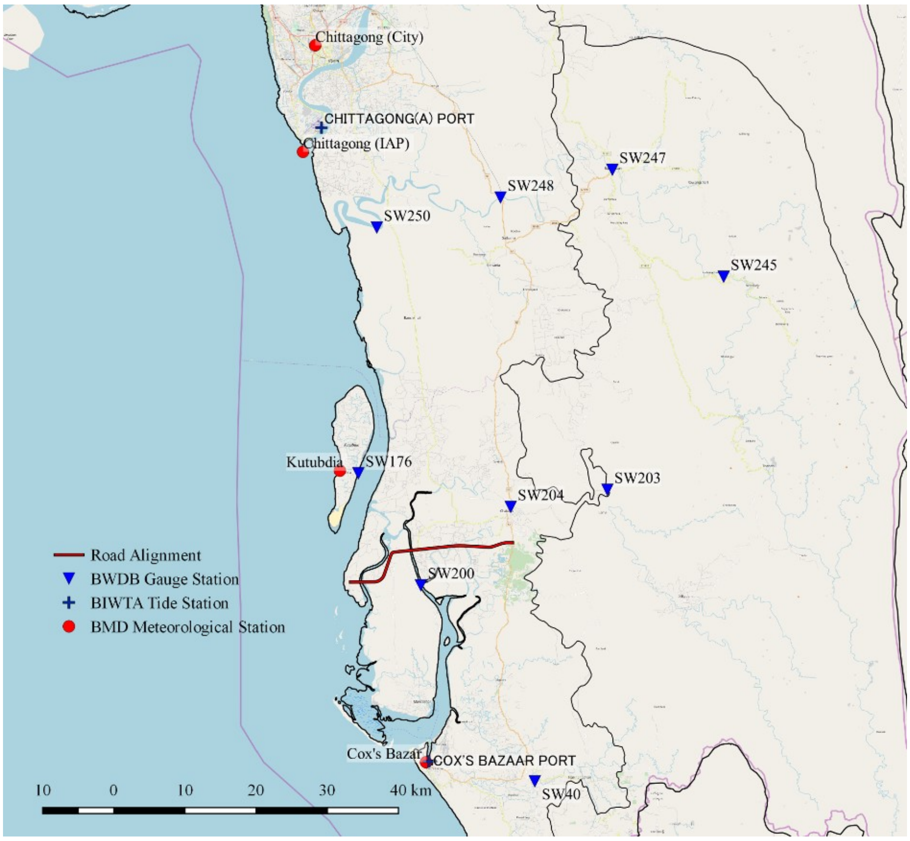
3) Data Collection

Regarding meteorology and hydrology in Bangladesh, the meteorological data is collected by Bangladesh Meteorological Department (BMD) under the Ministry of Defense (MoD), and the hydrological data is collected by Bangladesh Water Development Board (BWDB) under the Ministry of Water Resources (MoWR) and Bangladesh Inland Water Transport Authority (BIWTA) under the Ministry of Shipping (MoS).

There are 35 synoptic observation stations for climatic data under BMD. Of these stations, the climatic data of the related 4 stations closest to the proposed access road have been collected, in addition to the data of past JICA studies. The collection data items concerning the climate are temperature, relative humidity, wind speed/direction, sunshine hours and rainfall.

On the other hand, there are about 500 gauging stations for hydrological data under BWDB. Of these stations, the hydrological data of 9 stations closest to the proposed bridges have been collected, in addition to the data of past JICA studies. In addition, the tide data for Chittagong and Cox's Bazar Ports have been collected from BIWTA. The collection data items concerning the hydrology are annual maximum water level, annual maximum discharge, daily discharge, and past bathymetric survey results, etc. of related rivers. Also, in order to verify the hydrological characteristics etc., river topographic survey, field reconnaissance, interviews with the residents and bibliographic investigations have been surveyed, and the DEM (Digital Elevation Model) of related area have been acquired.

The data collection items and the locations of related stations are shown in Figure 3.4-37 and Table 3.4-44.



Source: JICA Survey Team

Figure 3.4-37 Location of Observed Stations Selected for Data Collection

Table 3.4-44 Collected Data and Surveyed Items

Survey Items	Unit	Survey Contents		Executing Agency
1. Meteorological Data				
1.1. Information of Meteorological Stations	-	4 stations - Chittagong (Patenga), Chittagong (Ambagan), Cox's Bazar, Kutubdia	Related Meteorological Stations: Station Code, Coordinates, Height, Period of Records, etc.	BMD (Bangladesh Meteorological Department)
1.2. Temperature (Max., Min.)	℃		Related 4 stations	
1.3. Relative Humidity	%	Daily data	Ditto	
1.4. Wind Speed, Direction	m/s	3 hourly data	Ditto	
1.5. Sunshine Hours	hr/day	Daily data	Ditto	
1.6. Rainfall	mm	Daily / 3 hourly data	Ditto	
2. Hydrological Data				
2.1. Information of Hydrological Stations	-	9 stations - Ramu (SW40), Lemsikhali (SW176), Saflapur_Moheshkhali (SW200), Lama (SW203), Chiringa (SW204), Ruma (SW245), Bandarban (SW247), Dohazari (SW248), Banigram (SW250)	Related Hydrological Stations: Station Code, Coordinates, Catchment Area, Type of Gauge, Height, Period of Records, River Cross-section at station, difference between zero of gauge and survey datum, etc.	BWDB (Bangladesh Water Development Board)
2.2. Annual Maximum Water Level	m	-		
2.3. Water Level		Daily / 3 hourly data		
2.4. Annual Maximum Discharge	m^3/sec	-		
2.5. Discharge	m^3/sec	Daily data		
2.6. Tide data	m	Chittagong and Cox's bazar port, 1 hourly data	Tide data, Tidal table, etc.	BIWTA (Bangladesh Inland Water Transport Authority)
3. Topographic Information and Others				
3.1. Bathymetry Survey Results for Related Rivers / Channels	-	5 sections data for Mathamuhuri River of BWDB, and other BIWTA bathymetry map	(Newest and Past bathymetric data)	BWDB, BIWTA
3.2. Nautical Chart	-	2 sheets		Bangladesh Navy
3.3. Topographic Map	-	6 sheets - 1/50000		SoB (Survey of Bangladesh)
3.4. Polder / Dike Information	-			BWDB
3.5. Water Facilities Information	-	Gates, Regulators		
3.6. Future Infrastructures Information	-			-
3.7. DEM	-	AW3D (JAXA), SRTM (NASA)	AW3D: 0.5m mesh, SRTM: 30 / 90m mesh	NTT-DATA, USGS
3.8. River Topographic Survey	-	Cross sectional Survey	52 cross-sections	Survey Team
3.9. Bibliographic Investigation	-	Various types of helpful documents / data		

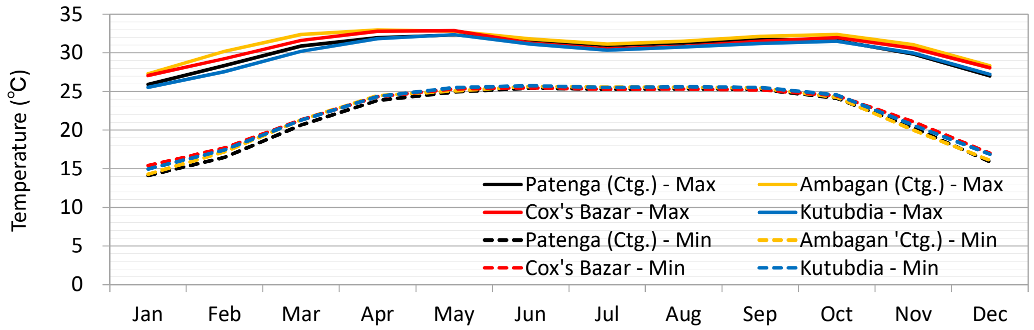
Source: JICA Survey Team

(2) Meteorology

1) General Weather Conditions

Temperature

The monthly mean maximum and minimum temperatures during the past 37 years at the 4 stations near the project site is shown in Table 3.4-28. Temperature is observed daily at 0:00, 3:00, 6:00, 9:00, 12:00, 15:00, 18:00 and 21:00 hours at each station. The temperature data of the 4 stations show similar trends. January is the coldest month and the peak for maximum temperature is observed in April-May.

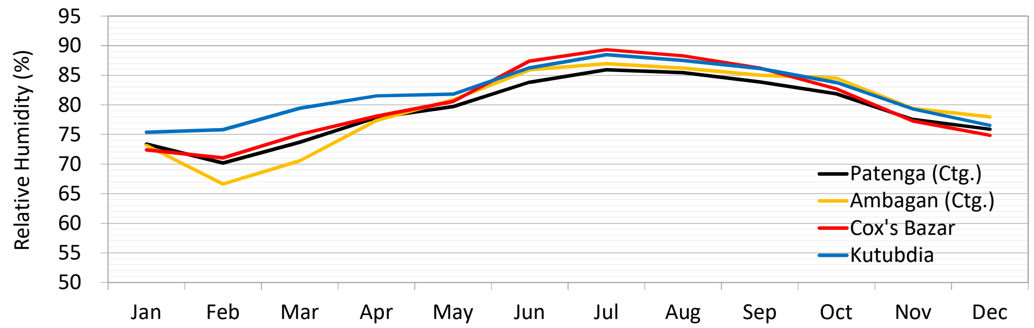


Source: JICA Survey Team, BMD

Figure 3.4-38 Monthly Mean Temperature (Daily Maximum and Minimum)

Relative Humidity

The monthly mean relative-humidity during past 37 years is shown in Figure 3.4-39. The daily fluctuation of relative-humidity is higher during dry season and is lower at rainy season. The lowest average monthly relative-humidity occurs from February to March and the highest average relative-humidity occurs throughout the rainy season. The relative-humidity is high throughout the entire year, and the maximum humidity reaches 100% a few times a year.

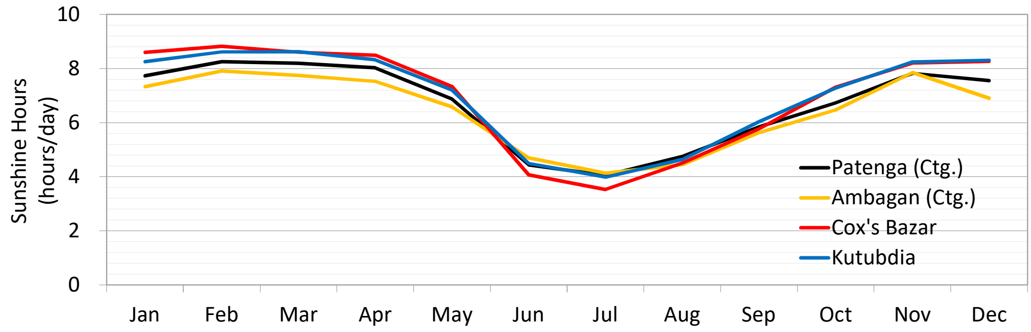


Source: JICA Survey Team, BMD

Figure 3.4-39 Monthly Mean Relative Humidity

Sunshine Hours

The monthly mean sunshine hours during past 38 years is shown in Figure 3.4-40. The sunshine-hours have two opposing seasonal patterns, coinciding with the winter monsoon and the summer monsoon. With the progression of the rainy season, the cloud-cover increases, and the sunshine hours decrease.



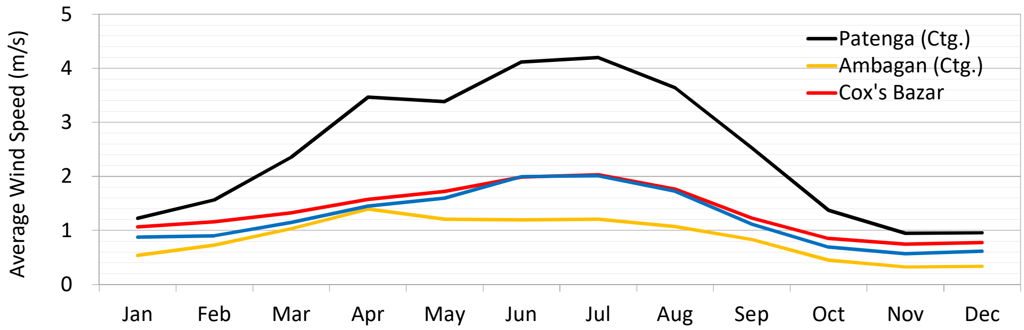
Source: JICA Survey Team, BMD

Figure 3.4-40 Monthly Mean Sunshine Hours

Wind Speed / Direction and Cyclones

The monthly mean wind speed during past 37 years is shown in Figure 3.4-41. The wind-direction in Bangladesh is characterized by seasonal alternation between summer and winter. During the winter season, a high-pressure center lies over northwestern India. A stream of cold air flows eastward from this high pressure and enters the country through its northeast corner by changing its course clockwise, almost at a right-angle. This wind is the part of the winter monsoon circulation of the South Asian subcontinent. During this season, the wind inside the country generally has a northerly component. On the other hand, during the summer season, a low-pressure center develops over the west-central part of India because of intense surface heat. As a result, a stream of warm and moist air from the Bay of Bengal flows toward the above-mentioned low pressure through Bangladesh. This wind is the part of the summer monsoon circulation of the sub-continent. Therefore, the prevailing wind direction in Bangladesh during the summer season generally has a southerly component.

The mean wind speed is within the range of 0.5-2.0 m/s, except in Patenga of Chittagong where it can reach maximum 4.2 m/s. The data given for the maximum wind speed has been affected by a cyclone in Southern Bangladesh, and the past maximum speed have recorded 49.0 m/s at Kutubdia in 1991 Cyclone as shown in Figure 3.4-55 (The data of 1991 Cyclone at other stations had missed). According to the list of major cyclonic storms of BMD, the maximum wind speed in 1991 Cyclone is mentioned as 62.51 m/s (225 km/hr, see Figure 3.4-56). Also, the maximum tidal surge height in 1991 Cyclone is 6.71 m (22 ft), and 10.06 m (33 ft) beyond 1991 has been recorded in 1970 Cyclone.



Source: JICA Survey Team, BMD

Figure 3.4-41 Monthly Mean Wind Speed

Table 3.4-45 Past Maximum Wind Speed at Surrounding Stations

Ranking	Wind Speed (m/s)	Wind Direction	Station Location	Occurrence Date	Remarks
1	49.0	SEE - SSW (13-20)	Kutubdia	1991.4.29	
2	47.0	SWS (21)	Cox's Bazar	1997.5.19	
3	46.0	SW (23)	Kutubdia	1997.5.19	
4	45.0	NE (5)	Patenga (Chittagong)	1997.5.19	
5	37.0	SSW (20)	Kutubdia	1998.5.20	

Source: JICA Survey Team, BMD

Table 3.4-46 List of Major Cyclonic Storms from 1960 to 2017

Date of Occurrence	Nature of Phenomenon	Landfall Area	Maximum Wind Speed in km/hr.	Direction of the Max. Wind Speed	Tidal Surge Height in ft.	Central Pressure (mbs)
11.10.60	Severe Cyclonic Storm	Chittagong	160	South-East	15	-
31.10.60	Severe Cyclonic Storm	Chittagong	193	South-East	20	-
09.05.61	Severe Cyclonic Storm	Chittagong	160	South-East	8-10	-
30.05.61	Severe Cyclonic Storm	Chittagong (Near Feni)	160	South-South-East	6-15	-
28.05.63	Severe Cyclonic Storm	Chittagong-Cox's Bazar	209	South-East	8-12	-
11.05.65	Severe Cyclonic Storm	Chittagong-Barisal Coast	160	South-South-East	12	-
05.11.65	Severe Cyclonic Storm	Chittagong	160	South-East	8-12	-
15.12.65	Severe Cyclonic Storm	Cox's Bazar	210	South-East	8-10	-
01.11.66	Severe Cyclonic Storm	Chittagong	120	South-East	20-22	-
23.10.70	Severe Cyclonic Storm of Hurricane intensity	Khulna-Barisal	163	South-West	-	-
12.11.70	Severe Cyclonic Storm with a core of hurricane wind	Chittagong	224	South-East	10-33	-
28.11.74	Severe Cyclonic Storm	Cox's Bazar	163	South-East	9-17	-
10.12.81	Cyclonic Storm	Khulna	120	South-West	7-15	989
15.10.83	Cyclonic Storm	Chittagong	93	South-East	-	995
09.11.83	Severe Cyclonic Storm	Cox's Bazar	136	South-East	5	986
24.05.85	Severe Cyclonic Storm	Chittagong	154	South-East	15	982
29.11.88	Severe Cyclonic Storm with a core of hurricane wind	Khulna	160	South-West	2-14.5	983
18.12.90	Cyclonic Storm (crossed as a depression)	Cox's Bazar Coast	115	South-East	5-7	995
29.04.91	Severe Cyclonic Storm with a core of hurricane wind	Chittagong	225	South-East	12-22	940
02.05.94	Severe Cyclonic Storm with a core of hurricane wind	Cox's Bazar-Teknaf Coast	220	South-East	5-6	948
25.11.95	Severe Cyclonic Storm	Cox's Bazar	140	South-East	10	998
19.05.97	Severe Cyclonic Storm with a core of hurricane wind	Sitakundu	232	South-East	15	965
27.09.97	Severe Cyclonic Storm with a core of hurricane wind	Sitakundu	150	South-South-East	10-15	-
20.05.98	Severe Cyclonic Storm with core of hurricane winds	Chittagong Coast near Sitakunda	173	South-South-East	3	-
28.10.00	Cyclonic Storm	Sundarban Coast near Mongla	83	South-South-West	-	-
12.11.02	Cyclonic Storm	Sundarban Coast near Rairnangal River	65-85	South-South-West	5-7	998
19.05.04	Cyclonic Storm	Teknaf-Akyab Coast	65-90	South-East	2-4	990
15.11.07	Severe Cyclonic Storm with core of hurricane winds (SIDR)	Khulna-Barisal Coast near Baleshwar river	223	South-West	15-20	942
25.05.09	Cyclonic Storm (AILA)	West Bengal-Khulna Coast near Sagar Island	70-90	South-South-West	4-6	987
16.05.13	Cyclonic Storm (MAHASEN)	Noakhali-Chittagong Coast	100	South-South-East	-	
30.07.15	Cyclonic Storm (KOMEN)	Chittagong-Cox's Bazar Coast	65	South-East	5-7	988
21.05.16	Cyclonic Storm (ROANU)	Barisal-Chittagong Coast near Patenga	128	South-South-West	4-5	992
30.05.17	Severe Cyclonic Storm (MORA)	Chittagong-Cox's Bazar Coast near Kutubdia	146	South-East	-	-

Note: 1 feet = 0.3048 m, 1km/hr = 0.2778 m/s

Source: BMD

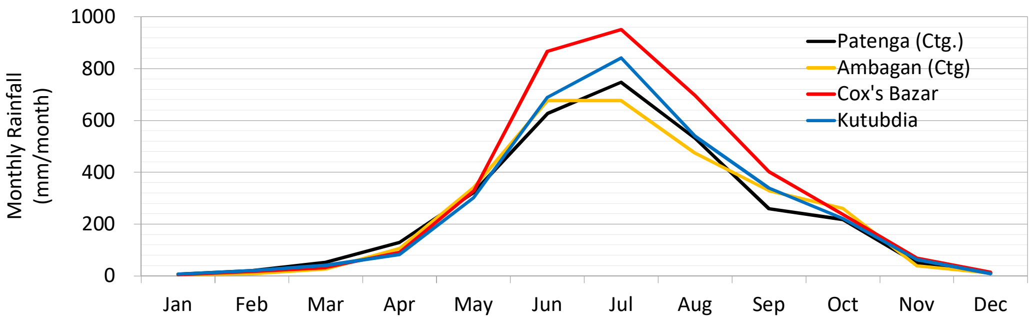
2) Rainfall

Daily, Monthly and Annual Rainfall

The monthly mean rainfall during past 37 years is shown in Figure 3.4-42 and Table 3.4-47. Also, the daily rainfall distribution during past 30 years is shown in Figure 3.4-43. Bangladesh is in a tropical monsoon region, the amount of rainfall is very high and, there is a distinct seasonal pattern in the annual cycle of rainfall, which is much more pronounced than the cycle of temperature. The winter season accounts for only 2-3 % of the total annual rainfall. Rainfall during the rainy season is caused by the tropical depressions that enter the country from the Bay of Bengal.

The past maximum daily rainfall is recorded 467 mm at Cox's Bazar in 2015, and 511 mm at Chittagong in 1983. The mean annual rainfall at Chittagong and Cox's bazar is about 2,974mm and 3,711mm. The long-term fluctuation of annual rainfall during the past 37 years at 4 stations is shown in Figure 3.4-44 and the fluctuation range is over 2000 mm. For example, Figure 3.4-44 includes the approximate optimization by linearization at Cox's Bazar. From this figure, it is recognized that a marginal upward trend in annual rainfall is going on.

Also, as an indicator of the annual workable days for construction planning, the annual total number of days with daily-rainfall more than 10 mm is counted as unworkable days. The annual mean rainy days with over 10 mm/day is shown in Table 3.4-48.



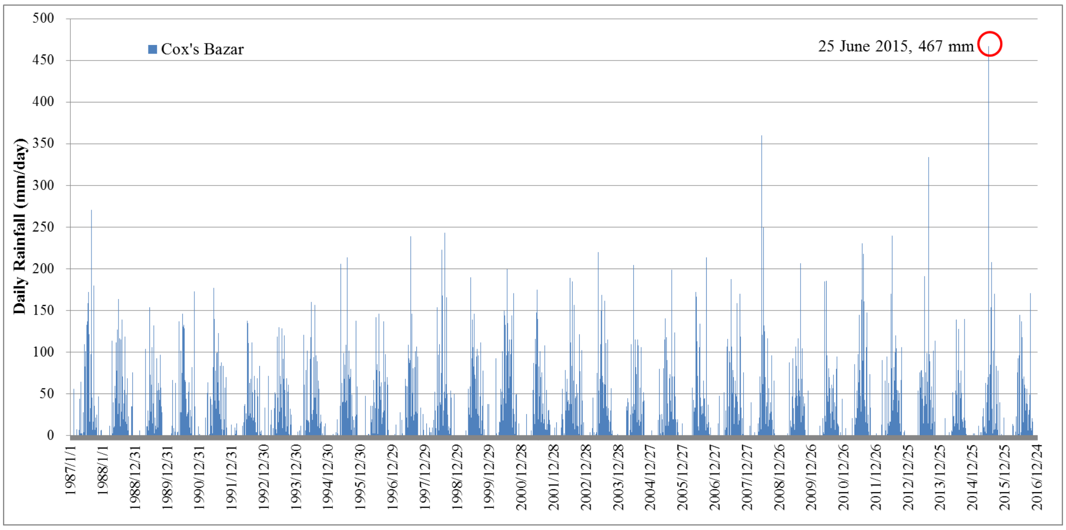
Source: JICA Survey Team, BMD

Figure 3.4-42 Monthly Mean Rainfall

Table 3.4-47 Monthly Mean Rainfall

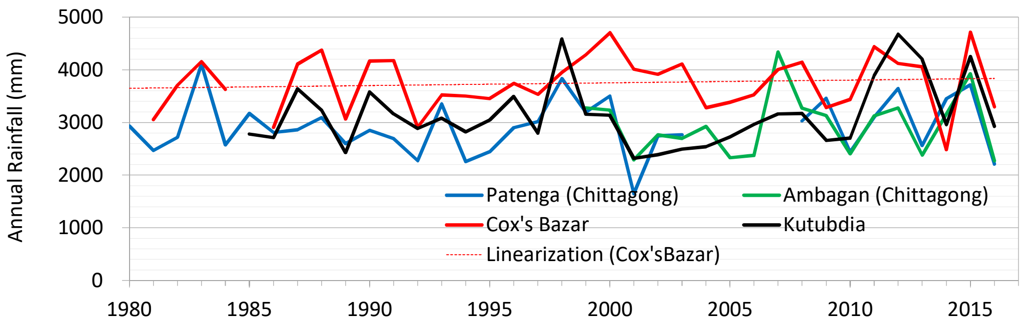
Satation	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annum	Remarks
Patenga (Chittagong)	6.3	20.7	52.2	129.3	321.0	626.5	747.4	531.5	259.8	217.7	49.9	11.8	2974.0	Coast (IAP)
Ambagan (Chittagong)	5.7	8.1	27.6	105.9	340.4	676.9	676.7	473.9	329.2	260.2	38.6	10.9	2954.1	City
Cox's Bazar	4.8	17.8	32.4	91.7	327.0	867.3	950.8	696.3	402.2	237.1	69.3	13.9	3710.6	
Kutubdia	6.5	20.3	41.0	82.9	301.5	689.7	841.5	539.5	339.2	221.8	63.4	8.5	3156.0	

Source: JICA Survey Team, BMD



Source: JICA Survey Team, BMD

Figure 3.4-43 Daily Rainfall at Cox's Bazar Station (1987-2016)



Source: JICA Survey Team, BMD

Figure 3.4-44 Annual Fluctuation of Rainfall

Table 3.4-48 Annual Mean Rainy Days (more than 10 mm/day)

Station Name	Chittagong (Patenga)		Chittagong (Ambagan)		Cox's Bazar		Kutubdia	
Organization	BMD		BMD		BMD		BMD	
Observed Year	Rainy Days							
	>0 mm	>10 mm	>0 mm	>10 mm	>0 mm	>10 mm	>0 mm	>10 mm
1980	109	61						
1981	117	58			122	78		
1982	118	56			102	72		
1983	124	64			139	86		
1984	110	54			120	74		
1985	111	58			112	62	95	66
1986	115	58			126	68	103	61
1987	111	70			127	74	109	71
1988	125	73			150	89	130	78
1989	101	50			120	70	102	57
1990	138	65			139	84	129	69
1991	115	61			138	82	113	79
1992	105	49			123	63	106	58
1993	134	68			139	82	120	71
1994	116	54			135	75	106	63
1995	111	41			124	71	108	62
1996	116	65			126	78	115	63
1997	109	68			129	80	111	67
1998	113	71			127	75	112	69
1999	106	60	114	64	140	89	119	76
2000	118	62	127	65	140	90	109	63
2001	112	60	123	61	145	89	113	62
2002	136	65	138	66	143	88	118	54
2003	118	59	119	57	128	81	95	60
2004			115	54	115	66	110	65
2005			120	61	122	75	118	59
2006			109	58	115	75	106	59
2007			132	80	130	84	129	78
2008	111	63	114	66	128	77	121	65
2009	114	63	116	64	121	72	114	67
2010	112	64	113	62	137	75	123	56
2011	123	71	121	72	129	82	119	76
2012	117	64	111	65	130	87	108	73
2013	119	56	128	62	122	76	108	75
2014	116	64	116	60	105	53	113	58
2015	123	66	119	62	115	69	123	70
2016	123	54	123	57	119	68	120	66
Average	116.55	61.06	119.89	63.11	127.28	76.64	113.28	66.13
Rate of Workable Days		83.3%		82.7%		79.0%		81.9%

Source: JICA Survey Team, BMD

<u>Exceedance Probability and Intensity Curve of Rainfall</u>

The annual maximum daily rainfall (extreme value) data are picked out from the raw data, and 3-hour / 24-hour probability rainfalls are calculated as shown in Table 3.4-49. In order to estimate the intensity of short duration rainfall from the 3-hour and 24-hour rainfalls, the Intensity-Duration-Frequency (IDF) curve is developed and approximated. Since the raw data for short duration rainfall is not available in Bangladesh, the rainfall intensity for duration of 15 minute (0.25 hour) which is indicated in 'Urban Drainage Manual (LGED)' was referred for the IDF curve's development in this study.

From Table 3.4-49, the probable 24-hour rainfall at each return period is higher in Chittagong area, followed by Cox's Bazar, Kutubdia in order. The study area is located between Kutubdia and Cox's Bazar, and the higher data out of the two, namely the values of Cox's Bazar, was adopted for this study.

The rainfall intensity formula at the study area (Cox's Bazar) is follows:

$$I = a/(T^b + c)$$

Where, I: Rainfall Intensity (mm/hour)

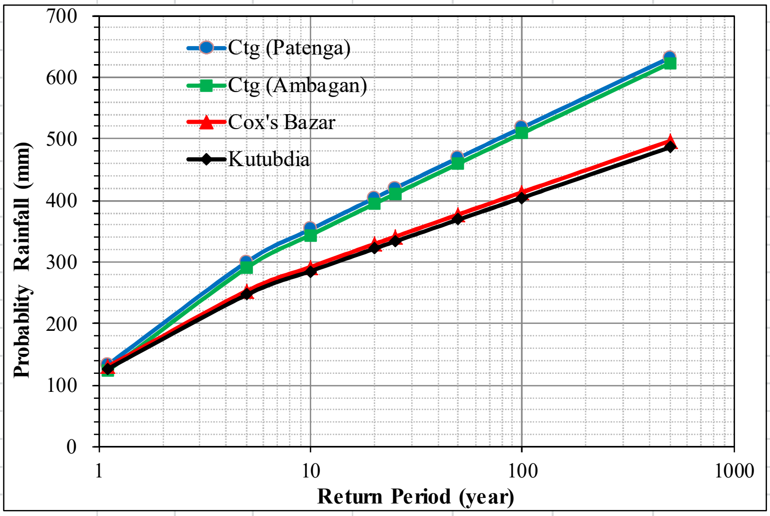
T: Concentration Time (hours, = Inlet Time + Travel Time)

A, b, c: Coefficient (Refer to Table 3.4-50.)

Figure 3.4-45 shows the IDF Curve developed in this study.

Table 3.4-49 Probable 24-hour Rainfall at 4 Stations

Station Name			Ctg (Patenga)	Ctg (Ambagan)	Cox's Bazar	Kutubdia	Remarks
Extreme Data No.			33	18	36	32	
Max. Value (mm/day)			511	438	467	422	
Min. Value (mm/day)			130	115	130	120	
Mean Value (mm/day)			235.2	225.9	205.6	200.3	
Probable Rainfall (mm)	(Year)	(%)					
	1.1	90.9%	133.1	123.7	130.7	126.7	
	5	20%	300.2	290.9	253.3	247.2	
	10	10%	353.0	343.7	292.0	285.2	
	20	5%	403.7	394.4	329.2	321.7	
	25	4%	419.7	410.5	341.0	333.3	
	50	2%	469.2	460.0	377.3	369.0	
	100	1%	518.4	509.2	413.4	404.4	
	500	0.2%	631.9	622.8	496.7	486.3	
Probabilistic Distributed model			Gumbel	Gumbel	Gumbel	Gumbel	

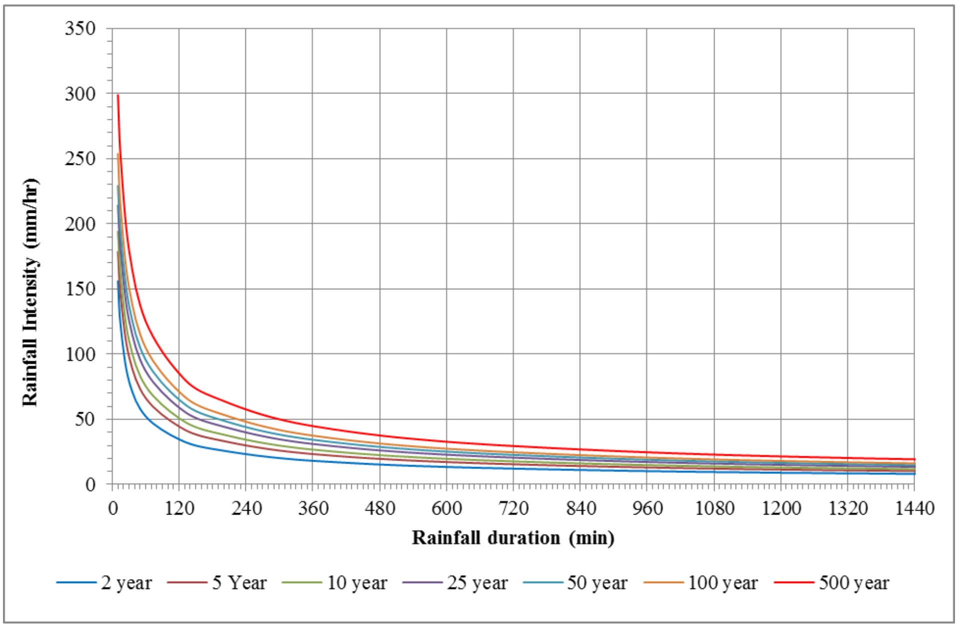


Source: JICA Survey Team

Table 3.4-50 Rainfall Intensity for each Duration at Cox's Bazar

Probable Rainfall for Cox'sBazar			Rainfall Intensity for Cox'sBazar													
Return Period T (years)	Probable Rainfall (mm)		Rainfall Intensity Formula			Rainfall Intensity (mm/hr) in given duration										Remarks
	3	24	I = a /(T^b + c), Cleveland			0.16667	0.25	0.5	1	2	3	6	12	24	48	(hrs)
	180	1440	a	b	c	10	15	30	60	120	180	360	720	1440	2880	(mins)
1.1	56.5	138.9	31.246	0.539	-0.149	(134.96)	96.29	(57.96)	(36.72)	(23.96)	18.83	(12.61)	(8.52)	5.79	(3.95)	
2	82.0	196.0	50.482	0.575	-0.034	(156.12)	121.04	(79.16)	(52.24)	(34.67)	27.33	(18.24)	(12.20)	8.17	(5.47)	
3	93.1	220.6	60.542	0.591	0.036	(158.34)	(127.11)	(86.55)	(58.45)	(39.25)	31.03	(20.72)	(13.81)	9.19	(6.11)	
5	105.4	248.0	69.169	0.596	0.044	(178.57)	143.71	(98.08)	(66.27)	(44.47)	35.13	(23.42)	(15.57)	10.33	(6.85)	
10	120.8	282.5	81.685	0.606	0.083	(194.09)	158.64	(110.35)	(75.42)	(50.90)	40.27	(26.84)	(17.80)	11.77	(7.77)	for Drainage Design
20	135.6	315.6	93.960	0.614	0.117	(209.00)	(172.83)	(122.01)	(84.16)	(57.06)	45.20	(30.13)	(19.95)	13.15	(8.64)	for Culvert Design
25	140.3	326.1	97.785	0.616	0.124	(214.26)	177.64	(125.82)	(86.96)	(59.03)	46.77	(31.17)	(20.63)	13.59	(8.92)	
50	154.8	358.4	109.981	0.622	0.152	(229.25)	191.65	(137.24)	(95.51)	(65.07)	51.60	(34.39)	(22.73)	14.93	(9.78)	
100	169.2	390.5	119.841	0.622	0.144	(253.79)	(211.61)	(150.95)	(104.74)	(71.20)	56.40	(37.54)	(24.78)	16.27	(10.64)	for Bridge design
500	202.4	464.7	145.243	0.627	0.161	(298.89)	(250.39)	(179.68)	(125.11)	(85.16)	67.47	(44.86)	(29.56)	19.36	(12.63)	

Source: JICA Survey Team



Source: JICA Survey Team

Figure 3.4-45 IDF (Intensity-Duration-Frequency) Curve

(3) Hydrological Analysis

In order to predict the flow rate and water level in flood season, it is necessary to collect and correlate the available data and conditions concerning the hydrology and hydraulics of the related rivers in the study area. In this section, river characteristics of main rivers only are examined.

1) Rivers and Flow Regime

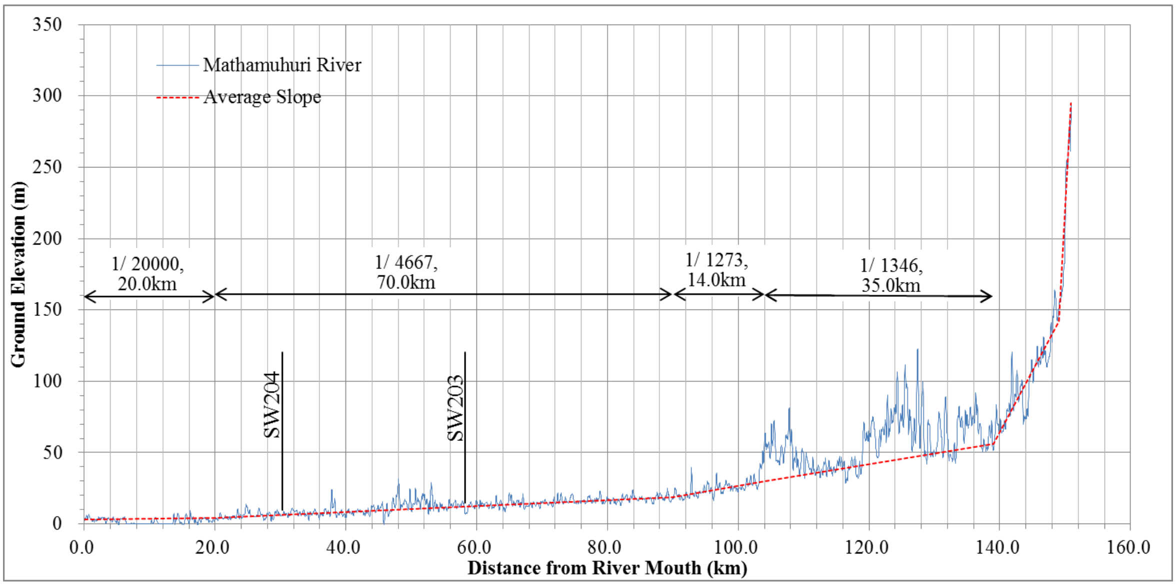
Related Rivers

As mentioned earlier, rivers related to the study area are comprised of Matamuhuri, Kohelia Rivers, Maheshkhali Channel and other small rivers. The Matamuhuri River, which is a transboundary river, has the largest basin and is generated from the Lusai hills in Myanmar. From the originating place, it flows in the north-west direction and flows through Alikadam and Lama towards Chakaria of Cox's Bazar district.

BWDB has presently two (2) hydrometric stations (SW203, SW204) at Matamuhuri River and has recorded since 1956. The SW203 is a streamflow gauging station (SGS) measuring water level and streamflow data, and the SW204 is a water level Gauging station (WLGS) measuring the water level data (the gauging stations of BWDB are classified into the non-tidal and tidal water level stations. SW204 station is influenced by the tide, but the fluctuation range of the tide is very small compared with the outer sea tide variability. Although SW203 is non-tidal station and SGS, the streamflow record is not measured at fixed intervals and is not enough to utilize in the study).

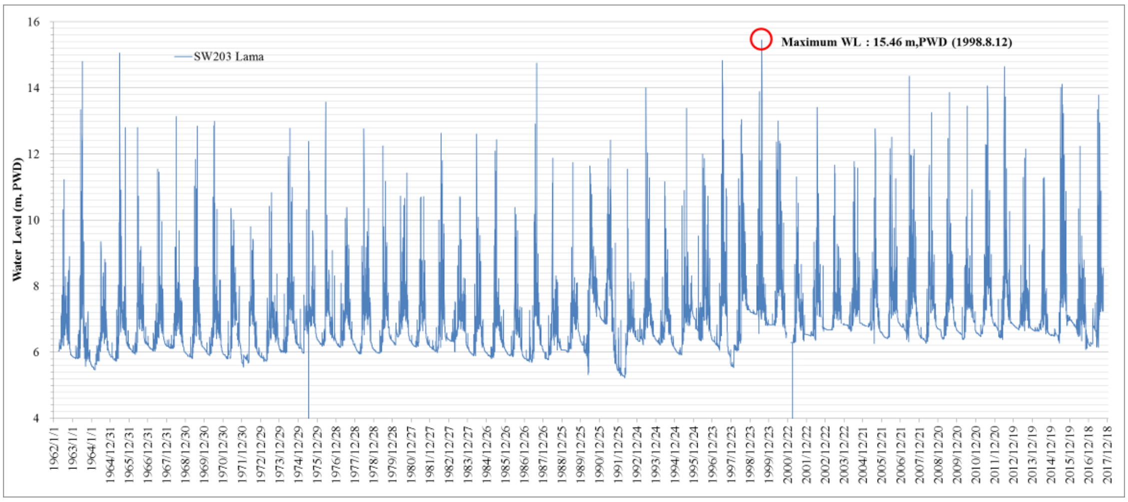
The downstream of the Matamuhuri River has a very gradual slope as shown in Figure 3.4-46 (according to the classification for river course characteristics in Japan, the study area can be classified into the very gentle river of 'segment-3'). Most of the flow channels of the other rivers are located in the downstream of the Matamuhuri River, and they have a very gentle slope same as the mainstream.

Figure 3.4-47 to Figure 3.4-49 show the daily water level hydrographs at SW203 and SW204 on Matamuhuri River and SW200 on Maheshkhali Channel. Past annual maximum water level is recorded as 15.46 mPWD at SW203, 7.63 mPWD at SW204 and 4.36 mPWD at SW200.



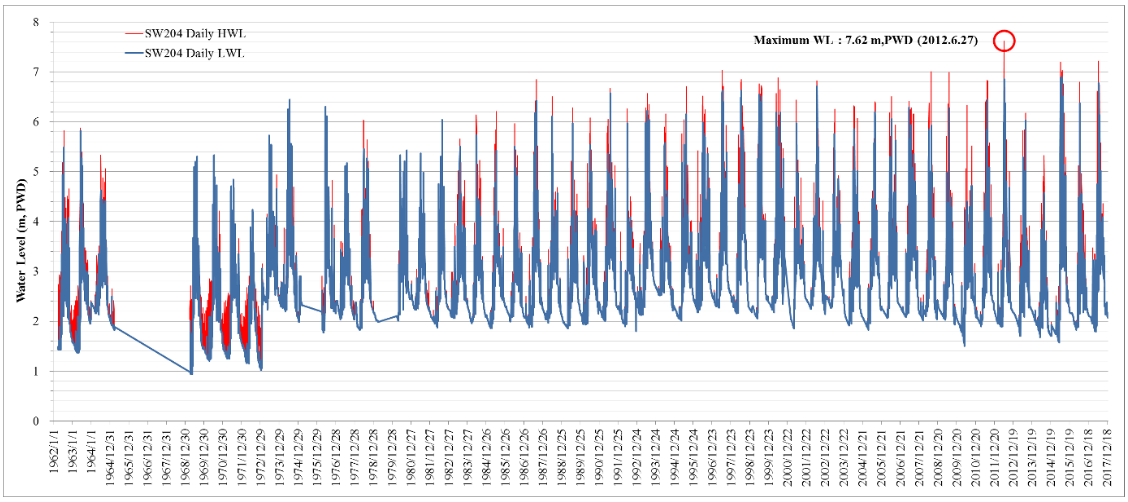
Source: JICA Survey Team, The slope is generated from GIS software by using SRTM DEM.

Figure 3.4-46 River Profile of Matamuhuri River



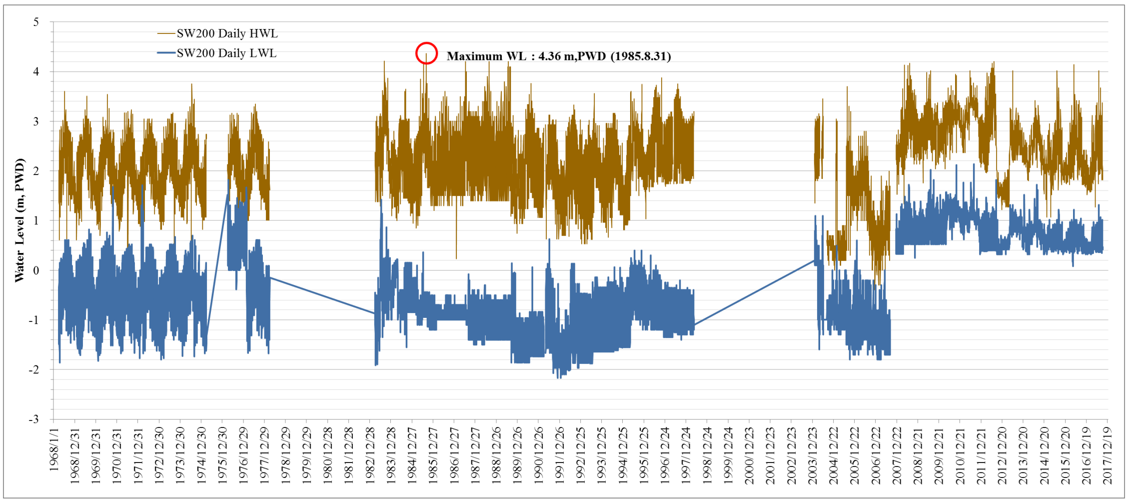
Source: JICA Survey Team

Figure 3.4-47 Daily Water Level at SW204 Station (Matamuhuri River–30.2km from Rivermouth)



Source: JICA Survey Team

Figure 3.4-48 Daily Water Level at SW203 Station (Matamuhuri River–58.3km from Rivermouth)



Source: JICA Survey Team

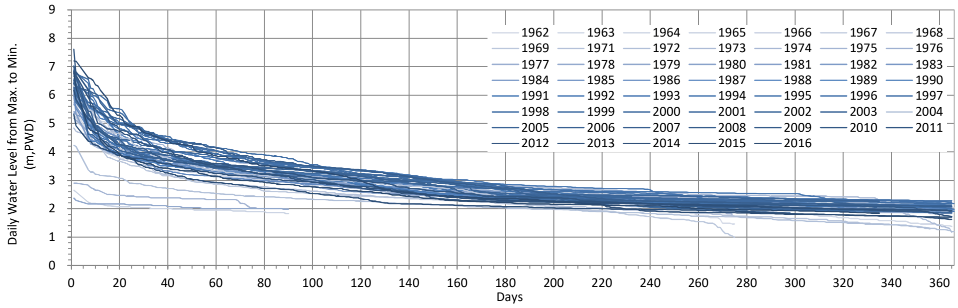
Figure 3.4-49 Daily Water Level at SW200 Station (Maheshkhali Channel)

River Flow Regime

The flow duration curve was used in order to understand the river flow characteristics throughout the year at the river crossing points of the project road. The flow regime shows the annual flow condition using the daily discharges at each hydrological station. However, the SW203 is the only available streamflow gauging station (SGS) and the streamflow data is limited. Therefore, the water-level data was used, and the stage duration curve was developed. The definition of annual flow regime are as follows;

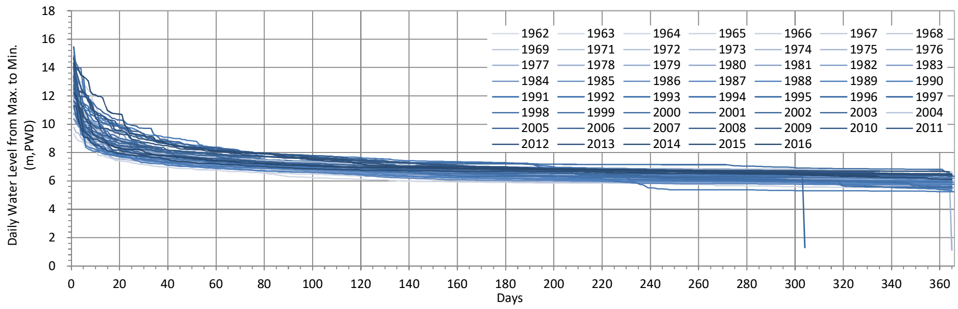
- High Water Level: 95th daily water-level from the greatest
- Normal Water Level: 185th daily water-level from the greatest
- Low Water Level: 275th daily water-level from the greatest
- Drought Water Level: 355th daily water-level from the greatest

The water-level duration curves at SW204 and SW203 for Matamuhuri River and at SW200 for Maheshkhali Channel are shown in Figure 3.4-50 to Figure 3.4-52, and the typical water levels of these stations are shown in Figure 3.4-50 to Figure 3.4-52 for selected years.



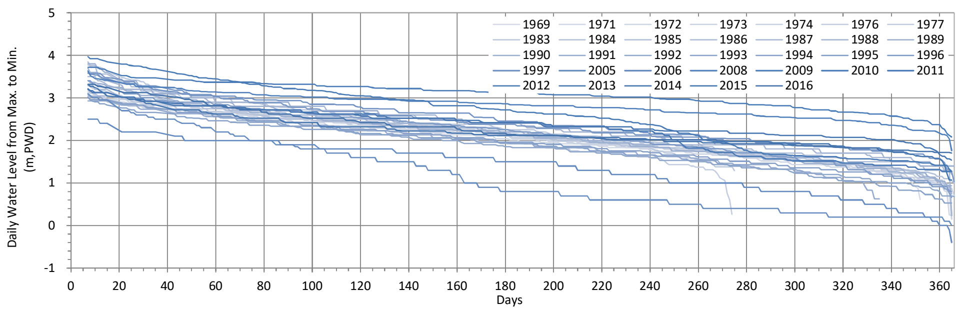
Source: JICA Survey Team

Figure 3.4-50 Water-Level Duration Curve at SW204 (Matamuhuri River)



Source: JICA Survey Team

Figure 3.4-51 Water-Level Duration Curve at SW203 (Matamuhuri River)



Source: JICA Survey Team

Figure 3.4-52 Water-Level Duration Curve at SW200 (Maheshkhali Channel)

Table 3.4-51 Typical Water Level of SW204 (Matamuhuri River, m PWD)

Year	Annual Maximum WL	Plentiful Water Level	OrdinaryWater Level	LowWater Level	Drought Water Level	Annual Minimum WL	Coefficient of River Rregime	Remarks
	1-day	95-day	185-day	275-day	355-day	365-day	Max/Min	
1962	5.82	2.93	2.38	1.46	-	-	∞	
1963	5.88	2.90	2.32	1.98	1.46	1.37	4.29	
1964	5.33	3.14	2.56	2.32	2.10	2.04	2.61	
1969	5.31	2.56	2.15	0.98	-	-	∞	
1970	5.34	2.71	2.27	1.89	1.33	1.20	4.45	
1971	4.85	2.66	2.19	1.74	1.31	0.00	∞	
1972	4.24	2.35	2.01	1.71	1.33	1.20	3.53	
1973	5.73	3.19	2.80	2.47	1.57	1.17	4.90	
1974	6.45	3.38	2.68	2.35	2.10	2.07	3.11	
1977	5.18	3.09	2.41	2.26	2.07	2.04	2.54	
1978	6.04	3.14	2.41	2.16	1.86	1.83	3.30	
1981	5.36	3.00	2.50	2.32	2.20	-	∞	
1982	6.04	2.88	2.30	2.07	1.95	1.90	3.18	
1983	5.66	3.10	2.58	2.23	2.08	2.06	2.75	
1984	6.14	3.04	2.31	2.12	1.98	1.96	3.13	
1985	6.21	3.15	2.45	2.04	1.87	1.86	3.34	
1986	5.96	2.88	2.32	2.18	2.00	1.99	2.99	
1987	6.85	3.24	2.35	2.07	1.92	1.88	3.64	
1988	6.51	3.21	2.34	2.15	2.05	1.98	3.29	
1989	6.28	3.08	2.32	1.98	1.87	1.86	3.38	
1990	6.08	3.08	2.48	2.13	1.96	1.94	3.13	
1991	6.67	3.53	2.59	2.22	2.04	2.00	3.34	
1992	6.26	3.06	2.41	2.19	1.96	1.91	3.28	
1993	6.57	3.51	2.84	2.30	2.14	2.12	3.10	
1994	6.16	3.22	2.68	2.55	2.26	2.22	2.77	
1995	6.71	3.20	2.69	2.19	2.05	2.01	3.34	
1996	6.52	3.39	2.75	2.40	2.21	2.18	2.99	
1997	7.03	3.50	2.65	2.42	2.28	2.27	3.10	
1998	6.85	3.66	2.56	2.30	2.18	2.16	3.17	
1999	6.76	3.46	2.43	2.19	-	-	∞	
2000	6.88	3.53	2.64	2.27	-	-	∞	
2001	6.44	3.26	2.71	1.93	-	-	∞	
2002	6.82	3.37	2.46	2.33	-	-	∞	
2003	6.25	3.37	2.52	2.39	-	-	∞	
2004	6.32	3.08	2.36	2.09	1.94	1.91	3.31	
2005	6.40	3.15	2.62	2.03	1.87	1.85	3.46	
2006	6.51	3.35	2.37	2.21	2.08	2.06	3.16	
2007	6.41	3.54	2.70	2.22	2.10	2.08	3.08	
2008	7.01	3.18	2.36	2.15	2.03	2.01	3.49	
2009	6.99	3.07	2.24	2.14	1.96	1.93	3.62	
2010	6.33	3.02	2.39	1.94	1.73	-	∞	
2011	6.83	3.18	2.42	2.07	1.96	1.91	3.58	
2012	7.62	3.01	2.26	2.08	1.95	1.91	3.99	
2013	6.17	2.97	2.29	2.11	1.89	1.72	3.59	
2014	5.32	2.61	2.07	1.91	1.71	1.70	3.13	
2015	7.20	3.38	2.34	1.86	1.69	1.62	4.44	
2016	6.80	2.78	2.18	1.97	-	-	∞	
Average	6.24	3.13	2.44	2.11	1.92	1.84	3.40	
Maximum	7.62	3.66	2.84	2.55	2.28	2.27	-	
Minimum	4.24	2.35	2.01	0.98	1.31	0.00	-	

Source: JICA Survey Team

Table 3.4-52 Typical Water Level of SW203 (Matamuhuri River, m PWD)

Year	Annual Maximum WL	Plentiful Water Level	OrdinaryWater Level	LowWater Level	Drought Water Level	Annual Minimum WL	Coefficient of River Rregime	Remarks
	1-day	95-day	185-day	275-day	355-day	365-day	Max/Min	
1962	11.24	6.83	6.16	5.87	-	-	∞	
1963	14.81	6.30	5.88	5.84	5.67	5.58	2.66	
1964	9.35	6.49	6.05	5.67	5.49	5.47	1.71	
1965	15.06	7.01	6.25	5.85	5.73	5.73	2.63	
1966	12.80	6.89	6.28	6.10	5.98	5.93	2.16	
1967	11.55	6.74	6.28	6.13	6.05	6.04	1.91	
1968	13.14	6.83	6.25	6.16	6.01	6.01	2.19	
1969	12.85	6.82	6.17	5.93	5.84	5.81	2.21	
1970	13.00	6.83	6.30	6.02	5.93	5.90	2.20	
1971	10.35	6.94	6.19	5.88	-	-	∞	
1972	9.80	6.86	5.95	5.81	5.61	5.55	1.77	
1973	10.83	6.76	6.39	5.85	5.73	5.72	1.90	
1974	12.78	7.16	6.39	6.07	6.01	5.98	2.14	
1975	12.38	7.12	6.79	6.16	5.98	1.12	11.02	
1976	13.58	7.05	6.43	6.33	6.22	6.20	2.19	
1977	10.38	7.07	6.43	6.24	6.16	6.14	1.69	
1978	12.77	7.28	6.28	6.08	5.96	5.95	2.15	
1979	12.26	6.85	6.37	6.04	5.98	5.98	2.05	
1980	11.43	7.29	6.66	6.36	6.22	6.22	1.84	
1981	10.71	7.00	6.33	6.22	6.16	6.16	1.74	
1982	12.64	6.79	6.39	6.11	6.03	5.98	2.11	
1983	10.71	7.03	6.60	6.35	6.13	6.01	1.78	
1984	12.61	6.75	6.14	6.00	5.92	5.90	2.14	
1985	12.43	6.84	6.22	5.95	5.85	5.82	2.14	
1986	10.39	6.63	6.04	5.93	5.86	5.83	1.78	
1987	14.75	7.00	6.16	5.80	5.72	5.70	2.59	
1988	11.87	6.63	6.18	6.00	5.77	5.77	2.06	
1989	11.74	6.93	6.37	6.06	6.00	6.00	1.96	
1990	11.64	7.78	7.22	6.22	5.48	5.33	2.18	
1991	12.42	7.60	6.96	6.50	5.44	5.44	2.28	
1992	11.55	6.83	6.40	5.37	5.27	5.23	2.21	
1993	14.01	7.04	6.56	6.41	6.33	6.22	2.25	
1994	11.16	6.93	6.43	6.30	6.13	6.10	1.83	
1995	13.38	7.24	6.93	6.03	5.93	5.92	2.26	
1996	12.00	7.26	6.97	6.61	6.37	6.35	1.89	
1997	14.83	7.34	6.34	6.16	5.64	5.60	2.65	
1998	13.05	7.74	7.25	6.10	5.55	5.53	2.36	
1999	15.46	7.41	7.21	7.08	6.81	-	∞	
2000	13.01	7.83	6.89	6.83	-	-	∞	
2001	11.31	7.19	6.54	6.29	-	-	∞	
2002	13.41	7.32	6.71	6.52	-	-	∞	
2003	11.67	7.34	6.84	6.67	-	-	∞	
2004	11.77	7.36	6.87	6.83	6.66	6.64	1.77	
2005	12.77	7.17	6.82	6.69	-	-	∞	
2006	12.51	7.61	6.89	6.48	6.42	6.41	1.95	
2007	14.35	7.83	6.84	6.70	6.44	6.32	2.27	
2008	13.26	7.40	6.73	6.42	6.32	6.28	2.11	
2009	13.87	7.56	6.77	6.58	6.41	6.40	2.17	
2010	13.45	7.55	6.93	6.59	6.40	6.39	2.10	
2011	14.06	7.76	6.94	6.58	6.50	6.48	2.17	
2012	14.65	7.60	6.87	6.71	6.44	6.43	2.28	
2013	12.16	7.15	6.79	6.74	6.69	-	∞	
2014	11.29	6.89	6.67	6.64	6.51	6.50	1.74	
2015	14.11	7.65	6.96	6.52	-	-	∞	
2016	12.24	7.09	6.86	6.58	6.21	6.12	2.00	
Average	12.50	7.13	6.55	6.25	6.04	5.87	2.13	
Maximum	15.46	7.83	7.25	7.08	6.81	6.64	-	
Minimum	9.35	6.30	5.88	5.37	5.27	1.12	-	

Source: JICA Survey Team

Table 3.4-53 Typical Water Level of SW200 (Maheshkhali Channel)

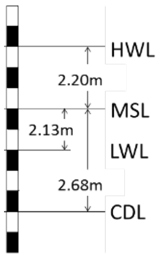
Year	Annual Maximum WL	Plentiful Water Level	OrdinaryWater Level	LowWater Level	Drought Water Level	Annual Minimum WL	Coefficient of River Rregime	Remarks
	1-day	95-day	185-day	275-day	355-day	365-day	Max/Min	
1969	3.51	2.56	2.13	1.69	-	-	∞	
1970	3.54	2.51	2.07	1.68	1.01	0.00	∞	
1971	3.23	2.44	2.01	1.65	1.07	0.24	13.74	
1972	3.05	2.50	2.07	1.62	1.00	0.88	3.47	
1973	3.17	2.50	2.04	1.65	1.01	0.15	21.13	
1974	3.75	2.53	2.07	1.71	1.16	0.25	15.00	
1976	3.16	2.37	1.92	-	-	-	∞	
1977	3.35	2.53	2.13	1.71	1.09	0.76	4.41	
1983	4.21	2.72	2.25	1.29	-	-	∞	
1984	3.65	2.50	2.15	1.65	1.25	1.05	3.48	
1985	4.36	2.60	2.20	1.80	1.10	0.85	5.13	
1986	3.60	2.60	2.20	1.80	1.30	1.20	3.00	
1987	4.20	2.80	2.40	1.90	1.50	0.23	18.26	
1988	4.20	2.80	2.40	1.90	1.40	1.40	3.00	
1989	4.20	2.70	2.20	1.70	1.14	0.74	5.68	
1990	3.76	2.54	2.14	1.64	1.16	0.94	4.00	
1991	3.13	2.33	1.93	1.43	-	-	∞	
1992	3.33	2.33	1.83	1.40	0.90	0.70	4.76	
1993	3.56	2.31	1.93	1.42	0.73	0.53	6.72	
1994	3.16	2.38	1.96	1.51	1.06	0.66	4.79	
1995	3.74	2.66	2.21	1.73	1.01	0.81	4.62	
1996	3.88	2.78	2.34	1.98	1.63	1.28	3.03	
1997	3.75	2.94	2.45	2.04	-	-	∞	
2005	3.70	1.80	0.80	0.40	0.20	0.00	∞	
2006	2.80	1.90	1.50	1.00	0.20	-0.40	-7.00	
2008	4.17	3.14	2.76	2.43	1.93	1.43	2.92	
2009	4.02	3.04	2.84	2.62	2.22	1.76	2.28	
2010	3.95	3.25	3.12	2.87	2.35	2.04	1.94	
2011	4.02	3.32	3.12	2.87	2.46	2.08	1.93	
2012	4.20	3.20	2.67	1.88	1.47	1.27	3.31	
2013	4.02	2.72	2.42	1.67	1.32	1.27	3.17	
2014	3.42	2.67	2.42	2.17	1.82	1.54	2.22	
2015	4.02	2.42	2.12	1.97	1.62	1.07	3.76	
2016	4.14	2.57	2.12	1.92	1.74	1.70	2.44	
Average	3.70	2.61	2.20	1.78	1.31	0.91	4.06	
Maximum	4.36	3.32	3.12	2.87	2.46	2.08	-	
Minimum	2.80	1.80	0.80	0.40	0.20	-0.40	-	

Source: JICA Survey Team

Tide, Wave at Project Site

The nearest tide-gauge stations of the study area are Chittagong, Kutubdia and Cox's Bazar Ports (located at Bengal Bay). The tidal observation gauge installed at Kutbudia was removed in the year of 2011.

According to the report of "Matarbari Urtra Super Critical Coal-fired Power Plant Project (MUSCCPP)" and "Data collection survey", tidal investigation was conducted in Kutubdia Island in order to analyze the tidal level at the project site, and the harmonic analysis was conducted to predict 60 tidal constituents (the harmonic constants are shown in Table 3.4-54). As the result of the analysis, the range of tide at the project site was estimated about 4.33m and the CDL was around 2.68m under the MSL. The tide chart diagram at project site is shown in Figure 3.4-53. Also, the tide prediction result at 2011 by using harmonic constants is shown in Figure 3.4-55.



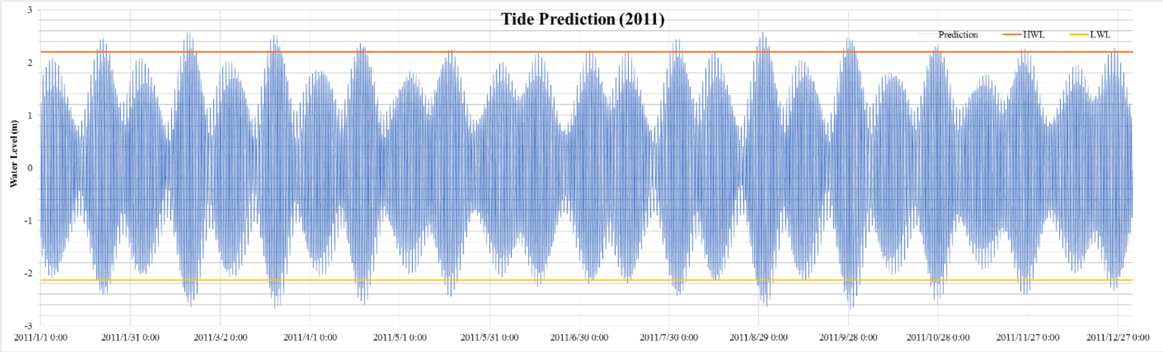
Source: Data collection survey on the Matarbari Port Development (JICA, 2018)

Figure 3.4-53 Tide Chart Diagram at Matarbari Port

Table 3.4-54 Tidal Harmonic Constants at Matarbari Port

No. of tidal constituents	Name	Amplitude	Phase	No. of tidal constituents	Name	Amplitude	Phase
1	Z0	0	0	31	M2	1.3375	9.41
2	SSA	0.0545	198.83	32	MKS2	0.179	318.29
3	MSM	0.0169	268.51	33	LDA2	0.0678	62.75
4	MM	0.0221	351.2	34	L2	0.0619	327.8
5	MSF	0.0316	35.08	35	S2	0.5275	340.15
6	MF	0.0144	7.36	36	K2	0.1454	327.18
7	ALP1	0.003	74.25	37	MSN2	0.0216	202.45
8	2Q1	0.0041	97.39	38	ETA2	0.0041	26.13
9	SIG1	0.0034	291.62	39	MO3	0.0038	315.31
10	Q1	0.0057	77.11	40	M3	0.0026	229.14
11	RHO1	0.0062	266.22	41	SO3	0.0056	221.33
12	O1	0.0806	23.27	42	MK3	0.0054	263.24
13	TAU1	0.0126	104.45	43	SK3	0.0137	196.5
14	BET1	0.0067	50.97	44	MN4	0.0066	40.32
15	NO1	0.0094	339.62	45	M4	0.0536	260.69
16	CHI1	0.0042	232.83	46	SN4	0.0025	76.32
17	P1	0.0404	339.91	47	MS4	0.0453	210.48
18	K1	0.1937	335.05	48	MK4	0.0267	180.73
19	PHI1	0.0024	317.27	49	S4	0.0123	171.36
20	THE1	0.0018	178.63	50	SK4	0.0068	150.75
21	J1	0.0045	340.51	51	2MK5	0.004	22.09
22	SO1	0.0039	107.9	52	2SK5	0.0021	76.63
23	OO1	0.0061	265.92	53	2MN6	0.0164	157.08
24	UPS1	0.0044	257.56	54	M6	0.0143	131.25
25	OQ2	0.009	125.94	55	2MS6	0.015	326.78
26	EPS2	0.0133	34.78	56	2MK6	0.0097	325.43
27	2N2	0.0254	90.75	57	2SM6	0.0079	321.21
28	MU2	0.0271	325.08	58	MSK6	0.0069	337.32
29	N2	0.2816	47.79	59	3MK7	0.0073	84.48
30	NU2	0.1029	315.17	60	M8	0.0116	70.52

Source: Data collection survey on the Matarbari Port Development (JICA, 2018)



Source: Data collection survey on the Matarbari Port Development (JICA, 2018)

Figure 3.4-54 Prediction of Astronomical Tide at Matarbari Port

Under the above two (2) studies, the following analyses were undertaken in order to evaluate the potential storm surge:

- Storm Surge Analysis: to verify the maximum storm surge height around the project site based on the historical cyclone records;
- Sensitive analysis: to apply the historically worst cyclone scale with shifted to worst course;
- Extreme statistical analysis: to examine the magnitude of corresponding return period against the historical storm surge height.

As a result of this analysis, the storm surge height corresponding to the probable return period was shown in Table 3.4-55. In addition, the associated effect of the water raise from the wave set-up generated by wave deformation and wave run-up generated by cyclone should be considered for the determination of land elevation of the Project site. As the conclusion, the design height for the land development at Matarbari Port in Table 3.4-56 was estimated in "Data collection survey".

Table 3.4-55 Probable Storm Surge Height at Matarbari Port

Return Period	Probable Storm Surge Height
10 years	1.58 m
20 years	2.59 m
25 years	2.98 m
30 years	3.32 m
50 years	4.41 m
100 years	6.29 m

Source: Data Collection Survey on the Matarbari Port Development (JICA, 2018)

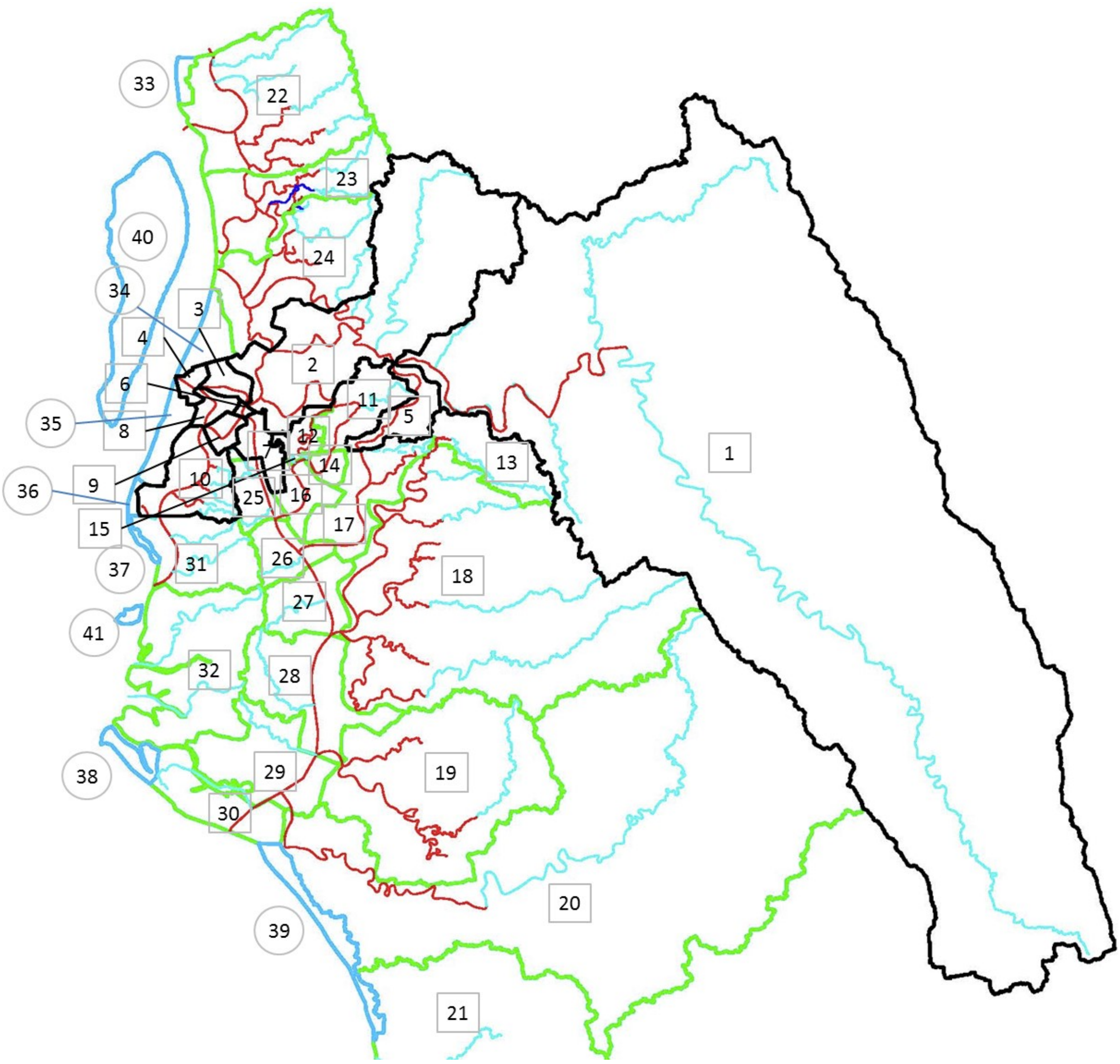
Table 3.4-56 Estimation of Height of Land Development at Matarbari Port

	1991 Cyclone Maximum	Anticipated Maximum	50 Years Return Period	100 Years Return Period
Storm Surge	5.9m	6.3m	4.5m	6.3m
H.W.L.	+2.2m MSL			
Wave Set-up	0.5m			
Total	+8.6m MSL	+9.0m MSL	+7.2m MSL	+9.0m MSL

Source: Data collection survey on the Matarbari Port Development (JICA, 2018)

<u>Drainage Basin</u>

The catchment areas for the study area are shown in Figure 3.4-55. The catchment areas were generated from the terrain analysis result by DEMs and GIS software with comparing and verifying the commercially available topographic maps. The range of the terrain analysis is set the range of 2-dimensional hydraulic analysis. The inventory of targeted catchment areas and the fluvial system diagram are shown in Table 3.4-57.

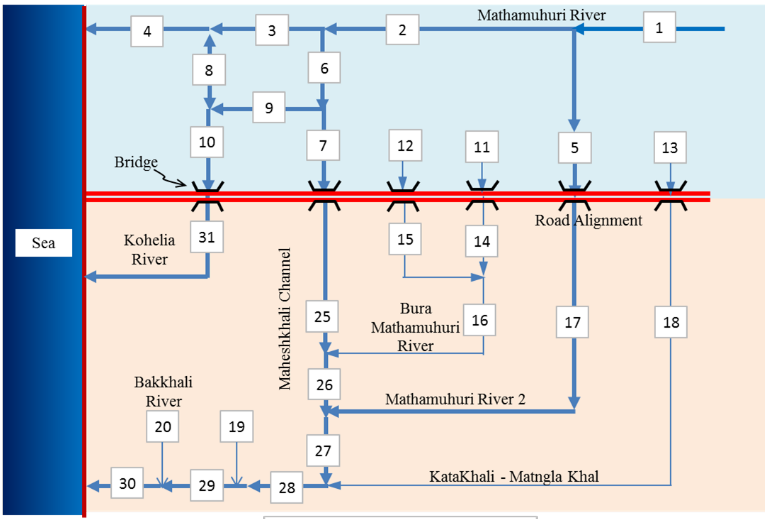


Source: JICA Survey Team

Figure 3.4-55 Related Catchment Area

Table 3.4-57 Related Catchment Area

ID	River	CA Name	Area (km2)	Remarks
1	Mathamuhuri	Mathamuhuri -Up	1385.57	
2	Mathamuhuri	Mathmuhuri -Middle	185.16	
3	Mathamuhuri	Mathamuhri -Down	7.66	
4	Mathamuhuri	Mathmuhuri -Estuary	4.27	
5	Mathamuhiri (2)	Mathmuhuri -Split-Up	18.62	
6	Maheshkhali	Maheshkhali -Up1	1.10	
7	Maheshkhali	Maheshkhali -Up2	11.66	
8	Kohelia	Kohelia -Mathamuhiri(Es)	6.04	
9	Kohelia	Maheshkhali -Kohelia	5.72	
10	Kohelia	Kohelia -Up	33.48	
11	Bura-Mathamuhuri	Bura-Mathamuhuri -Up	26.00	
12	Bura-Mathamuhuri (2)	Bura-Mathamuhuri (Tri.) -Up	9.10	
13	KataKhali-Mangla	KataKhali -Up	32.91	
14	Bura-Mathamuhuri	Bura-Mathamuhuri -Down1	5.82	
15	Bura-Mathamuhuri (2)	Bura-Mathamuhuri (Tri.) -Down	0.32	
16	Bura-Mathamuhuri	Bura-Mathamuhuri -Down	14.50	
17	Mathamuhiri (2)	Mathmuhuri -Split-Down	38.56	
18	KataKhali-Mangla	Mangla (KataKhali) -Down	336.35	
19	Bharuakhali	Bharuakhali -All	166.54	
20	Bakkhali	Bakkhali -All	593.97	
21	Reju	Reju -All	242.52	
22	Jal Kador	Jal Kador -All	121.09	
23	Dakshin	Dakshin -All	58.28	
24	Bhola	Bhola -All	83.35	
25	Maheshkhali	Maheshkhali -Down1	13.86	
26	Maheshkhali	Maheshkhali -Down2	18.25	
27	Maheshkhali	Maheshkhali -Down3	28.97	
28	Maheshkhali	Maheshkhali -Down4	58.58	
29	Maheshkhali	Maheshkhali -Down5	50.99	
30	Maheshkhali	Maheshkhali -Estuary	31.16	
31	Kohelia	Kohelia -Estuary	42.22	
32	Madardia-Baradia	Madardia-Baradia -All	90.29	
		Sub-Total-1 (Inland Area)	3722.91	
	Mathamuhuri -Maheshkhali - Kohelia		3127.38	Sum of 1-20, 25-31
33	Coast	Coast -1	3.68	
34	Coast	Coast -2	12.45	
35	Coast	Coast -3	12.79	
36	Coast	Coast -4	1.81	
37	Coast	Coast -5	1.65	
38	Coast	Coast -6	7.05	
39	Coast	Coast -7	17.21	
40	Island	Island -1 (Kutubdia)	69.06	
41	Island	Island -2	1.64	
		Sub-Total-2 (Island&Coastal Area)	127.35	
		Grand Total	3850.26	



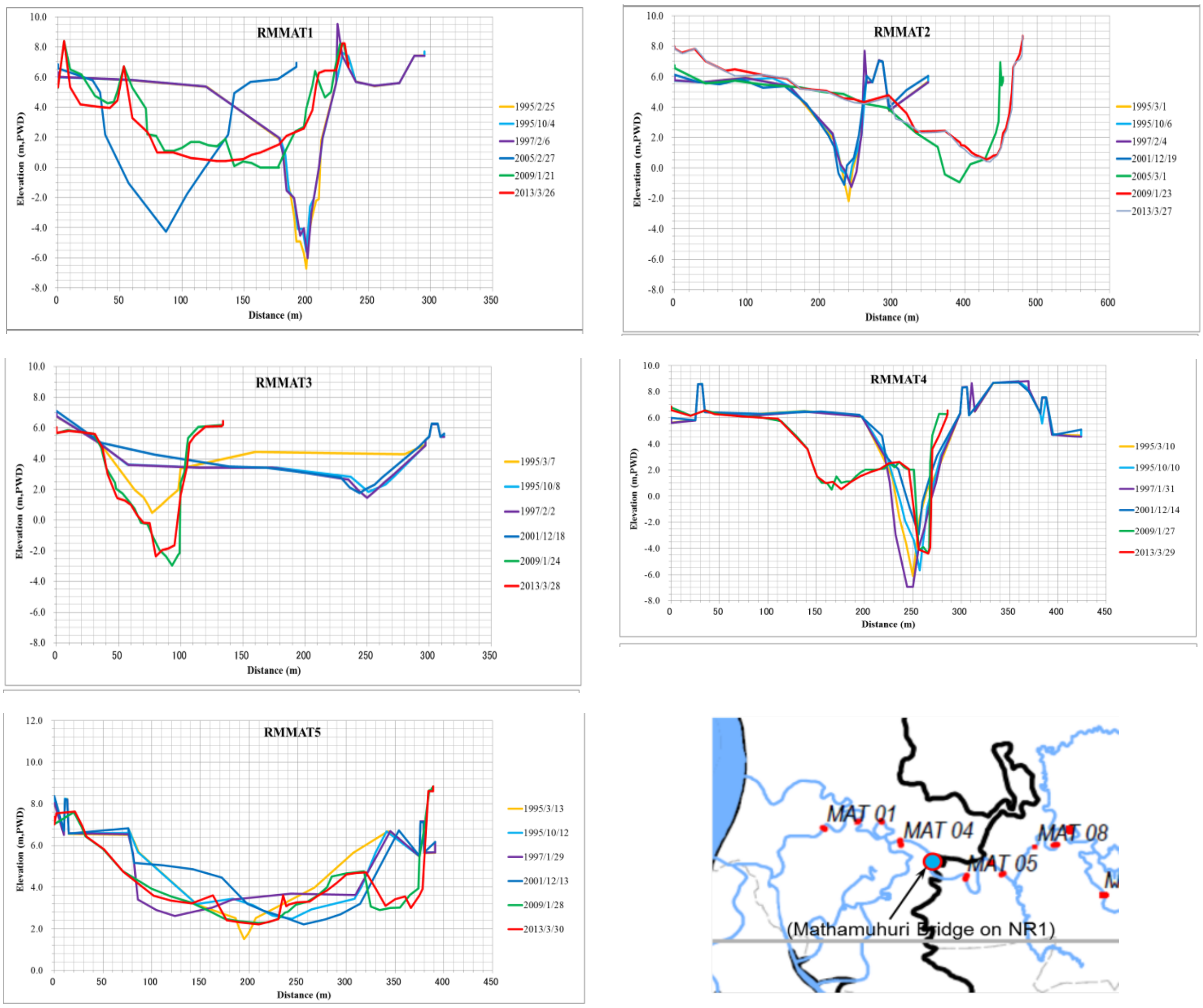
Fluvial System Diagram

Source: JICA Survey Team

River Morphology

The BWDB has undertaken the bathymetric survey at sections of the major rivers on a regular basis. Among these sections, there are five survey sections at near Matamuhuri as shown in Figure 3.4-56 (RMMAT5 at upstream of Matamuhuri Bridge of N1 and RMMAT1 to 4 at downstream of the bridge). These cross-section data were useful to check and understand the change of cross-sectional and longitudinal profile, such as aggradations and degradations process in the river channel.

According to the bathymetric survey results, it is possible to recognize that the cross-section of the river has continued to fluctuate over the years, although there is a difference in surveying accuracy (the river bank erosion and riverbed degradation are proceeding, and specially, only the surrounding of bridge is deep and has been eroded). The existing revetment and embankment would not be robust. Therefore, the river bank protections for bridges are required for keeping good condition of the project road embankment and bridges.



Source: JICA Survey Team, BWDB

Figure 3.4-56 River Bed Fluctuations at Matamuhuri River

2) Design Flood Discharges and Levels

Design Discharges (Probability Floods) at Gauging Station SW203

There are many methods and procedures utilized for the flood estimation and the theories for many methods have been developed by various institutions and researchers either based on measured statistical data, deterministic basis or empirical relationships. Except for the statistical method, these methods need to be calibrated for certain regions and flood events, and are limited in terms of the size of catchment areas for which they could be applied. In case if stream gauging records are not available or inadequate for streamflow estimation, design floods can be estimated by evaluating precipitation that would produce flood, and then converting the precipitation into discharge by simple runoff formulas and unit hydrograph. However, flow-based methods (i.e. frequency analyses) are preferred over conversion of precipitation to runoff in general because the flood flow rate is desired for larger catchment areas.

The observation data of SW203, which is a SGS located at the upstream of the Matamuhuri Bridge on N1, can be used for analyzing the flood discharge by frequency analyses. Although, the other basins are un-gauged basin because there is no streamflow gauging station at the areas, the basins are small and negligible. Therefore, simple formula from the probable-rainfall was applied for the other areas and was calculated by the rational formula which is the most commonly used worldwide.

The flood frequency analysis is calculated using the annual maximum discharge data of SW203 streamflow gauging stations (SGS), as shown in Table 3.4-58. However, the correlation at SW 203 is not so good due to the poor observation conditions and existence of many gaps of data.

Table 3.4-58 Probable Flood Discharge at SW 203

Station Name			Lama	Remarks
River Name			Mathamuhuri	
Station ID			SW203	
Long. (X)			92.2124	
Lat. (Y)			21.7926	
Catchment Area (km2)			1,010	
Data No. of Extreme Value			33	
Probable Discharge (m3/s)	(Year)	(%)		
	2	50%	293	
	3	33.3%	438	
	5	20%	599	
	10	10%	802	
	20	5%	996	
	25	4%	1058	
	30	3.33%	1108	
	50	2%	1248	
	80	1.25%	1376	
	100	1%	1437	
	150	0.667%	1547	
	200	0.5%	1625	
	300	0.333%	1734	
	400	0.25%	1812	
	500	0.2%	1873	
X-COR(99%)			0.913	
P-COR(99%)			0.959	
SLSC(99%)			0.102	
Probabilistic Distributed model			Gumbel distribution	

Source: JICA Survey Team

Flood Frequency Analysis of Peak Water Levels at Gauging Stations

In this study, the design water levels are estimated through the following steps: i) to undertake frequency analysis of the surrounding area of all stations, ii) to interpolate and estimate the probable water-levels in the study area by Triangulated Irregular Network (TIN) interpolation method; and iii) to undertake 1-d and 2-d analyses in comparison with the interviewed water levels.

Peak water level frequency analysis was also undertaken using the records of peak water levels observed at the nearest Peak Water Levels at Gauging Stations (WLGSs). There are nine (9) WLGSs located near the project site (SW204 and SW203: Matamuhuri River, SW200: Maheshkhali Channel, SW176: Kutubdia Channel (in outer sea), SW40: Bakkhali River, and SW250, SW248, SW 247, SW245: Sangu River). The peak water level at the WLGS in BWDB was recorded in mPWD units, and this is generally converted into mMSL using the conversion factor (α).

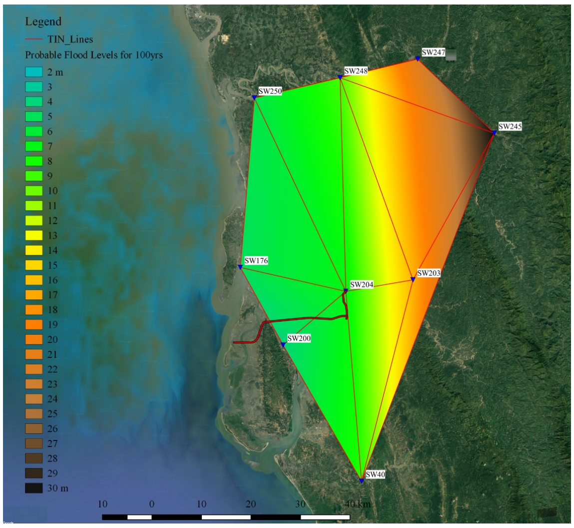
PWL(m MSL) = PWL(m PWD) + α

Although, the formal conversion value α of -0.46m is used by the Survey of Bangladesh (SoB) under Ministry of Defense, it was identified that the actual conversion factor at SW200 WLGS should be -0.945m based on the physical measurement at the SW200 by using the topographic survey results under this survey. The probable flood levels of each WLGSs are shown in Table 3.4-59. The water level distribution map of 100-year return period TIN interpolation method is shown in Figure 3.4-57 for reference.

Table 3.4-59 Probable Flood Levels at 9 Stations

No.	Station ID	Station Name	River Name	X (Longitude)	Y (Latitude)	1.1yr	5yrs	10yrs	20yrs	25yrs	50yrs	100yrs	500yrs	Remarks
1	SW40	Ramu	Bogkhali	92.11410	21.42580	5.58	6.80	7.19	7.56	7.68	8.05	8.41	9.24	
2	SW176	Lemsikhali	Kutubdia Channel	91.87501	21.81328	2.18	3.37	3.75	4.11	4.22	4.57	4.92	5.73	
3	SW200	Saflapur_Mohes	Moheshkhali Channel	91.95966	21.67253	2.25	3.05	3.30	3.55	3.62	3.86	4.10	4.64	
4	SW203	Lama	Matamuhuri	92.21240	21.79260	10.47	13.07	13.90	14.69	14.94	15.71	16.47	18.24	
5	SW204	Chiringa	Matamuhuri	92.08141	21.77101	5.42	6.38	6.57	6.72	6.77	6.89	7.01	7.23	
6	SW245	Ruma	Sangu	92.37000	22.06000	16.40	22.39	24.28	26.09	26.67	28.44	30.20	34.26	
7	SW247	Bandarban	Sangu	92.21920	22.19410	11.49	15.98	17.40	18.77	19.20	20.53	21.85	24.90	
8	SW248	Dohazari	Sangu	92.06766	22.15937	5.19	6.87	7.40	7.91	8.07	8.56	9.05	10.19	
9	SW250	Banigram	Sangu	91.90000	22.12160	3.05	4.06	4.38	4.68	4.78	5.08	5.38	6.06	

Source: JICA Survey Team



Source: JICA Survey Team, in m MSL

Figure 3.4-57 Distribution Map of Probable Flood Level (100-year Return Period, for Reference)

Interviewed Water Levels

In order to analogize the correlation between the flood-level around study area, the probability values at the gauging stations and calculated water-levels by hydraulic analyses, interview survey to the residents in the study area was conducted at 20 locations. Interviewed water levels are measured based on the SoB datum in MSL by survey devises (total station).

The locations of the interview survey and the interviewed historical high flood levels are shown in Figure 3.4-58 and the results of the interview survey are summarized in Table 3.4-61 to Table 3.4-64. The interviewed historical high water level (HHWL) at point-4 of sea side on the Maheshkhali Hill is 5.60 m in 1991 cyclone, and HHWL at near the end point of proposed access road is 4.28 m in 1991 cyclone.

Source: JICA Survey Team

Figure 3.4-58 Interviewed Locations and Interviewed Historical High Water Levels

Design Water Levels

The estimated design water-levels at the major bridge are shown in Table 3.4-60.

Table 3.4-60 Design Water Levels

ID	Station	River Name	Design Water Level		
			20-Year	50-Year	100-Year
M-2	1+423	Kohelia River	4.08	6.01	8.09
M-14	11+227	Maheshkhali Channel	2.89	4.31	5.59
M-16	14+270	Bara-Matamufuri 1	2.48	3.48	4.93
M-17	16+625	Bara-Matamufuri 2	2.39	3.38	4.78
M-18	18+730	Matamufuri River	2.34	3.27	4.61
M-26	24+475	Kata Khali (Mangla River)	2.57	3.26	4.51

Source: JICA Survey Team

Table 3.4-61 Interview Survey Results (1/4)

SL	Inerview Spot Number	X (Easting)	Y (Northing)	HHWL	AHWL	Impact of Monsoon / Rainy Seasons	Impact of Storm Surge (1991)	Flood on GL	Other Note	Photo
1	1	383770	2401514	5.58	2.15	Every year 3 to 4 times flood occured. During each flood, water stay 4 to 5 days. The average depth of flood water are 1.0m to 1.10m.	Historical flood occured during 1991 cyclone. During that cyclone the tide level was appx. 5.582mMSL. That time, we have lost our all crop, all domestic animal & 10 nos. relatives. During that time, all road net-work and bridges has been damaged.	Yes	Observed erosion in the Sea coast. Water level rising the tidal effect of Sea water. During high tide embankment is over flooded by spring tide.	
2	2	389472	2401622	5.56	?	This area is flood free area as it is on the hill top. Only during heavy rain fall some stagnent water observed on the low land which depth 0.30 m (max 0.50 m).	No impact of storm surge	Yes	-	
3	3	388710	2400335	5.51	?	Flood don't observed here. This is Hilly area. Only during heavy rain fall some stagnent water observed on the low land which depth 0.30m.	No impact of storm surge	Yes	-	
4	4	388412	2397262	5.60	?	Flood don't observed here. This is Hilly area. Only during heavy rain fall some stagnent water observed on the low land which depth 0.30m.	No impact of storm surge	Yes	-	
5	5	388364	2392133	5.52	?	Flood don't observed here. This is Hilly area. Only during heavy rain fall some stagnent water observed on the low land which depth 0.30m.	No impact of storm surge	Yes	-	
6	6	392567	2380321	5.34	3.59	Flood observed 4 to 5 times in every year by rising of Sea water. This flood water stay 3 to 4 days, which average depth 1.0m (max 2.0 m).	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, domestic animal lost. Some peopole also died. At that time water depth was about 3.5 to 4.0m. HFL level was 5.342 mMSL.	Yes	River became silted up day by day. During spring tide of monsoon, water level over top road & housing area.	

Source: JICA Survey Team

Table 3.4-62 Interview Survey Results (2/4)

SL	Inerview Spot Number	X (Easting)	Y (Northing)	HHWL	AHWL	Impact of Monsoon / Rainy Seasons	Impact of Storm Surge (1991)	Flood on GL	Other Note	Photo
7	7	394436	2389440	4.53	2.96	Flood don't observed in this area. River water don't over flow the land.	No impact of storm surge at here. Historical flood occured during 1991 cyclone. But during that cyclone don't observed any effect in this area, as it is opposit of hill. During that time HFL was 4.53 mMSL.	Yes	-	
8	8	391897	2397021	4.57	2.06	Flood observed 2/3 time in every year. Water stay 3 to 4 days only during spring tide. During that time water depth 0.80m over the agriland.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, domestic animal had damaged. And some people also died during that cyclone. As of my knowledge that time HFL was 4.568 mMSL.	Yes	River became silted up. Flood water don't overtop the road & housing area.	
9	9	391218	2401792	5.39	2.60	Flood observed 6/7 times in every year. Water stay 3 to 4 days only during spring tide. During that time water depth 1.50m over the agriland.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, domestic animal had damaged. And some people also died during that cyclone. As of my knowledge that time HFL was about 5.386 mMSL.	Yes	River became silted up. Water level overtop in the year 1991 due to Sea Cyclone.	
10	10	392447	2398820	4.52	2.58	Flood observed 3/4 times in every year. Water stay 3 to 4 days only during spring tide. During that time water depth 1.50m over the agriland.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, domestic animal had damaged. And some people also died during that cyclone. As of my knowledge that time HFL was 4.521 mMSL.	Yes	Flood water damaged seasonal carops and some time damaged fishing project. River became silted up. Water level overtop in the year 1991 by Sea Cyclone.	
11	11	392871	2405040	4.06	2.02	Flood observed 2/3 times in every year. Water stay 5 to 6 days. During that time water depth 1.00m over the agriland. Up-stream hilly flood water damaged seasonal carops and some time damaged fishing project. This flood occured if embankment damaged.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, domestic animal had damaged. And some people also died during that cyclone. It was about 3.00m high over the land. As of my knowledge that time HFL was 4.057 mMSL.	Yes	River became silted up. Water level overtop in the year 1991 by Sea Cyclone.	
12	12	397420	2405902	4.39	1.50	Flood observed 2/3 times in every year. Water stay 7 to 8 days. During that time water depth 1.00m over the agriland. Up-stream hilly flood water damaged seasonal carops and some time damaged fishing project. This flood occured if embankment damaged.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, domestic animal had damaged. It was about 3.0m high over the land. HFL level was 4.388 mMSL.	Yes	River became silted up. Water level overtop the road if embankment damage by up-stream hilly flood.	

Source: JICA Survey Team

Table 3.4-63 Interview Survey Results (3/4)

SL	Inerview Spot Number	X (Easting)	Y (Northing)	HHWL	AHWL	Impact of Monsoon / Rainy Seasons	Impact of Storm Surge (1991)	Flood on GL	Other Note	Photo
13	13	399487	2405412	4.39	1.50	Flood observed 4/5 times in every year. Water stay 7 to 8 days. Up-stream hilly flood water damaged seasonal carops and some time damaged fishing project. This flood occured due to embankment damaged by heavy rainfall and up-stream hilly flood. During that time water comes 1.1 - 1.80m over the agri-land.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, Fishing project, domestic animal had damaged. It was about 3.50m to 4.00 m high over the land. HFL level was 4.390mMSL.	Yes	River became silted up by the hilly flood. Water level overtop the road if embankment damage by up-stream hilly flood.	
14	14	400258	2403210	4.33	2.26	Every year 5/6 times water from river over flow the land. Water stay 4 to 5 days. Up-stream hilly flood water damaged seasonal carops and some time damaged fishing project. This flood occured due to embankment damaged by heavy rainfall and up-stream hilly flood. During that time water comes 1.1- 1.80m over the agri-land.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, Fishing project, domestic animal had damaged. It was about 4.00m high over the land. HFL level was 4.334 mMSL.	Yes	River became silted up by the hilly flood. Water level overtop the road if embankment damage by up-stream hilly flood. It's occured in the month of August or September, when heavy rain fall in the up-stream hilly area.	
15	15	402449	2401946	4.12	1.98	Every year 4/5 times flood observed by over flow the river water. Water stay 3 to 4 days. During that time water depth 0.80m to 0.90m over the agriland.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, Fishing project, domestic animal had damaged. It was about 4.00m high over the land. HFL was 4.120 mMSL.	Yes	River became silted up. Water level overtop the road if embankment damage by up-stream hilly flood. It's occured in the month of August or September, when heavy rain fall in the up-stream hilly area.	
16	16	403762	2402857	4.22	?	Every year 3/4 times flood observed by hilly water. Water stay 2 to 3 days. During that time water depth 0.70m to 1.0 m over the agriland. Flood water damaged seasonal carops and some time damaged fishing project. This flood occured due to embankment damaged by heavy rainfall and up-stream hilly flood.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, Fishing project, domestic animal had damaged. It was about 4.00m high over the land. As of my knowledge that time HFL was 4.220 mMSL.	Yes	River became silted up. Water level overtop the road if embankment damage by up-stream hilly flood. It's occured in the month of August or September, when heavy rain fall in the up-stream hilly area.	
17	17	404381	2402737	4.26	2.36	Every year 2/3 times flood observed by over flow the river water. Water stay 2 to 3 days. During that time water depth 0.40m to 0.50m over the agriland. Flood water damaged seasonal carops and some time damaged fishing project. This flood occured due to embankment damaged by heavy rainfall and up-stream hilly flood.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, Fishing project, domestic animal had damaged. As of my knowledge that time HFL was 4.260 mMSL.	Yes	River became silted up. Water level overtop the road if embankment damage by up-stream hilly flood. It's occured in the month of July or August, when heavy rain fall in the up-stream hilly area.	

Source: JICA Survey Team

Table 3.4-64 Interview Survey Results (4/4)

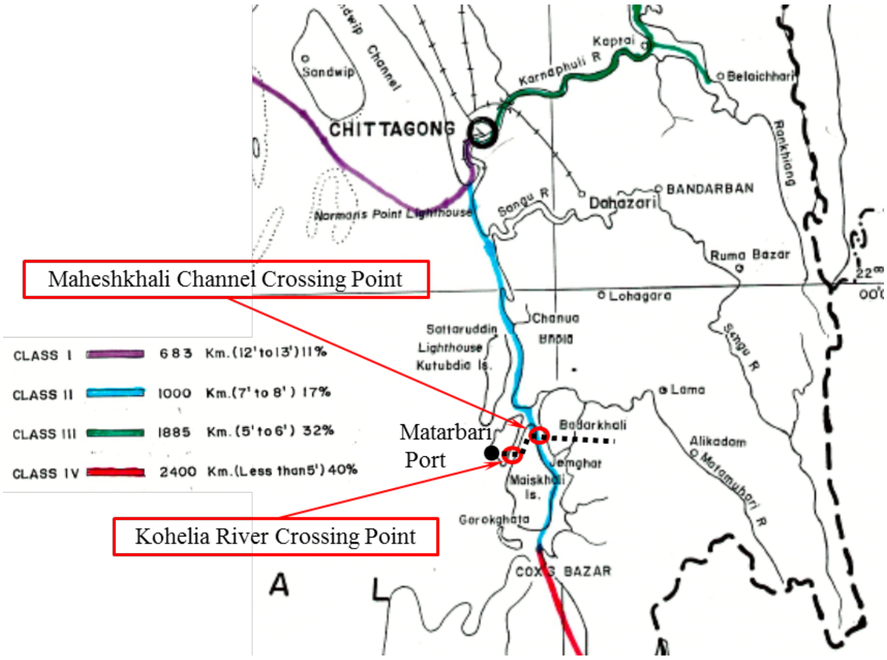
SL	Inerview Spot Number	X (Easting)	Y (Northing)	HHWL	AHWL	Impact of Monsoon / Rainy Seasons	Impact of Storm Surge (1991)	Flood on GL	Other Note	Photo
18	18	405221	2402604	4.28	2.13	Every year 1/2 times flood observed over the land. Water stay 1 to 2 days. During that time water depth 0.40m to 0.50m over the agriland. Mentionable damage don't occured.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, Fishing project, domestic animal had damaged. As of my knowledge that time HFL was 4.275 mMSL	Yes	River became silted up. No sevier flood observed in this area.	
19	19	403653	2393463	3.64	1.98	Every year 1/2 times flood observed over the land. Water stay 01 to 02 days. During that time water depth 0.50m to 0.60m over the agriland. Mentionable damage don't occured.	historical flood occured during 1991 cyclone. During that cyclone trees, crops, Fishing project, domestic animal had damaged. It was about 3.00m high over the land. As of my knowledge that time HFL was 3.635 mMSL.	Yes	The area is far from the river. No sevier flood observed in this area. Water level overtop the road by up-stream hilly flood, when heavy rain fall in the up-stream hilly area. It's occured in the month of July/August.	
20	20	402246	2383242	4.24	2.11	Flood don't observed in this area. If embankment damage than flood observed and water depth 0.60m to 0.80m over the agri land.	Historical flood occured during 1991 cyclone. During that cyclone trees, crops, Fishing project, domestic animal had damaged. It was about 2.00m high over the land. As of my knowledge HFL was 4.241 mMSL.	Yes	River became silted up by up-stream hilly flood. No sevier flood observed in this area. Water level overtop the road if embankment damage by up-stream hilly flood. It's occured in the month of August or September, when heavy rain fall in the up-stream hilly area.	
21	21	404862	2407525	6.42	-	I've seen the maximum flood at 1997. (Normal annual maximum flood did not damage any thing.) The Highest Flood Lavel is 6.429 m from MSL. Most of the agricultural land and low housing area, village road etc. went under water. All crops were damaged by that flood.	No impact of storm surge	Yes	Yes, historical changes around the river was noticed. The small river was widen it's width.	

Source: JICA Survey Team

3) Navigation Clearance

The Standard High Water Level (SHWL) is known as the overhead clearance datum which will seldom to be exceeded. SHWL is defined as the Fortnightly Mean Water Levels (FML) with 5% exceedance (once in 20 years return period) by Bangladesh Inland Water Transport Authority (BIWTA).

The BIWTA's waterways in the project site are classified as shown in Figure 3.4-59. The minimum vertical and horizontal clearances for the waterway classification are given in Table 3.4-65. The project road will cross Maheshkhali Channel, which is corresponded to Class II as it can be seen from Figure 3.4-59. Additionally, BIWTA specified Kohelia River to be considered as Class II waterway as the result of joint site investigation with BIWTA. BIWTA requires the consultation and approval against bridge construction having length of 100 m or more. Furthermore, as a Bangladesh common practice, the girder bottom (soffit level) of new bridges should be higher than these of existing bridges.



Source: BIWTA

Figure 3.4-59 Classification of Inland Waterways

Table 3.4-65 Fairway Limitation in Bangladesh

Classification of Waerways	Minimum Vertical Clearance	Minimum Horizontal Clearance	Remarks
Class-I	18.30m (60ft)	76.22m (250ft)	
Class-II	12.20m (40ft)	76.22m (250ft)	Proposed Kohelia and Maheshkhali Bridges
Class-III	7.62m (25ft)	30.48m (100ft)	
Class-IV Including seasonal rivers	5.00m (16.5ft)	20.00m (66ft)	

Source: BIWTA, 1991

(4) Hydraulic Analyses

1) General

The hydraulic phenomena at tidal compartment of the river (such as rising tide, falling tide, storm surge, etc. as well as the river own flood) are needed to simulate all temporal motions, as the tide level changes from moment to moment. Therefore, the range of numerical calculation shall be targeted all of the tidal area from river mouth to the non-tidal area. In the boundary of downstream, the tidal curves are necessary for hydraulic calculation, and the tidal curve of Figure 3.4-53 was input in the calculation model.

Also, since the river surveying range conducted in this study is limited, the channel topography was assumed and interpolated by using the following data and GIS software (however, the flow channels are limited to the main river channels). The river length and river plane shape was measured from the river route on the topographic map and Google Earth map. The total length to each upstream boundary from each river-mouth on the calculation is 218 km. In order to evaluate the topography of flood-plain necessary for two-dimensional analyses, the purchased high-definition digital terrain model (0.5m DTM) was used.

Table 3.4-66 Topographic Data used for Hydraulic Analyses

Item	Description
DEM Data for the Seabed Topography	Level-III data of "Data Collection Survey", and GEBCO[1] data
Riverbed Topography	Cross-section data of river topographic survey result of this study, past survey documents by BIWTA and Bangladesh Navy, etc.
DEM data for the floodplain topography	AW3D of JAXA – 0.5 m mesh data, SRTM[2] - 30 m mesh data,

[1] – GEBCO (General Bathymetric Chart of the Oceans), International Hydrographic Organization
[2] – JAXA (Japan Aerospace Exploration Agency), Japan
[3] – SRTM (Shuttle Radar Topographic Mission), NASA
Source: JICA Survey Team

The 2-dimensional analysis was performed for re-producing the hydraulic phenomena by the storm-surge of cyclone, such as flood velocity/ velocity-vector, inundation height/ range, flooding-/ receding-time at the floodplains and flow channels. And the 1-dimensional analyses were performed in order to calculate the hydraulic quantities (water-depth, velocity, shear-stress, etc.) and/or the scour depth for the surrounding of bridges.

2) Analysis Software

Hydraulic analysis was carried out to simulate the tidal and flood phenomena at the study area using HEC-RAS (Hydrologic Engineering Center - River Analysis System) developed by US Army Corps of Engineers, USA. HEC-RAS has the capability to compute one- and two-dimensional water surface profiles for both steady and unsteady flow. Sub-critical, supercritical and mix flow regime profiles can be calculated.

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

3) Hydraulic Analyses and the Precondition

The hydraulic analyses are conducted by following procedure.

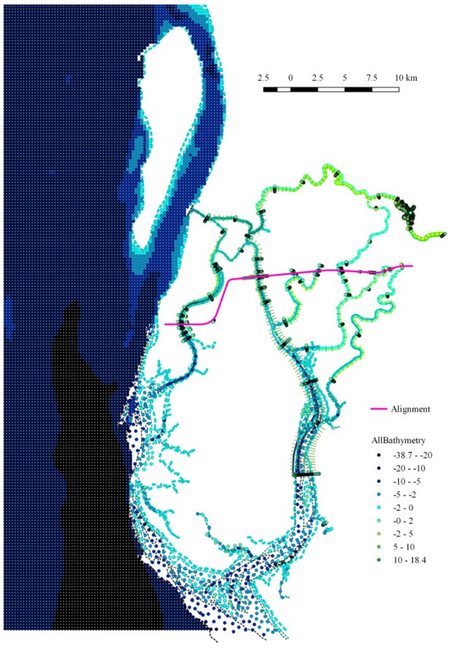
- To estimate the roughness coefficient of the river channel by simulating from the calculated astronomical tide levels at the proposed Matarbari Port site (see Figure 3.4-54) and past observation water level at SW200 (Maheshkhali Channel) and SW204 (Matamuhuri River) in 2011. The water level at WLGSs located in upstream can be calculated from the astronomical tide of downstream by the hydraulic calculation, if the water level from downstream is hydraulically continuous (influence of backwater). Specifically, the water level on calculation can approximate/ calibrate to the observed water level at WLGSs locations by changing the roughness-coefficient of each tidal-reach (SW200 and SW204 are tidal-stations, and the daily maximum and minimum water levels have observed).
- To calculate two flow regimes (subcritical-flow and mixed-flow regimes) for the case of storm-surge, since the flow regime may show a mixture of sub-critical and super-critical flow by the run-up for storm surge.
- To conduct the calculation case at the time of flood and storm-surge (of each return period) by using the above roughness coefficient.

Also, preconditions of the calculation case are as follows:

- The cross-sections and the terrain model for the hydraulic calculation are based on the bathymetric/ topographic survey results, other channels data (BIWTA, etc.) and the detailed DEM (AW3D, SRTM). As the preparation procedure, the DEM model for the river and sea areas first, and then, it is synthesized with the DEM model for the land. Figure 3.4-60 shows the contour lines and elevation points for the sea and river which was generated by the GIS software. Also, the generated terrain model for 2-dimensional analysis by the GIS software is shown in Figure 3.4-61. (2m mesh.) And the total number of hydraulic cross sections for interpolating is 325 cross sections. The software for 2-dimensional analysis is designed to use unstructured and/or computational meshes, and meshes are closely arranged along river center-lines in this model as seen in Figure 3.4-62 (unstructured meshes are limited to elements with up to eight sides). In this calculation model, there are 349669 meshes having mesh-areas of 10 m^2 to 10000m^2 in case of "without Road Embankment", and there are 376163 meshes in case of "with Road Embankment".
- The downstream boundary for hydraulic calculation during calculation period was given by the predicted tide-levels (at Matarbari Port) which vary from hour to hour (hence, the flow becomes the unsteady flow). At the external upstream boundary for 2-d calculation area and the internal boundaries for each river reach are given the monthly low runoff and/or flood runoff as the steady flow (in total, 57 inflow points were input to the model). The discharge to the inflow points is given as proportional

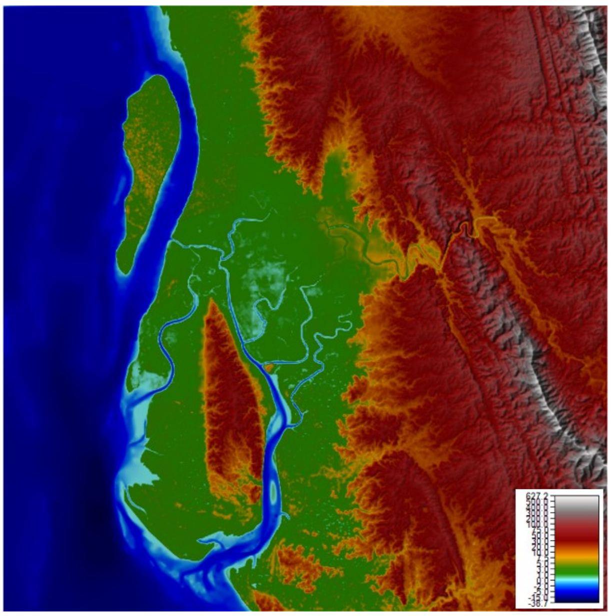
distribution between catchment area at inflow points and total area.

- The downstream boundary for the flood simulation calculation by storm surge is given by the predicted tide-level in 2011 plus probable storm surge height.
- In 1-dimensional analysis, the analysis ranges are limited to the surrounding areas of the targeted bridges, and flow conditions are limited to the steady flow only. For the boundary condition, the water-level at downstream-end and the discharge to the upstream-end of each bridge are given by using 2-dimensional analysis results. If the flood volume by the rational formula is large, it is checked using both values of the 2-dimensional result and the rational formula.



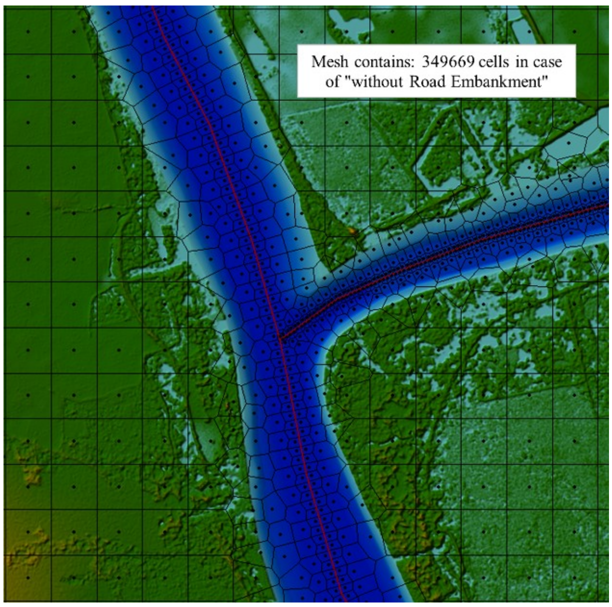
Source: JICA Survey Team

Figure 3.4-60 Contour Line and Elevation Points of River and Sea Area



Source: JICA Survey Team

Figure 3.4-61 Detailed Terrain Model (2m mesh DEM)



Source: JICA Survey Team

Figure 3.4-62 Unstructured and Structured Meshes on Calculation

4) Design of External Force and Calculation Cases

The reference tide level condition shall be the high tide level (+ 2.2 m) indicated by "Data Collection Survey". However, the impact of climate change is not considered. In the inflowing river, it cannot be denied that floods to some extent occur at the occurrence time of storm surge. However, it is not assumed that the maximum possible flood can occur at the same time as the maximum possible storm surge with high tide (from the recommendations of the Japanese government committee). Design external force assumes the following three (3) events:

- Storm Surge only: Maximum possible storm surge (Bridge: 100 years, Road embankment: 50 years return period), at the time of high-tide (surge height is shown in Table 3.4-55).
- Flood only: Maximum possible flood (Bridge: 100 years, Road embankment: 50 years return period.)
- Co-occurrence with flood and storm surge: Maximum possible storm surge (Bridge: 100 years, Road embankment: 50 years return period, with high-tide) and a certain scale of flood.

The calculation of combination cases with the design external force are shown in Table 3.4-67.

Table 3.4-67 Calculation Cases

Case ID	Name	Dimen-sion	Flow Regime	Existing Geometry (Terrains)	Future Geometry (Road & Bridge)	Return Period for Flood (Upland flow)					Return Period for Storm Surge			Remarks
						Low flow	10 years	20 years	50 years	100 years	20 years	50 years	100 years	
1-1	Calibration for Manning's n	2-d	Un-steady flow	● (1-1e)	● (1-1f)	●								For check tide-generating force
2-1	Only Flood by Storm Surge	2-d	Un-steady flow	● (2-1e)	● (2-1f)							●		
2-2		2-d	Un-steady flow	● (2-2e)	● (2-2f)								●	
3-0a	Only Flood by Upland Flow	2-d	Un-steady flow	● (3-0ae)	● (3-0af)		●							
3-0b		2-d	Un-steady flow	● (3-0be)	● (3-0bf)			●						
3-1		2-d	Un-steady flow	● (3-1e)	● (3-1f)				●					
3-2		2-d	Un-steady flow	● (3-2e)	● (3-2f)					●				
4-1	Co-occurrence with Flood and Storm Surge	2-d	Un-steady flow	● (4-1e)	● (4-1f)			●			●			for checking Culvert
4-2		2-d	Un-steady flow	● (4-2e)	● (4-2f)			●				●		for checking Embankment
4-3		2-d	Un-steady flow	● (4-3e)	● (4-3f)			●					●	for checking Bridge
5-1	Hydraulic calculation by using discharge (Q) which is calculated the 2-d analyses	1-d	Steady flow	● (5-1e)	● (5-1f)	●					for checking Bridge Hydraulics (Hydraulic Quantity, Scour Depth) with Brige Geometry (Pier / Deck Shape, Dimension)			
5-2		1-d	Steady flow	● (5-5e)	● (5-5f)			●						
5-3		1-d	Steady flow	● (5-6e)	● (5-6f)				●					
5-4		1-d	Steady flow	● (5-7e)	● (5-7f)					●				

Source: JICA Survey Team

5) 2-Dimensional Flood Analysis Results

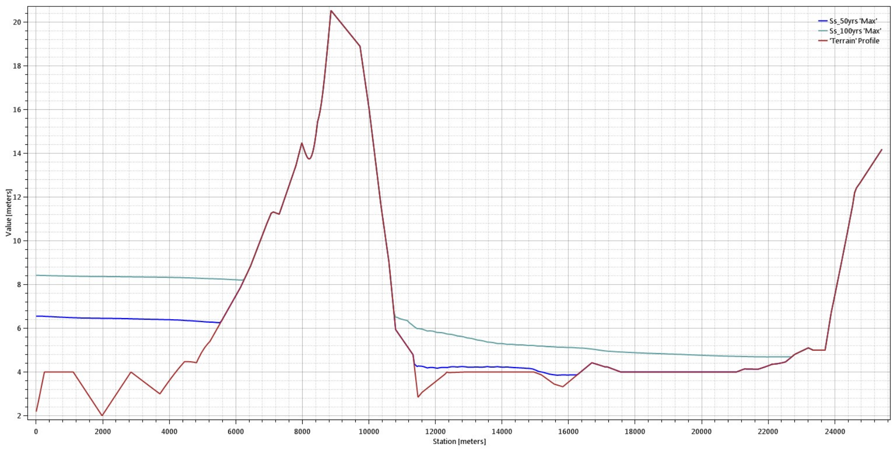
Pre-Analysis

The pre-analysis using the simplified DEM (GEBCO) was conducted for the purpose of setting the analysis range (this result of the pre-analysis is just for the purpose of setting the analysis range, and it is not used as the output for the hydraulic design).

The results of 2-dimensional analysis as a pre-analysis are shown in Figure 3.4-63 to Figure 3.4-64. GEBCO's simplified DEM expresses the ocean and land elevations with 900 m mesh, and if there is only design external force, the hydraulic analysis can be easily performed with only GEBCO's DEM. However, there is a big difference between the DEM elevation of GEBCO and the actual surveying elevation, and detailed landforms such as river / waterway, swamp, road etc. are not expressed.

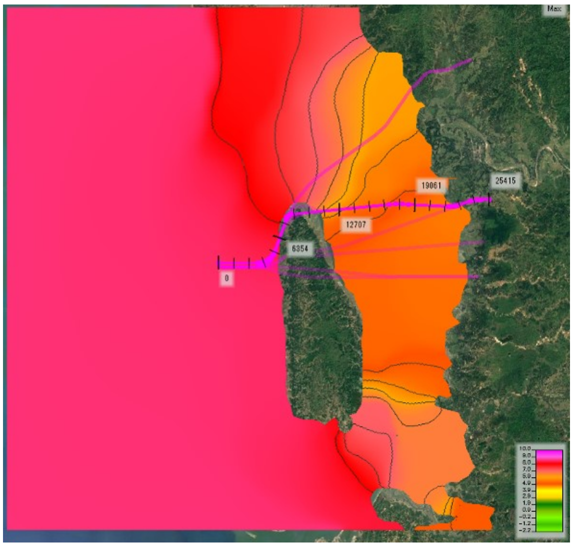
As calculation results, the flood levels due to the storm surge are gradually decayed towards the inland, and the farthest reached points of flood water are stopped short of 1km of N1 roughly, since GEBCO's elevations are relatively higher than AW3D's elevations. (The inundation area in calculation case of AW3D is beyond the N1, as described below.) Therefore, the detailed DEM which accurately expresses heights of rivers / roads should be used in order to reproduce detailed hydraulic phenomena around rivers / roads, etc., and the necessary range of detailed DEM recommends the range which takes into account a margin against the calculated flood area by the pre-analysis.

From the above, the detailed DEM of AW3D for the part related to the comparative examination area of the study road was purchased as shown in エラー! 参照元が見つかりません。. Regarding other calculation areas, the DEM of SRTM was obtained (the elevation of the AW3D-DEM data was adjusted in reference to the surveyed benchmarks installed under this survey. The DEM elevation of the SRTM was also adjusted to make it close to the elevation of the AW3D).

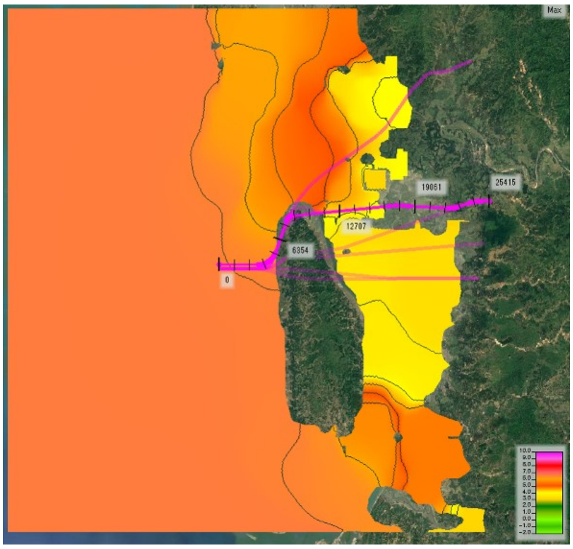


Source: JICA Survey Team

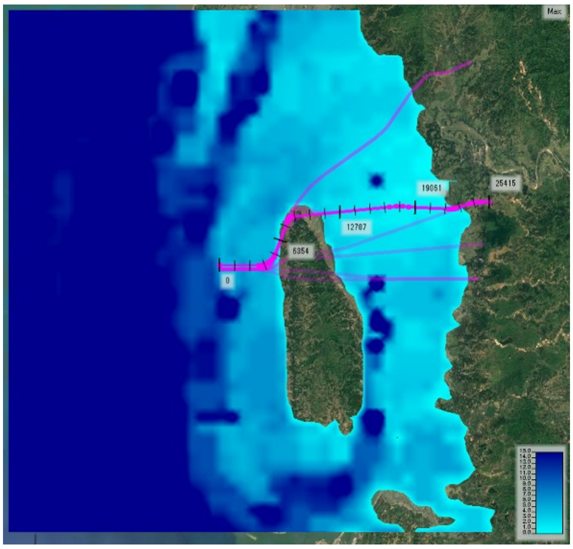
Figure 3.4-63 Hydraulic Profile on Proposed Road Alignment (100 / 50 year Storm Surge only)



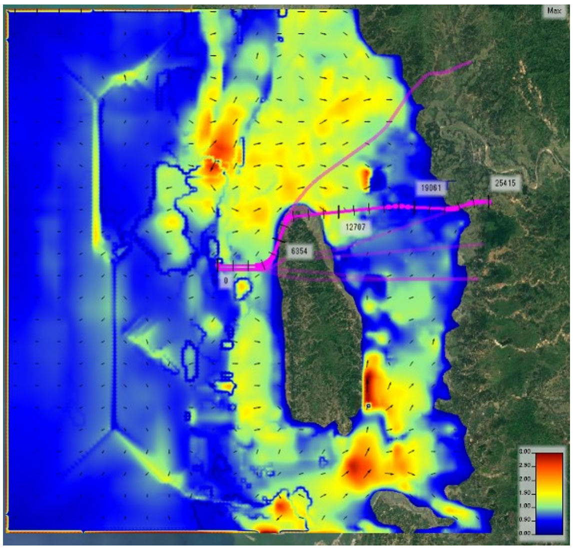
Maximum Water Surface Elevation Distribution (100 year Storm Surge only)



Maximum Water Surface Elevation Distribution (50 year Storm Surge only)



Distribution of Maximum Inundation Height (100 year Storm Surge only)



Flow Velocity / Velocity-Vector Distribution (100 year Storm Surge only)

Source: JICA Survey Team

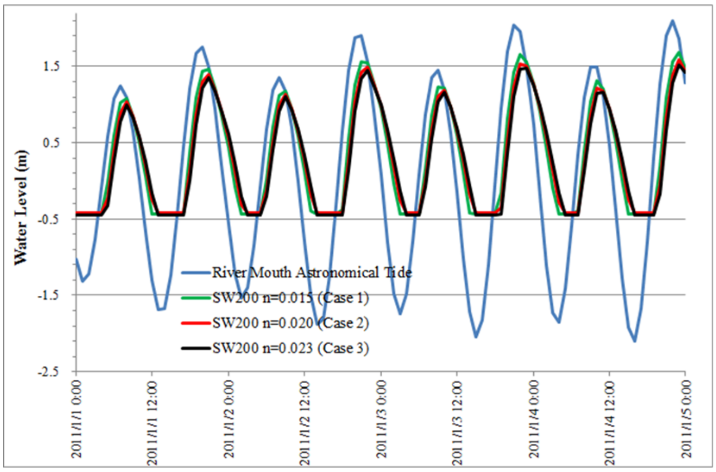
Figure 3.4-64 Results of 2-dimensional Pre-Analyses

Specifying the Roughness Coefficient (Calculation Case 1-1)

As shown in the river profile of the Matamuhuri Figure 3.4-46, the study area is a very low-lying area. (According to the classification of river characteristics in Japan, the river of survey area can be classified as a very gentle river of "segment-3.") According to Japanese standards, the roughness coefficient of the river classified as "segment-3" is generally set to the range of 0.015 - 0.023. In this study, the roughness coefficient that approaches the observed water-levels of SW 200 and SW 204 is set from 2-dimensional analyses by the detailed terrain model (however, the daily fluctuation of the water level at SW 204 is rarely observed in the non-flood period).

The stream gradient changing point is a "20 km point from the estuary" as shown in Figure 3.4-46. The roughness coefficient is divided into two reaches of the upstream and downstream from "20 km point from the estuary", and it is examined by two-dimensional analysis. The calculation shall be the calculation period of four days from 0 o'clock on January 1, 2011 to 0 o'clock on January 5, 2011, using the forecasted astronomical tide in 2011. Figure 3.4-65 shows the setting case for the roughness coefficient and the calculation results. As a result, the case where the roughness coefficient is Case 3 (downstream river reach: n = 0.023, upstream river reach: n = 0.025) is the closest to the observed water level. According to Japanese standards ("Guidance for study of river channel planning"), 0.023 is adopted as the standard value of the roughness coefficient in case of the river of segment-3, when the particle diameter component of riverbed material of 0.1 mm or more is 10% or less. (Figure 3.4-66 shows the grain size distribution curve of the riverbed material, the bed material of this area also has a particle size component of 0.1 mm or more in 10% or less.) From the above, it is considered as a combination of roughness coefficients of Case 3.

The roughness coefficient of other than river channels, is assumed to be 0.06 for the hilly area and 0.035 of other floodplains. (There are many ways to set the roughness coefficient, but in this study, the above values are adopted with reference to the standards of the United States and Japan.) The residential area is a low density area, and 0.04 is adopted in the Japanese standards. However, these residential areas in the study area are sparse residential areas, and 0.035 was applied same as the floodplain.) Figure 3.4-67 shows the distribution map of the roughness coefficient.



Roughness Coefficient

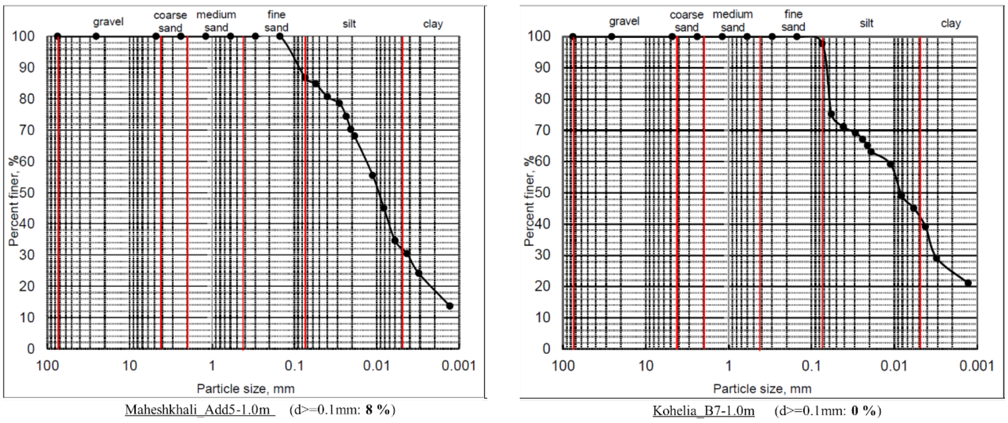
Location from Rivermouth	Case 1	Case 2	Case 3	Remarks
～21.5 km	0.015	0.020	0.023	SW200
21.5 km ～	0.020	0.025	0.025	SW204

Difference between HWL and LWL of Daily Water Level

Date	Calculated WL Difference at SW200			SW200 Record Difference
	Case 1	Case 2	Case 3	
2011/1/1	1.892	1.816	1.807	1.95
2011/1/2	1.989	1.907	1.899	1.92
2011/1/3	2.083	1.951	1.910	1.92
2011/1/4	2.115	2.002	1.974	1.90
Date	**Calculated WL Difference at SW204**			**SW204 Record Difference**
	Case 1	Case 2	Case 3	
2011/1/1	-	-	-	0.00
2011/1/2	0.031	0.017	0.015	0.01
2011/1/3	0.034	0.014	0.010	0.01
2011/1/4	0.045	0.020	0.014	0.00

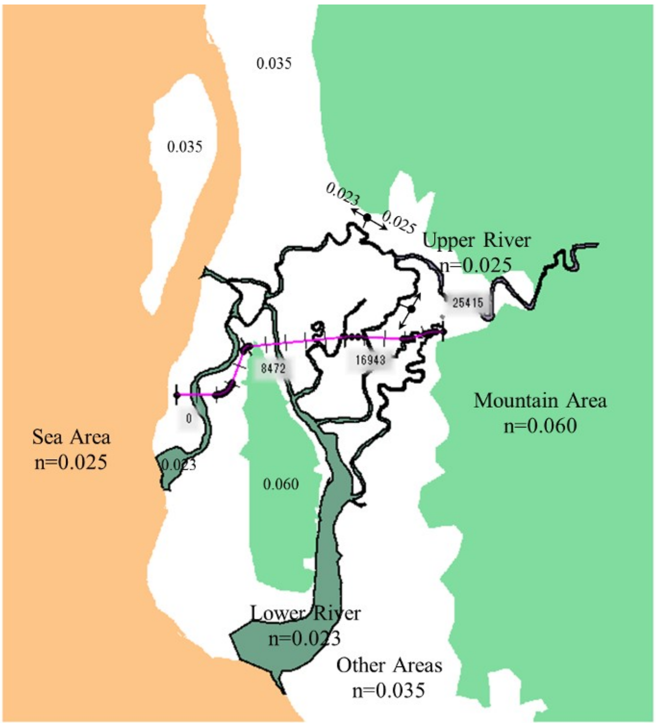
Source: JICA Survey Team

Figure 3.4-65 Calculated Water-Levels each Case of Roughness Coefficient at SW200 Station



Source: JICA Survey Team

Figure 3.4-66 Grain Size Distribution Curve for the Riverbed Materials



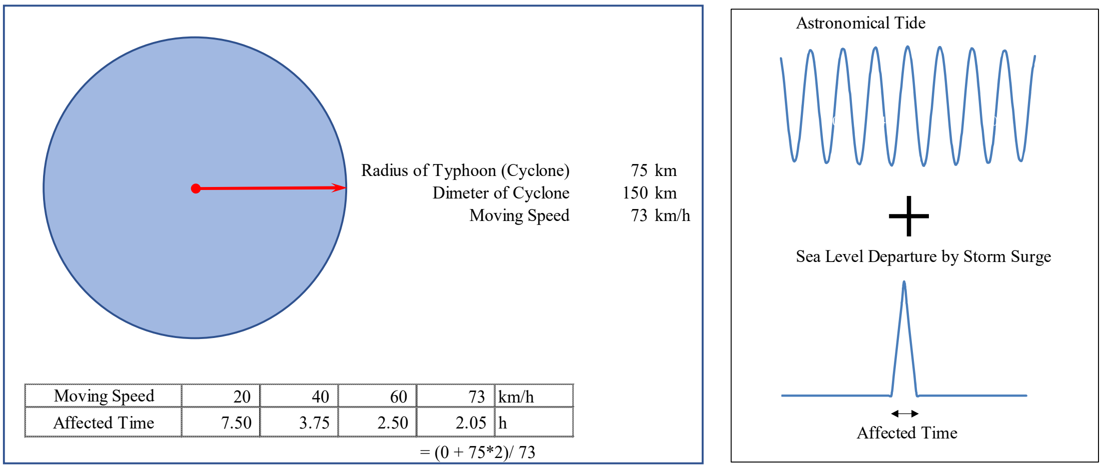
Source: JICA Survey Team

Figure 3.4-67 Setting Area for Roughness Coefficient

<u>Setting of Moving Speed of the Cyclone (Affected Time of the Storm Surge)</u>

The calculated tidal level of the ocean area which becomes the design external force adds the storm-surge deviations each design return-period to the predicted astronomical tide of 2011. Specifically, it is added to the peak high tide-level (2.208 m) at 9 o'clock on March 19 of 2011 which becomes nearly at the average high tide-level (+2.2 m). The calculation period is 4 days from 15 o'clock on March 18, 2011 to 15 o'clock on 22nd. According to Japanese standards ("Guidelines for creating storm-surge hazard map"), if the typhoon model on calculation is not dependent on the existing typhoon observation data, it is assumed that "the maximum typhoon wind speed radius is 75 km and the moving speed is constantly 73 km/h", and then analysis is performed. In this study also, two-dimensional analysis is performed due to the difference in moving speed and the difference in its influence is judged. (See Figure 3.4-68 and Figure 3.4-69.) This time, 2-dimensional analysis is performed by assuming the affected time to the storm-surge, in 3 cases of 2 hr, 4 hr and 6 hr, and it is confirmed the difference.

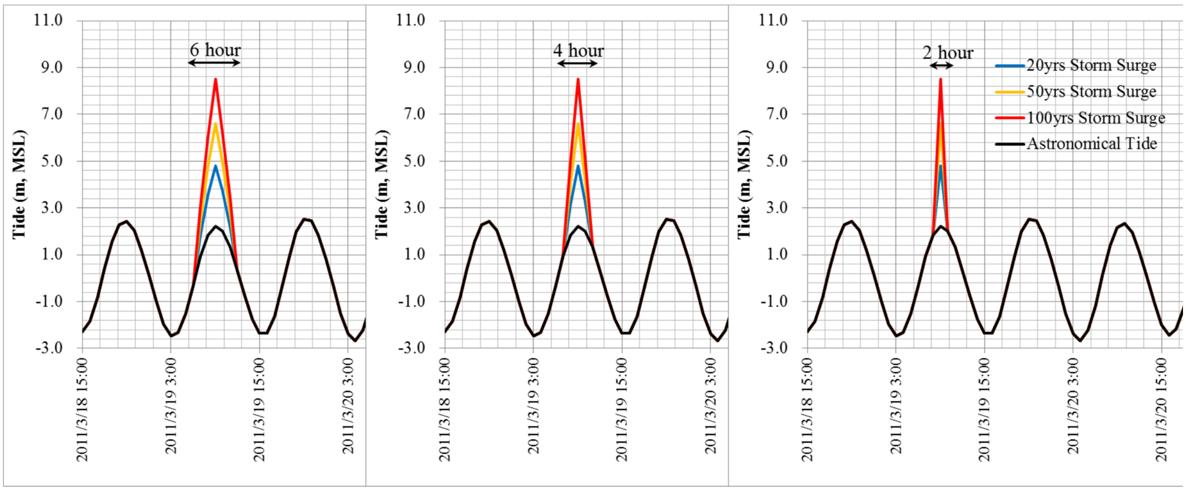
As a result, since the influence by the storm surge to the inland is greatest in case of "affected time: 6 hr", the analysis is henceforth proceeded as "affected time: 6 hr", considering the design of the safety side. (See Figure 3.4-70)



Moving Speed	20	40	60	73	km/h
Affected Time	7.50	3.75	2.50	2.05	h

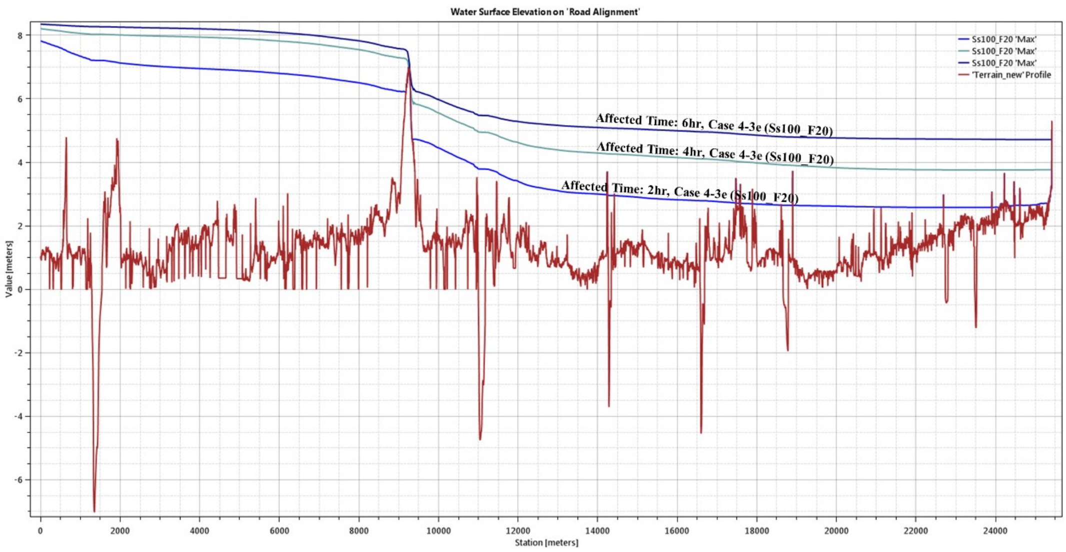
Source: JICA Survey Team

Figure 3.4-68 Relationship between Moving Speed and Affected Time



Source: JICA Survey Team

Figure 3.4-69 Affected Time of Storm Surge (Case of Input Condition)



Source: JICA Survey Team

Figure 3.4-70 Water Level Variation by Cyclone Cases (Calculation Case 4-3)

2-dimensional Hydraulic Analyses Results (Calculation Case 2 - 4)

2-dimensional hydraulic analyses are performed for the calculation cases shown in Table 3.4-67 with the conditions of "with/ without the project road embankment". For the case with the project road embankment, the embankment of CPGCBL's power plant and the proposed Matabari port sites, seawall and some culverts were also considered (see Figure 3.4-71). In addition, the calculation mesh along the main road embankment is subdivided, and the movement of water between the meshes is limited to the road height.

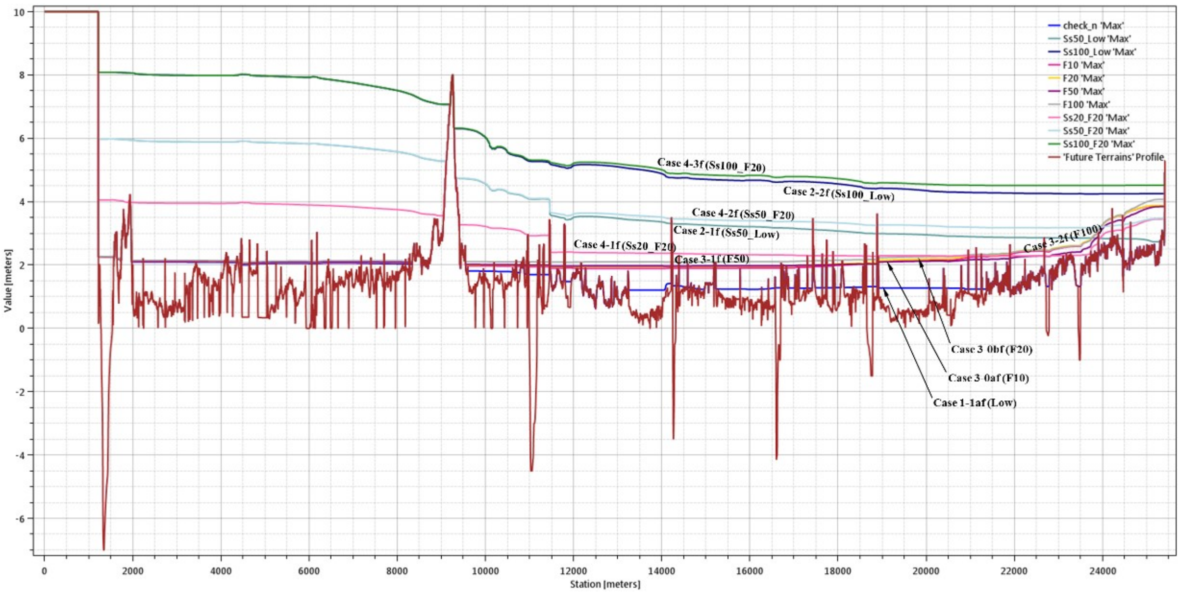
Source: JICA Survey Team

Figure 3.4-71 DEM used for the Case "with Project Road Embankment"

The water level calculation results with/ without embankment on the proposed road alignment are shown in Figure 3.4-72. From the calculation results, in any case, the calculated water-levels of Case 4-3 (simultaneous occurrence of 100-years probability storm-surge and 20-years probability flood) for the 100-years probability and Case 4-2 (simultaneous occurrence of 50-years probability storm-surge and 20-years probability floods) for the 50-years probability, are highest. Therefore, the road embankment height and the bridge height will be planned to take into account the freeboard against the Case 4-2 and Case 4-3 respectively (the design flood levels are shown in Table 3.4-60). The maximum water surface elevation distribution, maximum flooded depth distribution and flow velocity vector diagram of Case 4-3 and Case 4-2 with/ without the road embankment are shown in Figure 3.4-73 and Figure 3.4-74

Also, the contour map of maximum water level with/ without road embankment is shown in Figure 3.4-75. By filling the road embankment, the south side of the road embankment shows the water level lowering by about 1.0 m at the maximum 100-years probability storm-surge (Case 4-3), compared to the case without road embankment. From this result, it can be said that the road embankment will block the run up of storm-surge and has a mitigation effect against the storm surge. It should be noted that the elevations of the high-definition digital terrain model (0.5m DTM) is lower as a whole than the elevations of GEBCO used in the pre-analysis, so the influence of storm-surge run-up has reached inland area from the inundation area of pre-analysis (in the same Case 2-2 as pre-analysis, the run-up influence around end-point of proposed road outreaches the N1, and it is reached the point of 1.5 km upstream of the N1, compared with "1 km downstream of the N1" of the pre-analysis).

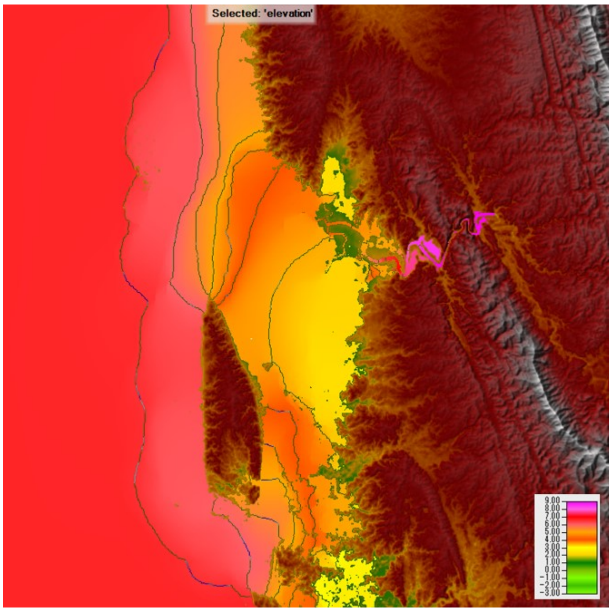
without Road Embankment



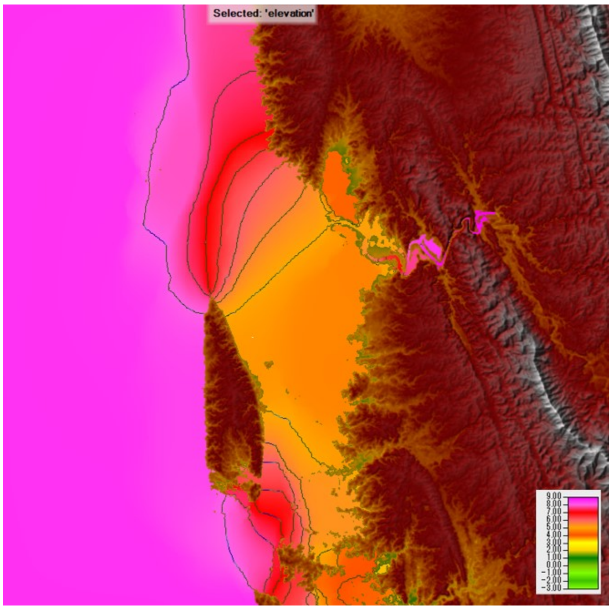
with Road Embankment

Source: JICA Survey Team

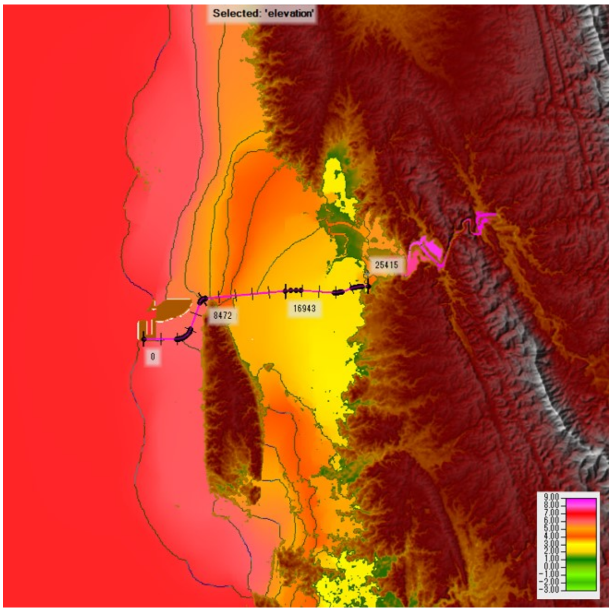
Figure 3.4-72 Hydraulic Profile on the Road Alignment



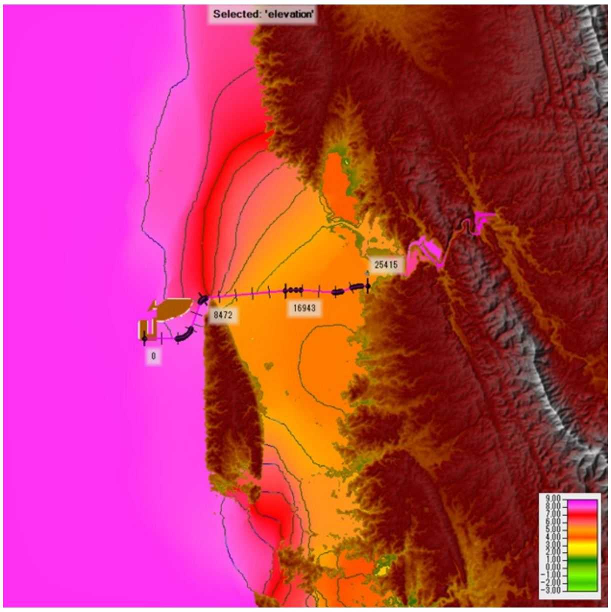
Maximum Water Surface Elevation Distribution (Case 4-2e, 50-years Storm Surge + 20-years Flood, without Road)



Maximum Water Surface Elevation Distribution (Case 4-3e, 100-years Storm Surge + 20-years Flood, without Road)



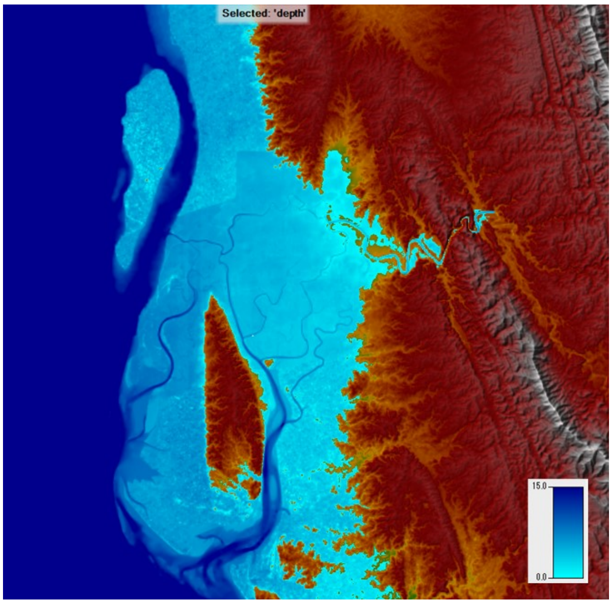
Maximum Water Surface Elevation Distribution (Case 4-2f, 50-years Storm Surge + 20-years Flood, with Road)



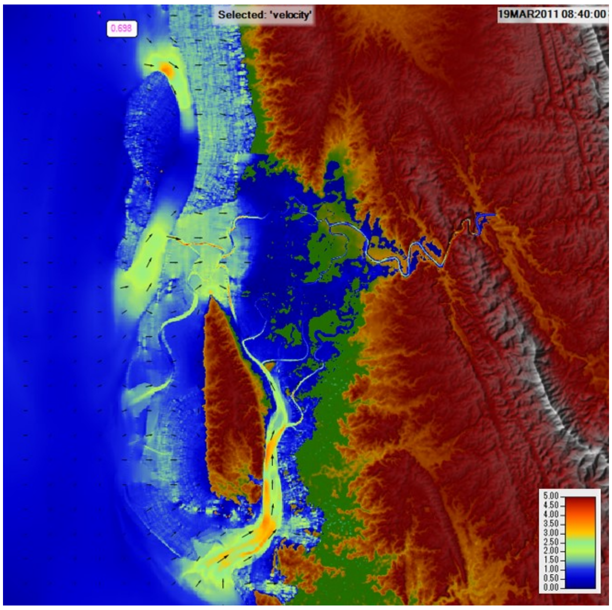
Maximum Water Surface Elevation Distribution (Case 4-3f, 100-years Storm Surge + 20-years Flood, with Road)

Source: JICA Survey Team

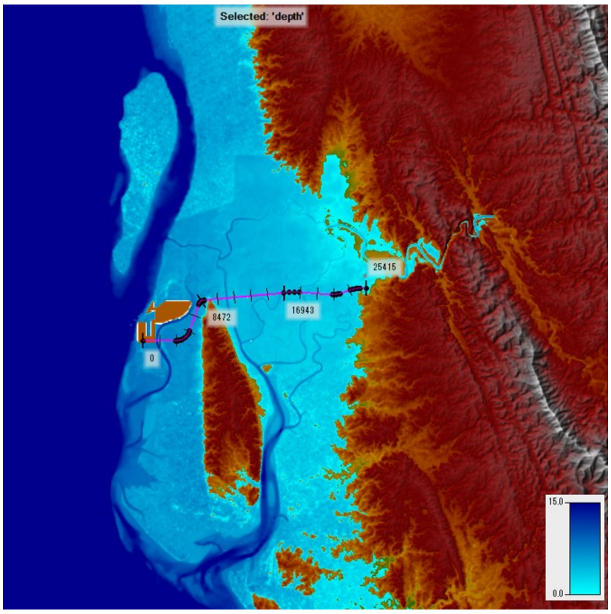
Figure 3.4-73 2-dimensional Hydraulic Analyses (1)



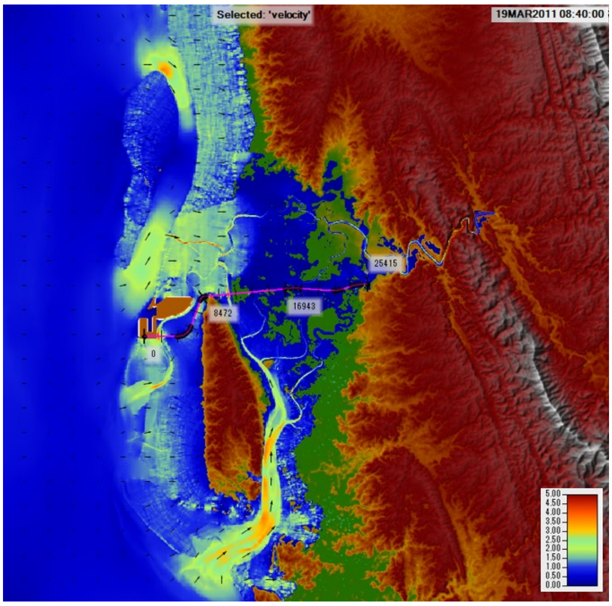
Distribution of Maximum Inundation Height (Case 4-3e, 100-years Storm Surge + 20-years Flood, without Road)



Flow Velocity / Velocity-Vector Distribution (Case 4-3e, 100-years Storm Surge + 20-years Flood, without Road)



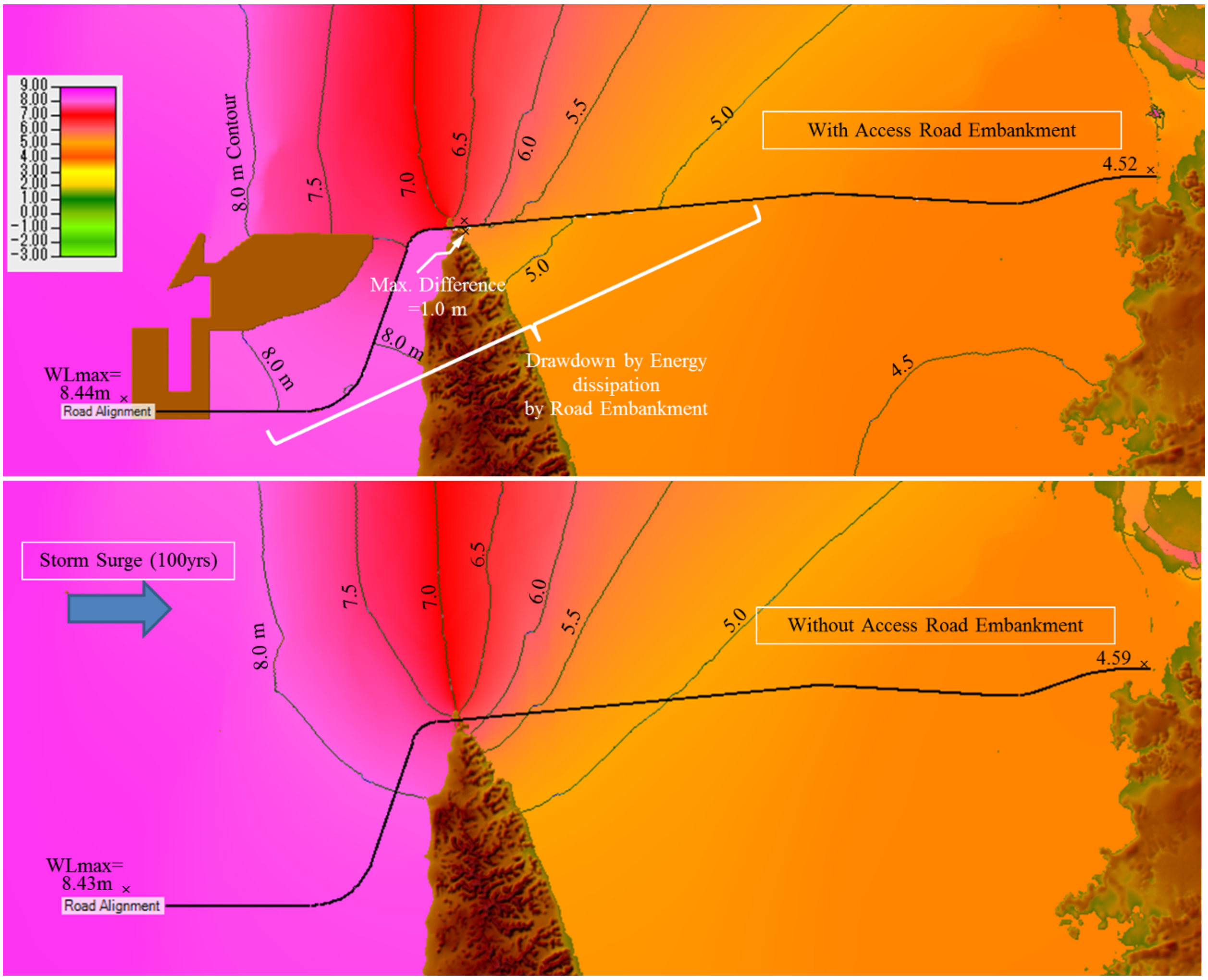
Distribution of Maximum Inundation Height (Case 4-3f, 100-years Storm Surge + 20-years Flood, with Road)



Flow Velocity / Velocity-Vector Distribution (Case 4-3f, 100-years Storm Surge + 20-years Flood, with Road)

Source: JICA Survey Team

Figure 3.4-74 2-dimensional Hydraulic Analyses (2)

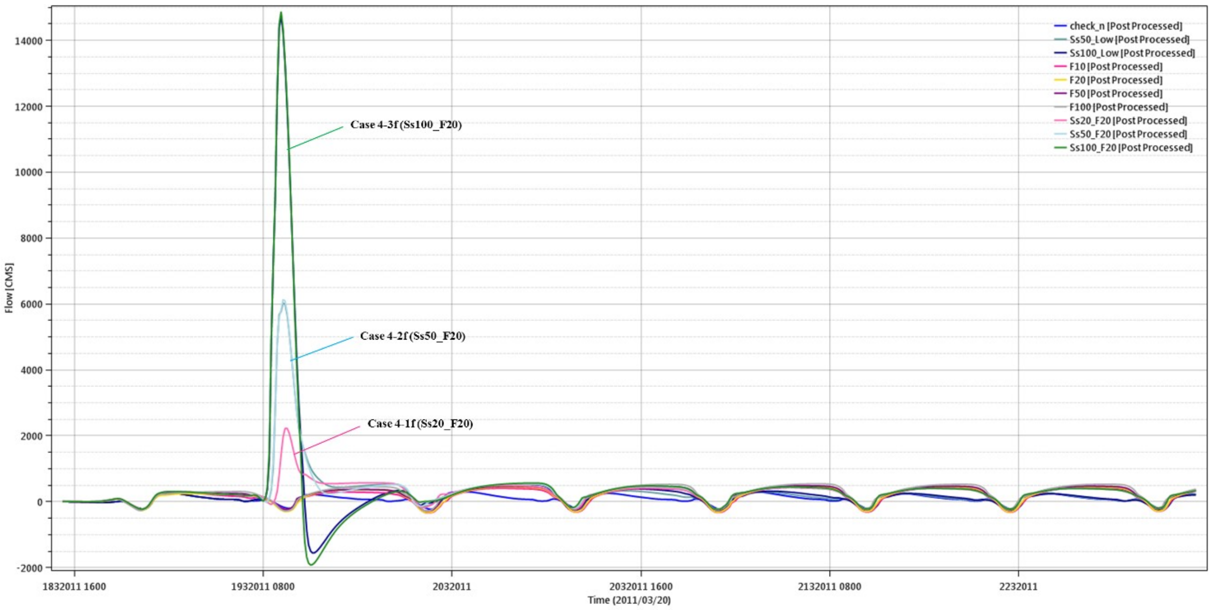


Source: JICA Survey Team

Figure 3.4-75 Maximum Water Surface Elevation Distribution with / without Road Embankment (Case 4-3, 100-years Storm Surge + 20-years Flood)

Hydraulic Quantities and Slope Protections along the Access Road Embankment

The hydraulic quantities (water level, flow rate, flow velocity, shear stress) at the main embankment openings (bridge sections) of the access road are calculated. As an example, Figure 3.4-76 shows the change in flow rate for each case at the opening portion at the proposed bridge of the Maheshkhali Channel. Similarly, the hydraulic quantities at the other bridge opening portions are calculated, and the maximum values are used as the input conditions for one-dimensional analyses. The waterway and river networks and flood plain in this study area generate highly complicated hydrological phenomena in which unpredictable flows and river split- and merging-flows interlaced to the flood plain, so the results of 2-dimensional hydraulic analysis were used.



Source: JICA Survey Team

Figure 3.4-76 Discharge Variation of Proposed Bridge Opening Portion at Maheshkhali Channel (with Road Embankment)

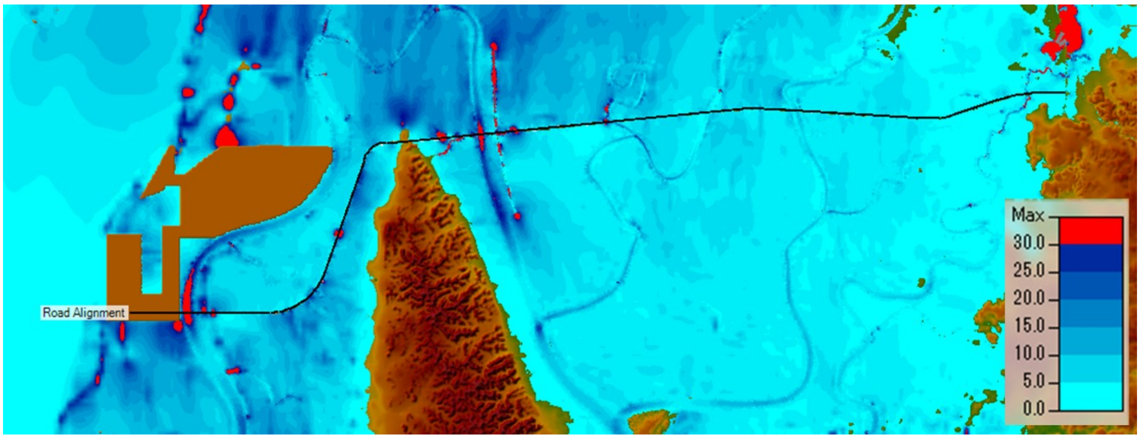
The necessity of the slope protection on the road alignment is judged by many indicators, such as permissible flow velocity and shear stress, but this time, the permissible shear stress is used as an indicator. The road embankment slope will have protection with vegetation and need extra protection for the range of 30 N/m^2 or more (a list of permissible shear stress generally used is shown in Table 3.4-68 and the maximum shear stress distribution at 50-years probability is shown in Figure 3.4-77). For reference, Figure 3.4-78 shows the maximum flow velocity distribution map at the 50-years probability with the threshold value of 1.8 m/sec, which is the permissible flow velocity value used as an indicator of the necessity of slope protection in Japan.

Figure 3.4-77 shows the simulated maximum shear stress distribution and the red color area will need special protection. The most sections that need such protection occur at the bridge opening portions. The riverbeds of Kohelia River, Maheshkhali Channel and Mangla River were found to be eroded easily, and it is desirable to protect such riverbeds. Locations that require the slope protection beyond permissible shear stress 30 N/m^2 are identified as shown in Table 3.4-69 (for reference, locations beyond permissible velocity 1.8 m/s are also shown in Table 3.4-70).

Table 3.4-68 Permissible Shear Stress of Fill Material (N/m^2)

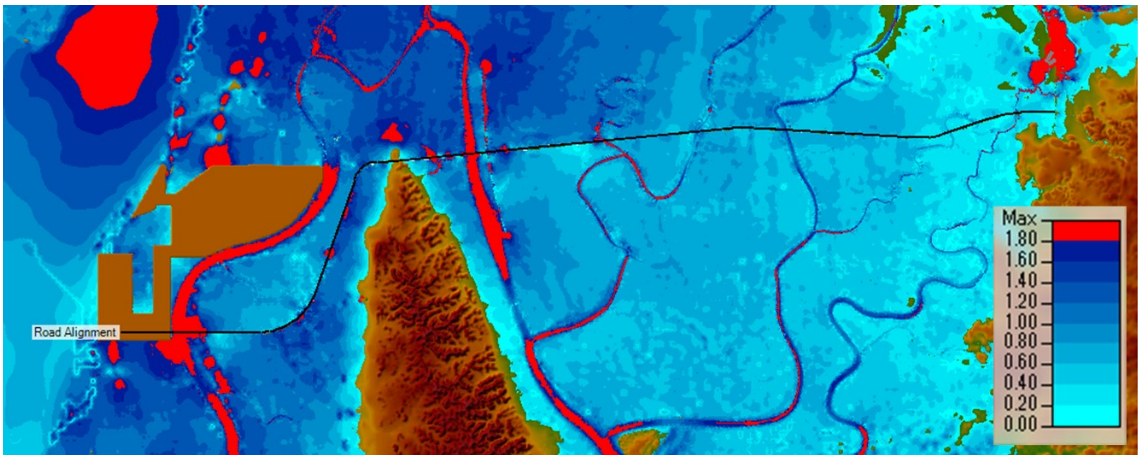
Description	Allowable shear stress [N/m2] **Without Vegetation**	Allowable shear stress [N/m2] **With Vegetation**
Fine sand	3.5	-
Sand and gravel	15.3	-
Coarse gravel	32	-
Cobbles and shingles	52.6	-
Stiff Clay (cohesive)	22	-
Shales (cohesive)	32	-
Silts w/cobbles (cohesive)	38	-
Grass mats	(no grass) 10	30
Cutting shrubs	10	60
Brush mats w/willow	50	300
Riparian wattles	10	50
Willow protections	20	100
GabionMats 0.30m	336	450
Gabions 0.50m	470.4	500
Gabions 1.00m	500	500
Riprap/Rock wall	300.8	350
Articulated blocks	250	350

Source: General Value of Software "Macra1" by Maccaferri Ltd.



Source: JICA Survey Team

Figure 3.4-77 Maximum Shear Stress Distribution (50-year Storm Surge + 20-year Flood)



Source: JICA Survey Team

Figure 3.4-78 Maximum Flow Velocity Distribution (50-year Storm Surge + 20-year Flood)

Table 3.4-69 Locations Requiring Slope Protection (Shear Stress > 30 N/m²)

Bridge ID.		Station No. from	Station No. to	Slope Protection Length (m)
1	(Bridge Opening)	1+ 307	2+ 039	(Bed Protection)
	Embankment	2+ 039	2+ 052	13.0
3	Embankment	6+ 053	6+ 081	29.0
	(Bridge Opening)	6+ 081	6+ 204	-
4	Embankment	10+ 039	10+ 091	52.0
	(Bridge Opening)	10+ 091	10+ 249	-
5	Embankment	10+ 478	10+ 491	13.0
	(Bridge Opening)	10+ 491	11+ 961	(Bed Protection)
6	Embankment	14+ 071	14+ 091	21.0
	(Bridge Opening)	14+ 091	14+ 204	-
15	(Bridge Opening)	24+ 456	24+ 494	-
	Embankment	24+ 494	24+ 496	2.0
Total				130.0

Source: JICA Survey Team

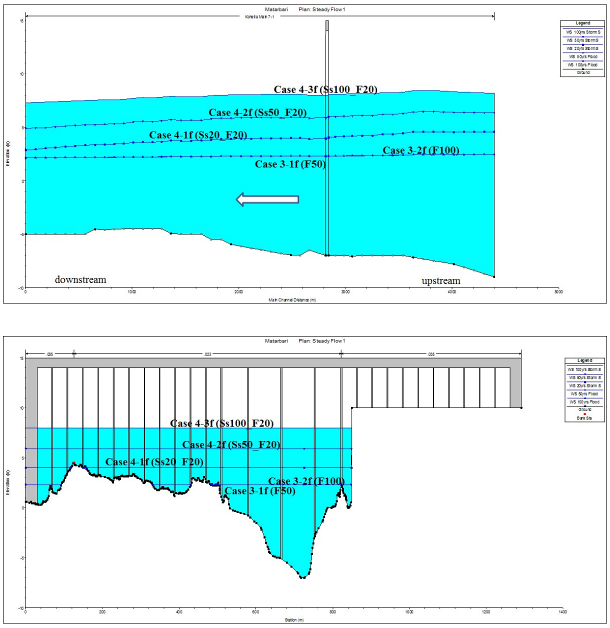
Table 3.4-70 Locations Requiring Slope Protection (Velocity > 1.8m/s)

Bridge ID.		Station No. from	Station No. to	Slope Protection Length (m)
1	(Bridge Opening)	1+ 252	2+ 039	(Bed Protection)
	Embankment	2+ 039	2+ 072	34.0
2	Embankment	4+ 429	4+ 431	2.0
	(Bridge Opening)	4+ 431	4+ 651	-
3	Embankment	6+ 048	6+ 081	33.0
	(Bridge Opening)	6+ 081	6+ 206	-
4	Embankment	10+ 021	10+ 091	70.0
	(Bridge Opening)	10+ 091	10+ 249	-
	Embankment	10+ 249	10+ 254	5.0
5	Embankment	10+ 470	10+ 491	22.0
	(Bridge Opening)	10+ 491	11+ 963	(Bed Protection)
	Embankment	11+ 963	11+ 968	6.0
6	Embankment	14+ 063	14+ 091	28.0
	(Bridge Opening)	14+ 091	14+ 308	-
15	(Bridge Opening)	24+ 458	24+ 494	-
Total				200.0

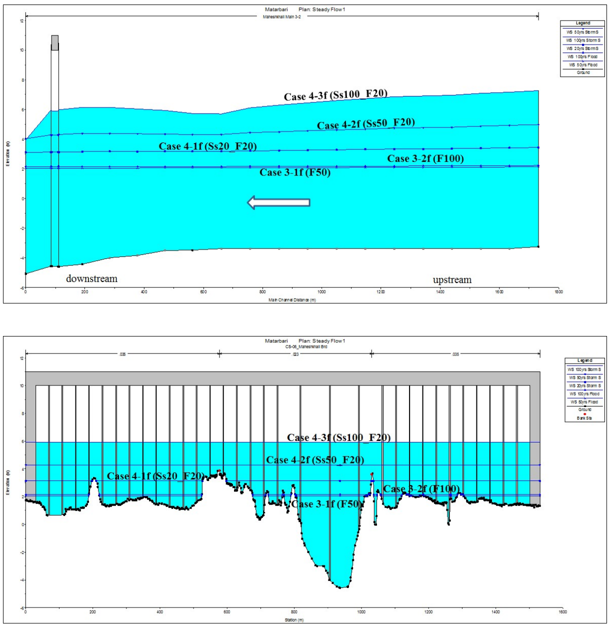
Source: JICA Survey Team

6) 1-Dimensional Flood Analysis Results (Calculation Case 5)

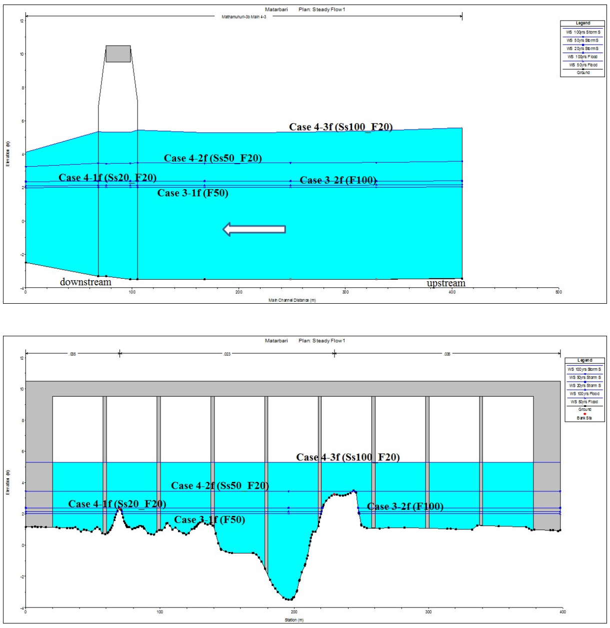
Using hydraulic quantities at the main embankment opening portions (at bridges) of 2-dimensional analyses, the 1-dimensional flood analysis is performed to calculate the hydraulic quantities around bridges. The hydraulic profiles and cross-sections at each bridge position are shown in Figure 3.4-79 and Figure 3.4-80.



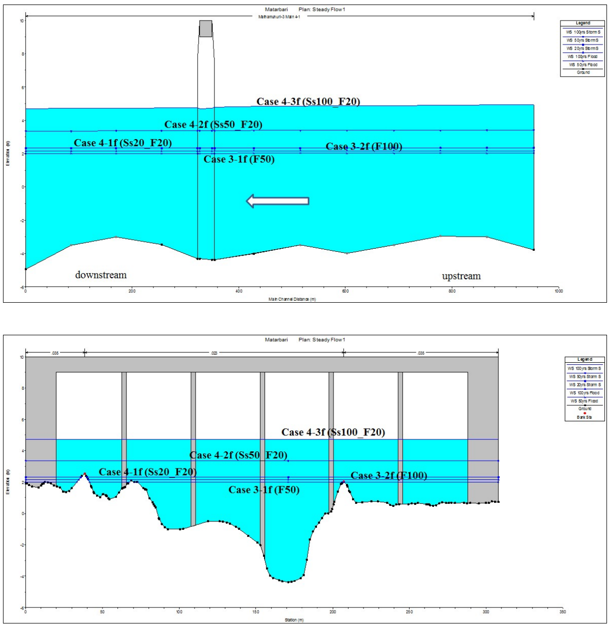
Kohelia River Bridge (No.1+423)



Maheshkhali Channel Bridge (No.11+227)



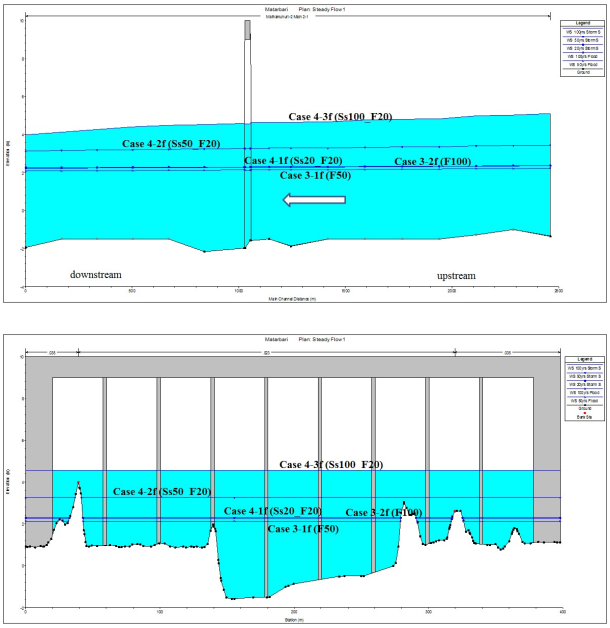
Old Matamuhuri River Tributary Bridge (No.14+270)



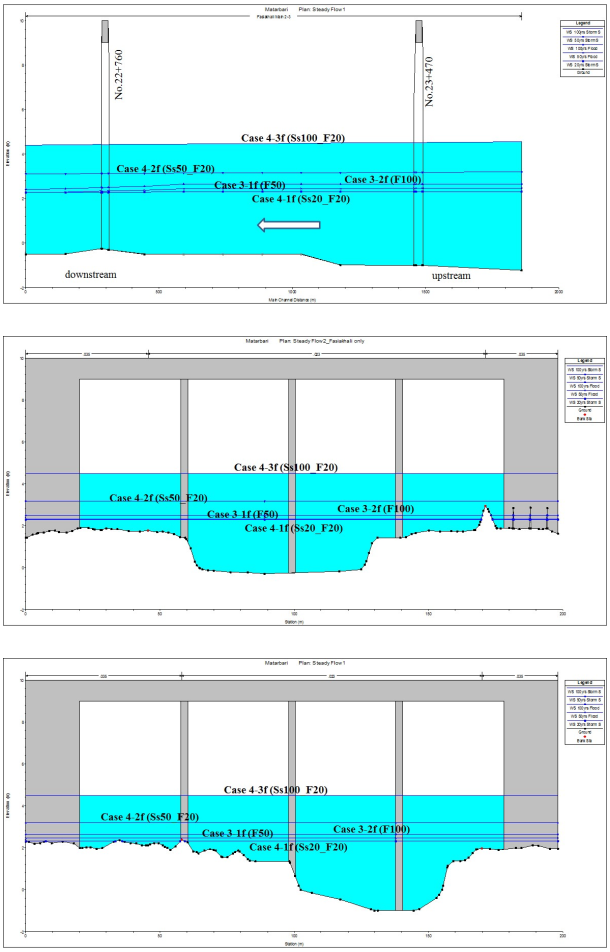
Old Matamuhuri River Bridge (No.16+625)

Source: JICA Survey Team

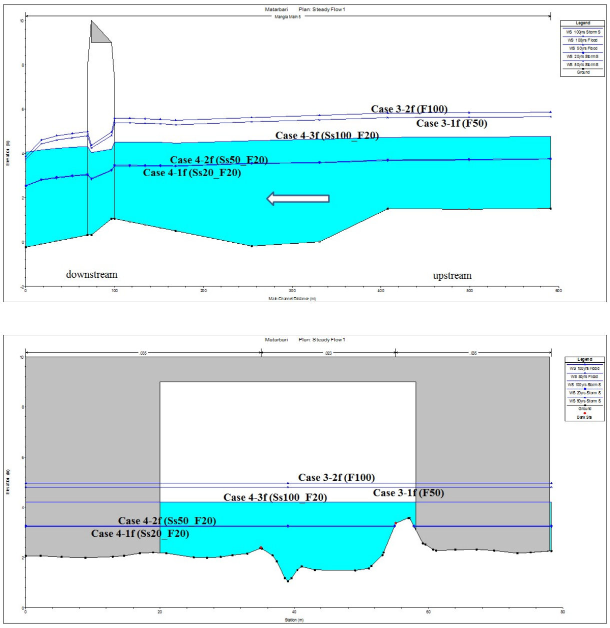
Figure 3.4-79 Hydraulic Profile and Cross-Section (1)



Matamuhuri River Bridge (No.18+730)



Fasiakhali Khal Bridge (No.22+760, No.23+470)



Mangla River Bridge (No.24+475)

Source: JICA Survey Team

Figure 3.4-80 Hydraulic Profile and Cross-Section (2)

(5) Scour Estimation and Protection Method

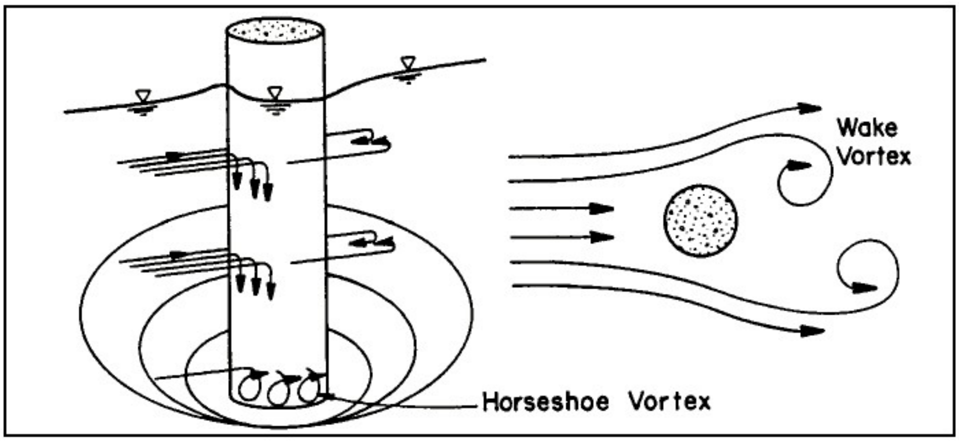
Scour Estimation

Using the 1-dimensional flood analyses results, the scour depth around the bridge is estimated. The calculation cases are 5 cases; Flood cases of Case 3-1 (50-years) and Case 3-2 (100-years) and Storm-surge cases of Case 4-1 (20-years), 4-2 (50-years) and 4-3 (100-years). Table 3.4-72 to Table 3.4-79 show the results of estimated scour depth at the piers and abutments of each bridge.

As mentioned earlier, the scour at the bridges were evaluated by the design scale of 50-years return period. In this study, both events of the storm surge and flood are considered for protection of the main channel. And the flood event only is considered for protection of the right and left banks, since the visual inspection is possible after disaster. In Table 3.4-72 to Table 3.4-79, either 50-yeras cases of storm-surge or flood, whichever is greater, was applied for the design total scour depth of the main channel and the 50-yeras case of flood was applied for the river banks. The maximum total scour depth represents the maximum value among total scour depth of each return-period case for reference.

Although the footing depths of bridge substructures are often installed deeper than the estimated scour depth, the deeper footing is more expensive and construction is more difficult. Full scale bed protections at whole riverbed would be required for the bridges across major rivers (Kohelia River, Maheshkhali Channel and Mangla River) but only partial bed protection around piers would be enough for the other bridges where the pier footings are under low-water-level.

As shown in Figure 3.4-81, the scouring phenomenon around bridge-piers is caused by the 3-dimensional flow accompanied by a horseshoe vortex and a wake vortex and such complex 3-dimensional water behavior cannot be elucidated, without performing 3-dimensional analysis. However, even if its behavior can be elucidated, the calculation method of the scour-depth has not been systematically established. Considering that the 1-dimensional analysis of US standards is used worldwide, the scour depth was estimated by the 1-dimensional analysis under this study. Table 3.4-71 shows the dimension-based comparison of hydraulic models.



Source: Evaluating Scour at Bridges (2012 Fifth edition), Hydraulic Engineering Circular No. 18 (HEC 18), FHWA, USA

Figure 3.4-81 Simple Schematic Representation of Scour at a Cylindrical Pier

Table 3.4-71 Dimension-based Comparison of Hydraulic Models for Scour Estimation

Hydraulic Model			Reproducibility against Special Eddies around Pier on Hydraulic Analysis		Applicability and Reliability for the Scour Depth Estimation in view of the Change in Hydraulic Quantities / Internationally Authorized Standard	Evaluation / Internationally Known Software
Hydraulic Dimension	Riverbed Geometry	Bridge Geometry	Wake Vortex	Horseshoe Vortex		
1d-model	Three or more 2d-cross sections	Pier plane shape / width / length / height	Impossible	Impossible	Many equations based on many experiment and observation results / Many standards (HEC-series, SETRA, etc.)	Good for this study / HEC-RAS (USACE), MIKE11 (DHI)
2d-model	3d-riverbed geometry	Ditto	Possible	Impossible	Still studying / No standard*	Good / Still developing
3d-model	3d-riverbed geometry	Ditto	Possible	May be possible	Still studying / No standard	Still developing / Still developing

Note. *For 2d-model, 1d ideas to the scour depth estimation are applicable, but the effect by wake vortex by 2d-model is considered.
Source: JICA Survey Team

Table 3.4-72 Estimated Scour Depth at Kohelia River (No.1+423)

Kohelia Bridge (1+423)																				
Id	HEC id	Location	Ss. 100yrs			Ss. 50yrs			Ss. 20yrs			F. 100yrs			F. 50yrs			Max. Total Scour Depth (m)	Design Total Scour Depth (m)	Necessary Bed Protection Length from Footing Edge (m)
			Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour			
A2	A1			(17.80)	(0.00)		(13.19)	(13.19)		(8.56)	(8.56)		(3.41)	(3.43)		(3.44)	(3.44)	(13.19)	(3.44)	Slope Protection
P26	P1		0.00	3.89	3.89	0.00	3.53	3.53	0.00	2.93	2.93	0.02	1.49	1.50	0.01	1.50	1.50	3.89	1.50	3.01
P25	P2			3.72	3.72		3.28	3.28		2.41	2.41		-	0.00		-	0.00	3.72	0.00	0.00
P24	P3			4.70	5.12		4.16	4.55		2.30	2.60		-	0.00		-	0.00	5.12	0.00	0.00
P23	P4			4.82	5.23		4.35	4.74		3.45	3.75		-	0.00		-	0.00	5.23	0.00	0.00
P22	P5	Left Bank		4.89	5.31		4.47	4.86		3.71	4.02		-	0.00		-	0.00	5.31	0.00	0.00
P21	P6			4.82	5.24		4.36	4.76		3.48	3.79		-	0.00		-	0.00	5.24	0.00	0.00
P20	P7			4.91	5.32		4.49	4.88		3.76	4.06		-	0.00		-	0.00	5.32	0.00	0.00
P19	P8			4.98	5.40		4.59	4.99		3.93	4.24		2.29	2.40		2.30	2.41	5.40	2.41	4.83
P18	P9		0.41	5.04	5.45	0.39	4.67	5.07	0.30	4.06	4.36	0.11	2.56	2.67	0.11	2.57	2.68	5.45	2.68	5.38
P17	P10			4.95	5.36		4.55	4.94		3.86	4.17		2.01	2.12		2.03	2.14	5.36	2.14	4.29
P16	P11			4.88	5.29		4.45	4.84		3.67	3.98		-	0.00		-	0.00	5.29	0.00	0.00
P15	P12	Main Channel		6.28	6.70		5.83	**6.22**		5.07	5.38		3.22	3.33		3.23	3.34	6.70	**6.22**	12.48
P14	P13			6.50	6.91		6.11	**6.50**		5.44	5.75		3.69	3.80		3.71	3.82	6.91	**6.50**	13.04
P13	P14			6.86	7.28		6.53	**6.93**		5.94	6.24		4.16	4.27		4.18	4.29	7.28	**6.93**	13.90
P12	P15			6.70	7.11		6.34	**6.74**		5.72	6.03		3.97	4.08		3.99	4.10	7.11	**6.74**	13.52
P11	P16			4.69	4.69		4.34	4.34		3.63	3.63		1.45	1.45		1.46	1.46	4.69	1.46	2.93
P10	P17			-	-		-	-		-	-		-	-		-	-	0.00	0.00	0.00
P9	P18			-	-		-	-		-	-		-	-		-	-	0.00	0.00	0.00
P8	P19			-	-		-	-		-	-		-	-		-	-	0.00	0.00	0.00
P7	P20			-	-		-	-		-	-		-	-		-	-	0.00	0.00	0.00
P6	P21	Right Bank	0.00	-	-	0.00	-	-	0.00	-	-	0.00	-	-	0.00	-	-	0.00	0.00	0.00
P5	P22			-	-		-	-		-	-		-	-		-	-	0.00	0.00	0.00
P4	P23			-	-		-	-		-	-		-	-		-	-	0.00	0.00	0.00
P3	P24			-	-		-	-		-	-		-	-		-	-	0.00	0.00	0.00
P2	P25			-	-		-	-		-	-		-	-		-	-	0.00	0.00	0.00
P1	P26			-	-		-	-		-	-		-	-		-	-	0.00	0.00	0.00
A1	A2			-	-		-	-		-	-		-	-		-	-	0.00	0.00	Slope Protection
		Max	0.41	6.86	7.28	0.39	6.53	6.93	0.30	5.94	6.24	0.11	4.16	4.27	0.11	4.18	4.29	13.19	6.93	13.90

Note. "Ss" means the calculation case which is occurred "Storm Surge for each return period" and "20 years return period flood" at the same moment. "F" means the calculation case which is occurred "Flood only" for each return period.

Source: JICA Survey Team

Table 3.4-73 Estimated Scour Depth at Maheshkhali Channel Bridge (No.11+227)

Maheshkhali Bridge (11+227)																				
Id	HEC id	Location	Ss. 100yrs			Ss. 50yrs			Ss. 20yrs			F. 100yrs			F. 50yrs			Max. Total Scour Depth (m)	Design Total Scour Depth (m)	Necessary Bed Protection Length from Footing Edge (m)
			Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour			
A2	A1	Left Bank	1.79	(10.92)	(12.71)	1.16	(6.11)	(7.26)	0.78	(3.08)	(3.86)	0.00	(0.84)	(0.84)	0.00	(0.71)	(0.71)	(12.71)	(0.71)	Slope Protection
P32	P1			3.67	5.46		2.77	3.92		1.98	2.76		1.03	1.03		0.97	0.97	5.46	0.97	1.95
P31	P2			3.67	5.46		2.77	3.92		1.98	2.76		1.03	1.03		0.97	0.97	5.46	0.97	1.95
P30	P3			3.63	5.42		2.72	3.88		1.93	2.71		0.99	0.99		0.92	0.92	5.42	0.92	1.85
P29	P4			3.50	5.29		2.57	3.72		1.73	2.51		-	0.00		-	0.00	5.29	0.00	0.00
P28	P5			3.60	5.38		2.68	3.84		1.88	2.66		0.94	0.94		0.87	0.87	5.38	0.87	1.74
P27	P6			3.59	5.38		2.68	3.83		1.88	2.66		0.93	0.93		0.87	0.87	5.38	0.87	1.74
P26	P7			3.56	5.35		2.64	3.80		1.83	2.61		0.87	0.87		0.80	0.80	5.35	0.80	1.60
P25	P8			3.54	5.32		2.61	3.77		1.80	2.58		0.79	0.79		0.70	0.70	5.32	0.70	1.40
P24	P9			3.59	5.38		2.68	3.83		1.88	2.66		0.93	0.93		0.87	0.87	5.38	0.87	1.74
P23	P10			3.61	5.40		2.70	3.86		1.91	2.69		0.97	0.97		0.90	0.90	5.40	0.90	1.81
P22	P11			3.63	5.41		2.72	3.88		1.93	2.71		0.99	0.99		0.92	0.92	5.41	0.92	1.85
P21	P12			3.58	5.36		2.66	3.81		1.85	2.63		0.90	0.90		0.83	0.83	5.36	0.83	1.66
P20	P13			3.34	5.13		2.31	3.47		-	0.00		-	0.00		-	0.00	5.13	0.00	0.00
P19	P14	Main Channel	1.84	4.33	6.18	1.16	3.07	**4.23**	0.69	-	0.00	0.10	-	0.00	0.02	-	0.00	6.18	**4.23**	8.48
P18	P15			4.55	6.40		3.46	**4.61**		2.39	3.08		-	0.00		-	0.00	6.40	**4.61**	9.25
P17	P16			4.55	6.40		3.46	**4.61**		2.38	3.08		-	0.00		-	0.00	6.40	**4.61**	9.25
P16	P17			4.79	6.64		3.77	**4.92**		2.82	3.51		1.81	1.91		1.75	1.75	6.64	**4.92**	9.87
P15	P18			5.87	7.71		4.57	**5.72**		3.37	4.06		2.08	2.17		1.99	1.99	7.71	**5.72**	11.47
P14	P19			6.06	7.90		4.79	**5.95**		3.62	4.31		2.37	2.47		2.30	2.30	7.90	**5.95**	11.93
P13	P20			6.52	8.37		5.27	**6.43**		4.08	4.77		2.77	2.87		2.70	2.70	8.37	**6.43**	12.90
P12	P21			6.13	7.97		4.87	**6.02**		3.70	4.39		2.45	2.55		2.38	2.38	7.97	**6.02**	12.07
P11	P22	Right Bank	0.59	4.44	5.02	0.27	3.30	3.57	0.06	2.21	2.27	0.00	1.05	1.05	0.00	0.97	0.97	5.02	0.97	1.95
P10	P23			3.61	4.19		2.70	2.97		1.83	1.89		0.91	0.91		0.85	0.85	4.19	0.85	1.70
P9	P24			3.52	4.10		2.59	2.86		1.70	1.76		0.66	0.66		-	0.00	4.10	0.00	0.00
P8	P25			3.51	4.09		2.57	2.85		1.68	1.74		-	0.49		-	0.00	4.09	0.00	0.00
P7	P26			3.57	4.16		2.66	2.93		1.79	1.85		0.86	0.86		0.79	0.79	4.16	0.79	1.58
P6	P27			3.63	4.21		2.72	2.99		1.86	1.91		0.93	0.93		0.87	0.87	4.21	0.87	1.74
P5	P28			3.51	4.09		2.58	2.85		1.69	1.75		-	0.57		-	0.00	4.09	0.00	0.00
P4	P29			3.57	4.15		2.65	2.92		1.78	1.84		0.84	0.84		0.78	0.78	4.15	0.78	1.56
P3	P30			3.57	4.15		2.65	2.93		1.78	1.84		0.85	0.85		0.78	0.78	4.15	0.78	1.56
P2	P31			3.59	4.17		2.67	2.95		1.81	1.87		0.88	0.88		0.82	0.82	4.17	0.82	1.64
P1	P32			3.58	4.17		2.67	2.94		1.80	1.86		0.87	0.87		0.81	0.81	4.17	0.81	1.62
A1	A2			(11.29)	(11.87)		(6.44)	(6.71)		(3.29)	(3.35)		(1.01)	(1.01)		(0.88)	(0.88)	(11.87)	(0.88)	Slope Protection
		Max	1.84	6.52	8.37	1.16	5.27	6.43	0.78	4.08	4.77	0.10	2.77	2.87	0.02	2.70	2.70	8.37	6.43	12.90

Note. "Ss" means the calculation case which is occurred "Storm Surge for each return period" and "20 years return period flood" at the same moment. "F" means the calculation case which is occurred "Flood only" for each return period.

Source: JICA Survey Team

Table 3.4-74 Estimated Scour Depth at old Matamuhuri River Tributary Bridge (No.14+270)

Tributary of Bara-Mathamuhuri Bridge (14+270)																				
Id	HEC id	Location	Ss. 100yrs			Ss. 50yrs			Ss. 20yrs			F. 100yrs			F. 50yrs			Max. Total Scour Depth (m)	Design Total Scour Depth (m)	Necessary Bed Protection Length from Footing Edge (m)
			Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour			
A2	A1	Left Bank	0.00	(10.60)	(10.60)	0.00	(5.20)	(5.20)	0.00	(2.08)	(2.08)	0.00	(1.74)	(1.74)	0.00	(1.52)	(1.52)	(10.60)	(1.52)	Slope Protection
P8	P1			3.58	3.58		2.54	2.54		1.43	1.43		1.42	1.42		1.34	1.34	3.58	1.34	2.69
P7	P2	Main Channel	0.00	4.60	4.60	0.00	3.36	**3.36**	0.13	2.04	2.17	0.19	2.07	2.26	0.23	1.98	2.21	4.60	**3.36**	6.74
P6	P3			4.55	4.55		3.30	**3.30**		1.97	2.10		1.98	2.17		1.89	2.12	4.55	**3.30**	6.62
P5	P4			4.89	4.89		3.70	**3.70**		2.35	2.48		2.42	2.62		2.35	2.58	4.89	**3.70**	7.42
P4	P5			4.49	4.49		3.21	**3.21**		1.85	1.99		1.82	2.01		1.70	1.92	4.49	**3.21**	6.44
P3	P6	Right Bank	5.25	3.52	8.78	3.69	2.44	6.13	0.86	1.39	2.25	0.74	1.36	2.10	0.53	1.28	1.81	8.78	1.81	3.63
P2	P7			3.53	8.78		2.44	6.13		1.40	2.25		1.36	2.11		1.28	1.81	8.78	1.81	3.63
P1	P8			3.50	8.76		2.41	6.10		1.37	2.22		1.33	2.07		1.24	1.77	8.76	1.77	3.55
A1	A2			(4.98)	(10.23)		(2.04)	(5.73)		(0.89)	(1.74)		(0.70)	(1.44)		(0.59)	(1.12)	(10.23)	(1.12)	Slope Protection
		Max	5.25	4.89	8.78	3.69	3.70	6.13	0.86	2.35	2.48	0.74	2.42	2.62	0.53	2.35	2.58	8.78	3.70	7.42

Note. "Ss" means the calculation case which is occurred "Storm Surge for each return period" and "20 years return period flood" at the same moment. "F" means the calculation case which is occurred "Flood only" for each return period.

Source: JICA Survey Team

Table 3.4-75 Estimated Scour Depth at old Matamuhuri River Bridge (No.16+625)

Bara-Mathamuhuri Bridge (16+625)

Id	HEC id	Location	Ss. 100yrs			Ss. 50yrs			Ss. 20yrs			F. 100yrs			F. 50yrs			Max. Total Scour Depth (m)	Design Total Scour Depth (m)	Necessary Bed Protection Length from Footing Edge (m)
			Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour			
A2	A1	Left Bank	0.00	(6.91)	(6.91)	0.00	(2.96)	(2.96)	0.00	(0.89)	(0.89)	0.00	(0.65)	(0.65)	0.00	(0.42)	(0.42)	(6.91)	(0.42)	Slope Protection
P5	P1	Main Channel	0.78	4.58	5.35	0.42	2.98	**3.40**	0.21	2.03	2.24	0.17	2.02	2.19	0.13	1.91	2.04	5.35	**3.40**	6.82
P4	P2			4.95	5.72		3.36	**3.78**		2.49	2.70		2.57	2.74		2.52	2.65	5.72	**3.78**	7.58
P3	P3			5.11	5.89		3.51	**3.93**		2.64	2.85		2.72	2.89		2.68	2.81	5.89	**3.93**	7.88
P2	P4			4.83	5.61		3.25	**3.67**		2.38	2.59		2.44	2.61		2.39	2.52	5.61	**3.67**	7.36
P1	P5	Right Bank	0.97	3.62	4.59	0.47	2.34	2.81	0.20	1.60	1.80	0.14	1.61	1.75	0.10	1.53	1.64	4.59	1.64	3.29
A1	A2			(10.46)	(11.42)		(5.05)	(5.52)		(2.63)	(2.83)		(2.44)	(2.58)		(2.18)	(2.29)	(11.42)	(2.29)	Slope Protection
		Max	0.97	5.11	5.89	0.47	3.51	3.93	0.21	2.64	2.85	0.17	2.72	2.89	0.13	2.68	2.81	5.89	3.93	7.88

Note. "Ss" means the calculation case which is occurred "Storm Surge for each return period" and "20 years return period flood" at the same moment. "F" means the calculation case which is occurred "Flood only" for each return period.

Source: JICA Survey Team

Table 3.4-76 Estimated Scour Depth at Matamuhuri River Bridge (No.18+730)

Mathamuhuri Bridge (18+730)

Id	HEC id	Location	Ss. 100yrs			Ss. 50yrs			Ss. 20yrs			F. 100yrs			F. 50yrs			Max. Total Scour Depth (m)	Design Total Scour Depth (m)	Necessary Bed Protection Length from Footing Edge (m)
			Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour			
A2	A1	Left Bank	0.00	(7.12)	(7.12)	0.00	(3.88)	(3.88)	0.00	(1.26)	(1.26)	0.00	(1.30)	(1.30)	0.00	(1.12)	(1.12)	(7.12)	(1.12)	Slope Protection
P8	P1	Main Channel	0.23	3.81	4.04	0.12	2.56	**2.68**	0.06	1.75	1.81	0.05	1.85	1.90	0.05	1.80	1.84	4.04	**2.68**	5.38
P7	P2			3.79	4.03		2.55	**2.67**		1.73	1.80		1.83	1.88		1.77	1.82	4.03	**2.67**	5.36
P6	P3			3.65	3.89		2.38	**2.50**		1.47	1.54		1.55	1.60		1.42	1.47	3.89	**2.50**	5.01
P5	P4			4.08	4.32		2.83	**2.95**		2.02	2.08		2.13	2.18		2.09	2.14	4.32	**2.95**	5.92
P4	P5			4.00	4.24		2.75	**2.87**		1.95	2.01		2.06	2.11		2.02	2.07	4.24	**2.87**	5.76
P3	P6			3.97	4.20		2.72	**2.84**		1.92	1.98		2.03	2.08		1.98	2.03	4.20	**2.84**	5.70
P2	P7			3.80	4.03		2.55	**2.67**		1.74	1.80		1.83	1.88		1.78	1.83	4.03	**2.67**	5.36
P1	P8	Right Bank	0.29	2.91	3.20	0.15	1.87	2.02	0.08	1.19	1.27	0.08	1.25	1.33	0.05	1.19	1.24	3.20	1.24	2.49
A1	A2			(8.45)	(8.74)		(3.92)	(4.07)		(1.79)	(1.87)		(1.84)	(1.92)		(1.60)	(1.65)	(8.74)	(1.65)	Slope Protection
		Max	0.29	4.08	4.32	0.15	2.83	2.95	0.08	2.02	2.08	0.08	2.13	2.18	0.05	2.09	2.14	4.32	2.95	5.92

Note. "Ss" means the calculation case which is occurred "Storm Surge for each return period" and "20 years return period flood" at the same moment. "F" means the calculation case which is occurred "Flood only" for each return period.

Source: JICA Survey Team

Table 3.4-77 Estimated Scour Depth at Fasiakhali Khal Bridge (No.22+760)

Fasiakhali 2 Bridge (22+760)

Id	HEC id	Location	Ss. 100yrs			Ss. 50yrs			Ss. 20yrs			F. 100yrs			F. 50yrs			Max. Total Scour Depth (m)	Design Total Scour Depth (m)	Necessary Bed Protection Length from Footing Edge (m)
			Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour			
A2	A1	Left Bank	0.66	(3.86)	(4.51)	0.46	(1.66)	(2.12)	0.06	(0.46)	(0.51)	0.42	(1.40)	(1.82)	0.33	(0.97)	(1.31)	(4.51)	(1.31)	Slope Protection
P3	P1	Main Channel	0.18	2.56	2.73	0.05	2.00	**2.06**	0.00	1.71	1.71	0.05	2.59	2.64	0.03	2.38	2.41	2.73	**2.41**	4.83
P2	P2			2.71	2.89		2.19	**2.25**		1.98	1.98		2.94	2.99		2.74	2.77	2.99	**2.77**	5.56
P1	P3			2.56	2.73		2.00	**2.06**		1.71	1.71		2.59	2.64		2.38	2.41	2.73	**2.41**	4.83
A1	A2	Right Bank	0.00	(4.96)	(4.96)	0.00	(1.97)	(1.97)	0.00	(0.65)	(0.65)	0.00	(1.33)	(1.33)	0.00	(0.94)	(0.94)	(4.96)	(0.94)	Slope Protection
		Max	0.66	2.71	2.89	0.46	2.19	2.25	0.06	1.98	1.98	0.42	2.94	2.99	0.33	2.74	2.77	2.99	2.77	5.56

Note. "Ss" means the calculation case which is occurred "Storm Surge for each return period" and "20 years return period flood" at the same moment. "F" means the calculation case which is occurred "Flood only" for each return period.

Source: JICA Survey Team

Table 3.4-78 Estimated Scour Depth at Fasiakhali Khal Bridge (No.23+470)

Fasiakhali 1 Bridge (23+470)

Id	HEC id	Location	Ss. 100yrs			Ss. 50yrs			Ss. 20yrs			F. 100yrs			F. 50yrs			Max. Total Scour Depth (m)	Design Total Scour Depth (m)	Necessary Bed Protection Length from Footing Edge (m)
			Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour			
A1	A1	Left Bank	0.43	(4.44)	(4.88)	0.28	(1.72)	(2.00)	0.00	(0.28)	(0.28)	0.00	(0.65)	(0.65)	0.00	(0.44)	(0.44)	(4.88)	(0.44)	Slope Protection
P1	P1	Main Channel	1.55	2.63	4.18	0.83	1.74	**2.57**	0.10	0.00	0.00	0.00	0.93	0.93	0.20	0.87	1.07	4.18	**2.57**	5.15
P2	P2			2.79	4.34		1.96	**2.79**		1.14	1.24		1.15	1.15		1.19	1.40	4.34	**2.79**	5.60
P3	P3			2.98	4.53		2.15	**2.98**		1.31	1.41		1.28	1.28		1.35	1.55	4.53	**2.98**	5.98
A2	A2	Right Bank	0.58	(4.23)	(4.81)	0.36	(2.11)	(2.48)	0.00	(0.39)	(0.39)	0.00	(0.72)	(0.72)	0.00	(0.56)	(0.56)	(4.81)	(0.56)	Slope Protection
		Max	1.55	2.98	4.53	0.83	2.15	2.98	0.10	1.31	1.41	0.00	1.28	1.28	0.20	1.35	1.55	4.53	2.98	5.98

Note. "Ss" means the calculation case which is occurred "Storm Surge for each return period" and "20 years return period flood" at the same moment. "F" means the calculation case which is occurred "Flood only" for each return period.

Source: JICA Survey Team

Table 3.4-79 Estimated Scour Depth at Mangla River Bridge (No.24+475))

Mangla Bridge (24+475)

Id	HEC id	Location	Ss. 100yrs			Ss. 50yrs			Ss. 20yrs			F. 100yrs			F. 50yrs			Max. Total Scour Depth (m)	Design Total Scour Depth (m)	Necessary Bed Protection Length from Footing Edge (m)
			Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour	Contraction Scour	Local Scour	Total Scour			
A2	A1	Left Bank	0.84	(5.80)	(6.64)	0.38	(2.98)	(3.36)	0.39	(3.91)	(4.30)	1.44	(7.81)	(9.26)	1.34	(7.47)	(8.80)	(9.26)	(8.80)	Slope Protection
		Main Channel	1.62	-	1.62	0.85	-	0.85	0.88	-	0.88	2.56	-	2.56	2.40	-	**2.40**	2.56	**2.40**	4.81
A1	A2	Right Bank	0.00	(4.61)	(4.61)	0.00	(0.98)	(0.98)	0.00	(1.05)	(1.05)	0.00	(6.49)	(6.49)	0.00	(6.15)	(6.15)	(6.49)	(6.15)	Slope Protection
		Max	1.62			0.85			0.88			2.56			2.40			2.56	2.40	

Note. "Ss" means the calculation case which is occurred "Storm Surge for each return period" and "20 years return period flood" at the same moment. "F" means the calculation case which is occurred "Flood only" for each return period.

Source: JICA Survey Team

Method of Riverbed Protection

There are many countermeasures for protecting the riverbed and the surrounding of bridge piers and abutments against scouring phenomenon. The protection works act as a resistant layer to hydraulic shear stresses providing protection to the erodible riverbed and fill material. Revetments and riverbed protection can be classified as either rigid or flexible articulating by material types (riprap, gabion, grouted-riprap, geo-bag, soil-cement, concrete-pavement, interlocking blocks, cable-tied blocks, etc.).

Rigid revetments and bed protection do not have the ability to conform to changes in the supporting surface and these are subject to failure due to undermining. Flexible articulating revetments and bed armoring can conform to changes in the supporting surface and adjust to settlement. Therefore, the flexible materials such as riprap, gabion, geo-bag, etc. are generally desired for the riverbed/ riverbank protections and pier protection, because of the easiness of material procurement and the flexibility to supporting surface.

Due to the low availability of stone materials, the geo-bag and/or concrete cube block have been widely used for the protection works around the river in Bangladesh. Therefore, the combination of geo-bag below low water level (LWL) and the concrete cube blocks above LWL would be the most appropriate riverbed protection for the project road. In order to prepare for future unforeseeable changes of the river channel, monitoring of riverbed protection should be periodically undertaken.

Method of Embankment Slope Protection

The method of embankment slope protection around bridge abutment is basically the same as the above measures. Considering that the ground around road embankment is relatively good, rigid material can be applied for slope protection works around the road embankment. Therefore, concrete cube blocks would be recommended for the slope protection of the embankment around bridge abutments.

(6) Hydrological Assessment

As the result of hydrological and hydraulic analyses, the following findings are obtained.

Hydrological Statistical Analyses, Hydraulic Analyses and Scour Estimation

- Based on the calculation results of the bottom shear-force of the 2-dimensional analysis, values exceeding the permissible shear stress are seen in the vicinity of the bridge openings and the riverbeds of Kohelia River, Maheshkhali Channel and Mangla River. For these three (3) rivers, geobag below LWL and concrete cube blocks above LWL would be recommended for riverbed protection (for Mangla River, increasing of flow-area by extending the bridge length would be also be an alternative measure of riverbed protection). Also, concrete cube blocks would also be recommended for the slope protection around the bridge abutments.
- In addition, there are many existing road embankments (where flooding water would overflow) in the vicinity of the project area and the permissible shear stress at these location would exceed the standard value. Therefore, the protection of existing embankment would also be necessary against the overtopping of flooding water. But such protection works outside of the ROW of the project road would not be included in the scope of the project.
- The scour calculation under the case of storm surge resulted in that the large contraction scouring will occur at the left-bank and main-channel of the Maheshkhali Channel and at the right-bank of the tributary of the old Matamuhuri River. Therefore bed protection works for such riverbed would be recommended. Although scouring may occur even on the ground due to storm surge, protection works would not be necessary because of the easiness of repair. Therefore, the protective works to the floodplain outside of the embankment of the project road is not considered in this study.
- In addition, as a result of local scouring analysis, the scouring occurs in most of the bridge piers in the flood zone of each bridge. For the bridges excluding the two (2) bridges across Kohelia River and Maheshkhali Channel, the deeper foundation or appropriate riverbed protection works should be considered at the river-bed around piers having possibility of local scour.
- Topographic survey was undertaken in the project but the difference in elevation between "BWDB station's PWD datum level" and "Topographic survey datum level (MSL)" was measured at 3 locations relatively close to the distance between BWDB stations and proposed bridges. The official difference between those was 0.46 m, but the difference in measured survey values was 0.69 - 0.95 m. This result suggests that errors may also be included in the other BWDB stations water-level data other than 3 stations. In other words, it should be noted that the calculated probability water-level may also include some errors.

Assessment on Hydrology / Hydraulics

- Regarding flood flows in 2-dimensional analysis, the steady flow by specific discharge for each basin are given under this study, but the hydraulic analysis giving the hydrograph for each watershed in view of rainfall waveform and flood runoff process would be desired.
- Hydraulic calculations including scouring were carried out for only eight (8) bridges. In the detailed design stage, further detailed bridge hydraulic studies should be carried out for all bridges. Especially for two (2) bridges across Kohelia River and Maheshkhali Channel, which are greatly influenced by storm surge, this necessitates further detailed hydraulic investigations and studies to be undertaken for verification of the river bed fluctuation, the valid design runoff and turbulence fields around bridge-piers.
- There are various kinds of river bed protection works and revetment works. This study recommends

the conventional protection works often applied in Bangladesh but further comparative study may need to be carried out in the detailed design stage. In addition, it is necessary to further study the scour estimation with other prediction formulas including the HEC formula, and it is also necessary to take measures for scouring at storm surge.