CHAPTER 8

REHABILITATION AND RETROFITTING OF EXISTING BRIDGES

8. REHABILITATION AND RETROFITTING OF EXISTING BRIDGES

8.1 Background

8.1.1 General

Having examined the survey results described in the previous chapter, it can be said that the deteriorations of Kanchpur Bridge, Meghna Bridge and Gumti Bridge were only a result of normal aging except those of the expansion joints in all the bridges and the hinges in Megna Bridge and Gumti Bridge.

The reasons for the damages on the hinges and expansion joints in Meghna Bridge and Gumti Bridge were described in the last clause of this chapter. It is considered the reasons for the damages on the expansion joints in Kanchpur Bridge were the same as the other two bridges;

- Heavily overloaded trucks,
- Inappropriale maintenance.

8.1.2 Additional Information

RHD, consulting BUET, has requested the Army to execute emergency repair works such as;

for Meghna Bridge

- Protection of P7 or P8 against the river bed scouring,
- Replacement of the expansion joints, 9 nos.,

- Repair of the hinges involving 36 pot bearings,

for Gumti Bridge

- Replacement of the expansion joints, 15 nos.,
- Repair of the hinges involving 60 pot bearings.

The budget for the works planned is considered to be 1.5 billion BDT.

On 19 May 2012, RHD, BUET and the Army held a meeting and it was decided that the works will start after the rainy season has finished, around 3 months later.

8.2 Configuration of Existing Bridges

8.2.1 Existing Kanchpur Bridge

Basic data of existing Kanchpur Bridge are shown in Table 8.2.1.

Table 8.2.1 Basic Data of Existing Kanchpur Bridge

<table>
<tr><td colspan="3">Bridge</td><td>Existing Kanchpur Bridge</td></tr>
<tr><td colspan="3">Consultant</td><td>Anman and Whitny</td></tr>
<tr><td colspan="3">Contractor</td><td>Obayashi Corporation</td></tr>
<tr><td colspan="3">Period of Construction</td><td>September 1973 to September 1977</td></tr>
<tr><td colspan="3">River</td><td>Lakhya River</td></tr>
<tr><td colspan="3">Bridge length</td><td>396.5 m
(4×42.7 m+54.9 m+ 73.2 m+ 54.9 m+ 42.7 m)</td></tr>
<tr><td colspan="3">Bridge width</td><td>14.64 m
(12.81 m(road) + 2×0.686 m(sidewalk) + 2×0.229 (Railing))</td></tr>
<tr><td rowspan="5">Bridge components</td><td colspan="2" rowspan="2">Superstructure</td><td>Pre-stressed concrete I girder, continuous type
(54.9 m+ 73.2 m+ 54.9 m)</td></tr>
<tr><td>Pre-stressed concrete I girder, simply supported type
(42.7 m5 spans)</td></tr>
<tr><td rowspan="3">Substructure</td><td>Abutment</td><td>Inverted T-type</td></tr>
<tr><td>Pier</td><td>Rigid frame-type</td></tr>
<tr><td>Foundation</td><td>Abutment: Spread foundation
Pier: RC open caisson</td></tr>
<tr><td colspan="3">Construction method</td><td>Erection by staging</td></tr>
<tr><td>Loads</td><td colspan="2">Live load</td><td>AASHOTO (HS20-44 ASD)
(available at the time of construction)</td></tr>
<tr><td rowspan="3">Materials</td><td colspan="2">Concrete</td><td>Slab: 21 N/mm^2
Girder: 35 N/mm^2
Abutment and pier: 21 N/mm^2
Caisson: 18 N/mm^2</td></tr>
<tr><td colspan="2">Reinforcing bar</td><td>ASTM A 615, Grade 60
(available at the time of construction)</td></tr>
<tr><td colspan="2">Pre-stressing steel</td><td>SWPR7A 12S12.4 (assumed for restoration design)</td></tr>
</table>

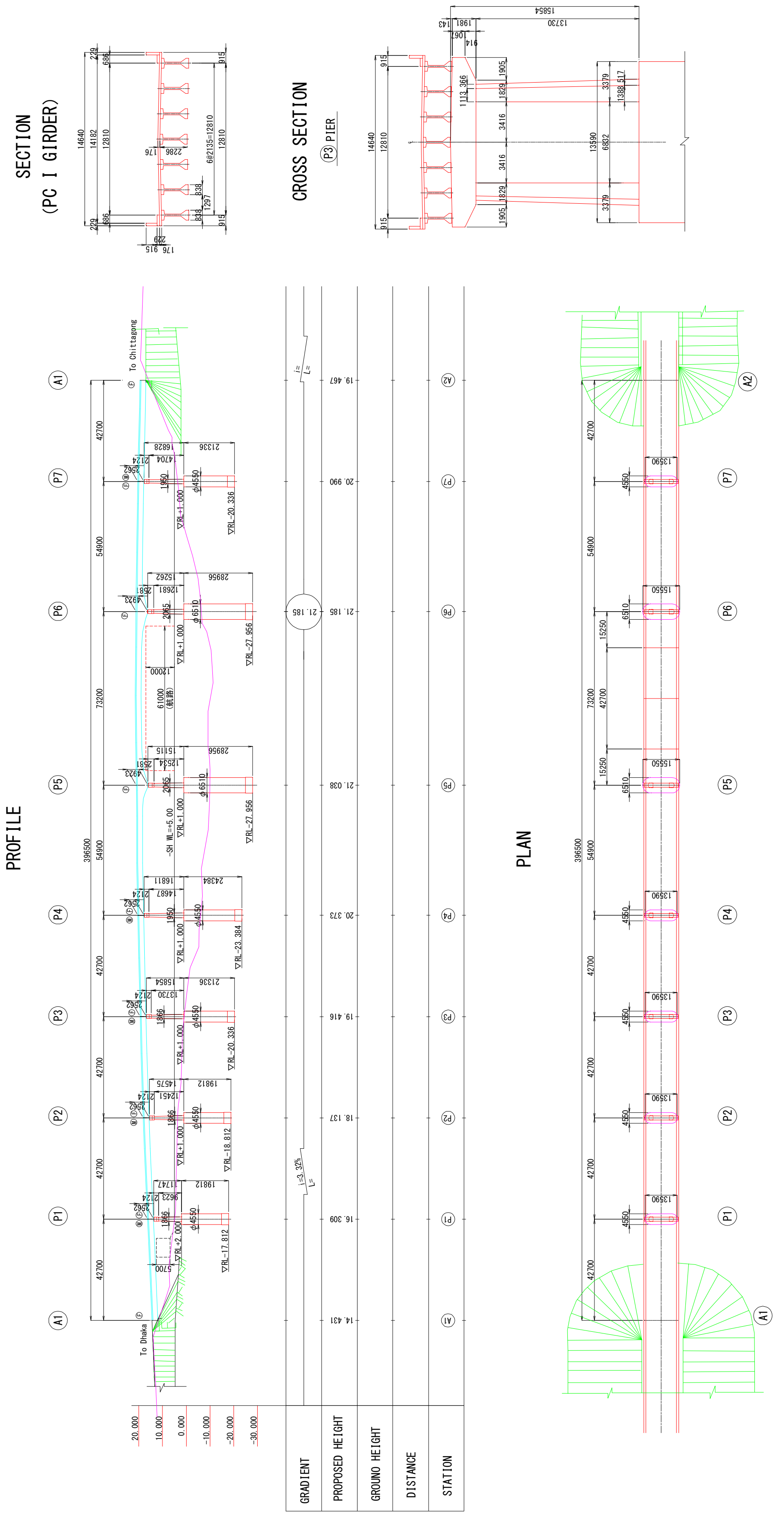


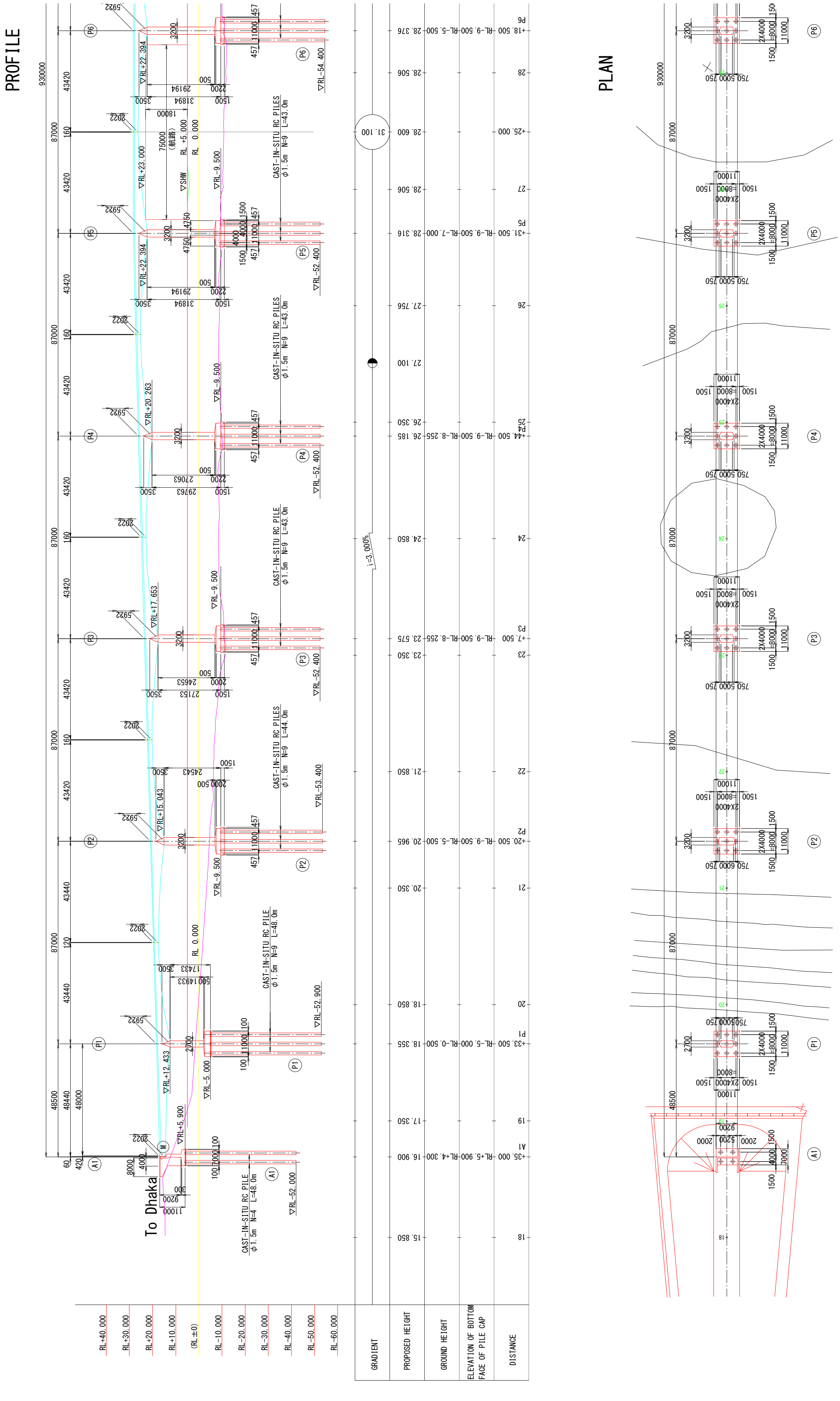
Figure 8.2.1 General View of Existing Kanchpur Bridge

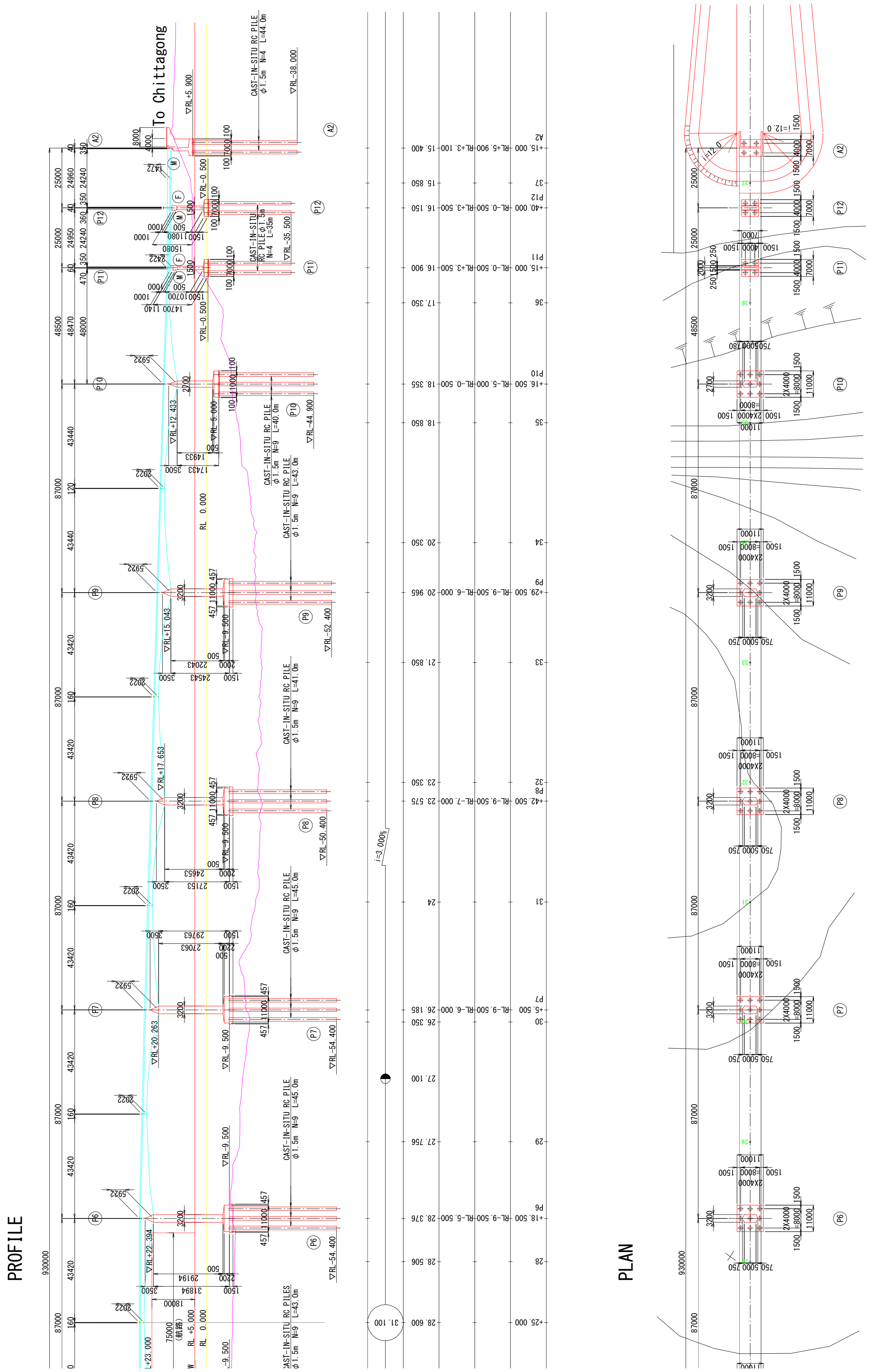
8.2.2 Existing Meghna Bridge

Basic data of existing Meghna Bridge are shown in Table 8.2.2.

Table 8.2.2 Basic Data of Existing Meghna Bridge

<table>
<tr><td colspan="3">Bridge</td><td>Existing Meghna Bridge</td></tr>
<tr><td colspan="3">Consultant</td><td>Pacific Consultants International in consortium
with Nippon Koei Co., Ltd.</td></tr>
<tr><td colspan="3">Contractor</td><td>Obayashi Corporation</td></tr>
<tr><td colspan="3">Period of Construction</td><td>March 1987 to February 1991</td></tr>
<tr><td colspan="3">River</td><td>Meghna River</td></tr>
<tr><td colspan="3">Bridge length</td><td>930 m
(48.5 m+ 9x87.0 m+ 48.5 m+ 2x25.0 m)</td></tr>
<tr><td colspan="3">Bridge width</td><td>9.2 m
(7.2 m(road) + 2×1.0 m(Sidewalk +Railing))</td></tr>
<tr><td rowspan="5">Bridge components</td><td colspan="2" rowspan="2">Superstructure</td><td>Pre-stressed concrete box girder, continuous rigid frame type
(48.5 m+ 9×87.0 m+ 48.5 m)</td></tr>
<tr><td>Pre-stressed concrete T girder, simply supported type
(2×25.0 m)</td></tr>
<tr><td rowspan="3">Substructure</td><td>Abutment</td><td>Inverted T-type</td></tr>
<tr><td>Pier</td><td>Columnar type</td></tr>
<tr><td>Foundation</td><td>Abutment: RC pile (φ1500)
Pier: RC pile (φ1500)</td></tr>
<tr><td colspan="3">Construction method</td><td>Balanced cantilever erection</td></tr>
<tr><td rowspan="4">Loads</td><td colspan="2">Live load</td><td>AASHOTO HS20-44 ASD</td></tr>
<tr><td colspan="2">Seismic load</td><td>Horizontal load coefficient taken as 0.05 g</td></tr>
<tr><td colspan="2">Wind load</td><td>Without live load (V=140 MPH): 0.479 T/m^2
With live load (V=100 MPH): 0.244 T/m^2</td></tr>
<tr><td colspan="2">Thermal force</td><td>Temperature range: 26 °C±17 °C</td></tr>
<tr><td rowspan="3">Materials</td><td colspan="2">Concrete</td><td>Cylinder compressive strength at 28 days
Pier, Abutment, Pile cap: 240 kgf/cm^2
PC box girder, PC- T beam, PC slab: 350 kgf/cm^2
RC pile: 300 kgf/cm^2</td></tr>
<tr><td colspan="2">Reinforcing bar</td><td>Tensile strength: 4,950 kgf/cm^2
Yield stress: 4,250 kgf/cm^2</td></tr>
<tr><td colspan="2">Prestressing steel</td><td>SWPR7A (12T12.4)
SWPR19 (1T19.3)
SWPR19 (1T21.8)
SBPR 95/110 (φ32)</td></tr>
</table>


PROFILE
PLAN
To Dhaka
GRADIENT
PROPOSED HEIGHT
GROUND HEIGHT
ELEVATION OF BOTTOM FACE OF PILE CAP
DISTANCE


PROFILE
PLAN
To Chittagong

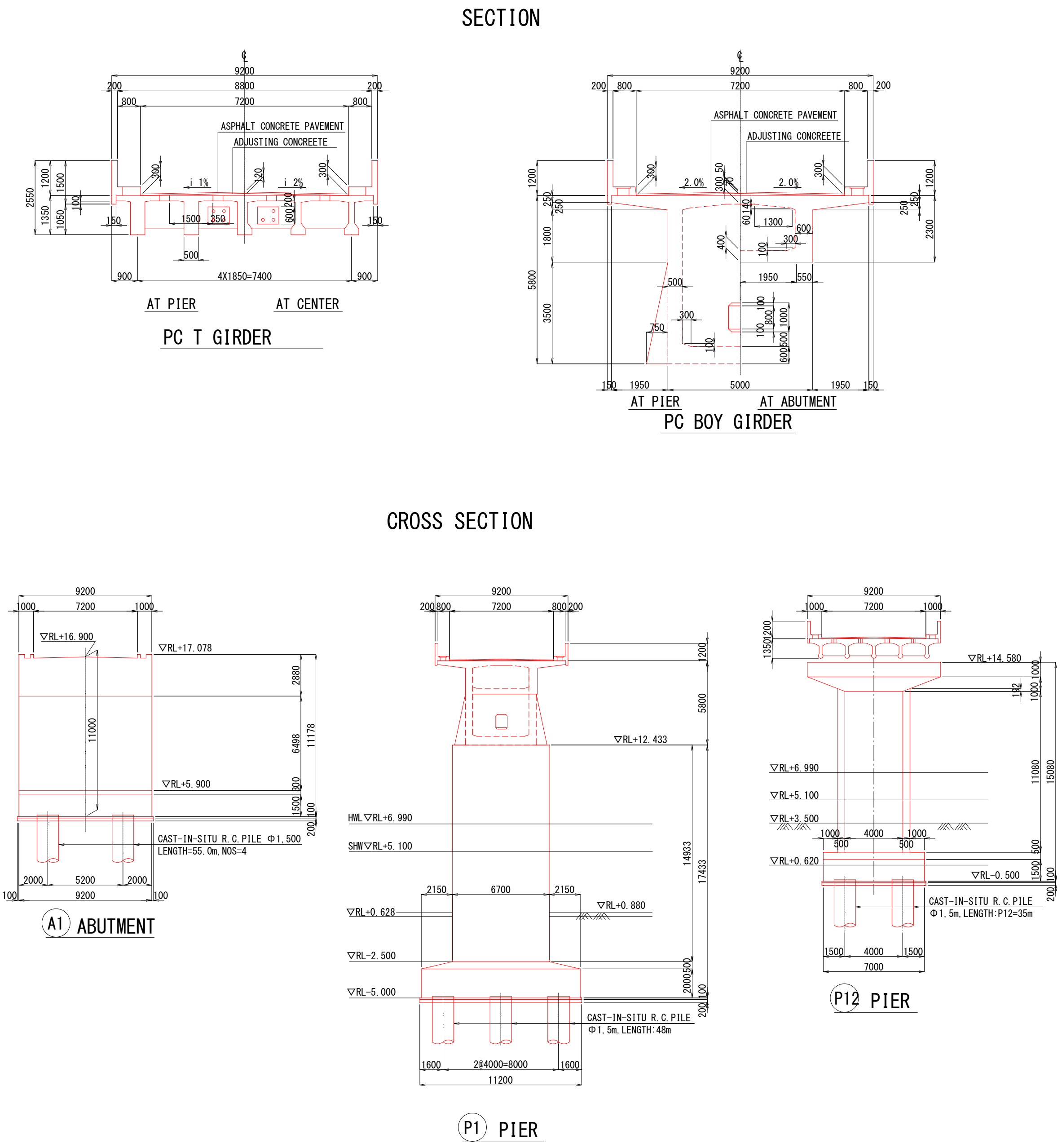


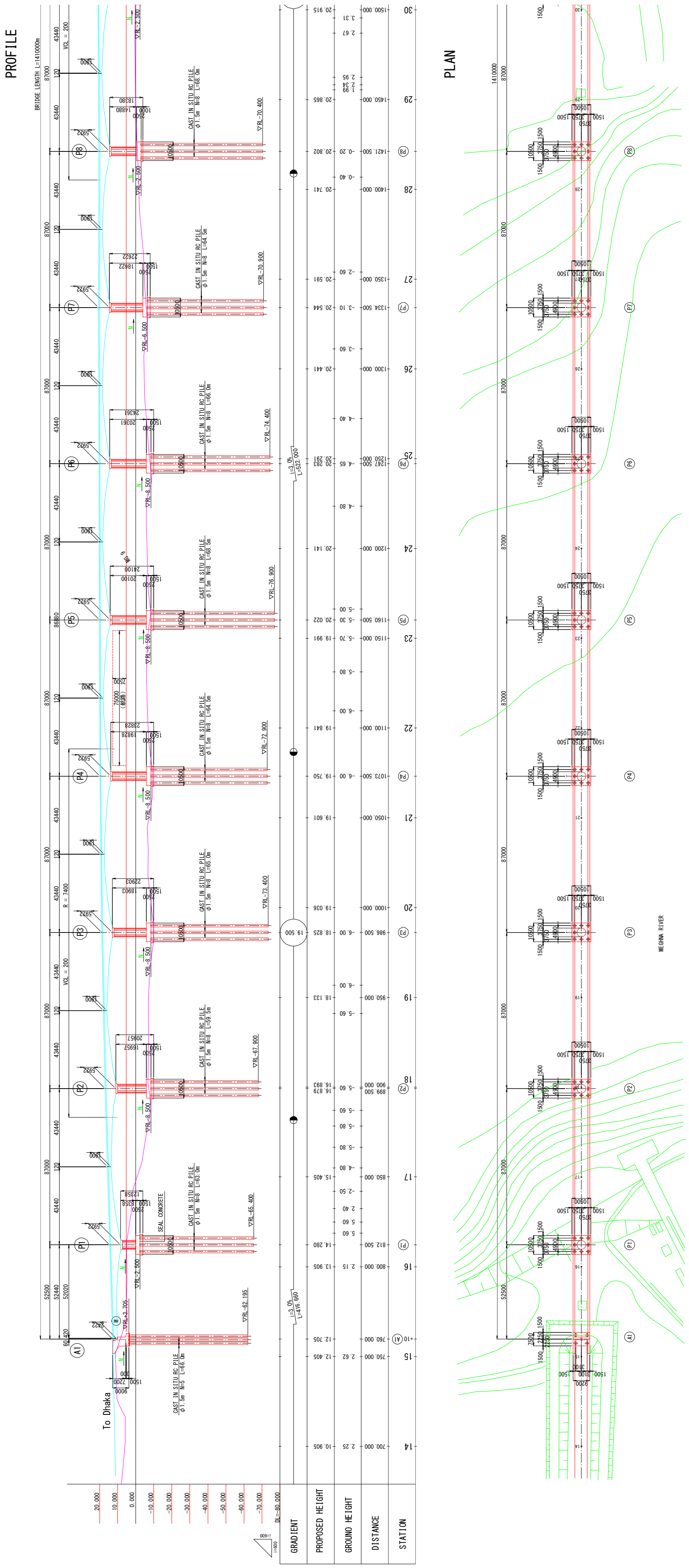
Figure 8.2.2 General View of Existing Meghna Bridge

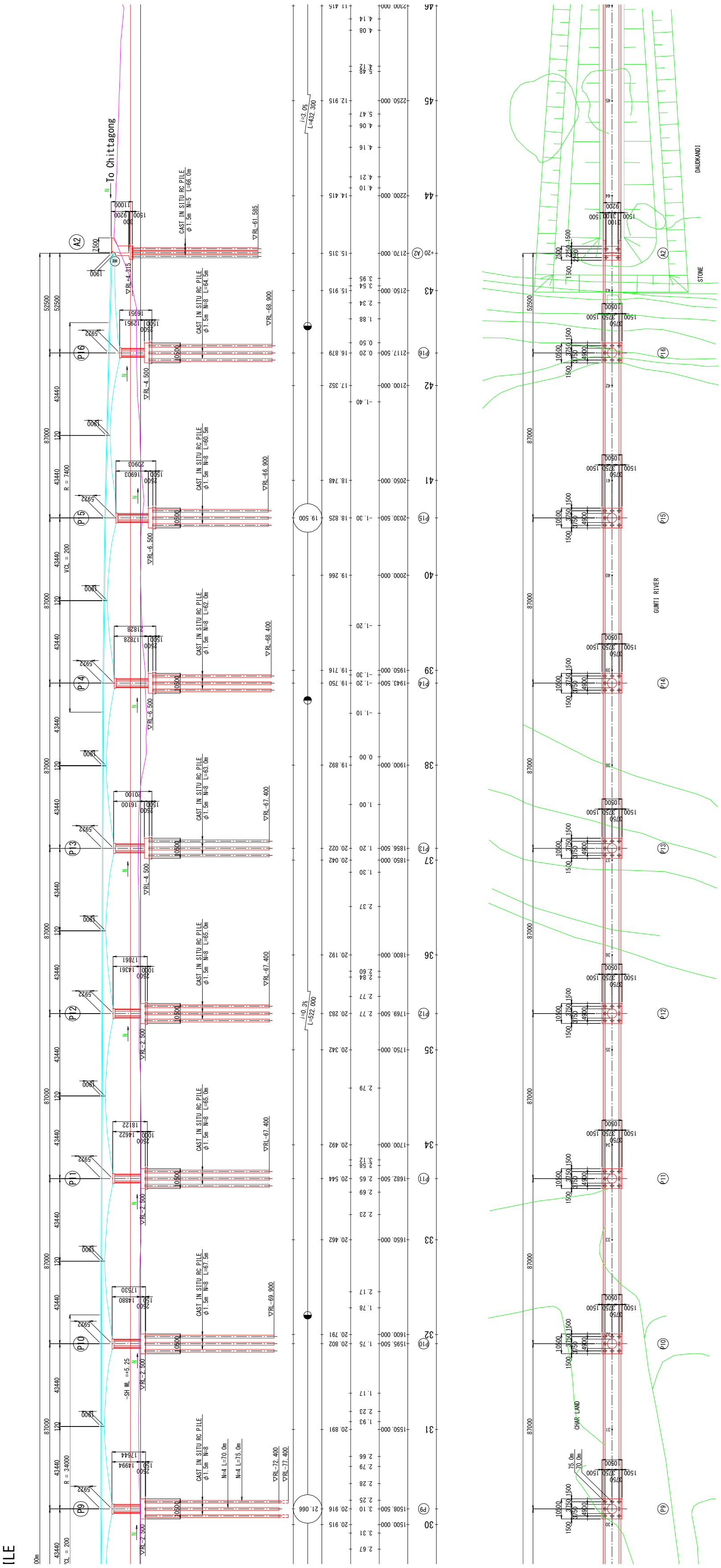
8.2.3 Existing Gumti Bridge

Basic data of existing Gumti Bridge are shown in Table 8.2.3.

Table 8.2.3 Basic Data of Existing Gumti Bridge

Bridge			Existing Gumti Bridge
Consultant			Pacific Consultants International in consortium with Nippon Koei Co., Ltd.
Contractor			Obayashi Corporation
Period of construction			March 1992 to February 1996
River			Meghna River and Gumti River
Overall length			1410 m (52.5 m + 15x87.0 m + 52.5 m)
Overall width			9.2 m (7.2 m (road) + 2×1.0 m (Sidewalk +Railing)
Bridge components	Superstructure		Pre-stressed concrete box girder, continuous rigid frame type
	Substructure	Abutment	Inverted T-type
		Pier	Columnar type
		Foundation	Abutment: RC pile (φ1500) Pier: RC pile (φ1500)
Construction method			Balanced cantilever erection
Loads	Live load		AASHOTO HS20-44 ASD
	Seismic load		Horizontal load coefficient taken as 0.05
	Wind load		Without live load (V=150 MPH): 0.513 T/m^2 With live load (V=100 MPH): 0.244 T/ m^2
	Thermal force		Temperature range: 26 °C±17 °C
Materials	Concrete		Cylinder compressive strength at 28 days Pier, Abutment, Pile cap: 240 kgf/cm^2 PC box girder, PC T beam, PC slab: 350 kgf/cm^2 RC pile: 300kgf/cm^2
	Reinforcing bar		Tensile strength: 4,950 kgf/cm^2 Yield stress: 3,000 kgf/cm^2 Allowable stress: 2,100kgf/cm^2
	Prestressing steel		SWPR7A (12T12.4) SWPR19 (1T19.3) SWPR19 (1T21.8) SBPR 95/110 (φ32).


PROFILE
PLAN
To Dhaka
GRADIENT
PROPOSED HEIGHT
GROUND HEIGHT
DISTANCE
STATION
MEGHNA RIVER


To Chittagong
GUMTI RIVER

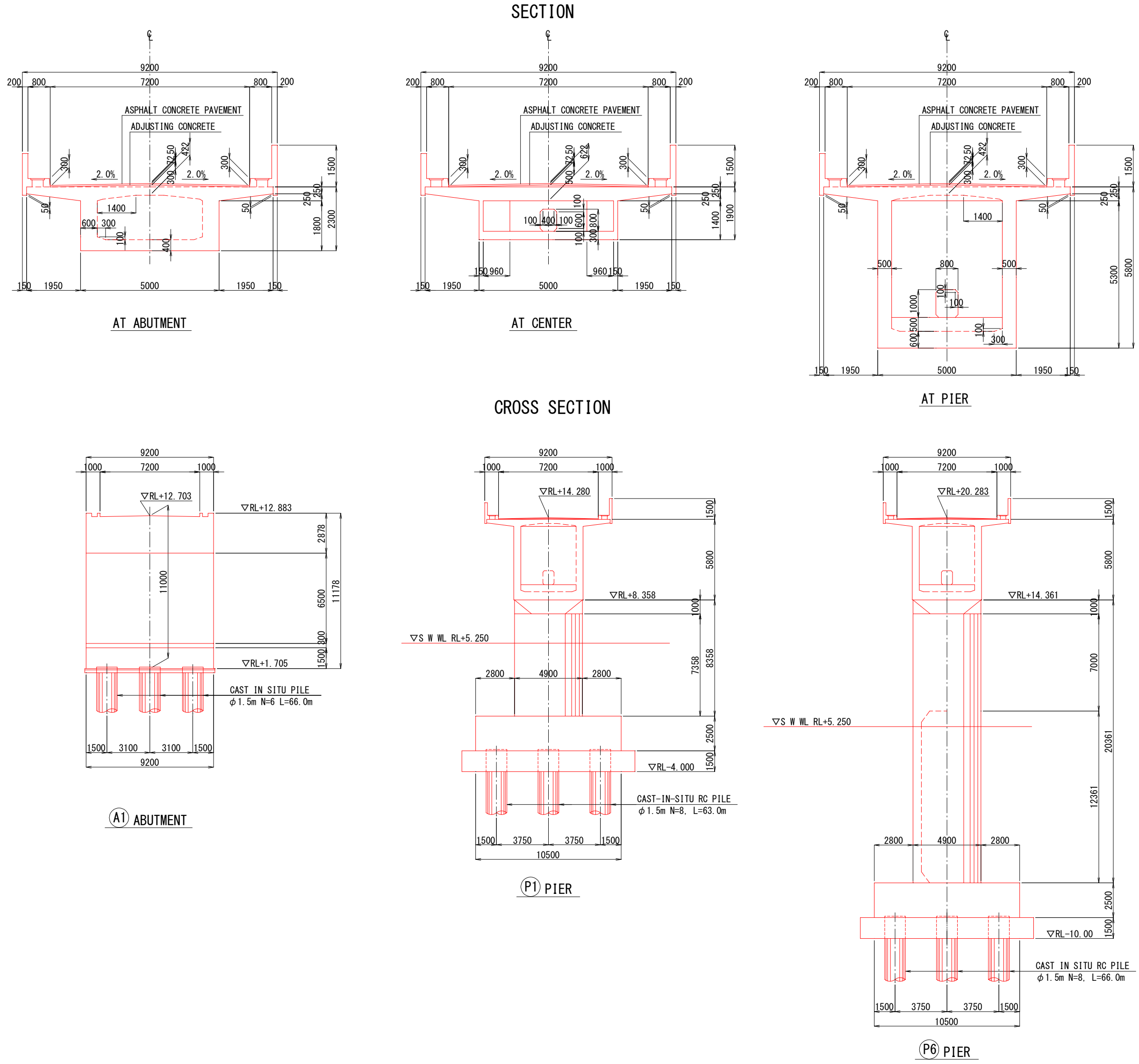


Figure 8.2.3 General View of Existing Gumti Bridge

8.3 Scope of the Retrofitting and Rehabilitation Works

8.3.1 Observation of the Damages

As shown in detail in 2.5.1 the damages to the three bridges have been observerd as follows;

Table 8.3.1 Observation of the Damages (Structures)

	Kanchpur	**Meghna**	**Gumti**
Sub-structure	Cracks and water leakage	Cracks Small dents No rebar deteriorations.	Cracks Water leakage Small dents No rebar deterioration.
Girders & Cross beams	No visible cracks.	Rebar exposures	Cracks over 0.2 mm Water leakage
Deck slab	No visible cracks.	A crack over 0.2 mm width with small amount of isolated lime	No visible cracks.
State	Normal	Normal	Normal
Causes of Deterioration	Aging	Aging	Aging

Table 8.3.2 Observation of the Damages (Accessories)

	Kanchpur	**Meghna**	**Gumti**
Expansion joints	Damaged.	Damaged.	Damaged.
Hinges	N/A	Damaged.	
State	Serious	Serious	Serious
Possible causes of damages	Overloaded trucks Insufficient maintenance	Overloaded trucks Insufficient maintenance	Overloaded trucks Insufficient maintenance

8.3.2 Scope of the Retrofitting and Rehabilitation Works

From the above mentioned background the scope of the rehabilitation works shall be defined as the repair works for the damaged expansion joints in Kanchpur Bridge, Meghna Bridge and Gumti Bridge and the repair works for the damaged hinges in Meghna Bridge and Gumti Bridge.

The retofitting works shall be defined as the works to renew the bridge structures to conform to the current bridge design standards and to cope with the current and future scouring conditions. As mentioned in the previous chapter the changes of the design seismic force should be reflected in all the bridges and the changes of the live load should be reflected in Kanchpur Bridge.

The repair works for the damaged hinges shall be connecting two cantilever girders to be monolithic, eliminating the hinge and expansion joint at each hinge section. As the hinge section shall remain in every 5 or 6 spans to avoid excessive restraint forces from being generated by the temperature change, the hinges and expansion joints at such sections shall be replaced by new ones.

To prevent the bridge from collapsing during earthquake the steel brackets will be attached to the substructures at the girder ends. At the halving joint of Kanchpur Bridge the girder ends are connected to each other.

Consequently, the scope of the retrofitting and rehabilitation works for Kanchpur Bridge, Meghna Bridge and Gumti Bridge are summarized below.

Table 8.3.3 Scope of the Rehabilitation and Retrofitting Works

			Kanchpur	Meghna	Gumti
Repair	Repair of cracks/rebar exposures		○	○	○
	Connecting girders (eliminating hinges/joints)		-	○	○
	Center hinge rehabilitation		-	○	○
	Expansion joint replacement		○	○	○
Retrofit	Steel brackets on the substructures		○	○	○
	Fail-safe connection		○	-	-
	Deck strengthening		○	-	-
	Piers	RC-lining	○	○	○
		Diaphragm wall	○	-	-
	Foundations	Pile cap integration	P1, P3, P5, P6	P1 to P10	P1 to P8
		Steel pipe sheet piles	P1 to P6	P3 to P10	P1 to P8
		Bored RC piles	-	P1, P2	-

8.4 Loads Applied in the Original Design and to be Applied in the Current Design

The difference between the original design standards and the current design standards is found to be changes in magnitude of;

- Earthquake loads
- Live loads

8.4.1 Earthquake Loads

According the existing records (as-built drawings and design calculation report of existing Meghna and Gumti Bridges), it is found that the seismic acceleration coefficient of 0.05 was used to calculate the horizontal seismic forces. But, that value according to BNBC (2006) has been increased to 0.15, which results in increasing the coefficient of response of spectral acceleration to 0.08~0.33. Therefore, it may necessitate revising the original design of the existing bridges.

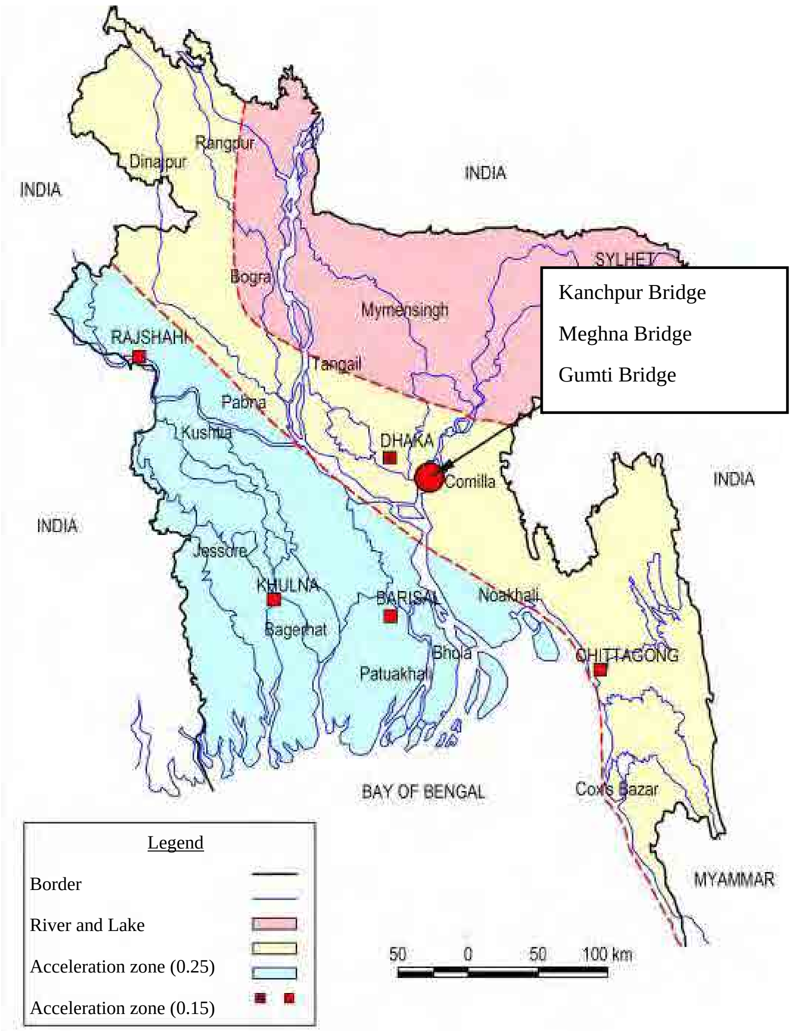


Figure 8.4.1 Seismic Zoning Map of Bangladesh

Due to the change of the seismic acceleration coefficient, the magnitude of the horizontal seismic force will be increased to 1.6~6.6 times the original seismic force.

Table 8.4.1 Horizontal Seismic Coefficient

Original design standard (1)	Current standard (2)	(2) / (1)
0.05	0.08～0.33	1.6～6.6

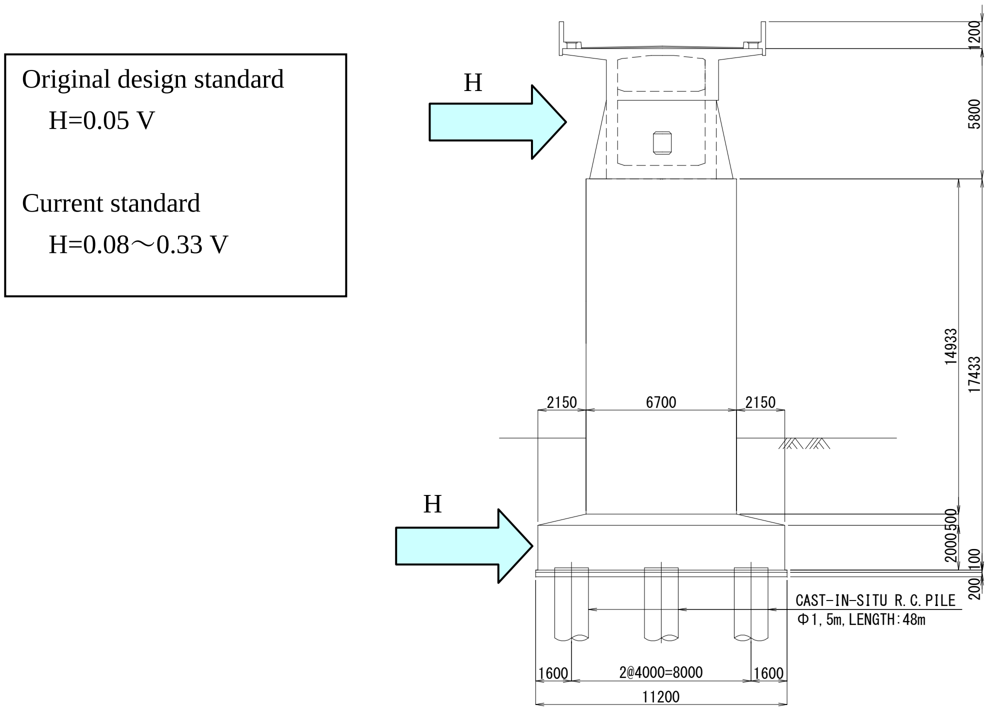


Figure 8.4.2 Earthquake Load Application

8.4.2 Live load Combination

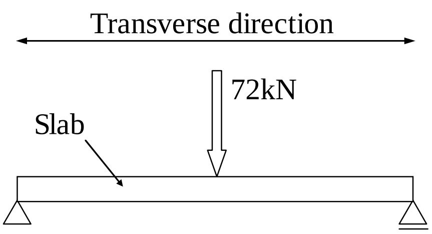
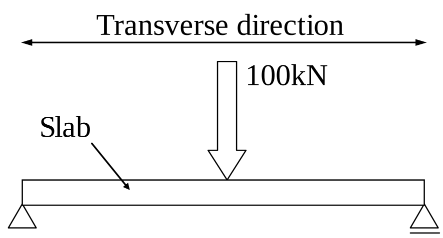
As is mentioned in Chapter 6, special consideration will be given to the design of the floor slab system due to accommodating the heavily loaded trucks. Accordingly, the live load combination from Japanese standards B-type live load will be followed, while the live load combination from AASHTO HS20-44 will be followed for girder design.

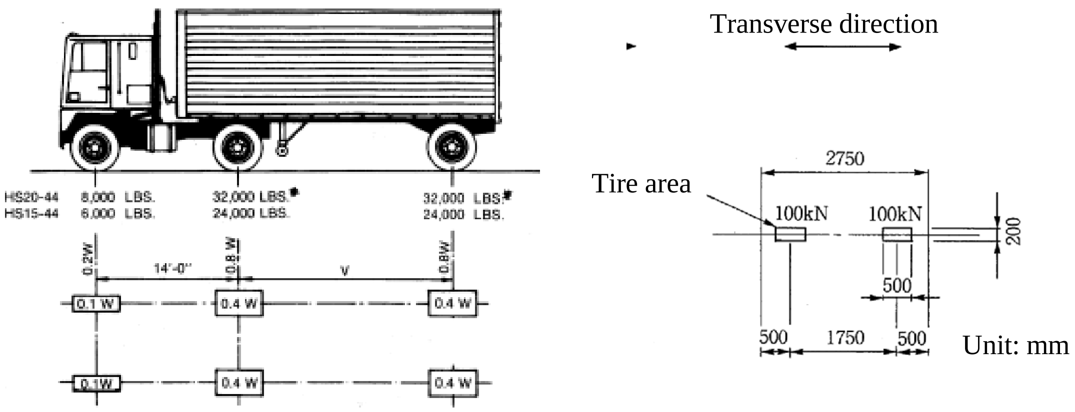
(1) Slab design

The restoration design by reverse analysis method shall be followed particularly for the existing Kanchpur Bridge. This is due to lack of information and no as-built drawings available. The floor slab design is recovered by restoration method applying the AASHTO ASD based HS20-44 live load. The computed results hereinafter are referred to as the capacity of the existing Kanchpur Bridge considering the bridge time span from construction. On the other hand, the results computed with B-type live loads are considered as the requirements for the exiting bridges. However, the rehabilitation plan along with retrofitting is finalized based on these two results.

Furthermore, the retrofitting plan for the floor slab system of the existing Meghna and Gumti Bridges is finalized considering the B-type live loads.

Table 8.4.2 Live Loads for Slab Design

	Original design standard (AASHTO HS20-44 ASD)	Current standard (Japanese standard B-type live load)
Truck Wheel Load	72 kN	100 kN
Load applica-tion		



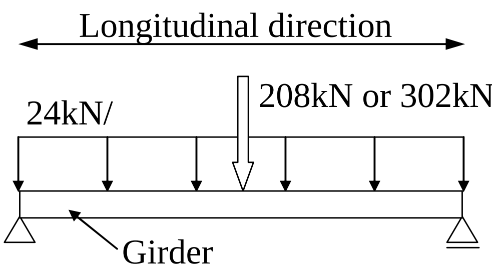
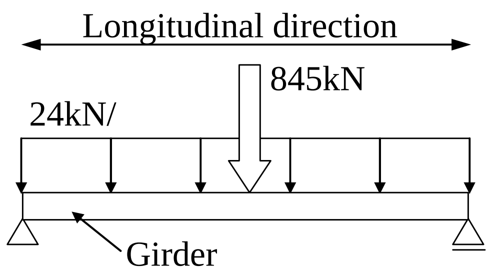
(a) AASHTO HS20-44 truck load **(b) JRA B-type truck load**

Figure 8.4.3 Truck Live Load Comparison

(2) Girder design

The differences in lane live loads, based on different AASHTO versions, are mainly focused on their magnitude, which are shown in Table 8.4.3 side by side. The lane load specification corresponding to AASHTO LRFD HS20-44 will be used for retrofitting of the existing bridge girders.

Table 8.4.3 Live Loads for Girder Design

	Original design standard (AASHTO ASD, HS20-44)	Current standard (AASHTO LRFD, HS20-44)
Live loads	Concentrated load: 208 kN ※1 or 302 kN※2 (4 Lanes) Uniform load: 24 kN/m (4 Lanes)	Concentrated load: 845 kN (4 Lanes) Uniform load: 24 kN/m (4 Lanes)
Load applicat-ion		

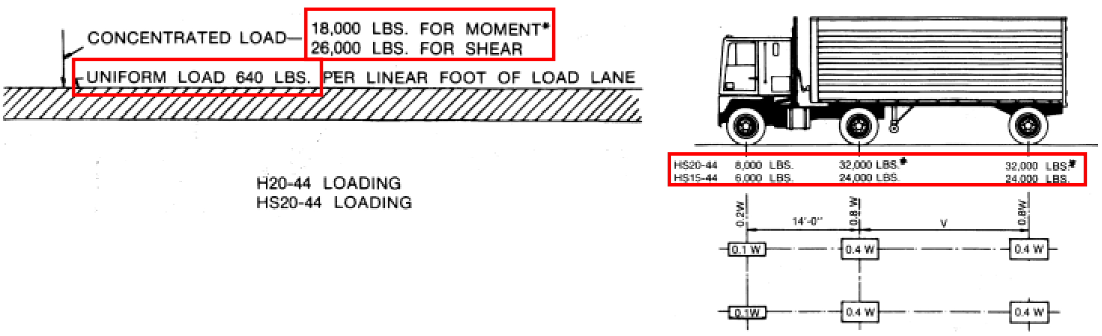


Figure 8.4.4 Live Loads (AASHTO HS20-44)

※1: Live load for moment calculation

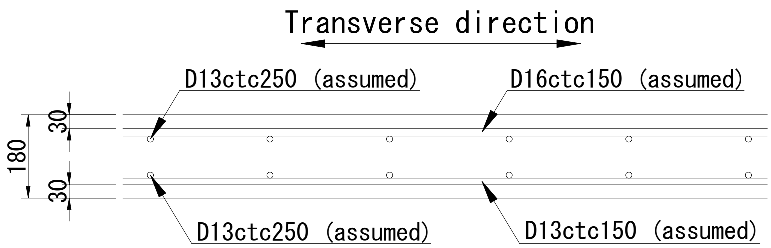
※2: Live load for shear calculation

8.5 Kanchpur Bridge

8.5.1 Evaluation and Recommendations for Slab Design

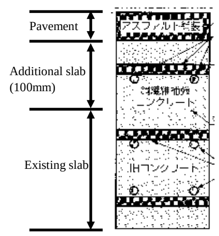
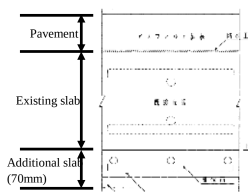
As is also discussed in the previous section, the slab strength of the existing Kanchpur Bridge shall be assessed by applying the restoration design concept. Then, the strength assessment in both longitudinal and transverse directions is compared with the allowable stress recommended by JRA. It can be found that the slab strength retained does not meet the allowable criteria when subjected to current design standards.

Table 8.5.1 Slab Strength Evaluation



			Allowable	Unit	Transverse direction Center span	Transverse direction Support	Longitudinal direction Center span	Check
Original design standard								
	Moment		-	kN・m	13.80	-25.24	8.25	-
	Stress	Concrete	8.0	N/mm2	4.41	7.12	3.75	OK
		Reinforcement	165	N/mm2	128.00	155.30	138.50	OK
Current standard								
	Moment		-	kN・m	19.42	-38.61	13.43	-
	Stress	Concrete	8.0	N/mm2	6.20	10.88	6.10	OUT
		Reinforcement	165	N/mm2	180.10	237.60	225.50	OUT

Table 8.5.2 Retrofitting Method for Slabs

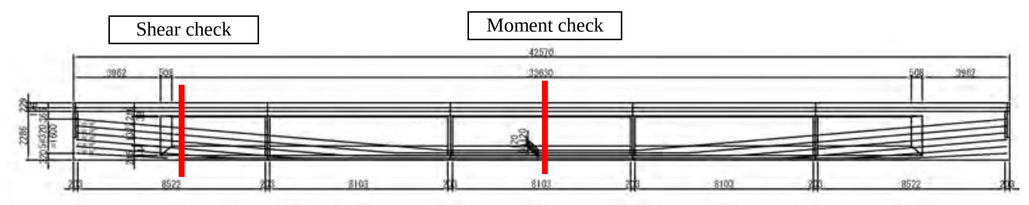
		A plan: Overlay method on the top of slab		B plan: Overlay method on the under of slab		C plan:Carbon fiber	
Image						 	
Record of usage		Many	◎	A little	△	Many	◎
Feature		Weight is increased Girder, pier and foundation need retrofitting about slab retrofitting	△	Weight is increased Girder, pier and foundation need retrofitting about slab retrofitting	△	No weight increase	◎
Constructability	Construction	Best (Downward construction)	◎	Better (Upward construction)	○	Better (Upward construction)	○
		Need scaffolding	△	Need scaffolding	△	Need scaffolding	△
	Construction period	Short	◎	Long	△	Short	◎
Traffic restriction		Need traffic restriction (Overlay and waterproof)	△	Need traffic restriction (waterproof)	△	traffic restriction isn't required	◎
Crack rehabilitation		Crack width under 0.2mm shall be coated Crack width over 0.2mm shall be crack grout	△	Crack rehabilitation isn't required	◎	Crack width under 0.2mm shall be coated Crack width over 0.2mm shall be crack grout	△
Cost		1.11	△	1.11	△	1.00	◎
Evaluation		△		△		◎	

Legend: ◎excellent, ○good, △poor

8.5.2 Evaluation and Recommendations for PC Girder Design

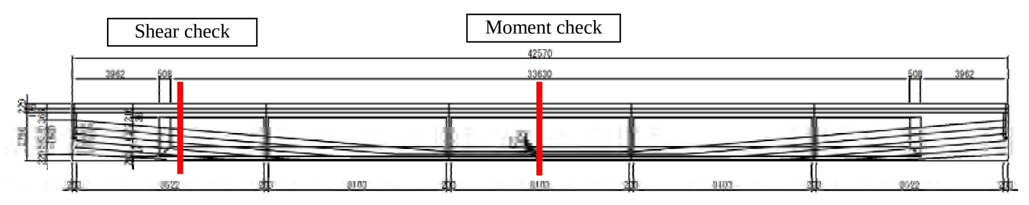
Based on the original design standards, the arrangement and profile of PC-bars are decided from the restoration design. Then, the computed results are compared with the results computed by current design standards specification in Table 8.5.3 - Table 8.5.4. It shows that the capacity of the PC-girder does not meet the requirements as per current design standards. Therefore, it requires retrofitting /additional reinforcement. On the other hand, PRC girder seems to be quite safe under current design standards and thereby, no further retrofitting is necessary.

Table 8.5.3 Restoration Design of PC Girder Based on Original Design Standards



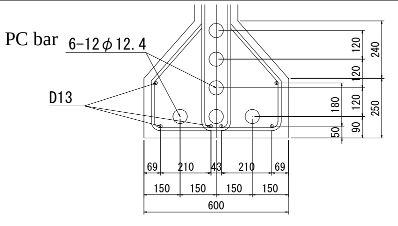
PC bar				SWPR7A 12S12.4-6				
				Unit	Allowable	Results	Check	Remarks
Moment	Compound stress due to							Center span
		Dead load		N/mm2	0.00<σ<12.5	3.82	OK	
		Live load		N/mm2	-1.35<σ<12.5	1.32	OK	
		Temperature load		N/mm2	-1.35<σ<14.3	1.57	OK	
	Flexural safety							
		Ultimate moment	Mu	kNm	-	17213	-	
		Moment capacity	Mr	kNm	-	22846	-	
			Mr/Mu	-	F>1.0	1.33	OK	
Shear	Diagonal tensile stress due to							From girder end to 4.6m
		Dead load		N/mm2	σi>-0.9	-0.05	OK	
		Live load		N/mm2	σi>-1.85	-0.16	OK	
	Web crushing capacity							
		Ultimate share	Sh	kN	-	1299	-	
		Crushing capacity	Suc	kN	-	2405	-	
			Suc/Sh	-	F>1.0	1.85	OK	

Table 8.5.4 State of Strength of PC-Girder



PC bar				SWPR7A 12S12.4-6				
				Unit	Allowable	Results	Check	Remarks
Moment	Compound stress due to							Center span
		Dead load		N/mm2	0.00<σ<12.5	3.82	OK	
		Live load		N/mm2	-1.35<σ<12.5	-1.45	OUT	
		Temperature load		N/mm2	-1.55<σ<14.3	-1.21	OK	
	Flexural safety							
		Ultimate moment	Mu	kNm	-	20816	-	
		Moment capacity	Mr	kNm	-	22846	-	
			Mr/Mu	-	F>1.0	1.10	OK	
Shear	Diagonal tensile stress due to							From girder end to 4.6m
		Dead load		N/mm2	σi>-0.9	-0.05	OK	
		Live load		N/mm2	σi>-1.85	-0.35	OK	
	Web crushing capacity							
		Ultimate share	Sh	kN	-	1581	-	
		Crushing capacity	Suc	kN	-	2405	-	
			Suc/Sh	-	F>1.0	1.52	OK	

Table 8.5.5 State of Strength of PRC-Girder



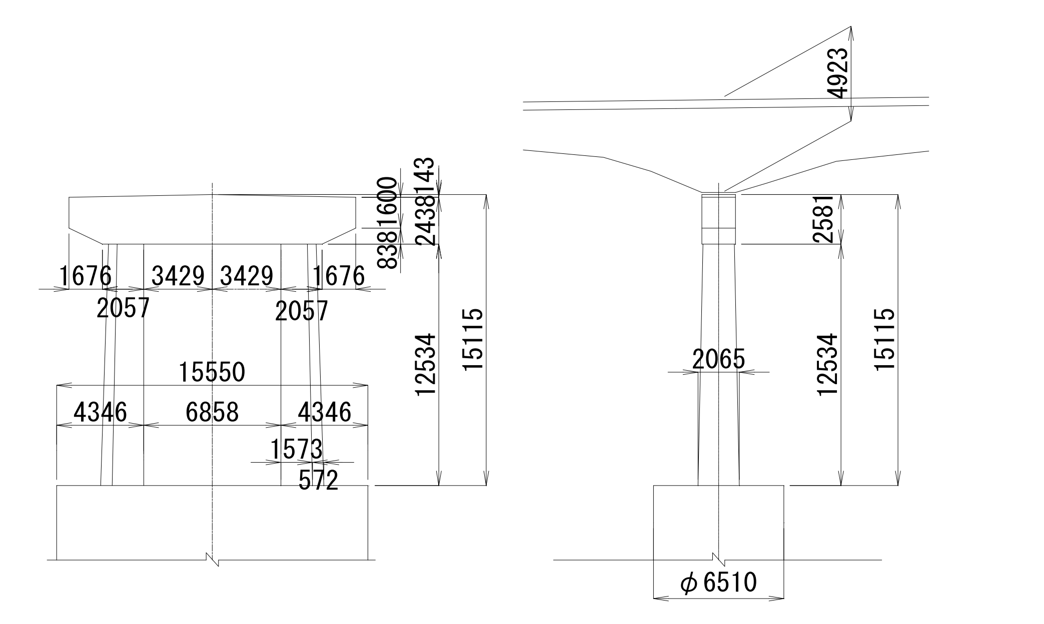
Reinforcement		SD390-4		
		Unit	Results	Check
Compound stress	Live load	N/mm2	-1.45	-
Tensile force		N	99180	-
Allowable tensile stress		N/mm2	200.00	-
Reinforcement required		mm2	495.90	-
Available reinforcement		mm2	506.80	-
	F>1.0	-	1.02	OK

There are two cantilever sections in the span between pier P5 and P6. It is found from the survey results that these cantilever sections are not severely damaged and the existing traffic can easily move over these cantilever sections without any hindrance. Therefore, it is planned not to rehabilitate and leave them unchanged.

8.5.3 Evaluation and Recommendations for Pier Design

In order to assess the state of strength of the bridge piers, the pier P5 is analyzed with the seismic loading specified by the original and current design standards. From the computed results, it can be seen that pier P5 does not meet the requirements for its resistance capacity. Therefore, it will lose its resistance capacity when subjected to strong earthquake excitation.

Table 8.5.6 State of Strength of Pier P5



		Unit	Periodic time	Spectral acceleration coefficient	Demand	Capacity	Check
Longitudinal direction	Moment	kNm	1.89	0.08	18420	17434	OUT
	Shear	kN			1290	1838	OK

It is necessary to increase the strength of the existing bridge piers so that they can withstand strong seismic excitation. Accordingly, the JICA study team adopted three plans;

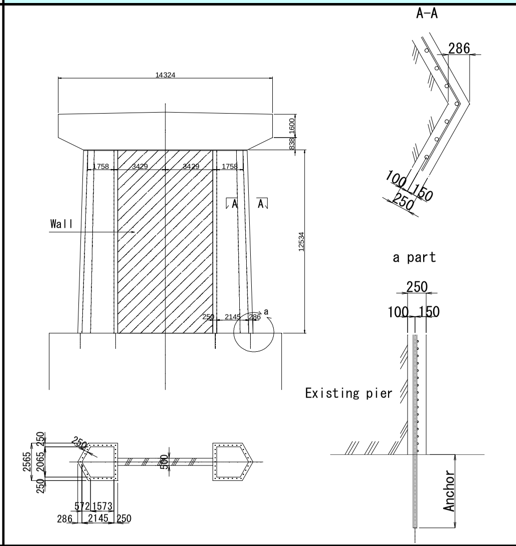
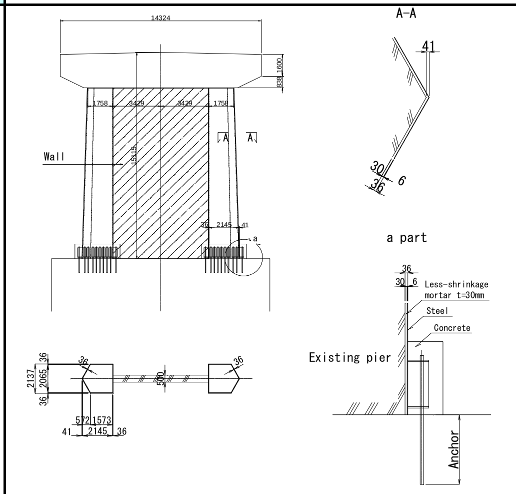
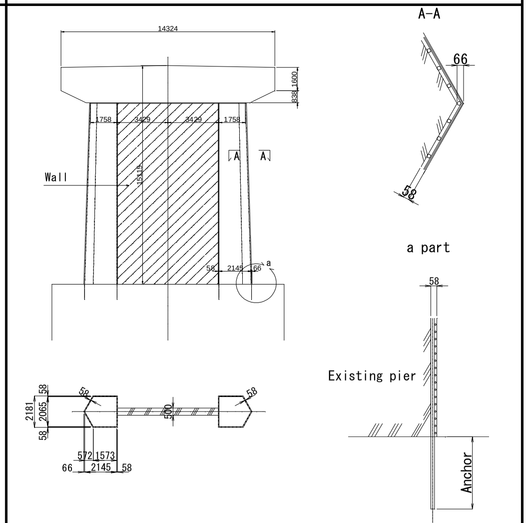
A-plan: RC-lining + Wall

B-plan: Steel plate lining

C-Plan: Polymer mortar lining

A comparative study of the four plans with their construction cost is shown in Table 8.4.7. From this comparison, the JICA study team recommends that the A-plan (RC-lining + Wall) is most preferable for seismic retrofitting of the existing Kanchpur Bridge piers. This is due to the least construction cost.

Table 8.5.7 Seismic Retrofitting of Existing Bridge Piers

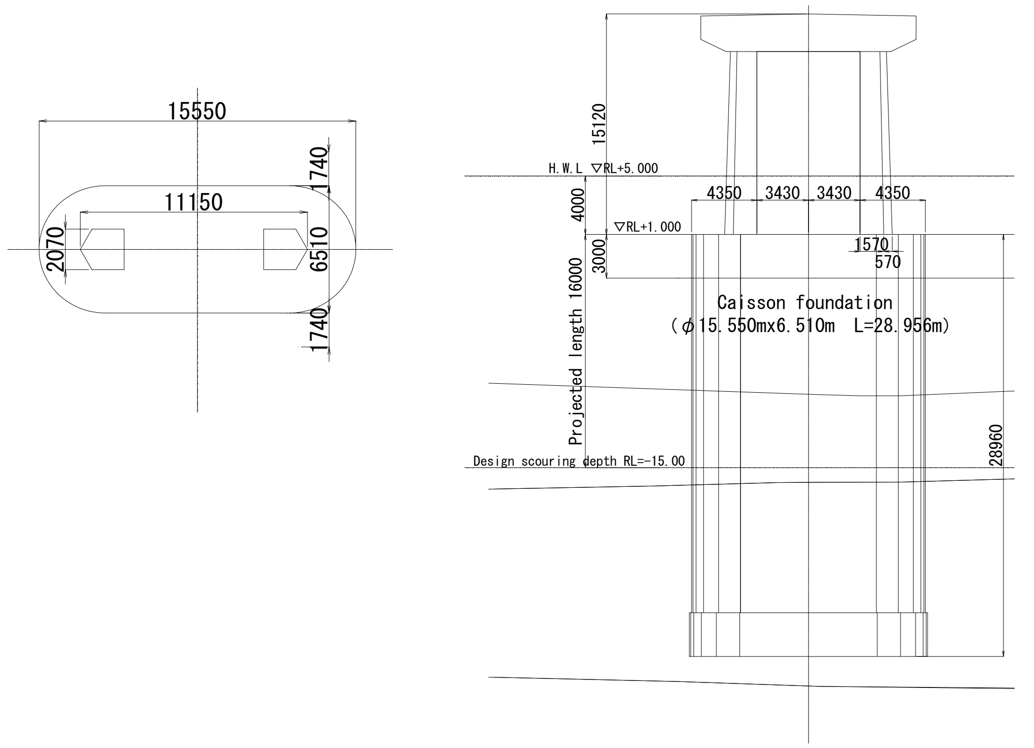
	A plan: RC lining + Wall		B plan: Steel plate lining		C plan: polymer mortar lining	
Image						
Structural performance						
Record of usage	Many	◎	Not so many	○	Few	△
Extra provision	No need protection from heavier floating bodies	◎	No need protection from heavier floating bodies	◎	Need protection from heavier floating bodies	△
Constructability						
Construction	Easy	◎	Easy	◎	Difficult	△
	Need cofferdam	△	Need cofferdam	△	Need cofferdam	△
Construction period	Long	△	Short	◎	Long	△
Maintenance						
Painting	No need	◎	Need	△	No need	◎
Inspection	Easy	◎	Difficult	△	Easy	◎
Landscape	Poor (No space in the wall)	△	Poor (No space in the wall)	△	Poor (No space in the wall)	△
Effect on acquatic environment	Good	◎	Good	◎	Good	◎
Cost	1.00	◎	1.26	○	1.49	△
Evaluation	◎		○		△	

Legend: ◎excellent, ○good, △poor

8.5.4 Evaluation and Recommendations for Foundation Design

As per the original design standards, the capacity of the existing bridge pier foundation is assessed by applying the restoration design concept. Then, the computed results are compared with the results computed by the current design specifications in Table 8.5.8. The current design specifications consider the seismic force as per BNBC along with the higher scouring level that is expected. It shows that the capacity of the foundation corresponding to pier P5 does not meet the requirements as per current design standards. Therefore, it requires retrofitting /additional reinforcement.

Table 8.5.8 State of Strength of Foundation Corresponding to Pier P5



	Unit	Periodic time	Spectral acceleration coefficient	Demand	Capacity	Check
Longitudinal direction						
Displacement	mm	1.89	0.15	29	50	OK
Subgrade reaction in axial direction	kN/m2			833	637	OUT
Subgrade shearing action	kN			7707	8292	OK
Transverse direction						
Displacement	mm	0.89	0.15	92	50	OUT
Subgrade reaction in axial direction	kN/m2			1894	734	OUT
Subgrade shearing action	kN			7967	5860	OUT

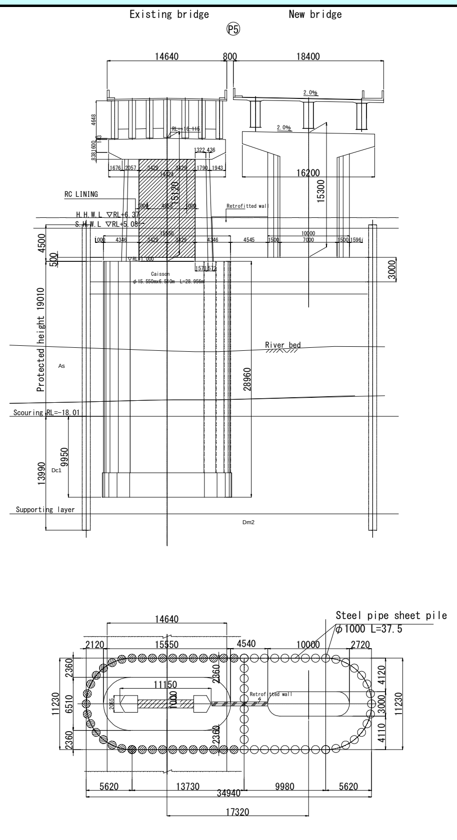
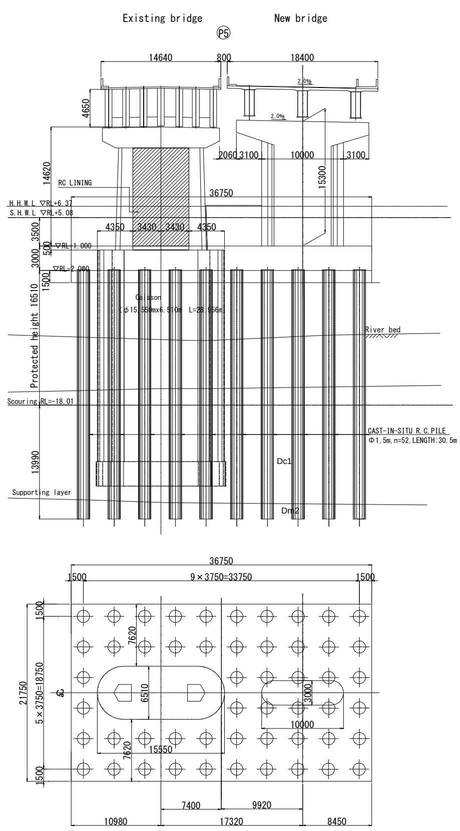
Based on the above results, it is necessary to increase the strength of the foundation for the existing bridge piers so that they can withstand strong seismic excitation. Accordingly, the JICA study team has adopted two plans;

A-plan: Steel Pipe Sheet Pile

B-plan: Additional RC-Pile

A comparative study of the two alternative plans highlighting their construction cost and scouring effect on the river bed is shown in Table 8.5.9. From this comparison, the JICA study team recommends that the A-plan (Steel Pipe Sheet Pile) is the most preferable option for retrofitting of the existing Kanchpur Bridge pier foundations. This is due to the least construction cost and small scouring effect on the river bed.

Table 8.5.9 Foundation Retrofitting

	Plan A: SPSP	Plan B: RC pile
Image		
Cofferdam	Not required	SP
Foundation size	Small	Large
Cost	1.00	1.92
Evaluation	◎	

8.5.5 Evaluation and Recommendations for Work to prevent the Bridge from collapsing

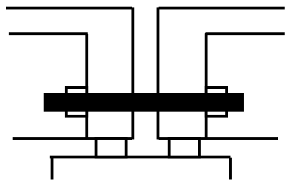
The end of the girder must be constructed in a way of preventing bridge from collapsing. The following are options for such prevention works. In the case of the halving joints section C plan will be recommended.

A plan: Steel brackets
Steel brackets are attached to the pedestal so as to ensure the foolproof length for preventing the girder's falling off the pedestal

B plan: Additional pedestal
Pedestal is extended so as to ensure the foolproof length for preventing the girder's falling off the pedestal

C plan: Fail-safe connection
Ends of the adjacent girders are connected by PC bars for preventing the girder's falling off the pedestal

Table 8.5.10 Prevention Work above the Substructure to prevent bridge collapse

	A plan: Steel brackets		B plan: Additional pedestal		C plan: Fail-safe connection	
Image						
Structure	Attach the steel bracket to the pedestal so as to ensure the foolproof length of the end of the girder against the girder's falling off the pedestal		Extend the pedestal so as to ensure the foolproof length of the end of the girder against the girders falling off the pedestal		Connect the ends of the adjacent girders by PC bars so as to prevent the girder's falling off if the end of the girder deviates from the pedestal	
Record of usage	Many	◎	Many	◎	Many	◎
Constructability	Nomal	○	Nomal	○	Nomal	○
	Need scaffolding	△	Need scaffolding	△	Need scaffolding	△
Aesthetic appearance	Not good	△	Not good	△	Not good	△
Maintenance	Nomal	○	Nomal	○	Nomal	○
Cost	1.00	◎	1.07	○	1.62	△
Evaluation	◎		○		△	

Legend: ◎excellent, ○good, △poor

8.6 Meghna Bridge

8.6.1 Evaluation and Recommendations for Girder Design

The JICA study team assumed that the reduction in the number of expansion joints and hinged shoes will increase the servicability of the existing bridges and reduce the maintenace cost in the future. This can be achieved by connecting the simply supported girders into the continuous girder at hinged sections. The computed results are shown in Figure 8.6.1.

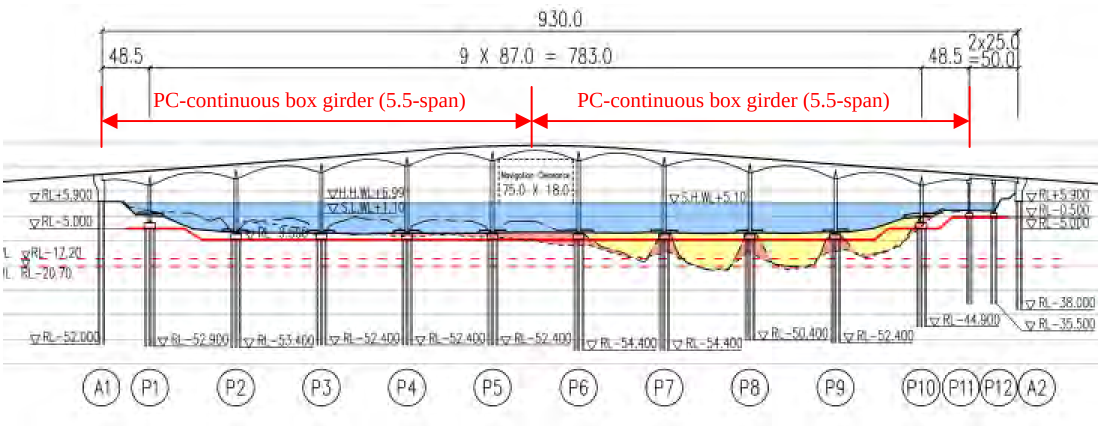


Figure 8.6.1 Concept of Continuity in Meghna Bridge Girder

In order to reduce the number of expansion joints and hinges in the floor slab, the existing cantilever box girder system is being planned to be continuous with a rigid frame at the hinged sections. The lengths of the two continuous sections for Meghna Bridge are shown in Figure 8.6.1. The lengths of the continuous sections is determined to meet the requirements that the stress due to change of temperature shall be less than the stress generated from horizontal earthquake excitation.

In order to rehabilitate the central hinges of Meghna and Gumti bridges, the JICA Study Team has considered three plans;

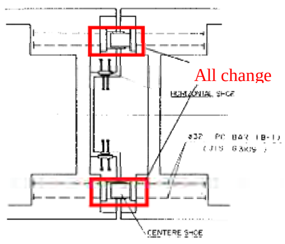
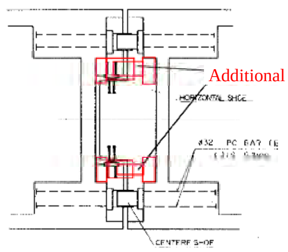
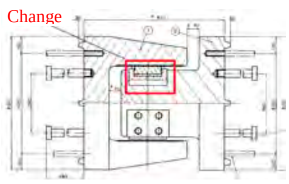
A plan: Entire change

B plan: Additional hinge

C plan: Partly change

Meghna bridge has one joint for this rehabilitation and the Gumti bridge has two joint for this rehabilitation. Among the three plans, C plan is more preferable. This is due to the fact that it is possible to manufacture and procure.

Table 8.6.1 Retrofitting by Center Hinge

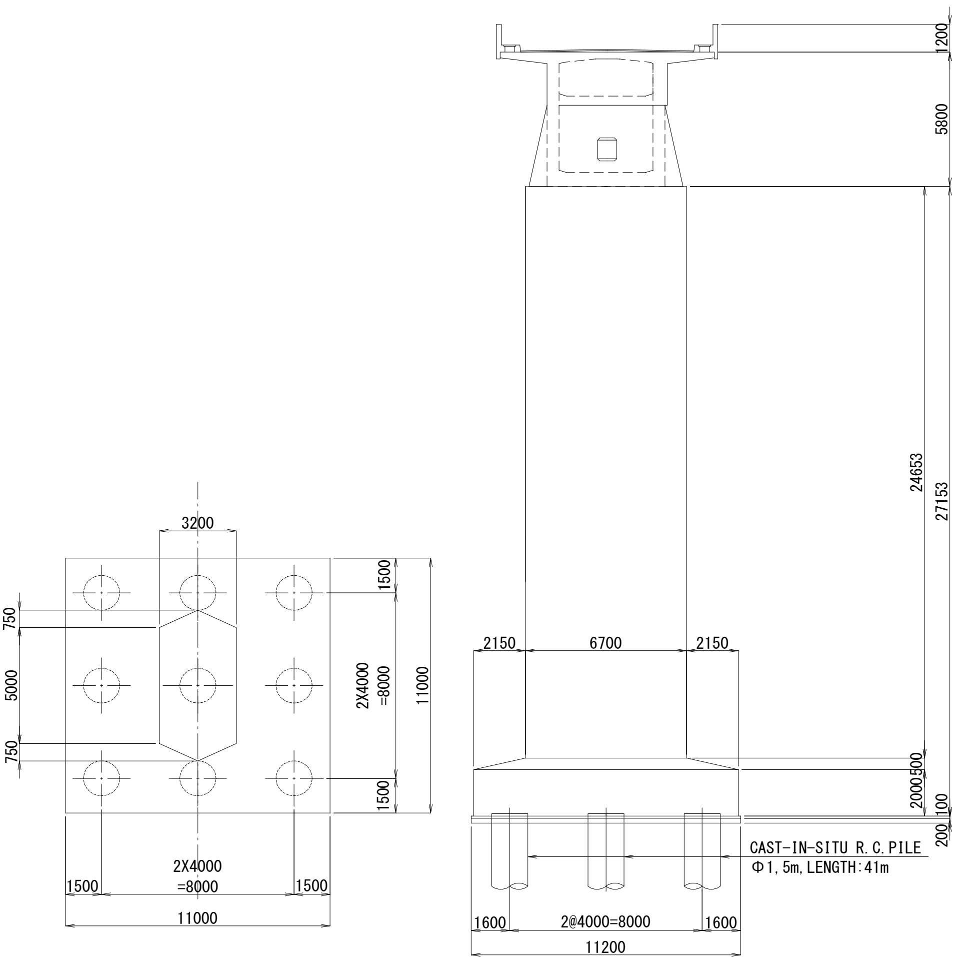
	A plan: Entire change	B plan: Addition at hinge	C plan: Partly change
Image			
Procurement	Impossible to manufacture	Impossible to manufacture	Possible to manufacture
Evaluation	△	△	◎

Legend: ◎excellent, ○good, △poor

8.6.2 Evaluation and Recommendations for Pier Design

In order to assess the state of strength of the bridge pier, pier P8 is analyzed with the seismic loading specified in the original and current design standards. From the computed results, it can be seen that pier P8 does not meet the requirements for its resistance capacity. This state of strength is similar to Kanchpur Bridge. Therefore, the Meghna Bridge pier will lose its resistance capacity when subjected to strong earthquake excitation.

Table 8.6.2 Strength Check for Pier P8



		Unit	Periodic time	Spectral acceleration coefficient	Demand	Capacity	Check
Longitudinal direction	Moment	kNm	0.6	0.17	140042	97856	OUT
	Shear	kN			12691	9923	OUT

Similar to Kanchpur Bridge, three of four plans are also considered for retrofitting of Meghna Bridge pier.

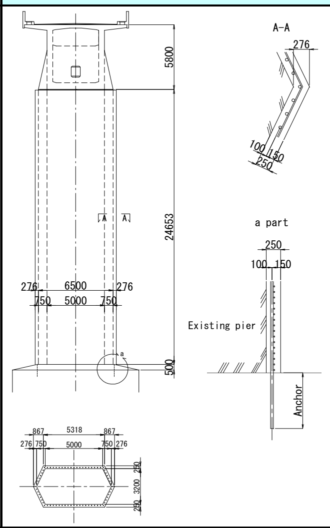
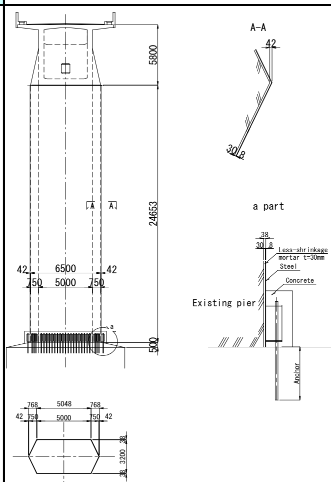
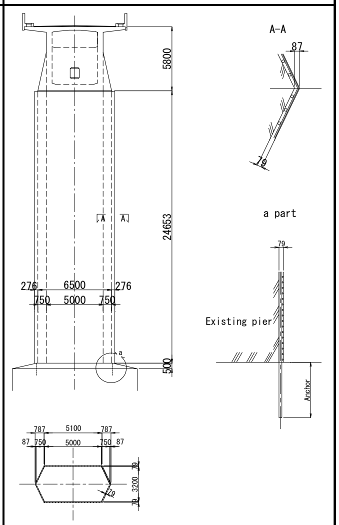
A-plan: RC-lining

B-plan: Steel plate lining

C-Plan: Polymer mortar lining

A comparative study of the three plans with their construction cost is shown in Table 8.6.3. From this comparison, the JICA study team recommends that the A-plan (RC-lining) is most preferable for seismic retrofitting of the existing Meghna Bridge piers. This is due to the least construction cost.

Table 8.6.3 Retrofitting by Pier

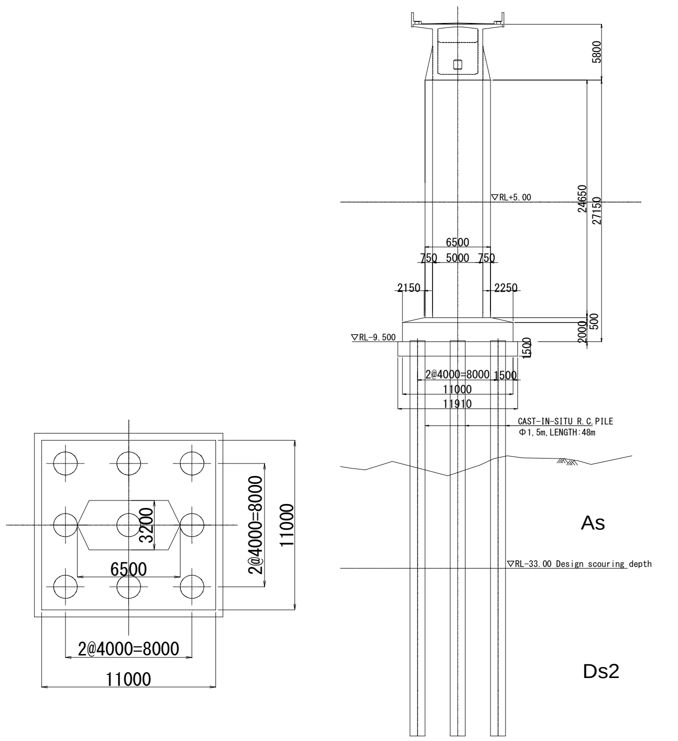
		A plan: RC lining		B plan: Steel sheet lining		C plan: Polymer mortar lining	
Image							
Structual performance							
	Record of usage	Many	◎	Not so many	○	Few	△
	Extre provision	No need protection from heavier floating bodies	◎	No need protection from heavier floating bodies	◎	Need protection from heavier floating bodies	△
Constructability							
	Construction	Easy	◎	Easy	◎	Difficult	△
		Need cofferdam	◎	Need cofferdam	◎	Need cofferdam	◎
	Construction period	Long	△	Short	◎	Long	△
Maintenance							
	Painting	No need	◎	Need	△	No need	◎
	Inspection	Easy	◎	Difficult	△	Easy	◎
Landscape		Good	◎	Good	◎	Good	◎
Effect on acquatic environment		Good	◎	Good	◎	Good	◎
Cost		1.00	◎	1.84	○	2.34	△
Evaluation		◎		△		△	

Legend: ◎excellent, ○good, △poor

8.6.3 Evaluation and Recommendations for Foundation Design

Following a method similar to that stated above for Kanchpur Bridge, the capacity of the Meghna Bridge foundations is verified under load specifications by current design standards. The current design specification considers the seismic force as per BNBC along with the higher scouring level that is expected. The decision is similar to Kanchpur Bridge. The capacity of the foundation corresponding to pier P8 does not meet the requirements and requires retrofitting /additional reinforcement.

Table 8.6.4 Strength Check for Foundation of Pier P8



			Unit	Periodic time	Spectral acceleration coefficient	Demand	Capacity	Check
Longitudinal direction								
	Displacement		mm	0.6	0.33	65	15	OUT
	Axial force	Max	kN			29149	26733	OUT
		Min	kN			-21582	-21432	OUT
	Moment	Mmax	kNm			71482	1920	OUT
Transverse direction								
	Displacement		mm	0.8	0.27	27	15	OUT
	Axial force	Max	kN			16862	26733	OK
		Min	kN			-6136	-21432	OK
	Moment	Mmax	kNm			29740	1920	OUT

Similarly, two retrofitting plans are also considered for Meghna Bridge foundation.

A-plan: Steel Pipe Sheet Pile

B-plan: Additional RC-Pile

A comparative study of the two alternative plans highlighting with their construction cost and scouring effect on the river bed was carried out as already seen in Table 7.3.1. From this comparison, the JICA study team recommends that the A-plan (Steel Pipe Sheet Pile) is the most preferable option for retrofitting of the existing Meghna Bridge pier foundations. This is due to the least construction cost and small scouring effect on the river bed.

The comparison shown in Table 7.3.1 as described above was for the severe scouring zone.

In the cases of the shallow and medium scouring zones results of the comparison are as shown in Table 8.6.5 and Table 8.6.6 Retrofitting the foundation, respectively. The piled foundation is advantageous in the shallow scouring zone.

Table 8.6.5 Retrofitting the foundation

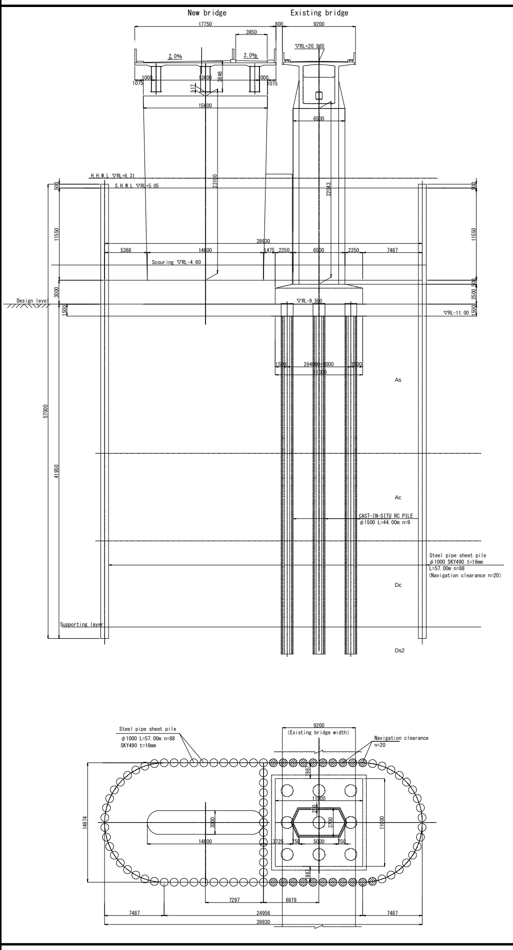
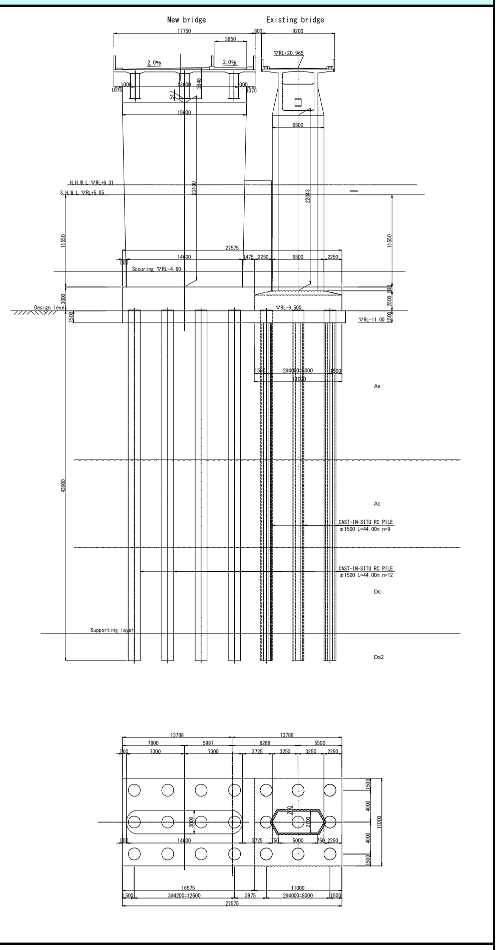
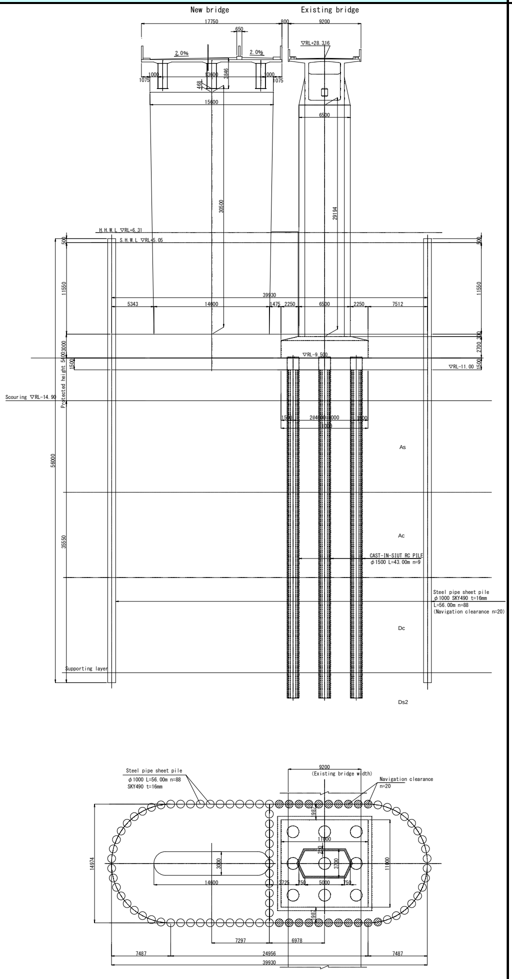
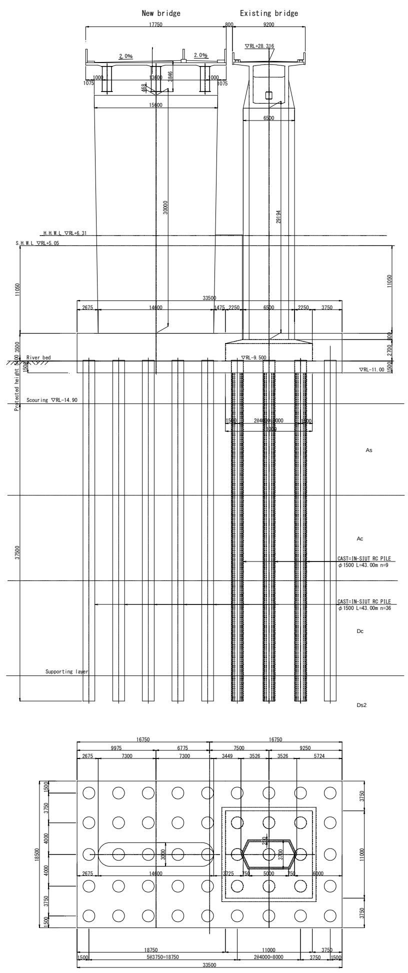
	Plan A: SPSP	Plan B: RC pile
Image		
Cofferdam	Not required	SP
Foundation size	Small	Small
Cost	1.00	0.66
Evaluation		◎

Table 8.6.6 Retrofitting the foundation

	Plan A: SPSP	Plan B: RC pile
Image		
Cofferdam	Not required	SP
Foundation size	Small	Large
Cost	1.00	1.07
Evaluation	◎	

8.6.4 Evaluation and Recommendations for Work to prevent Bridge collapse

The work to prevent bridge collapse that was already explained for Kanchpur Bridge in subsection 8.5.5 will also be used for Meghna Bridge.

8.7 Gumti Bridge

8.7.1 Evaluation and Recommendations for Girder Design

Similar to Meghna Bridge, the number of expansion joints and hinged shoes will be decreased in order to increase the serviceability of the existing bridges and reduce the maintenance cost in the future. This can be achieved by connecting the simply supported girders to become a continuous girder at the hinged sections. The computed results are shown in Figure 8.7.1

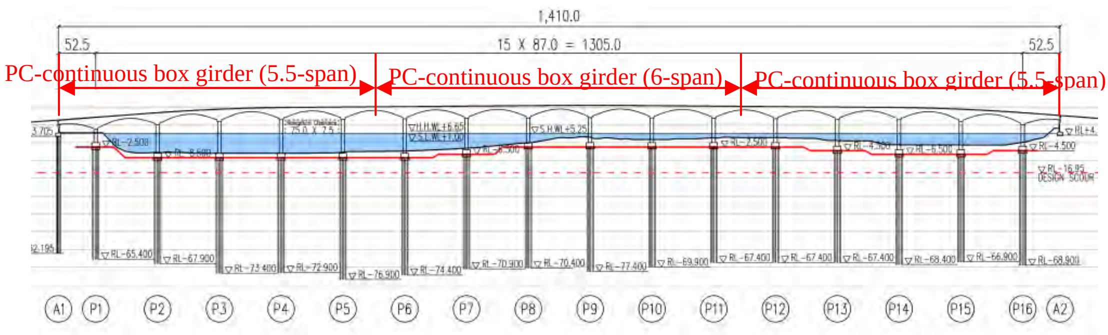
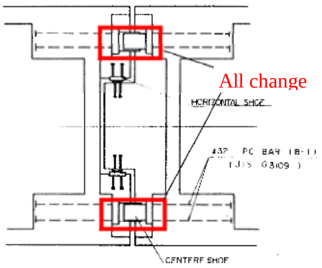
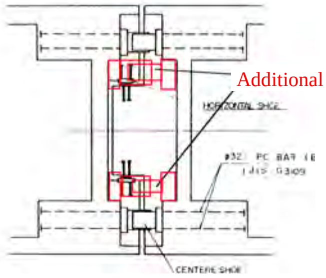
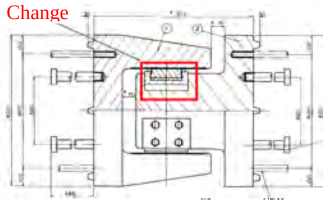


Figure 8.7.1 Concept of Continuity in Gumti Bridge Girder

The concept to determine the length of the continuous section of Gumti Bridge is the same as that used for Meghna Bridge. The existing cantilever system will be rehabilitated and converted into three continuous box sections so that the number of expansion joints and hinged shoes will be decreased and thereby increase its serviceability. The lengths of the three continuous sections for Gumti Bridge are shown in Figure 8.7.1.

Meghna bridge has one joint for this rehabilitation and the Gumti bridge has two joint for this rehabilitation.

Table 8.7.1 Retrofitting Center Hinge

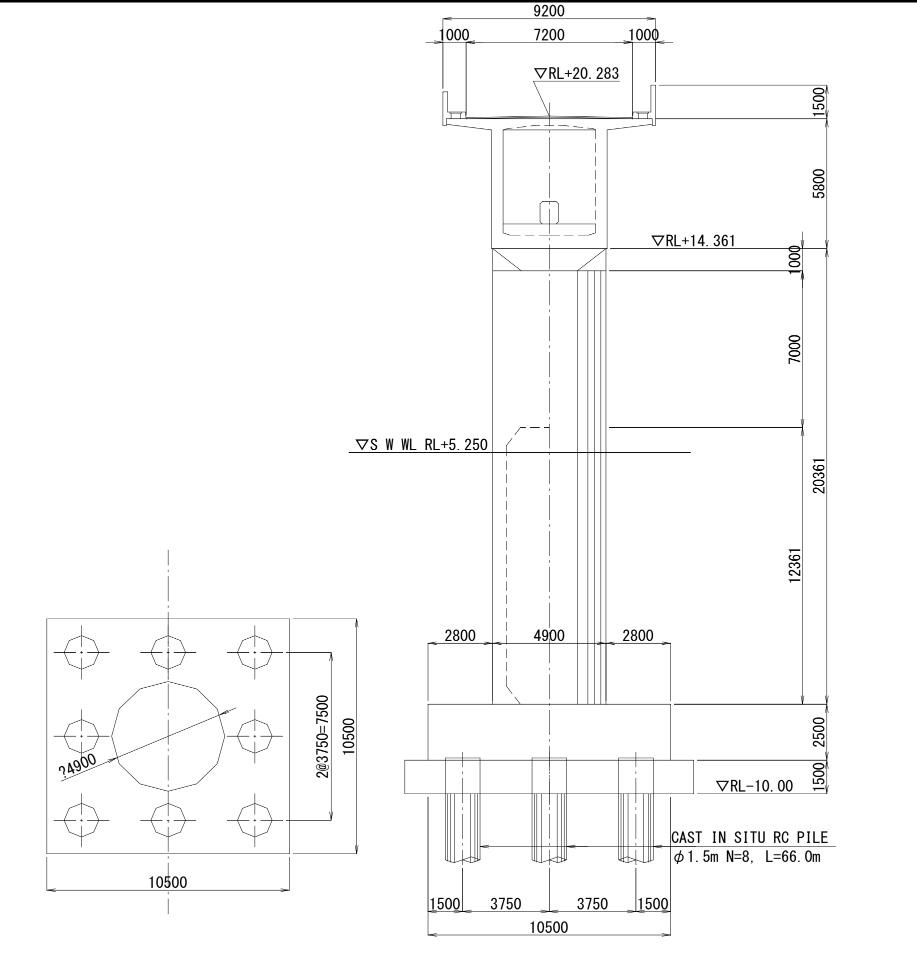
	A plan: Entire change	B plan: Addition at hinge	C plan: Partly change
Image			
Procurement	Impossible to manufacture	Impossible to manufacture	Possible to manufacture
Evaluation	△	△	◎

Legend: ◎excellent, ○good, △poor

8.7.2 Evaluation and Recommendations for Pier Design

In order to assess the state of the strength of the bridge piers, pier P6 is analyzed with the seismic loading specified in the original and current design standards. From the computed results, it can be seen that pier P6 does not meet the requirements for its resistance capacity. This state of strength is similar to Kanchpur and Meghna Bridges. Therefore, the Gumti Bridge piers will lose their resistance capacity when subjected to strong earthquake excitation.

Table 8.7.2 Strength Check for Pier P6



		Unit	Periodic time	Spectral acceleration coefficient	Demand	Capacity	Check
Longitudinal direction	Moment	kNm	0.6	0.17	140042	98221	OUT
	Shear	kN			12691	8587	OUT

Similar to Meghna Bridge, three plans are also considered for retrofitting of Gumti Bridge piers.

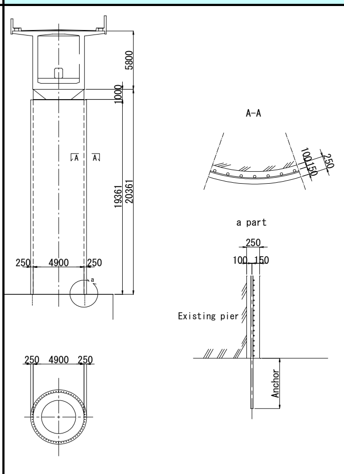
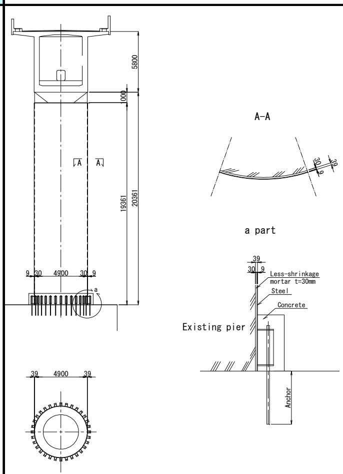
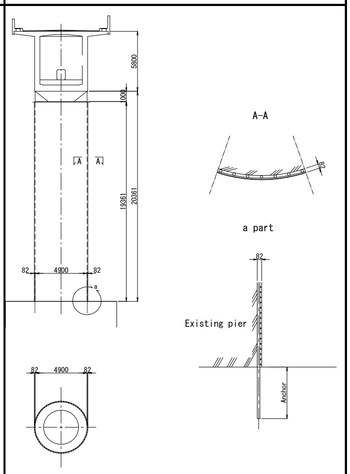
A-plan: RC-lining

B-plan: Steel plate lining

C-Plan: Polymer mortar lining

A comparative study among the three plans with their construction cost is shown in Table 8.7.3. From this comparison, the JICA study team recommends that the A-plan (RC-lining) is the most preferable for seismic retrofitting of existing Meghna Bridge piers. This is due to the least construction cost, which is the same reason considered for Meghna Bridge.

Table 8.7.3 Retrofitting Piers

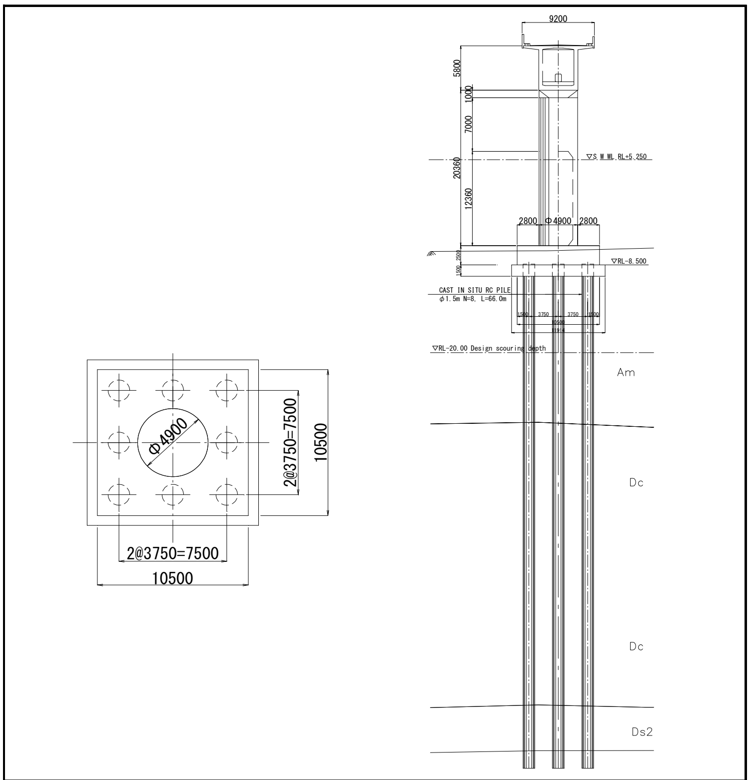
		A plan: RC casting reinforcement		B plan: Steel casting reinforcement		C plan: polymer mortar casting reinforcement	
Image							
Structural performance							
	Record of usage	Many	◎	Not so many	○	Few	△
	Extre provision	No need protection from heavier floating bodies	◎	No need	◎	Need	△
Constructability							
	Construction	Easy	◎	Easy	◎	Difficult	△
		Need cofferdam	◎	Need cofferdam	◎	Need cofferdam	◎
	Construction period	Long	△	Short	◎	Long	△
Maintenance							
	Painting	No need	◎	Need	△	No need	◎
	Inspection	Easy	◎	Difficult	△	Easy	◎
Landscape		Good	◎	Good	◎	Good	◎
Effect on acquatic environment		Good	◎	Good	◎	Good	◎
Cost		1.00	◎	2.04	○	2.43	△
Evaluation		◎		△		△	

Legend: ◎excellent, ○good, △poor

8.7.3 Evaluation and Recommendations for Foundation Design

Following a method similar to that stated above for Kanchpur and Meghna Bridges, the capacity of Gumti Bridge foundations is verified under load specifications in the current design standards. The current design specifications consider the seismic force as per BNBC along with the higher scouring level that is expected. The computed result shows a similar tendency that has been verified for Kanchpur and Meghna Bridges. The capacity of the foundation corresponding to pier P6 does not meet the demand requirements and requires retrofitting /additional reinforcement.

Table 8.7.4 Strength Check for Foundation of Pier P6



		Unit	Natural periods	Spectral acceleration coefficient	Demand	Capacity	Check
Longitudinal direction							
Displacement		mm	0.6	0.33	53	15	OUT
Axial force	Max	kN			24327	43479	OK
	Min	kN			-15814	-38177	OK
Moment	Mmax	kNm			40441	8400	OUT
Transverse direction							
Displacement		mm	0.8	0.27	22	15	OUT
Axial force	Max	kN			15256	43479	OK
	Min	kN			-3438	-38177	OK
Moment	Mmax	kNm			16826	4130	OUT

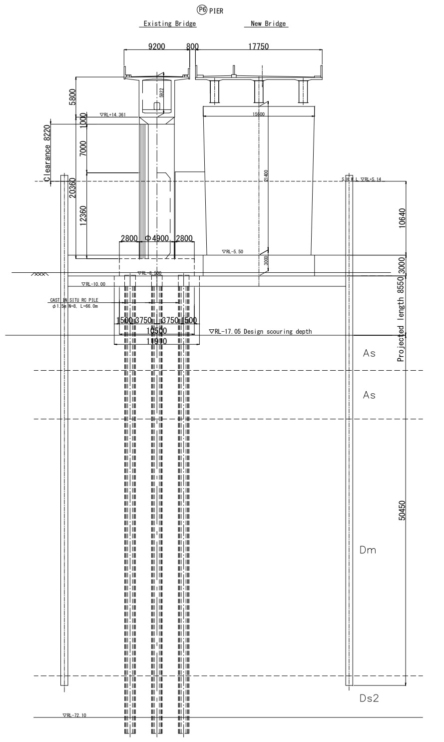
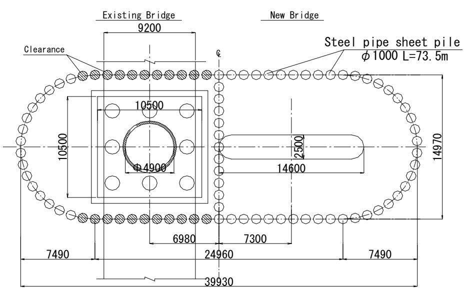
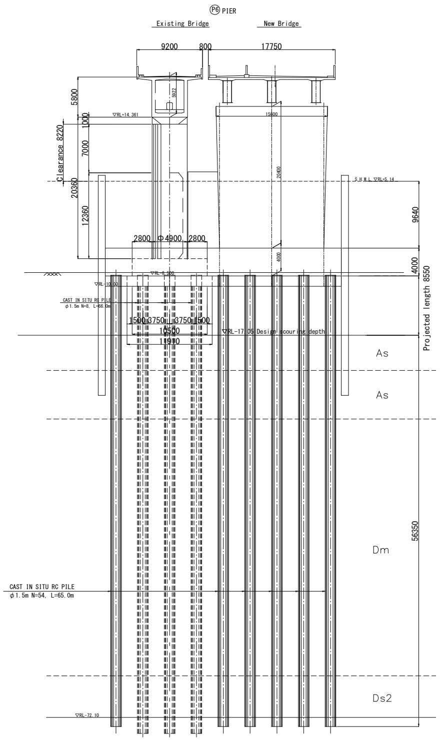
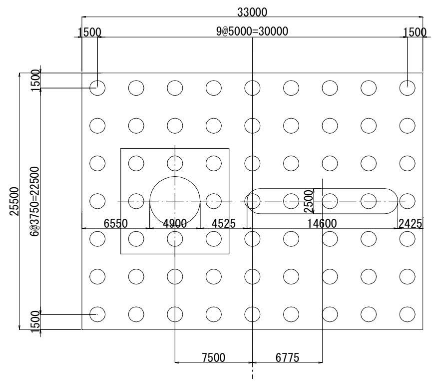
Similarly, two retrofitting plans are also considered for Meghna Bridge foundation.

A-plan: Steel Pipe Sheet Pile

B-plan: Additional RC-Pile

A comparative study of the two alternative plans highlighting their construction cost and scouring effect on the river bed is shown in Table 8.7.5. From this comparison, the JICA study team recommends that the A-plan (Steel Pipe Sheet Pile) is the most preferable option for retrofitting of existing Gumti Bridge pier foundation. This is due to the least construction cost and small scouring effect on the river bed.

Table 8.7.5 Retrofitting the Foundations

	A plan: Steel pipe sheet pile	B plan: Additonal RC pile
Image	 	 
Cofferdam	Not required	SP
Foudation size	Small	Large
Cost	1.00	1.48
Evaluation	◎	

8.7.4 Evaluation and Recommendations for Work to Prevent Bridge Collapse

The work to prevent bridge collapse that was already explained for Kanchpur Bridge in subsection 8.5.5 will also be used for Gumti Bridge.

8.8 Consideration of Damages on Meghna Bridge and Gumti Bridges

8.8.1 General

Meghna Bridge and Gumti Bridge are multi span continuous prestressed concrete rigid frame bridges having, in the middle of each span between piers, hinges and expansion joints where no bending moments occur. The hinges are fixed in the vertical direction to transmit the shearing forces between the cantilevers projecting from the piers whereas they are free in the horizontal direction to eliminate restraint forces due to creep, shrinkage and temperature change in the structures and thus greatly reduce the sizes of the substructures / foundations.

8.8.2 What Happened to the Hinges and Expansion Joints

(1) Damages to the hinges

It has been observed that the hinges lost their proper function to transmit the shearing forces between the cantilevers projecting from the piers, generating noises and unfavorable impact forces on the expansion joints when vehicles move from one cantilever to the other cantilever. Recent investigations showed the deteriorated rubber plates in the hinges as below.

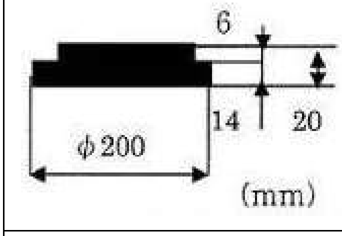
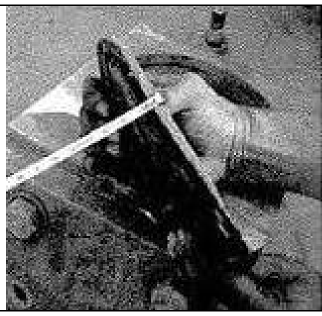
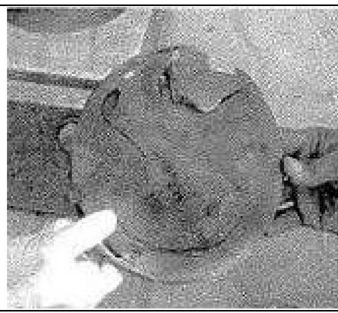
		
Rubber plate (Originally 20 mm thick)	Damaged rubber plate (Thickness 0 to 8 mm)	Damaged rubber plate (Plan view)

Figure 8.8.1 Deterioration in Hinges

(2) Damages to the expansion joints

As shown on the right in Figure 8.8.1 the expansion joints also deteriorated as a result of unfavorable impact forces acting frequently on them caused by the loss of proper function of the hinges.

Figure 8.8.2 Deterioration in Expansion Joints

8.8.3 Possible Causes of the Damages

(1) Possible causes of the damages to the hinges

Historically, the common types of hinges used to be the line touch type shown in Figure 8.8.3. As this type is vulnerable to abrasion from the steel components in the contact points due to the horizontal movement, there were many cases where the line touch type was replaced by the plane touch type shown in Figure 8.8.3 as the repair works.

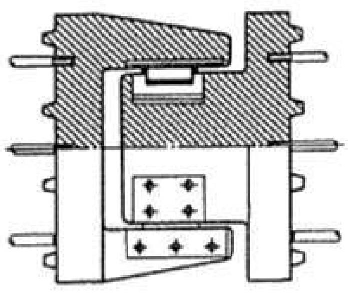
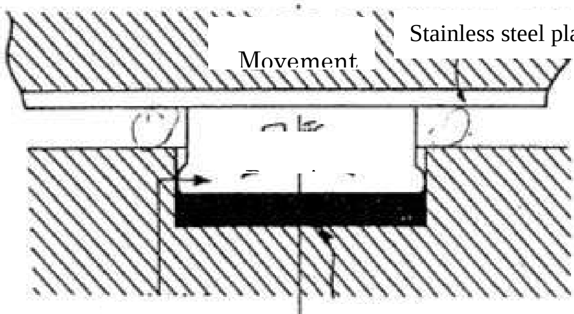
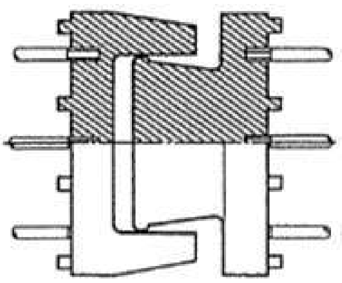
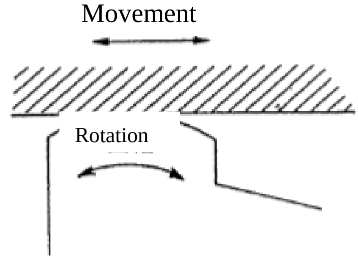
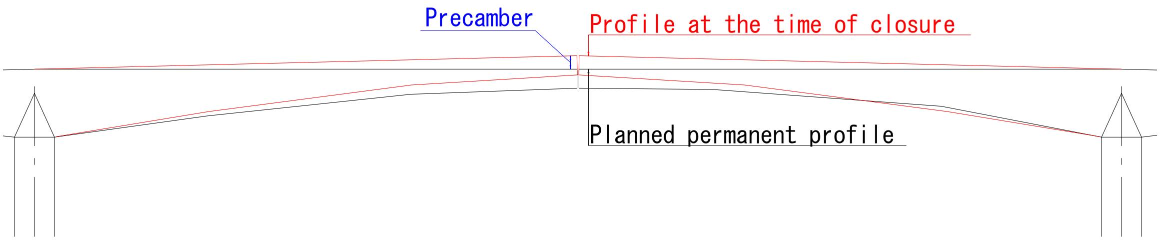
Conceptual drawing of the hinge	Mechanism of the hinge
 **Plane touch type**	
 **Line touch type**	

Figure 8.8.3 Types of Hinges

As can be seen in the figure, the plane touch type has improved resistance to abrasion due to horizontal movement. On the other hand, it should be noted that the plane touch type may have less capacity to accommodate the rotation than the line touch type.

When used for the repair works of an existing bridge where the creep deflection has finished and the rotation of the hinge will be relatively small because it is generated mainly by the live loads, adopting the plane touch type will cause no major problems due to the rotation.

In the case of a newly constructed bridge, however, a pre-camber is prescribed in the girder to accommodate the creep deflection and as a result of it a considerable amount of rotation will be generated between at the time of completion and at the time of reaching the permanent condition—the creep is finished. See Figure 8.8.4.



Pre-camber = creep deflection = profile at the time of closure – planned permanent profile

Figure 8.8.4 Creep Deflection of the Bridge Girders

Meghna Bridge and Gumti Bridge adopted the plane touch type of hinges as shown in Figure 8.8.5. There could have been a possibility that the rubber plates in the hinges had a distorted deformation due to the rotation caused by the creep deflection with an amount exceeding the planned value, since the actual magnitude of the creep could have been larger than the planned depending on various conditions such as not only the actual mix proportion and characteristics of the materials used for the concrete but also the climate or environmental conditions. If that was the case then the rubber plates which had already been stressed some amount could have been excessively stressed when they were subjected to the live loading effects.

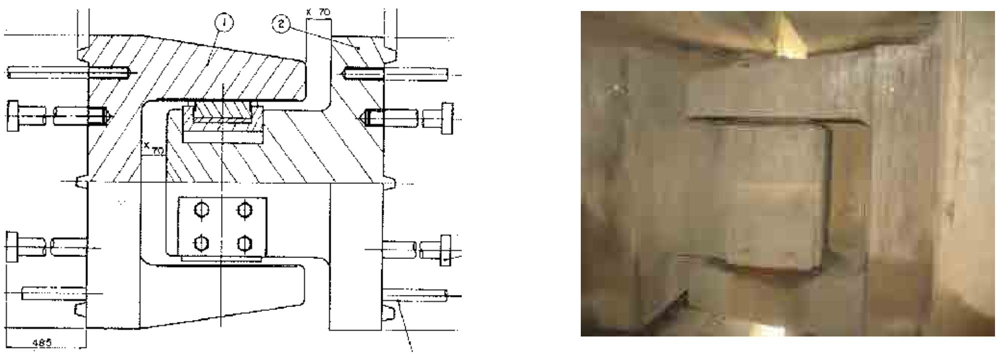


Figure 8.8.5 Hinge of Gumti Bridge

According to the current observations, strict control of the axle load of the trucks, with a limit of 20 tons, has just recently become effective at the entrance to the toll gate of Meghna

Bridge. It was observed before then that a number of trucks with remodeled suspension and tires which appeared to carry an axle load well over 20 tons, namely 30 or even 40 tons were passing over the roads and bridges.

In this way Meghna Bridge and Gumti Bridge have been often occupied by trucks heavily overloaded in comparison with the design truck so as to further increase the excessive stresses in the rubber plates.

Consequently, complex factors as described above may have hastened deterioration of the rubber plates in the hinges, gaps between the female (concaved) parts and the male (convex) parts having been formed accordingly.

(2) Possible causes of the damages in the expansion joints

Following the above consideration, it can be said that;

- Once a gap was formed in the hinges, unfavorable impact forces were generated when the vehicles move from one cantilever to the other cantilever over an expansion joint, which would be rapidly deteriorated, since such impact forces were considerably larger than the forces envisaged in the design.
- The heavily overloaded trucks further increased the unfavorable impact forces, which again further deteriorated the expansion joints.
- The type of the expansion joint adopted, consisting of steel and rubber, would have been appropriate if the gap in the hinge was not formed and the trucks moving over it were not overloaded. But in the case of a gap having been formed for reasons described above, this type would be much more vulnerable than a type like the steel finger joint.

In addition, it can be said that inappropriate maintenance, which might have not been done in a timely manner, spurred the deterioration of the expansion joints. It looks from the observation of the image on the right that the fixing bolts, crucial components, might have not been adequately taken care of.

8.8.4 Conclusions

(1) Possible causes of the damages to the hinges and expansion joints

It can be said that the damages to the hinges and expansion joints on Meghna Bridge and Gumti Bridge have been caused by;

- Creep deflection of the girder which may have been unexpectedly larger than the planned,
- Heavily overloaded trucks,
- Inappropriate maintenance, which might not have been done in a timely manner.

(2) Appropriateness of the design and construction

As described in Figure 8.8.4 the adoption of a structural system with the hinges in the mid span eliminated restraint forces due to creep, shrinkage and temperature change in the structures and thus greatly reduced the sizes of the substructures / foundations. Adoption of the continuous girder with sliding bearings on the piers to eliminate the restraint forces would have greatly increased the cost because of the expensive bearings. Adoption of the continuous girders with neither the mid span hinges nor the sliding bearings on the piers would have also increased the cost because of the increased substructures / foundations due to the restraint forces. Consequently it can be said the design was the most appropriate at the time.

From the viewpoint of constructability, there have been no particular observations showing any sign of faulty construction.

CHAPTER 9

OUTLINE OF THE DESIGN

9. OUTLINE OF THE DESIGN

9.1 Road

9.1.1 Geometric Design

(1) Design Principles

The following design principles were considered for setting alignment.

- Compliance with the required parameters of the adopted design speed
- Balancing the size and length of curves
- Balancing the horizontal, vertical and cross-sectional parameters
- Method of construction
- Constraints due to natural conditions such as terrain, geological features and existing property

The horizontal alignment is finalized considering mobility, safety and comfort which are derived from balanced design parameters. In addition to the above, the following conditions are taken into account for the vertical alignment design.

- Start point and end point of the horizontal alignment follow the existing NH-1 center line.
- New alignment is next to and parallel to the existing road and bridges, therefore, the new vertical alignment is set to almost the same level as the existing to secure adequate navigation clearance under the bridge section.

(2) Geometric Design Outline

The application of the design parameters are shown in Table 9.1.1.

Table 9.1.1 Geometric Design Outline

Parameters	Unit	Applied	Kanchpur	Meghna	Gumti
Length	m		STA 0+0.0 - STA 1+100.0 Road L=703,5 (Bridge L=396.5 m)	STA 0+0.0 - STA 1+800.0 Road L=870 (Bridge L=930.0 m)	STA 0+0.0 - STA 2+420.0 Road L=1010.0 (Bridge L=1410.0 m)
Maximum Slope Height	m		12.6	10.5	6.3
Design Speed	km/h	80	80		
Lane Width	m	3.65	3.65		
Outer Shoulder	m	1.8	1.8		
Inner Shoulder	m	0.3	0.3		
Cross fall of Travel Way	%	3.0	3.0		
Maximum Super Elevation	%	7.0	7.0	7.0	7.0
Minimum R. of Horizontal Curve	m	250	340	390	350
Maximum Gradients	%	5.0	4.5	3.0	3.0
Minimum Vertical Curve K Value	m	35	37	35	74

9.1.2 Pavement Design

(1) Introduction

Pavement design is carried out based on the consideration of the constraints of the three bridge sites.

(2) Equivalent Single Axle Load (ESAL) Calculations

For pavement design, all heavy commercial vehicles are expressed in the terms of the equivalent number of standard axles that they represent. 8,160 kg is taken for a standard axle. Based on axle load studies previously undertaken in Bangladesh, the following equivalence factors have been determined.

Table 9.1.2 Vehicle Equivalent Factor

Vehicle Category	Equivalence Factor
Large Truck	4.8
Medium Truck	4.62
Small Truck	1.0
Large Bus	1.0
Mini Bus	0.5

The cumulative ESAL for design is calculated using the following equation;

ESAL = Traffic volume of vehicle type for 20 years x DDF x LDF x VEF

DDF: Directional Distribution Factor = 0.5

LDF: Lane Distribution Factor = 0.7 for 3-lanes, 0.6 for 4-lanes

VEF: Vehicle Equivalent Factor

Table 9.1.3 Summary of ESAL

Section	10 years (million)	20 years (million)
Kanchpur site	102	207
Meghna, Gumti site	97	229

(3) Flexible Pavement Design

Flexible Pavement Design is carried out by two methods namely;

- AASHTO Guide for the Design of Pavement Structures
- RHD's Pavement Design Manual

(4) AASHTO Design Method

The AASHTO method for the design of pavement structure is based on the consideration that a pavement life is a function of;

- Traffic loading
- Pavement strength
- Pavement serviceability
- Sub-grade strength

The relationship between the above factors can be mathematically represented as;

Log ESAL = Z.S. + f_1 (SN) – 0.20 + f (P_0 – P_1) / f_2 (SN) + f (M_R)

Where

ESAL: Cumulative Equivalent Single Axle Loads

Z.S.: Adjustment factor to offset unreliability of traffic volumes and serviceability

SN: Structural Number

P_0: Initial Serviceability

P1: terminal serviceability

MR: resilient modulus of sub-grade

SN: $a_1D_1 + a_2D_2m_2 + a_3D_3m_3$

a: layer coefficient

D: layer thickness in inches

m: drainage coefficient of base and sub-base

The structural number (SN) is represented by the sum total of the multiplication of the layer coefficient, the thickness in inches, and drainage coefficient of each layer in the pavement structure.

The AASHTO equation can be solved by mean of a nomograph include in the AASHTO Guide for Design of Pavement method that involves certain input variables, which are as follows;

- Traffic or Cumulative Equivalent Single Loads (ESAL)
- Reliability (R)
- Standard Deviation (S_0)
- Resilient Modulus of Subgrade (M_R)
- Loss in Serviceability ($\triangle$PSI)
- Structural Layer Coefficients

1) Design Life

Design life is the number of years reckoned from the completion of pavement construction and application of traffic load until the time when major maintenance is required so that it can continue to carry traffic satisfactorily for a further period. A design life of 20 years is adopted for this project and is recommended in general as per Pavement Design Guide (RHD). For comparison of design life between 20 years and 10 years refer to Appendix 8.

2) Design CBR

A CBR value of 5 % is recommended in general as per the Pavement Design Guide (RHD). The CBR values at the site surveyed range from 4 to 8 and the average CBR at the site is beyond 5 %.

The soil parameter used in the AASHTO Guide for Pavement Design is the Modulus of Resilience (M_R). Therefore the design CBR needs to be converted into the corresponding M_R value. There is a widely used correlation for CBR and M_R up to 10. The correlation is below.

M_R=1500*CBR

3) Thickness Design

- Design Parameters

20 years ESAL	= 229 Million
Modulus of Resilience (MR)	= 12000 Psi
Reliability (R)	= 95 %
Standard Deviation (So)	= 0.45
Initial Serviceability Index (Po)	= 4.2
Terminal Serviceability Index (Pt)	= 2.5

- Design Thickness

Bituminous Wearing Course	5 cm
Bituminous Binder Course	20 cm
Aggregate Base Course	35 cm
Aggregate Subbase	40 cm

For detailed calculations, refer to Appendix 8.

(5) RHD Design Manual

The subject design guide is a simplified version of various international standards with due modifications keeping in view local conditions and practices. The thickness design shown in the table provides variable thicknesses of bituminous pavement, base and subbase for discrete ranges of ESAL from 3-4 million to 60-80 million. The target ESAL is beyond the range of ESAL value in the RHD Design Manual, therefore pavement thickness is adopted from the AASHTO Design Method.

9.1.3 Drawing

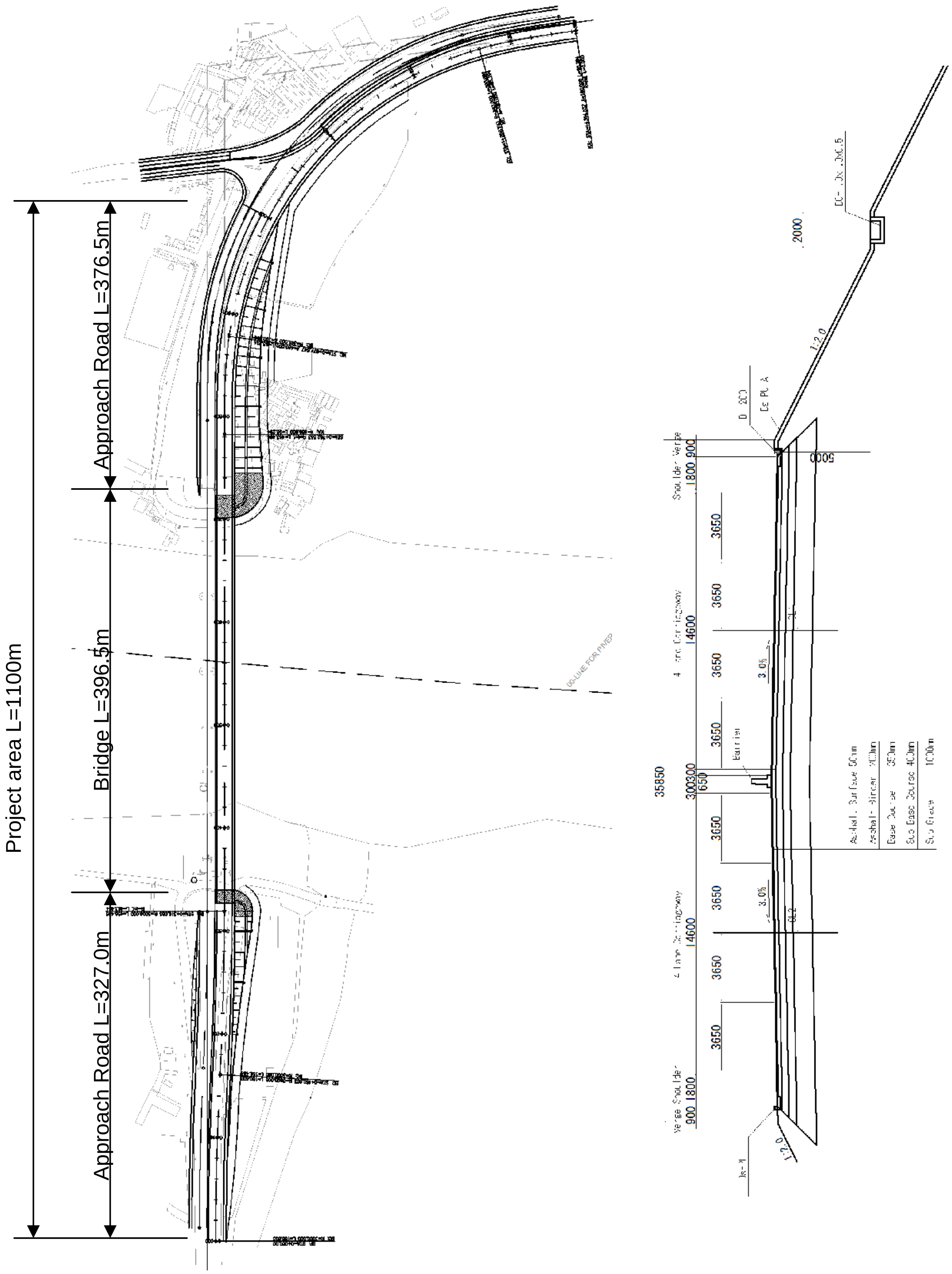


Figure 9.1.1 Plan and Typical Cross Section at Kanchpur

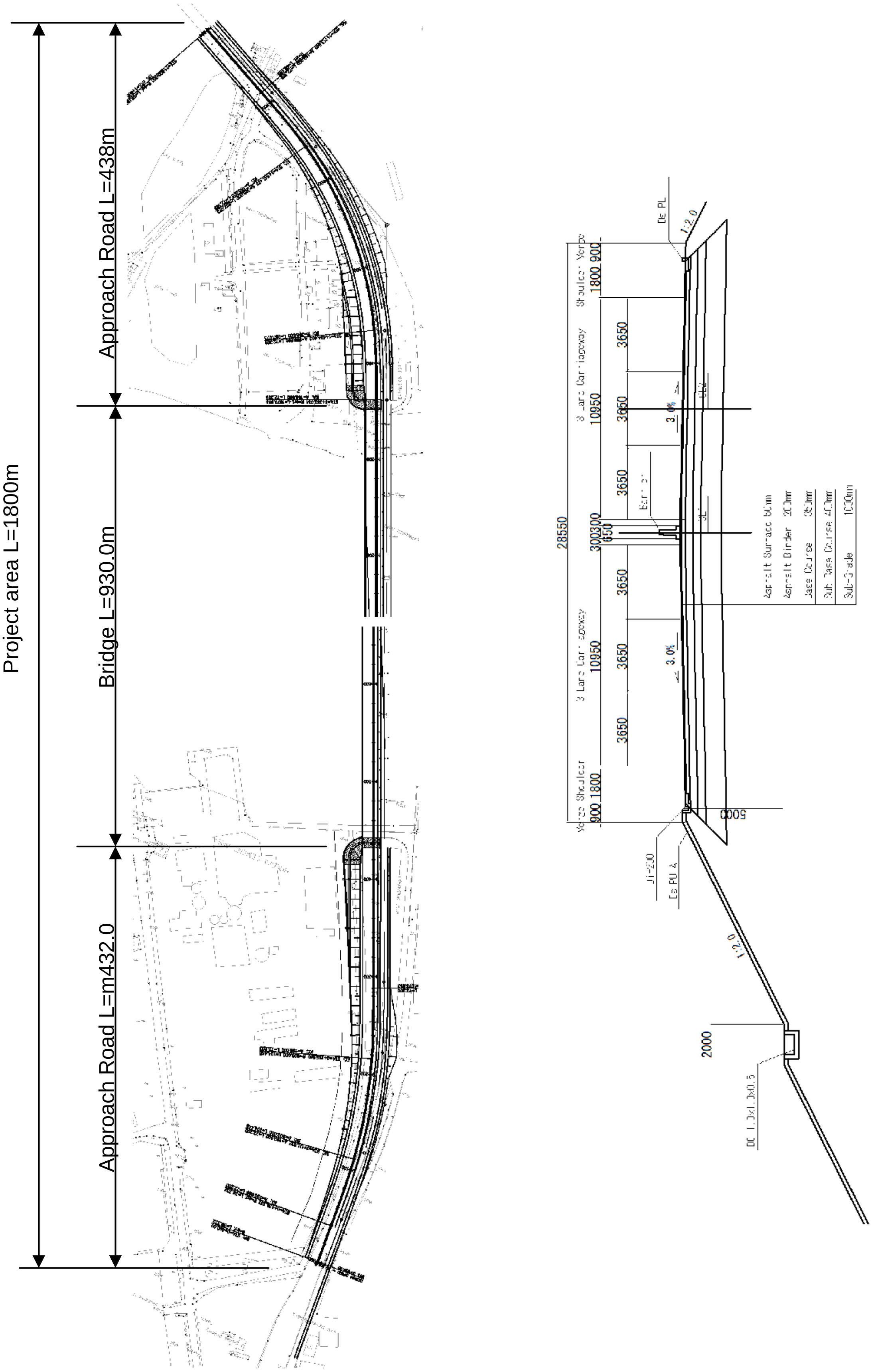


Figure 9.1.2 Plan and Typical Cross Section at Meghna

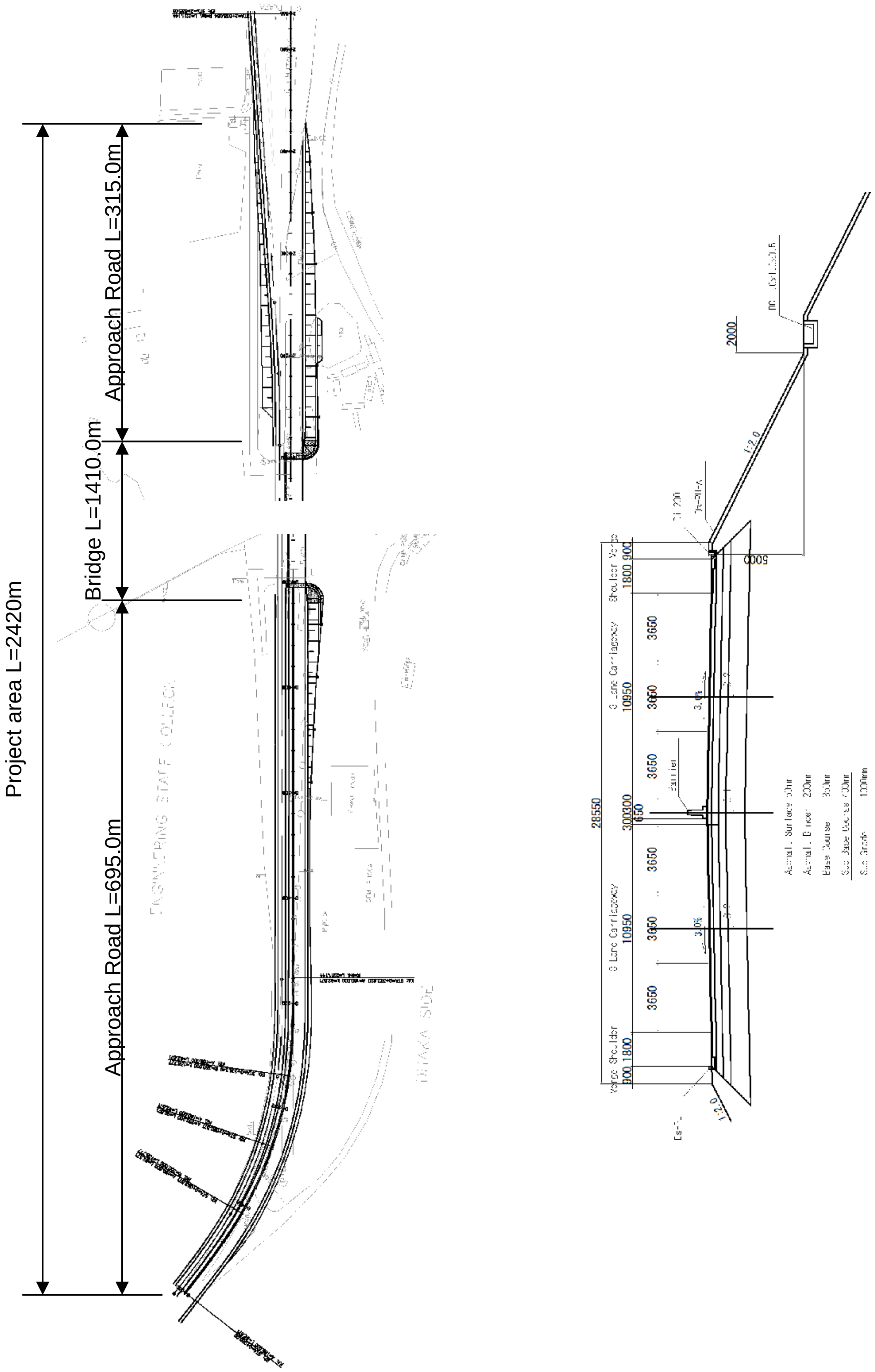


Figure 9.1.3 Plan and Typical Cross Section at Gumti

9.2 The 2nd Bridges

As per the clarification letter No.35.033.014.00.00.007.2012 -161 issued by Ministry of Communication (MoC) of Bangladesh on 13th August 2012, the type of the 2nd Bridges would be 'Steel Narrow Box Girder Bridge'. The reason behind this bridge type selection is its relatively least construction cost, least construction period and constructability. The Steel box girders have been chosen for the superstructure of the 2nd bridges; therefore the span for the 2nd Kanchpur Bridge will be larger than that of the existing one. But, for the 2nd Meghna and 2nd Gumti Bridges, the span allocation is kept the same as the existing one. This is because the same span allocation will ensure the same number of foundations for the 2nd and existing bridges. Therefore, it will be easier to unify the new foundations with the adjacent existing foundations, which will secure the least construction cost and minimize the scouring in the riverbed.

(1) Superstructure design

For the superstructure, steel narrow box girders is chosen in order to reduce the superstructure weight and increase the earthquake resistance capacity of the bridges.

(2) Foundation design

In order to determine the foundation type for the 2nd bridges, the following factors are taken into consideration.

-Design scouring level is revised in consideration of new research results and a river study.

-Cost factor

Cast-in-place RC pile foundation and Steel Pipe Sheet Pile (SPSP) foundation are taken into consideration and they are compared with respect to the cost aspect. These factors are studied for the respective 2nd Bridges and the computed results are shown in the following sections.

9.2.1 The 2nd Kanchpur Bridge

(1) Design Scouring Level

Using the bathymetric survey results, the scouring level along the river cross-section at the bridge center line is estimated and graphically shown as blue line in Figure 9.2.1. This figure also shows the design scouring level at each respective pier. The design scouring levels (red line) are categorized as follows;

- Shallow scouring: RL= - 1.4m
- Severe scouring: RL= - 18.0 m

It can be observed that piers P1, P2 and P3 are located in shallow scouring zones (RL=-1.4m), whereas piers P4, P5 and P6 are located in severe scouring zones (RL=-18.0m). The foundation type for each pier is examined in accordance with the design local scouring level.

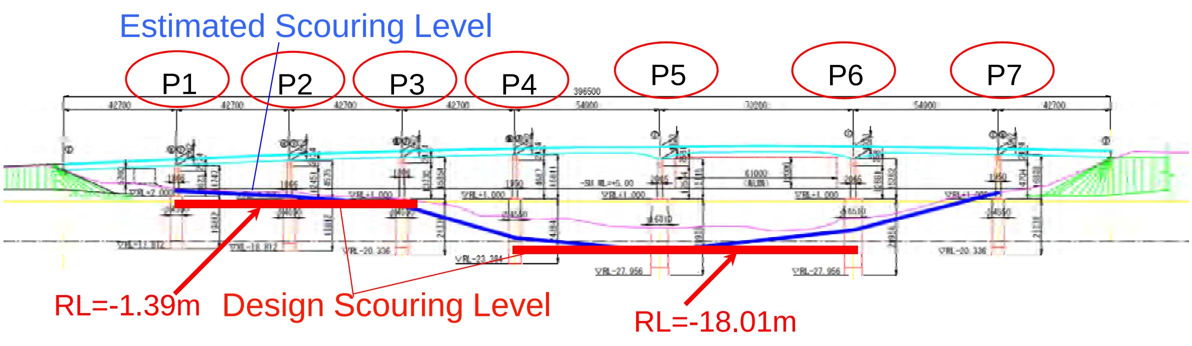


Figure 9.2.1 Riverbed Scouring and Design Scouring Level for the 2nd Kanchpur Bridge

(2) Foundation type selection

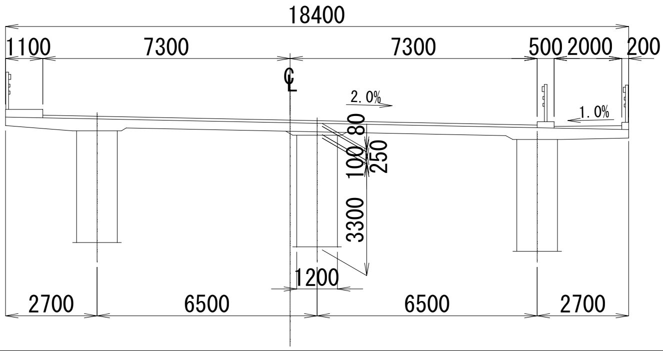
It can be seen from Figure 9.2.1 that the foundations of piers P4, P5 and P6 of 2nd and the existing bridges are located in severe scouring zones. A cast-in-place RC pile foundation and Steel Pipe Sheet Pile (SPSP) foundation are examined considering the severe scouring level (RL=-18.0m) parameter and then, their construction cost is compared as already seen in Table 8.5.9. It can be found that the construction of SPSP foundation in a severe scouring zone is cost effective. Moreover, implementing SPSP technology is a countermeasure to scouring.

It is also decided that the foundations for piers P1, P2 and P3, which are located in areas with shallow riverbed scouring should also be SPSP type. This is just to maintain conformity with the existing caisson foundation that needs to be retrofitted. Therefore, for the 2nd Kanchpur Bridge, the foundation of piers P1-to-P6 will be the SPSP type and their foundations will be unified with those of the adjacent existing foundations. On the other hand, the foundation of pier P7 only will be a cast-in-place RC pile type which is designed as a single type foundation that means that there will be no interference with the existing foundation.

(3) Results of the Study

The general outline of the design is summarized in Table 9.2.1. Consequently, the general view of the 2^{nd} Kanchpur Bridge is schematically shown in Figure 9.2.2.

Table 9.2.1 Outline of Design (The 2nd Kanchpur Bridge)

Bridge type				Continuous steel narrow box girder
Configuration of bridge superstructure				Bridge length = 396.5 m Girder length = 396.0m Span = 41.6 m + 85.4 m + 97.6 m + 73.2 m + 54.9 m + 41.6 m
Number of lanes				4-lanes
Cross section				18.4 m (1.1 m (sidewalk) + 14.6 m (road) + 2.7 m (sidewalk)) 
Bridge components	Superstructure			3-box girders (1.2 m x 3.3 m) with PC floor slab (t=25 cm)
	Substructure	Abutment	Inverted T-type	Number: 2 Height: 7.5 m
		Piers	Columnar type	Number: 5 Height: 10.64 m - 15.55 m
		Foundation	Abutment	Cast-in-place RC pile (φ1.5 m): A1&A2; n=6, L=36.0 m
			Piers	Cast-in-place RC pile (φ1.5 m): P7; n=8, L=18.0 m
				SPSP (φ1.0 m) P1 to P5; max 34.94 m x 11.23 m (including existing) min 33.7 m x 8.74 m (including existing) L = 33.0 m
Loads	Live load			JRA B-type (only for floor slab system) AASHTO HS20-44 (for girder and substructure)
	Seismic load			$C_{sm} = \frac{1.2ZS}{T_m^{2/3}} \le 2.5Z$; where Z=0.15 and S=1.5
	Wind load			3.0 kN/m^2
	Thermal force			Temperature range: +10 ^{o}c to +50 °c
Construction materials				Specification
	Superstructure	Steel		Grade-SM490A (JIS) σ_u = 490 MPa, σ_y = 315 MPa
		PC steel bar		Grade-SWPR7BL (JIS) σ_u = 1,850 MPa, σ_y = 1,600 MPa
		Concrete (Precast)		JIS σ_c = 50 MPa
	Substructure	Concrete (cast-in-place)		RHD σ_c = 25 MPa
		Rebar		Grade-60 (ASTM) σ_u : 620 MPa, σ_y : 420 MPa
		SPSP		SKY400 and SKY490 (JIS) σ_u = 400 MPa and 490 MPa σ_y = 235 MPa and 315 MPa

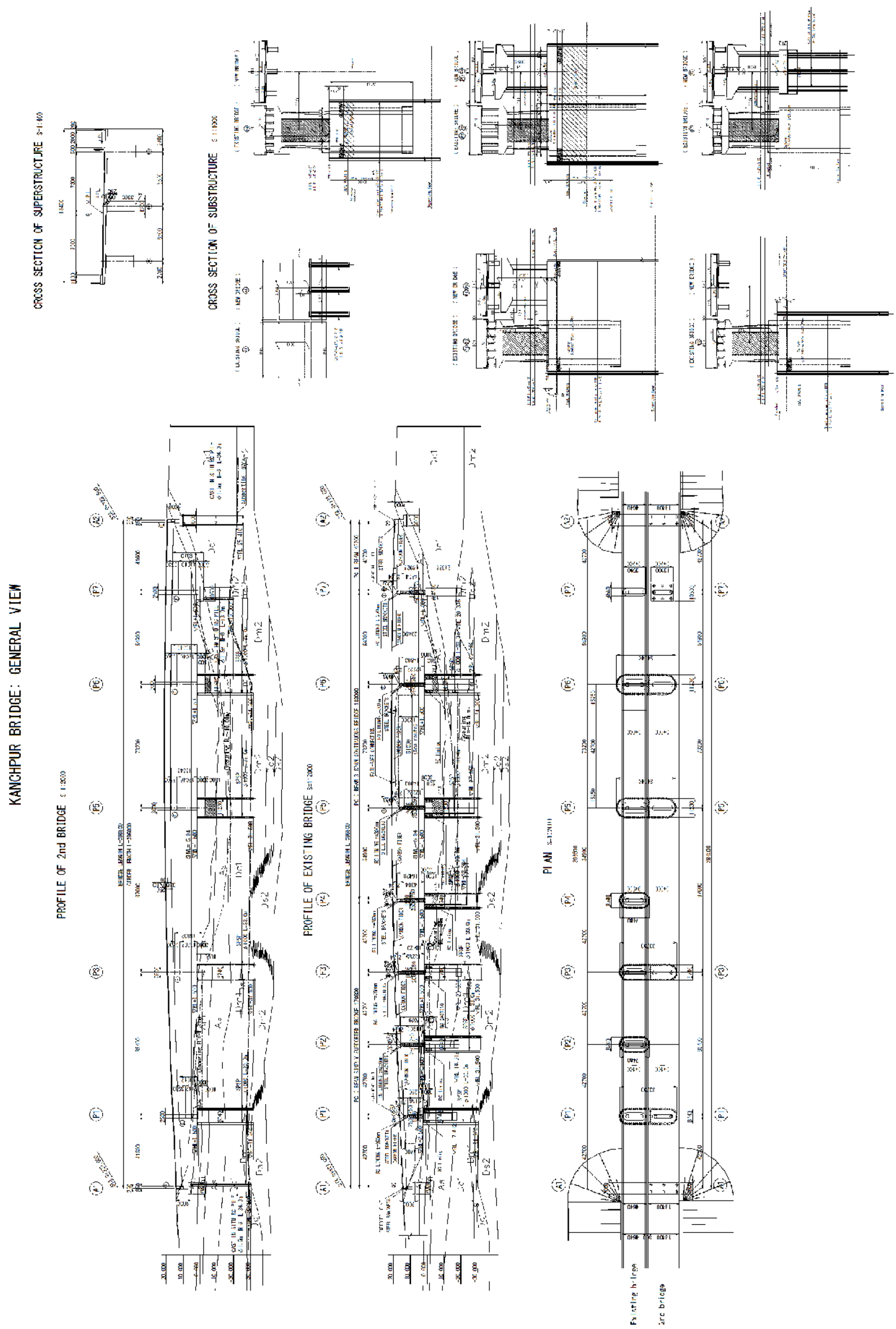


Figure 9.2.2 General View of Kanchpur Bridge

Figure 9.2.3 CG of Completed the 2nd Kanchpur Bridge (Dhaka side)

9.2.2 The 2nd Meghna Bridge

(1) Design Scouring Level

Using the bathymetric survey results, the scouring level along the river cross-section is estimated and graphically shown as blue line in Figure 9.2.4. This figure also shows the design scouring level for each respective pier. The design scouring levels (red line) are categorized as follows;

- Shallow scouring : RL=-4.6 m
- Medium scouring : RL=-14.90 m
- Severe scouring : RL=-26.2 m

It can be observed that piers P1 and P2 are in shallow scouring zones (RL = -4.6 m), whereas piers P3-P5 are in medium scouring zones (RL = -14.90) and P6-P11 are in severe scouring (RL = -26.2 m) zones. The foundation type for each pier is examined in accordance with these three categorized design scouring levels.

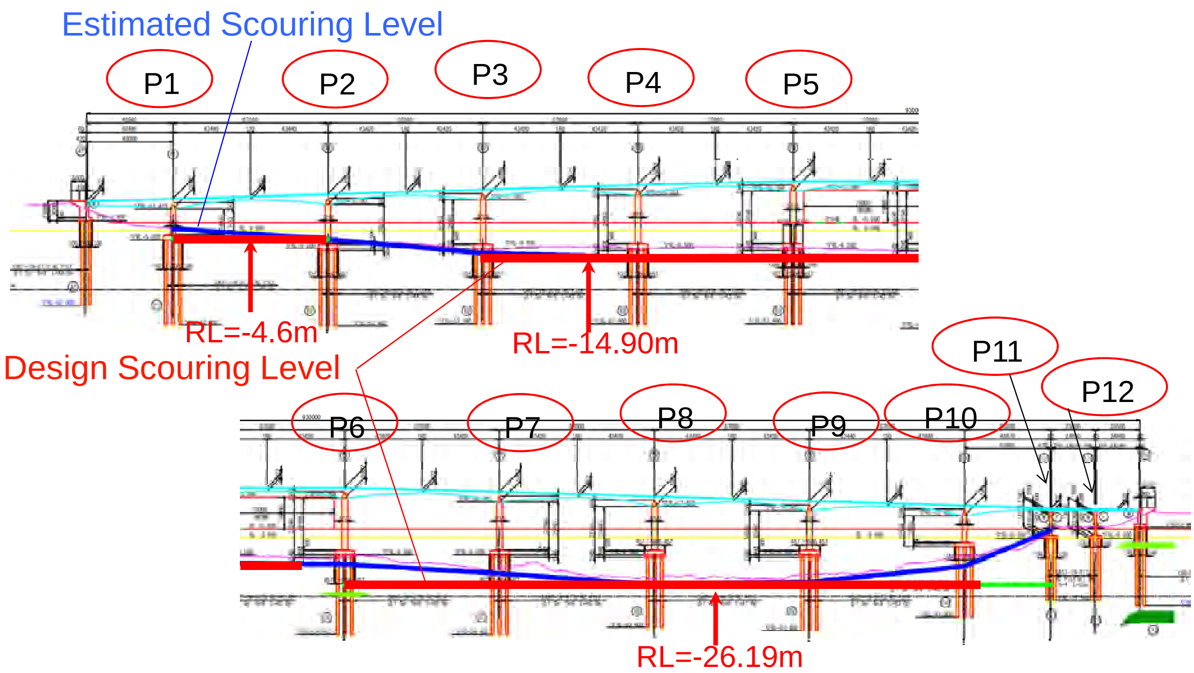


Figure 9.2.4 Riverbed Scouring and Design Scouring Level for the 2nd Meghna Bridge

(2) Foundation type selection

1) In case of shallow scouring

The foundation of pier P2 (new bridge) is close and parallel to that of P2 (existing bridge). Their foundations will be combined so as to secure the least construction cost. It is also observed from Figure 9.2.4 that they are located in a shallow scouring zone. A cast-in-place RC pile foundation and Steel Pipe Sheet Pile (SPSP) foundation are examined considering the shallow scouring zone (RL = -4.6 m) and then, their construction costs are compared as already seen in Table 8.6.5. The construction of cast-in-place RC pile foundation in a shallow scouring zone is cost effective.

2) In case of medium and severe scouring

It can be observed from Figure 9.2.4 that the foundations of pier P5 (2nd and existing bridges) are located in a medium scouring zone. A cast-in-place RC pile foundation and Steel Pipe Sheet Pile (SPSP) foundation are also examined considering the medium scouring level (RL=14.90 m) and then, their construction costs are compared as already seen in Table 8.6.6. From this comparison, it can be seen that the construction of an SPSP foundation in a medium scouring zone is cost effective. This is because no temporary cofferdam is required to construct an SPSP foundation and thereby it secures the least construction cost.

The foundation selected for a medium scouring zone is the SPSP type, therefore, the foundation for a severe scouring zone undoubtedly will be evaluated as an SPSP type. Therefore, no additional comparison between the said foundations in severe scouring zones needs to be shown.

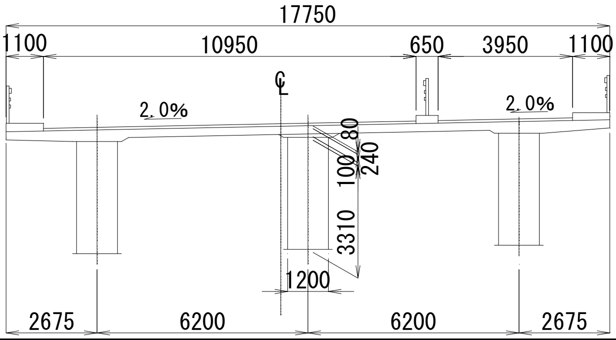
(3) Remarks on foundation type

From the above results, it can be concluded that where the riverbed scouring level is medium to severe, the SPSP type foundation will be economical and additionally it will act as a scouring countermeasure. On the other hand, where the riverbed scouring level is shallow, a cast-in-place RC pile foundation will be cost effective.

(4) Results of the Study

The general outline of the design and the obtained output results are summarized in Table 9.2.2. Consequently, the general view of the 2nd Meghna Bridge is schematically shown in Figure 9.2.5.

Table 9.2.2 Outline of Design (The 2nd Meghna Bridge)

Bridge type				Continuous steel narrow box girder
Configuration of bridge superstructure				Bridge length= 930.0 m Girder length= 929.1 m Span= 47.4 m +9@87.0 m + 73.5 m + 23.9 m
Number of lanes				4-lanes
Cross section				17.75 m (1.1 m (sidewalk) + 15.55 m (road) + 1.1 m (sidewalk)) 
Bridge components	Superstructure			3-box girders (1.2 m x 3.31 m) with PC floor slab (t=24 cm)
	Substructure	Abutment	Inverted T-type	Number: 2 Height: 8.0 m - 9.5 m
		Pier	Columnar type	Number: 11 Height: 9.9 m - 30.44 m
		Foundation	Abutment	Cast-in-place RC pile (φ1.5 m): A1&A2; n=6, L=48.0 m
			Pier	Cast-in-place RC pile (φ1.5 m): P1,P2&P12; n=6-12, L=35.0 m-44.0 m
				SPSP (φ1.0 m) P3-P10; 39.93 m x 14.97 m (including existing) L = 42.65 - 44.15 m
Loads	Live load			JRA B-type (only for floor slab system) AASHTO HS20-44 (for girder and substructure)
	Seismic load			$C_{sm} = \frac{1.2ZS}{T_m^{2/3}} \leq 2.5Z$; where Z=0.15 and S=1.5
	Wind load			3.0 kN/m^2
	Thermal force			Temperature range: +10 ^{o}c to +50 °c
Construction Materials	Superstructure			Specification
		Steel		Grade-SM490A (JIS) σ_u = 490 MPa, σ_y = 315 MPa
		PC steel bar		Grade-SWPR7BL (JIS) σ_u = 1,850 MPa, σ_y = 1,600 MPa
		Concrete (Precast)		JIS σ_c = 50 MPa
	Substructure	Concrete (cast-in-situ)		RHD σ_c = 25 MPa
		Rebar		Grade-60 (ASTM) σ_u : 620 MPa, σ_y : 420 MPa
		SPSP		SKY400 and SKY490 (JIS) σ_u = 400 MPa and 490 MPa σ_y = 235 MPa and 315 MPa

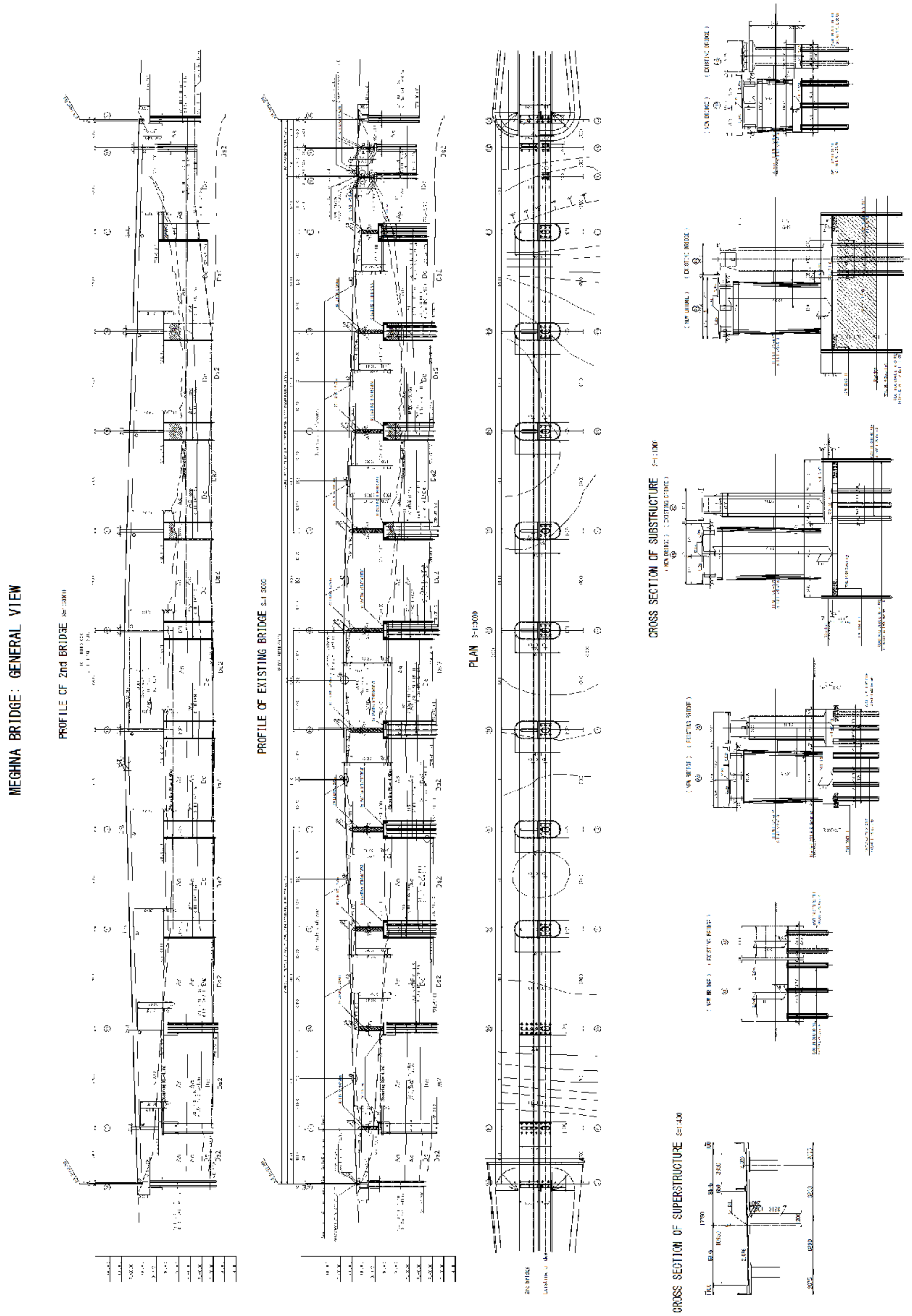


Figure 9.2.5 General View of Meghna Bridge

Figure 9.2.6 CG of Completed the 2nd Meghna Bridge (Chittagong side)

9.2.3 The 2nd Gumti Bridge

(1) Design Scouring Level

Using the bathymetric survey results, the scouring level along the river cross-section is estimated and graphically shown as blue line in Figure 9.2.7. This figure also shows the design scouring level (red line) for each respective pier. The design scouring levels are categorized as follows;

- Shallow scouring : RL= -0.7 mto -2.8 m
- Severe scouring : RL= -17.1 m

It can be observed that the piers P1-P8 are in severe scouring zones (RL = -17.1 m), whereas piers P9-P16 are located in shallow scouring zones (RL = -0.7 to -2.8 m). The foundation type for each pier is examined in accordance with these two categorized scouring levels.

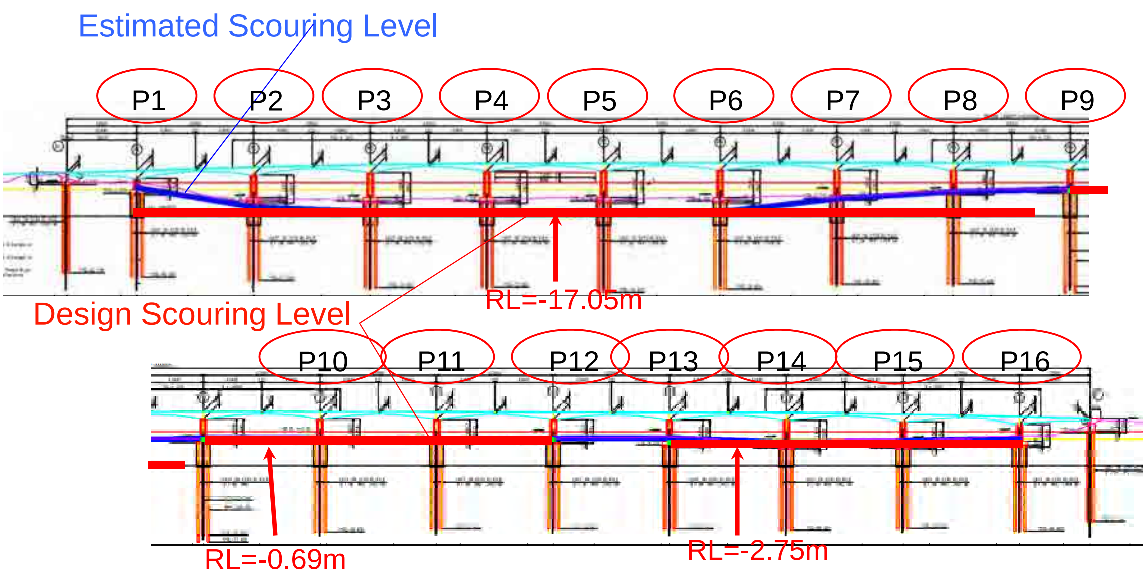


Figure 9.2.7 Riverbed Scouring and Design Scouring Level for 2nd Gumti Bridge

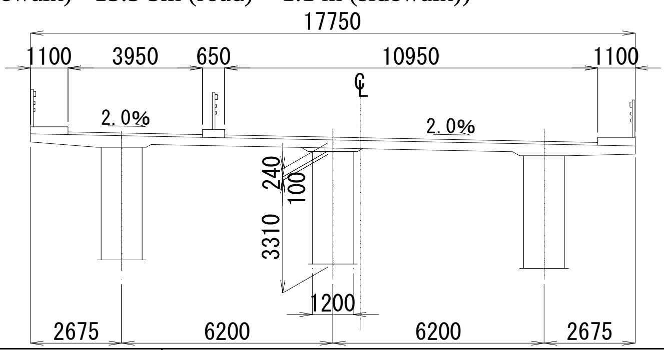
(2) Foundation type selection

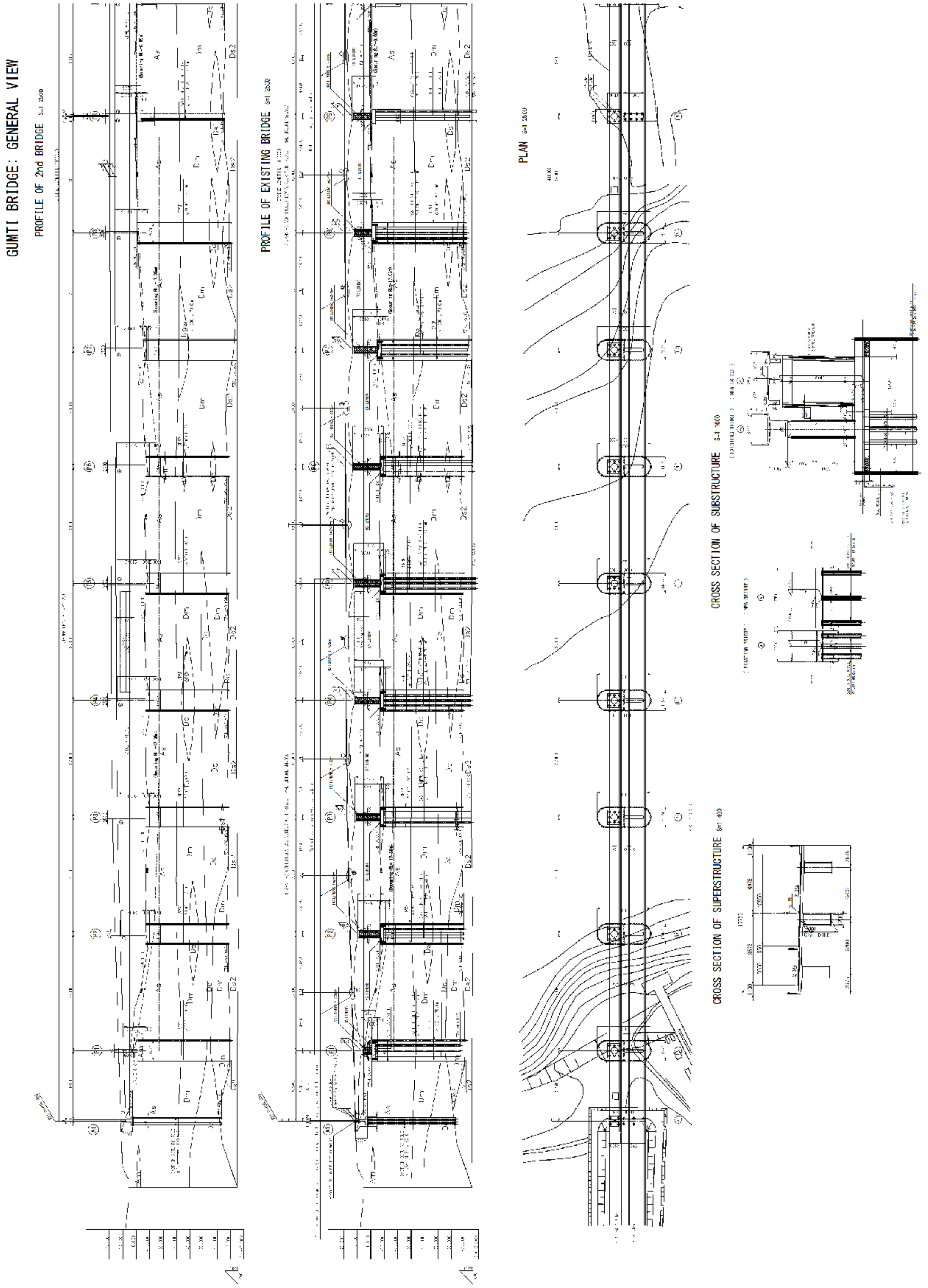
As is already stated for the 2nd Meghna Bridge, a combined cast-in-place RC pile foundation will secure least construction cost where the riverbed scouring is shallow, whilst the combined SPSP foundation is cost effective where the riverbed scouring is severe. The same results are also observed for the 2nd Gumti Bridge. Therefore, it is decided that the foundations for piers P1-P8 in severe scouring zones will be designed as SPSP foundations and that for piers P9-P16 in shallow scouring zones will be designed as cast-in-place RC pile foundation. The foundations of piers P1-to-P8 of the new bridge will be unified with that of the corresponding existing pier. But, the remaining foundations will be single type foundations and there will be no interference with the existing foundations.

(3) Results of the Study

The general outline of the design and the design outputs are summarized in Table 9.2.3. Consequently, the general view of 2nd Gumti Bridge is schematically shown in Figure 9.2.8.

Table 9.2.3 Outline of Design (The 2nd Gumti Bridge)

Bridge type				Continuous steel narrow box girder
Configuration of bridge superstructure				Bridge length = 1,410 m Girder length = 747.75 m + 660.75 m Span = 51.4 m + 7@87.0 m + 86.15 m, 86.15 m + 6@87.0 m + 51.4 m
Number of lanes				4-lanes
Cross section				17.75 m (1.1 m (sidewalk) +15.5 5m (road) + 1.1 m (sidewalk)) 
Bridge components	Superstructure			3-box girders (1.2 m x 3.31 m) with PC floor slab (t=24 cm)
	Substructure	Abutment	Inverted T-type	Number: 2 Height: 7.5 m - 9.5 m
		Pier	Columnar type	Number: 16 Height: 9.6 m - 21.60 m
		Foundation	Abutment	Cast-in-place RC pile (φ1.5 m): A1&A2; n=6, L=66.0-m
			Pier	Cast-in-place RC pile (φ1.5 m): P9-P16; n=8, L=62.0 m-
				SPSP (φ1.0 m) P1-P8; 34.93 m x 14.97 m (including existing) L=62.0 m - 70.0 m
Loads	Live load			JRA B-type (only for floor slab system) AASHTO HS20-44 (for girder and substructure)
	Seismic load			$C_{sm} = \frac{1.2ZS}{T_m^{2/3}} \le 2.5Z$; where Z=0.15 and S=1.5
	Wind load			3.0 kN/m^2
	Thermal force			Temperature range: +10 ^{o}c to +50 °c
Construction Materials				Specification
	Superstructure	Steel		Grade-SM490A (JIS) σ_u = 490 MPa, σ_y = 315 MPa
		PC steel bar		Grade-SWPR7BL (JIS) σ_u = 1,850 MPa, σ_y = 1,600 MPa
		Concrete (Precast)		JIS σ_c = 50 MPa
	Substructure	Concrete (cast-in-situ)		RHD σ_c = 25 MPa
		Rebar		Grade-60 (ASTM) σ_u : 620 MPa, σ_y : 420 MPa
		SPSP		SKY400 and SKY490 (JIS) σ_u = 400 MPa and 490 MPa σ_y = 235 MPa and 315 MPa


GUMTI BRIDGE: GENERAL VIEW
PROFILE OF 2nd BRIDGE
PROFILE OF EXISTING BRIDGE
PLAN
CROSS SECTION OF SUBSTRUCTURE
CROSS SECTION OF SUPERSTRUCTURE

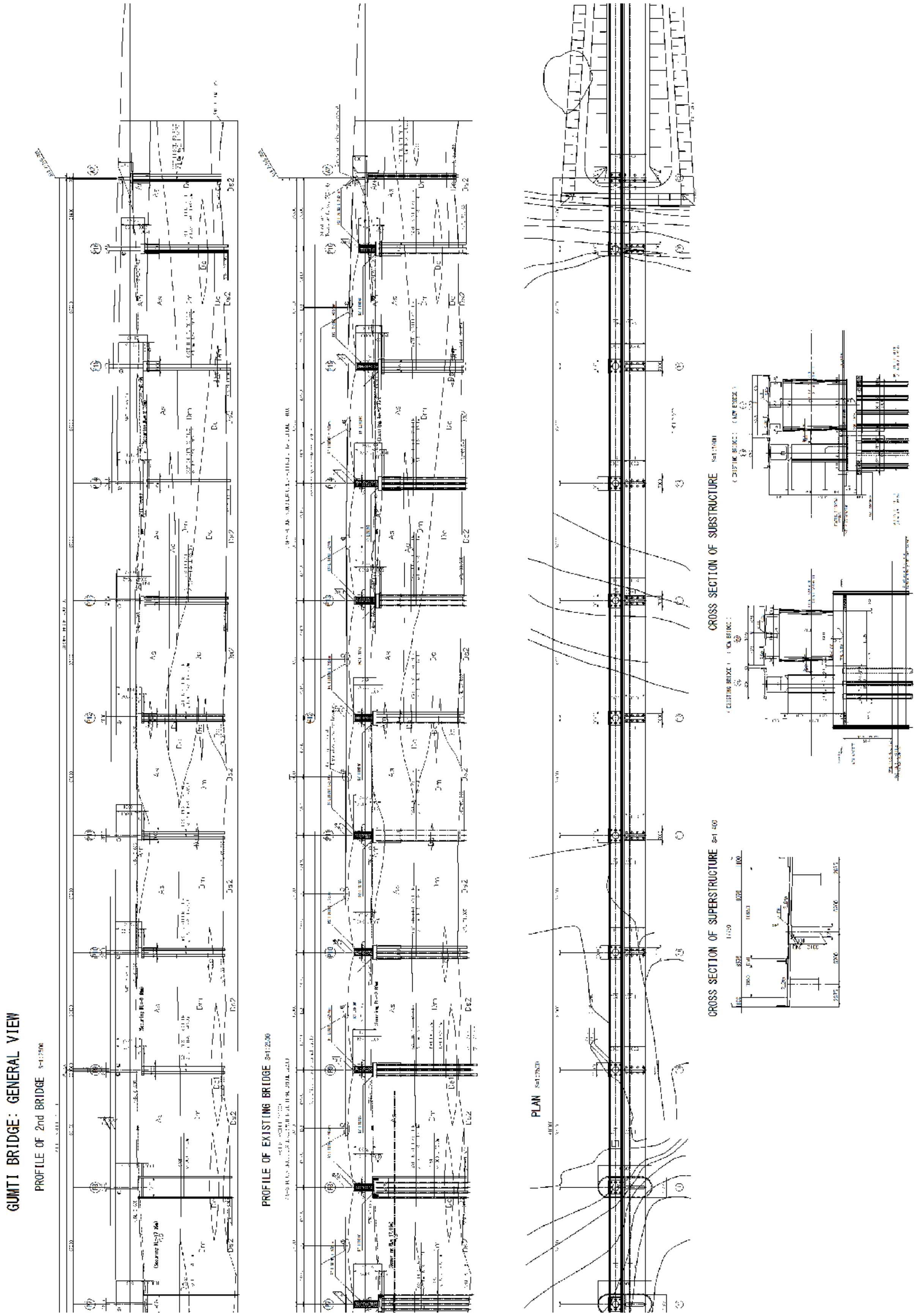


Figure 9.2.8 General View of Gumti Bridge

Figure 9.2.9 CG of Completed the 2nd Gumti Bridge (Dhaka side)

9.2.4 Bridge Accessories

As the bridges are planned as the steel structure, therefore the bridge accessories should be considered as water proof at the design stage in order to avoid corrosion in the girders.

(1) Expansion joints

Table 9.2.4 Horizontal Movement of Expansion Joint

Bridge	Horizontal movement	Type
Kanchpur	±170 mm	Finger non-drainage type
Meghna	±400 mm	
Gumti	±300 mm	

The expansion joints should be designed as non-drainage type in order to avoid the water leakage into the end of girder and also near the abutment. The water will be passed through drainage system from the outside of the girder and from the water tight rubber drain under the fingers.

The finger type is chosen from the view point of durability against heavy loading trucks. In that case, the steel finger is more durable than the rubber top expansion joints.

(2) Water drain system

The road surface catchment should be located at the end of surface and the collected water will directly drop to the river. The pipe end is set 1 m below the girder in order to avoid water spray back to the girder.

Table 9.2.5 Drainage System (Direct Drop)

Item	Material	Remarks
Catchment drain	Cast Steel	Buried in the PC deck
pipe	Vinyl chloride pipe	
Pipe support	Steel	Directly fixed to the girder

(3) Bearing shoes

The bearing shoes are rubber bearings for the movement and seismic resistance. The durability of rubber bearing is secured by a lapping layer over the rubber against ultraviolet for 100 years. And the rust prevention for bearing accessories should be achieved by metallic thermal spraying which is expected to be lasting as 100 years.

Table 9.2.6 Bearing Movement

Bridge	Movement	Maximum Reaction Force
Kanchpur	±170 mm	1250 t
Meghna	±400 mm	1150 t
Gumti	±300 mm	1300 t

(4) Inspection way

The inspection ways should be made of galvanized steel structure in order to inspect the piers and the girders at some time intervals.

- Type: Girder with railing
- Size: Width 60 cm, railing height 120 cm
- Material: Galvanized steel

9.3 Existing Bridges

9.3.1 General

The present conditions observed and investigated have been examined and the latest design criteria to be conformed to have also been considered. As a result, the existing bridges, Kanchpur Bridge, Meghna Bridge and Gumti Bridge have been determined to need to be retrofitted and rehabilitated in the manner described as follows.

9.3.2 Scope of the Retrofitting and Rehabilitation Works

It has been determined that the scope of the rehabilitation works shall be defined as the repair works for the damaged expansion joints in Kanchpur Bridge, Meghna Bridge and Gumti Bridge and the repair works for the damaged hinges in Meghna Bridge and Gumti Bridge.

The retrofitting works shall be defined as the works to renew the bridge structures to conform to the current bridge design standards and to cope with the current and future scouring conditions. As mentioned in the previous chapter the changes of the design seismic force should be reflected in all the bridges and the changes of the live load should be reflected in Kanchpur Bridge.

The repair works for the damaged hinges shall be connecting two cantilever girders to become monolithic, eliminating the hinge and expansion joint, at each hinge section. As the hinge section shall remain in every 5 or 6 spans to avoid excessive restraint forces to be generated by the temperature change, the hinges and expansion joints at such sections shall be replaced by new ones.

To prevent the bridge from collapsing during earthquake the steel brackets will be attached to the substructures at the girder ends. At the having joint of Kanchpur Bridge the girder ends are connected to each other (Fail-safe connection).

Consequently, the scope of the retrofitting and rehabilitation works for Kanchpur Bridge, Meghna Bridge and Gumti Bridge are summarized below.

Table 9.3.1 Scope of the Rehabilitation and Retrofitting Works

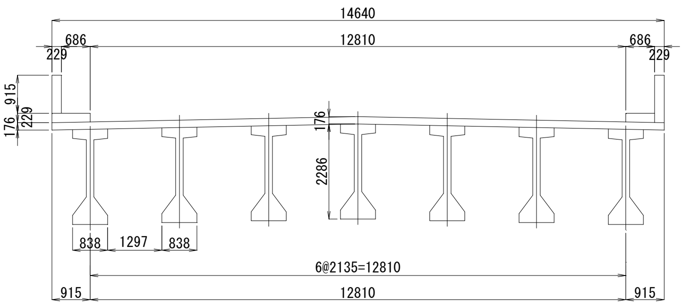
		Kanchpur	Meghna	Gumti
Repair of cracks/rebar exposures		○	○	○
Connecting girders (eliminating hinges/joints)		-	○	○
Center hinge rehabilitation		-	○	○
Expansion joint replacement		○	○	○
Steel brackets on the substructures		○	○	○
Fail-safe connection		○	-	-
Deck strengthening		○	-	-
Piers	RC-lining	○	○	○
	Diaphragm wall	○	-	-
Foundations	Pile cap integration	P1, P3, P5, P6	P1 to P10	P1 to P8
	Steel pipe sheet piles	P1 to P6	P3 to P10	P1 to P8
	Bored RC piles	-	P1, P2	-

9.3.3 Retrofitting and Rehabilitation Works

(1) Existing Kanchpur Bridge

The general outline of the design and the design outputs are summarized in Table 9.3.2.

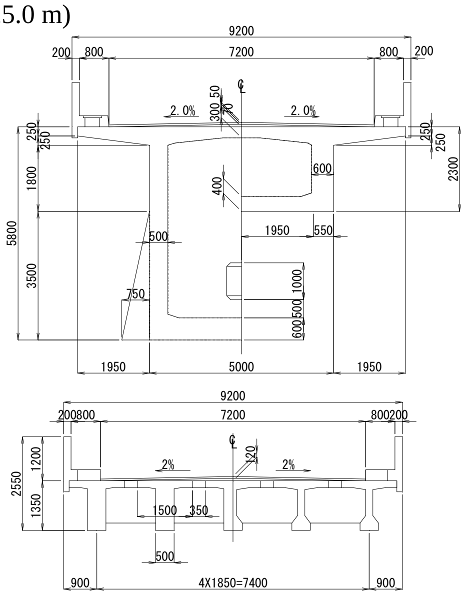
Table 9.3.2 Outline of Design (Existing Kanchpur Bridge)

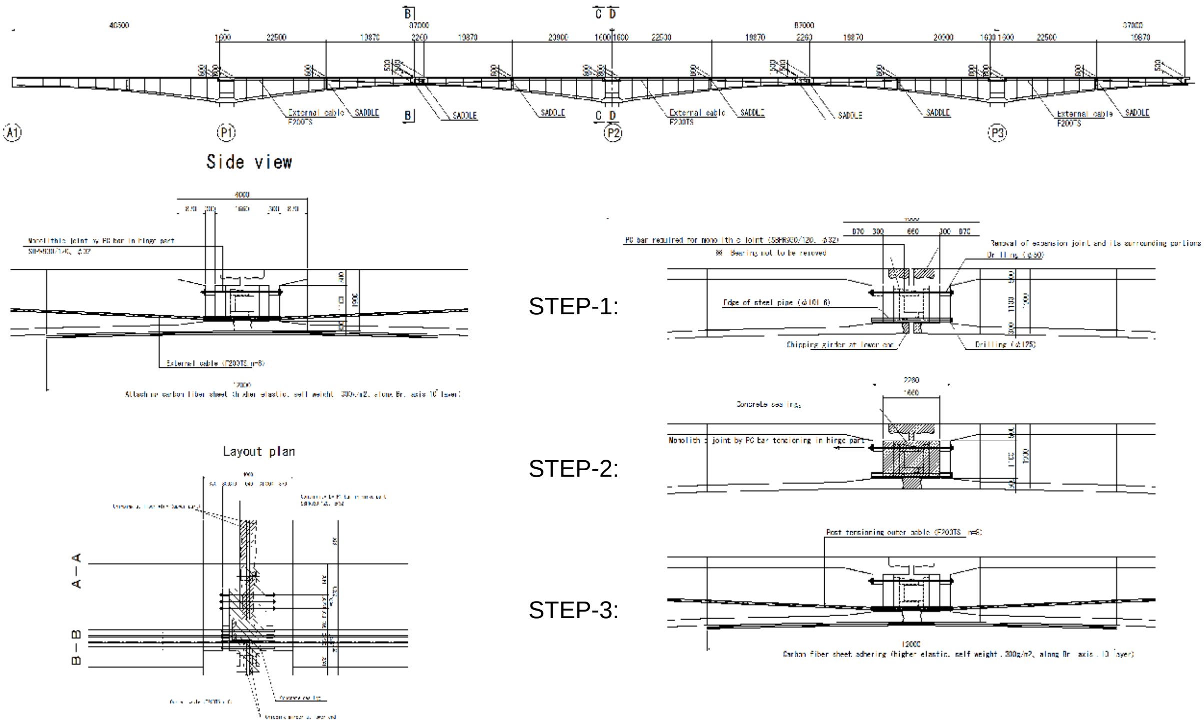
Bridge data			Length = 395.6 m : 4@42.7 m + 54.9 m + 73.2 m + 54.9 m+42.7 m Width = 14.64 m
Bridge components	Superstructure		Pre-stressed concrete I girder, continuous type (54.9 m + 73.2 m + 54.9 m) Pre-stressed concrete I girder, simply supported type (42.7 m5 spans) 
	Substructure	Abutment	Inverted T-type
		Pier	Rigid frame-type
		Foundation	Abutment: Spread foundation Pier: RC open caisson
Loads	Live load		JRA B-type (only for floor slab system) AASHTO HS20-44 (for girder and substructure)
	Seismic load		$C_{sm} = \frac{1.2ZS}{T_m^{2/3}} \le 2.5Z$; where Z=0.15 and S=1.5
	Wind load		3.0 kN/m^2
	Thermal force		Temperature range: $26^{\circ}c \pm 17^{\circ}c$
Girder and deck	Repair of cracks/rebar exposures		N/A
	Deck strengthening		Carbon fiber sheet adhering A1 to A2
Bridge accessories	Expansion joint replacement		A1 to P4, P7, A2
	Steel brackets		A1 to P4, P7, A2
Pier	RC-lining		P1 to P7
	Diaphragm wall		P1 to P7
Foundation	Steel pipe sheet piles		P1 to P6

(2) Existing Meghna Bridge

The general outline of the design and the design outputs are summarized in Table 9.3.3. The connection of the hinges should be the same as Figure 9.3.1.

Table 9.3.3 Outline of Design (Existing Meghna Bridge)

Bridge data			Length = 930 m : 48.5 + 9@87.0 + 73.5 + 25.0 m Width = 9.2 m
Bridge components	Superstructure		Pre-stressed concrete box girder, continuous rigid frame type (48.5 m + 9*87.0 m + 48.5 m) Pre-stressed concrete T girder, simply supported type (2*25.0 m) 
	Substructure	Abutment	Inverted T-type
		Pier	Columnar type
		Foundation	Abutment: Cast-in-place RC pile (φ1,500) Pier: Cast-in-place RC pile (φ1,500)
Loads	Live load		AASHTO HS20-44 (for girder and substructure)
	Seismic load		$C_{sm} = \frac{1.2ZS}{T_m^{2/3}} \leq 2.5Z$; where Z=0.15 and S=1.5
	Wind load		3.0 kN/m^2
	Thermal force		Temperature range: $26^{\circ}c \pm 17^{\circ}c$
Girder and deck	Repair of cracks/rebar exposures		P12 to A2
	Connecting girders (eliminating hinges/joints)		All except P5 to P6
Bridge accessories	Center hinge rehabilitation		P5 to P6
	Expansion joint replacement		A1, P5 to P6, A2
	Steel brackets		A1, A2
Pier	RC-lining		P1 to P12
Foundation	RC casting reinforcement		P1 to P10
	Steel pipe sheet piles		P3 to P10
	Bored RC piles		P1,P2



STEP-1: Removal of expansion joint in top slab and chip bottom slab.
Weld rebars for connecting to longitudinal rebar in top and bottom slab.
Drill holes for external cables and PC bars.
Arrange steel pipes for external cables and PC bars for connecting.

STEP-2: Cast concrete for connecting.
Having confirmed hardened concrete strength, PC bar to be connected

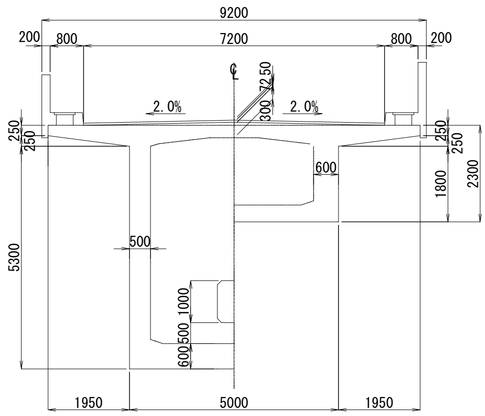
STEP-3: Arrange and stress the external cables.
Adhere carbon fiber sheets on the bottom slab

Figure 9.3.1 Hinge Connection of Existing Meghna Bridge

(3) Existing Gumti Bridge

The general outline of the design and the design outputs are summarized in Table 9.3.4. The connection of the hinges should be the same as Figure 9.3.1.

Table 9.3.4 Outline of Design (Existing Gumti Bridge)

Bridge data			Length = 1,410 m: 52.5 + 15@87.0 + 52.5 m Width = 9.2 m
Bridge components	Superstructure		Pre-stressed concrete box girder, continuous rigid frame type 
	Substructure	Abutment	Inverted T-type
		Pier	Columnar type
		Foundation	Abutment: Cast-in-place RC pile (φ1,500) Pier: Cast-in-place RC pile (φ1,500)
Loads	Live load		AASHTO HS20-44 (for girder and substructure)
	Seismic load		$C_{sm} = \frac{1.2ZS}{T_m^{2/3}} \leq 2.5Z$; where Z=0.15 and S=1.5
	Wind load		3.0 kN/m^2
	Thermal force		Temperature range: $26^{\circ}c \pm 17^{\circ}c$
Girder and deck	Repair of cracks/rebar exposures		P5 to P7, around P10
	Connecting girders (eliminating hinge/joints)		All except P5 to P6 and P11 to P12
Bridge accessories	Center hinge rehabilitation		P5 to P6, P11 to P12
	Expansion joint replacement		A1, P5 to P6, P11 to P12, A2
	Steel brackets		A1, A2
Pier	RC-lining		P1 to P16
Foundation	RC casting reinforcement		P1 to P8
	Steel pipe sheet piles		P1 to P8
	Bored RC piles		N/A