CHAPTER 5

BRIDGE AND STRUCTURE DESIGN

5. BRIDGE AND STRUCTURE DESIGN

5.1 Applicable Design Standards and Design Criteria

5.1.1 Applicable Design Standards

The applicable standards and codes for bridge design in Pakistan are listed as follows:

SFRP	Standards for Roads in Pakistan	【NHA－1992】
DBNH	Designing of Bridges on National Highways	【NHA-July/2006】
SBS	Standardization of Bridge Superstructures	【NHA-March/2005】
PCPHB	Pakistan Code of Practice for Highway Bridges	【PAKISTAN－1967】
AASHTO	Standard Specifications for Highway Bridges	【USA－2004】
UBC	Uniform Building Code	【USA－1996】

In this study, DBNH and AASHTO (17^{th} Edition, 2004), are the main code references based on standard practice in Pakistan. In addition to the above standards, Japanese standards, "Specifications for Highway Bridges (Japan Road Association)" is also applied for seismic design. Said specification presents seismic design principles and provides suitable details of structures subjected to earthquakes. These were based from vast records and study results on severe earthquakes.

5.1.2 Applicable Design Criteria

The design criteria including bridge cross section, loadings, seismic parameters and material properties are established to comply with the above standards, with due consideration to related specific site conditions.

(1) Bridge Cross Section

The bridge cross section was originally designed and agreed with ERRA and AJK during the Preliminary Study. Prior to the commencement of the optimization of bridge components in this Study, it was reconfirmed with ERRA and AJK to consider provision of sidewalks for Naluchi Bridge, as also presented in the geometric design discussed in Chapter 4. The cross section of the bridge is shown in the **Figure 5.1.1**.

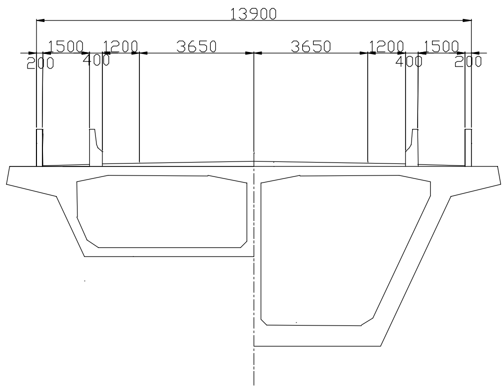


Figure 5.1.1 Bridge Cross Section of Naluchi Bridge

(2) Design Loading

a. Dead Load

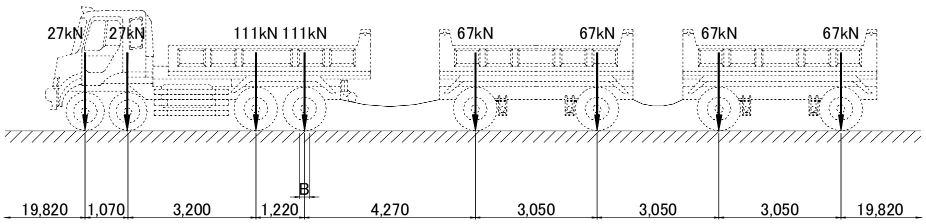
The following dead load applied in the design is in accordance with AASHTO 17th Edition:

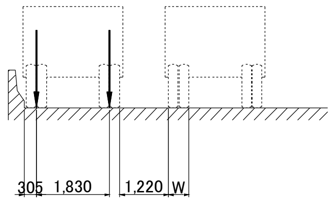
Reinforced Concrete	: 24.5 kN/m^3
Plain Concrete	: 23.0 kN/m^3
Steel	: 77.0 kN/m^3
Asphalt	: 22.5 kN/m^3

b. Live Load

As per DBNH, the most severe stresses that will govern after considering separate applications of live load type “AA” or “A” will be applied for design against traffic loading. Type “AA” load is specified in Section 4 of West Pakistan Code of Practice for Highway Bridges (WPCPHB-1967). Type “A” load is a standard truck-train highway live load for bridges or incidental structures. A standard truck-train consists of a four – axle truck, and two of two-axle trailers. For the bridge deck slab, punching shear shall be checked considering a wheel load of 95 kN, applied on a contact area of 0.25 x 0.50 m^2 of tire.

In order to confirm suitability of applying “A” or “AA” Loading in Naluchi Bridge design, comparison of bending moments due to various live load standards was conducted. Resulting from this comparison, it was concluded that application of “A” loading is suitable and appropriate since the bending moment due to “A” loading has been almost the same effect as that due to HS20-44 in ASSHTO and A loading in Japanese Specification. It is noted that effect due to AA loading is less than that due to “A” Loading in Pakistan.





Class of Loading	Axle Load	Ground Contact Area	
		B	W
	kN	mm	mm
A	111	254	508
	67	203	381
	27	152	203

Figure 5.1.2 Design vehicle Load (Type "A")

c. Impact Load

The amount of impact allowance is determined from the following formula, in accordance with AASHTO 3.8.2.

$$I = \frac{15.24}{L+38}$$

where,

I = impact coefficient (maximum 0.30)

L = Length in meters of the portion of the span that is loaded to produce the maximum stress in the member.

d. Brake Load

The bridge axis direction load is considered as an influence to the braking of vehicle. 30% of the weight of the trailer is considered as a brake load on one lane.

e. Sidewalk Live Load

The following Sidewalk Live Load for design of girders and other components is applied in accordance with AASHTO 3.14.

Span length	Sidewalk Live Load
0 to 7.8m	4070 Pa (= 4.07 kN/m^2)
7.81m to 30.50 m	2870 Pa (= 2.87 kN/m^2)
Over 30.50 m	$P = \left(143.5 + \frac{43800}{L}\right)\left(16.7 - \frac{W}{15.2}\right)$

where,

P = sidewalk live load in Pa. < 2870 Pa

L = loaded length of sidewalk in meters.

W = width of sidewalk in meters.

f. Wind Loads

Wind load is normally considered only for bridges with significant spans such as suspension bridges, cable stayed bridges, etc., due to their wind resistant requirements. Although extra-dosed bridges are known to be more stable against wind load compared to bridges with significant spans, effects of wind loads will still be verified for Naluchi Bridge. Due to site topographical conditions, wind velocity of 30m/s is applicable, in accordance with provision 3.15 of AASHTO.

g. Effect of Temperature Change

The temperature range is given below based on recorded monthly temperature in Muzaffarabad, as discussed in Chapter 3.

Range of Temperature : -5°C ~ + 45°C
Average Temperature : +20°C,

h. Stream Current Forces on Piers

The effect of water flow against piers is calculated using the following formula, based on AASHTO provision 3.18.1.

$$P_{avg} = K\left(V_{avg}\right)^2$$

in which,

P_{avg} = average river stream pressure (kN/m^2)
V_{avg} = average water velocity (m/s)
K = Resistance coefficient (varies depending on shape of pier)

(3) Seismic design

a. Applicable Standard

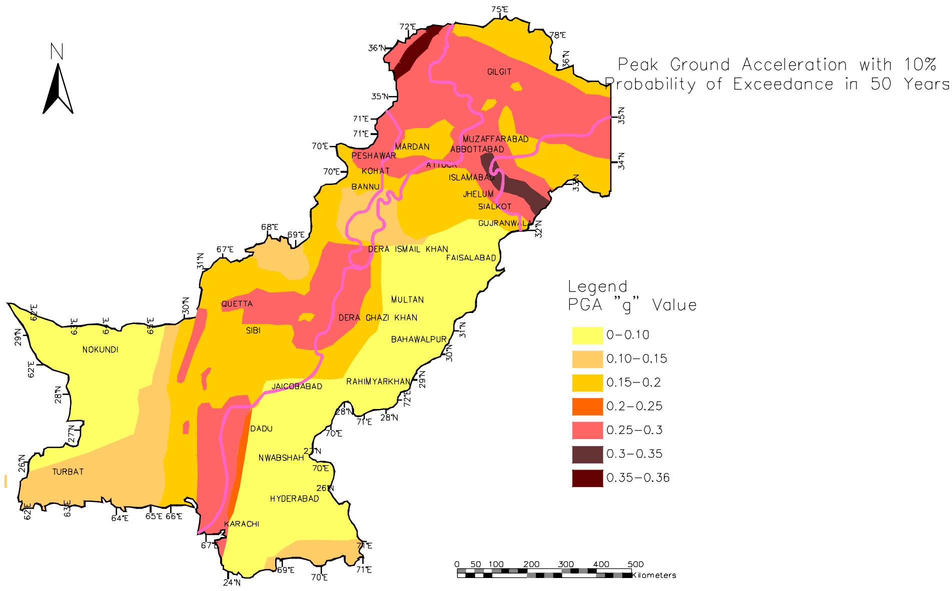
The standard considered for seismic design shall be in accordance with Japan Highway and Bridge Standards (Japan Road Association).

b. Design Policy

Muzaffarabad, AJK is one of the areas in Pakistan identified to be prone to high seismic risks. The Naluchi Bridge and viaducts have significant infrastructure importance due to their strategic locations. These structures will be difficult to rehabilitate/reconstruct once damaged by earthquake. Therefore, special attention should be rendered in the design of Naluchi Bridge and its approach viaducts. For seismic design, structural analysis will be performed considering earthquake forces defined in Pakistan guidelines on earthquakes (in view of the recent earthquake) and relevant AASHTO provisions on low probability earthquakes by active fault (an inland direct strike type). The analysis procedure will be based on relevant provisions of both AASHTO and the Specifications for Highway Bridges (Japan Road Association).

c. Seismic ground acceleration

Figure.5.1.3 shows the latest seismic ground acceleration map in Pakistan after 2005 Earthquake. Design seismic ground acceleration to be considered is g = 0.35 based on the map.

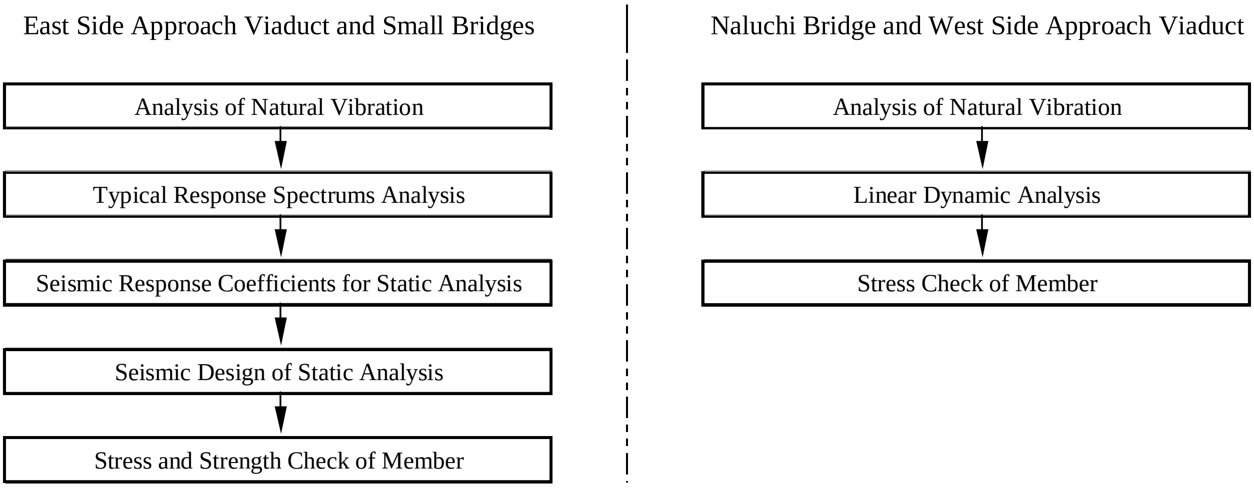


(Source: Peak Ground Acceleration Map, NHA, 2006)

Figure 5.1.3 Seismic Ground Acceleration Map

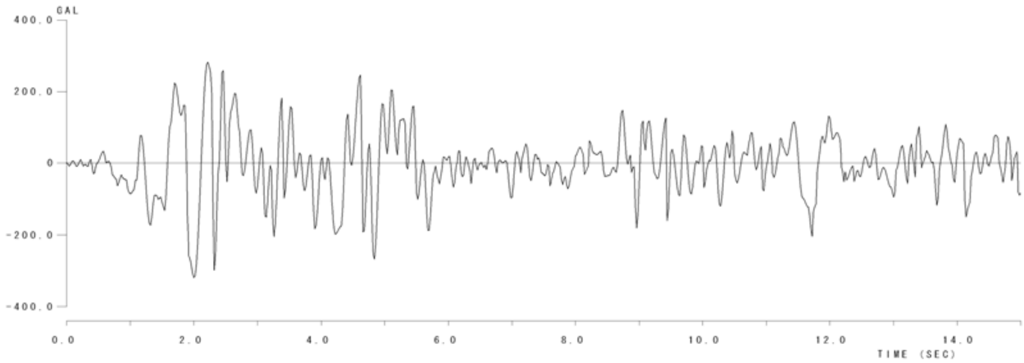
d. Seismic Design Method

The flowchart for seismic design method is shown in **Figure 5.1.4** while the applied design earthquake wave for dynamic analysis is depicted in **Figure 5.1.5**.



(Source: JICA Study Team)

Figure 5.1.4 Seismic Design Flowchart



(Source: Standard Specifications for Highway Bridges,2004)

Figure 5.1.5 Applied Earthquake Wave (El Centro Wave 1940 USA)

e. Load Combinations

The members will be designed considering both Service Load Design (Allowable Stress Design) and Strength Design (Load Factor Design). The following tables show load cases for both Service Load Design and Load Factor Design, in accordance with AASHTO provision 3.22.

Table 5.1.1 Loading Case Table for Service Load Design

Group	γ	B												%
		D	(L+I)n	(L+I)p	CF	E	B	SF	W	WL	LF	R+S+T	EQ	
I	1.0	1	1	0	1	βE	1	1	0	0	0	0	0	100
II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	125
III	1.0	1	1	0	1	βE	1	1	0.3	1	1	0	0	125
IV	1.0	1	1	0	1	βE	1	1	0	0	0	1	0	125
V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	140
VI	1.0	1	1	0	1	βE	1	1	0.3	1	1	1	0	140
VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	133

Source: AASHTO Standard Specifications for Highway Bridges ,17th Edition, 2004

2 Check in close load coefficient method

Table 5.1.2 Loading case table for Load Factor Design

Group	γ	β											
		D	(L+I)n	(L+I)p	CF	E	B	SF	W	WL	LF	R+S+T	EQ
I	1.3	β_D	1.67	0	1.0	βE	1	1	0	0	0	0	0
II	1.3	β_D	0	0	0	βE	1	1	1	0	0	0	0
III	1.3	β_D	1	0	1	βE	1	1	0.3	1	1	0	0
IV	1.3	β_D	1	0	1	βE	1	1	0	0	0	1	0
V	1.25	β_D	0	0	0	βE	1	1	1	0	0	1	0
VI	1.25	β_D	1	0	1	βE	1	1	0.3	1	1	1	0
VII	1.3	β_D	0	0	0	βE	1	1	0	0	0	0	1

Source: AASHTO Standard Specifications for Highway Bridges ,17th Edition, 2004

D : dead load
L : live load
I : live load impact
E : earth pressure
B : buoyancy
W : wind load on structure
WL : wind load on live load
LF : longitudinal force
CF : centrifugal force
R : rib shortening
S : shrinkage
T : temperature
EQ : earthquake
SF : stream flow pressure
*% : percentage to be applied for the basic unit stress
* (load factor) for all cases, is 1.0.

(4) Material Properties

a. Concrete

Classification of concrete considered in the design is as follows:

Table 5.1.3 Classification of Concrete

Class	28 days Cylinder Strength (N/mm^2)	Max. Size of coarse aggregate (mm)	Structural Members Applied
A_{0-1}	21	25	Non-Reinforced Concrete Structures such as side ditch and curb. Leveling concrete
A_{0-2}	21	10	Non-reinforced concretes structures with Max. size of aggregate 10mm for crib
A_{1-1}	24	25	Reinforce concrete members such as abutment wall, pier column, except pier 3, approach slab barrier (Longitudinal concrete wall), concrete box culvert, pipe culvert and reinforce concrete retaining wall
A_{1-2}	24	40	Reinforced concrete members such as footing , pile cap Shinso foundation (Chicago bored method)
A_{2-1}	30	25	Reinforced concrete members with a large amount of reinforcing bars such as deck slab
A_{2-2}	30	25	Concrete deposited in water such as concrete pile cast in situ
A_{2-3}	30	25	Pier shaft for pier No.3 in Naluchi Bridge
A_3	40	25	Pylon for Naluchi Bridge
D_1	35	25	Prestressed concrete box girder (cast in situ) of east side approach bridge and Westside approach viaduct
D_{2-1}	40	25	Prestressed concrete section girder for small bridges
D_{2-2}	40	25	Prestressed concrete box girder of Naluchi bridges fabricated using high early strength cement and erected by balanced cantilever method
Lean	10	40	Lean concrete

Note: Class A1-1Concrte further subdivided into A1-1 and A1-1s in the drawings for payment use

Source : JICA Study Team

b. Reinforcing steel

The requirements for reinforcing steel bar is as follows:

- Specification AASHTO M31, Grade 420 (ASTM A615)
- Specified Strengths:

Minimum Yield Strength: 420 MPa

Minimum Tensile Strength: 620 MPa

c. Prestressing (PS) Steel

The requirement of prestressing steel is as follows:

Table 5.1.4 Specification of Prestressing Steel

Tendon	Specification	Grade	Minimum Yield Strength (kN)	Minimum Breaking Strength (kN)
4S12.7	AASHTO M203 (ASTM A416)	1860	661.3	734.8
3S15.2		1860	703.9	782.1
4S15.2		1860	938.5	1,042.8
12S12.7		1860	1,984.0	2,204.4
12S15.2		1860	2,815.6	3,128.4
19S15.2		1860	4,458.0	953.3
27S15.2		1860	6,335.0	7,038.9

Note: Minimum Yield strength is equivalent to 90% of Minimum Breaking Strength

Source : AASHTO M203

d. Prestressing Bar

Prestressing bar shall be Type I, plain surface, with minimum ultimate tensile strength of 1,035 MPa. This shall be uncoated high-strength steel bar for prestressing concrete conforming to AASHTO M275 (ASTM A-722),

e. Structural Steel

Specification AASHTO M270 (ASTM A709) covers carbon and high-strength low-alloy steel structural shapes, plates, and bars and quenched-and-tempered alloy steel for structural plates intended for use in bridges. It is steel Grade 250 with minimum yield strength shall be $f_y = 250$ MPa and minimum tensile strength f_{pu}=400 MPa. Structural steel for “saddle” shall conform to STKM 13 A, with minimum yield strength of 215 MPa, and a minimum tensile strength of 370 MPa, in accordance with JIS G3445.

5.2 Optimizaton of Naluchi Bridge

5.2.1 General

Based on the Preliminary Design Study conducted by JICA, extra-dosed bridge was selected as the most suitable type for Naluchi Bridge. Extra-dosed is a bridge type that is actually a similar structural system with post-tension box girder bridge rather than a conventional cable-stayed bridge. The appearance of this extra-dosed bridge is similar to cable-stayed bridge while its structural behaviour is similar to post-tension box girder bridge.

Because of a lower main tower in extra-dosed bridges, vertical loads are partly resisted by main girders and stress variations in stay cables induced by live loads are smaller than those in cable-stayed bridges. This is quite similar to the behaviour of box-girder bridges, where the main girder itself has a decisive influence on the structure's rigidity, and live loads produce only limited stress variations in tendons. Comparing to cable stayed bridges, the height of the main tower in extra-dosed bridges is lower, hence, a reduction in material/equipment costs of construction can be achieved. This cost-effectively of extra-dosed bridge has been justified when clear span is about 200 m.

In the Basic Design Phase, the structural optimization of Naluchi Bridge was studied to determine the initial dimensions and position. The detailed structural analysis meanwhile is carried out in the Detailed Design Phase. The components of the bridge subject to optimization include refinement of span lengths, foundation type, pier configuration, sections of box girders, type of stay cables, and erection method. These are described in the subsequent sections.

5.2.2 Refinement of Span Length

The Naluchi Bridge crosses the deep valley of Jhelum River. The road level is 60 meters higher than the river bed. The gap between the cliff edge on the left bank (east side) and the edge on the west bank is about 400 meters. An extra-dosed bridge (main bridge) and viaduct (approaches) is selected to connect these edges.

For the refinement of its span length, initial consideration is attributed to the most suitable location of pier on the river. Considering an extra-dosed type structure with two continuous spans, position of the pylon resting on the river pier should be located closer to river water edge of Jhelum River, while the pier on the left bank should be located flushed to its cliff edge to minimize span length over the Jhelum River.

Foundation of the pier on the left bank would be located at the top of a precipitous river terrace, provided safe distance is achieved to the steep slope, maintaining stability of the pier foundation.

The total span length of the bridge, which is one of the factors in optimization, is affected by the foundation type of left bank pier and river pier. Piled foundation is finally selected for left bank pier instead of spread foundations due to the following significant advantages:

- Since the width of a piled foundation is expected to be narrower than a spread footing, the span lengths of the bridge is minimized.
- Casting of pile foundation is determined to have least traffic disturbance during the construction.

The span arrangement of the bridge is preferred to be symmetrical to achieve structural balance. The length of the bridge was estimated to be a total of 246 m.

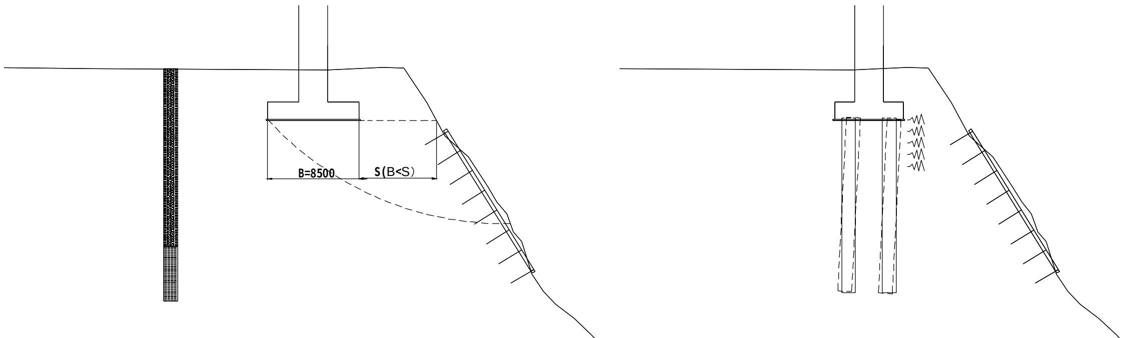


Figure 5.2.1 Options of Foundation on Left-bank Pier

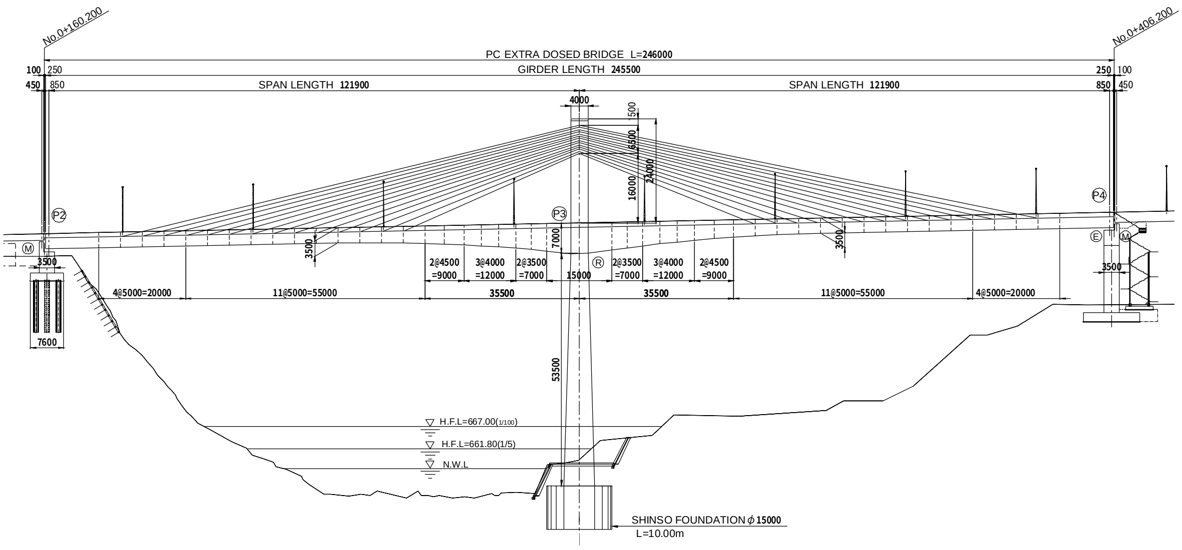


Figure 5.2.2 Bridge Profile

5.2.3 Refinement of Substructure

(1) Alternative Study for Pier Foundation

As mentioned in the previous section, one of the considerations for selecting type pier at the right bank should be that which could be casted almost flushed to the cliff edge, provided that no significant dewatering works will be required.

For foundation on the river pier, spread foundation is one of alternatives since the slate rock is exposed at shallow bearing stratum. However, caisson type pile foundation could be more suitable from structural point-of-view. A further comparative study for the river pier foundation for two alternatives (the caisson type pile and spread foundation) is thus conducted.

Based from study results shown in the **Figure.5.2.3**, the caisson type pile is preferred due to cost effectiveness and reliable seismic stability.

(2) Alternative Study for River Pier Configuration

The total pylon height of Naluchi Bridge is about 80 m from the foundation to its top level. Since the bridge site is located in Muzaffarabad City, one of the areas in Pakistan identified to be prone to high seismic risks, it is important to ensure the structural integrity of the bridge. Moreover, since the bridge is located at near the entry of Muzaffarabad City aesthetics is also another significant as it will serve as the City's landmark. Thus, the shape of pylon on the river pier is determined based on structural and architectural aspects. The key consideration point in evaluating the preferred shape of the pylon is to consider the least self-weight of pier, with adequate structural stiffness of pier head.

The required pylon dimension is about 7 m long and 13 m wide. Moreover, to reduce affect of water current forces, the shape of the pylon is either elliptical type or trapezoid. The comparative study for the type of pylon is presented in **Figure 5.2.4**. Resulting from the Study, alternative-2 (triangular section with uniform-hollow column) was selected as most suitable based on construction cost, structural aspect, and construction duration.

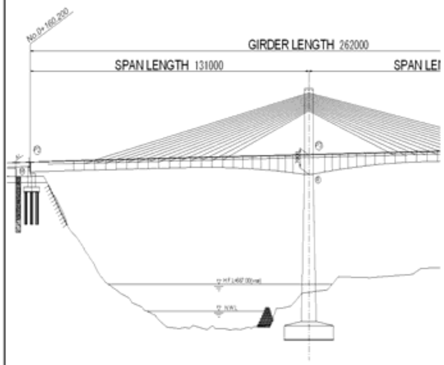
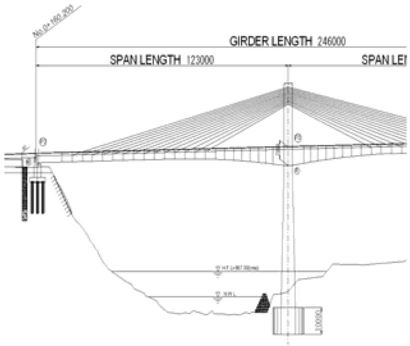
Description		Alternative 1		Alternative 2	
		Spread Footing Foundation		Well Type Pile Foundation(ϕ 15.00m)	
Configuration of Foundation					
General Feature		The location of pier is view of the size of spread foundation (23.0mx25.0m)and the footing be constructed with dry condition considering the tip of footing set beside water line with simple coffer dam ,so that effective span is 131.0 meter long.		The location of pier is identified in view of the size of well foundation (15m Diameter)and the footing be constructed with dry condition considering the tip of well set beside water line with simple coffer dam, so that effective span is 123.0meters long. This type foundation is big diameter well /caisson type by mechanical excavation.	
Main Bridge Length(m)		262.00		246.00	
Quantity	Concrete(m^3)	3187.0		2474.0	
	Excavation (m3)	11856.0		3802.4	
	Caisson Pile ϕ 15.0(m)	-		14.0	
Construction Cost Ratio (Include of Super Structure Cost)		1.147		1.000	
Construction Aspect		Normal construction method	Good	Excavation work in bellow the riverbed is required with possible need of water tied treatment.	Poor
Structural Aspect		It is possible to encounter weathered and deteriorated rock surface of bearing stratum. In that case, it needs design change in footing.	Poor	High liability of structural stability including quake resistance.	Excellent
Construction Period		Construction period is shorter than alternative	Good	Construction period is longer than alternative 1	Poor
Hydraulic Aspect		For a local scouring on floods and riverbed degradation.it still concerns about possible reduction of durability.	Bad	High safety against local scouring and riverbed aggradations.	Excellent
Technology Transfer Aspect		For normal construction method, there is no transfer technology elements.	Fair	It is a suitable industrial method for the mountains area and good opportunity of transferring technology.	Good
Overall Aspect		Discarded		Recommended considering total cost, structural aspect,Hydraulic aspect	

Figure 5.2.3 Comparison for Pier Foundation Type

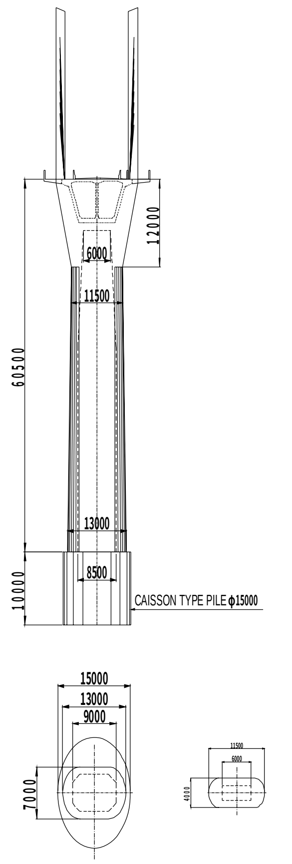
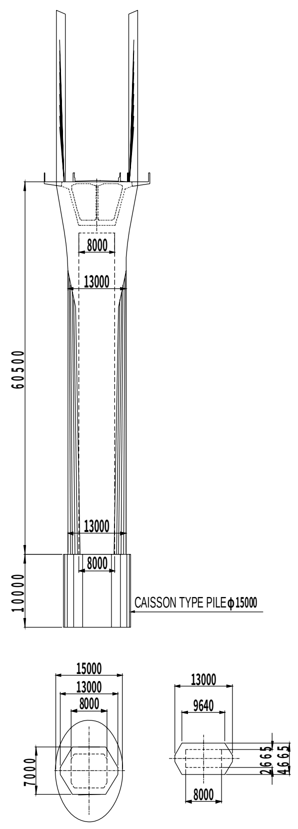
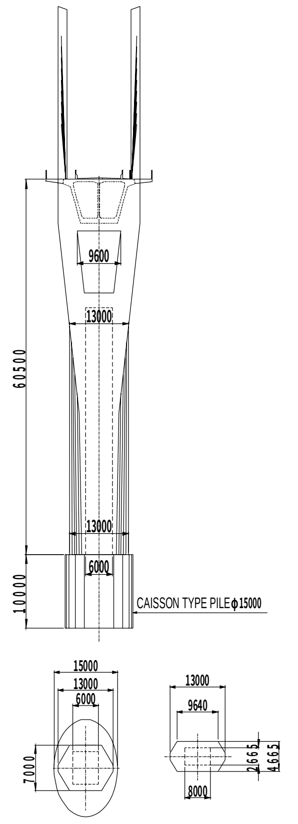
	Altenative					
	Alternative 1		Alternative 2		Alternative 3	
Configuratiuon of Pylon						
General Feature	Elliptical sections that change in the vertical - Hollow column		Triangular sections with uniformity - Hollow column		Triangular sections with uniformity (top of the pier with 2 pillars)- Hollow column	
Construction Cost	The most expensive alternative.(Ratio:1.06)	Poor	The most economical alternative.(Ratio:1.00)	Excellent	2nd best economical alternative.(Ratio:1.04)	Fair
Construction Aspect	It is too difficult to construct pier with a circular form that changes in the section in the vertical direction.	Bad	Easy construction because of uniformity of cross section	Good	The structure of bottom part has no difficult, but that of top part with a variety of section has drawbacks in the construction.	Poor
Structural Aspect	Highly quake-resistant, because of dead weight of pier is the lightest among the others (Inertia force at the earthquake is less).	Good	Relatively high earthquake-resistant and the connection with tower and pier is harmonious.	Fair	Less earthquake resistant because Self weight of pier is heavy , inertia force at the earthquake is large.	Poor
Construction Period	Middle level	Fair	The shortest period	Excellent	The longest period	Poor
Aesthetic Aspect	It is shape with a light feeling.	Fair	Sharp configuration is harmonious with superstructure.	Good	Massive pier head is not harmonious with superstructure.	Bad
Technology Transfer Aspect	Construction technology of high pier that has a few example in Pakistan.	Good	The same comments as Alt-1	Good	The same comment as Alt-1	Good
Overall Aspect	Rejected due to cost and difficulty of construction		Recommended because of no negative items		Rejected because of Poor rating of Construction aspect, structural aspect and bad aesthetic aspect	

Figure 5.2.4 Comparison for River Pier Configuration

5.2.4 Refinement of Superstructure

(1) Alternative Study for Stay Cable Arrangement

In the extra-dosed bridge, the main function of stay cable is both to minimize bending moment at the center span, by inducing negative moment to lift up the girders. The pylon height is an important factor for determining the stay cable arrangement because it controls the magnitude of negative moment. Another considered factor is the amount of PS steel. This is calculated based on tensile force and stress range. The appropriate stay cable arrangement is determined by the comparing the amount of PS Steel required relative the pylon height (20-26m). Although the significant difference of the PS cable weight in stay cable and girder cannot be appreciated from the study result, the 23 m pylon height is found to require more appropriate cable arrangement, with less weight of required PS steel materials. The comparison is shown in the **Figure 5.2.5**.

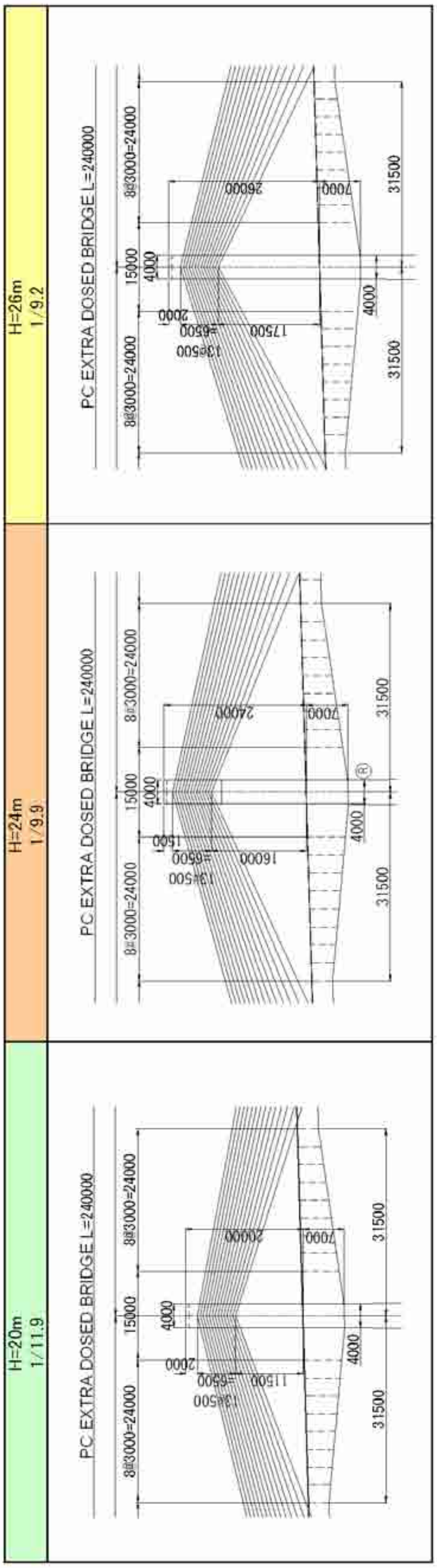
(2) Refinement of Main Girder

The highly rigid box girder is basically necessary as a girder type because it can accommodate longer clear span, i.e., 120 m or more for the Naluchi Bridge. A box type which is supported by both transversal ends with stay cable is selected in consideration of the deck width of 13.3 m.

Comparison of box girder type (1-cell or 2-cell) is presented in **Figure 5.2.6**. The advantage of 1-Cell type is that its void is wide, which will ease construction and maintenance. However, the required PS steel material in transverse direction of girder will consequently increase due to the span length of deck slab required for 1-cell girder as compared to the 2-cell type. Furthermore, the 2- cell types is more high durable and rigid. Thus, it was finally found that, Alternative -1:2-Cell Type is the most cost-effective girder type.

(3) Study on Stay Cable

The PS-strand cable (15.2 mm) of stay cable was considered as diagonal cables for Naluchi Bridge. Normally, double or treble anti-corrosion treatment is required for the stay cables of extra-dosed bridges, because of its exposure to air. The anti-corrosion methods is studied in **Figure 5.2.7** to determine the most effective corrosion prevention method, considering local climates, construction conditions and cost effectiveness. “PE tube” + Polyethylene coating is selected as suitable anti-corrosion method of stay cable in consideration of vast track records and expected high reliability against corrosion.



STAY CABLE LENGTH and WEIGHT

	STAY CABLE LENGTH			STRAND			STAY CABLE WEIGHT(kg)		
	H=20m	H=24m	H=26m	H=20m	H=24m	H=26m	H=20m	H=24m	H=26m
S1	212.8	215.6	219.9	27S15.2	27S15.2	27S15.2	6326.4	6409.7	6537.9
S2	203.0	205.6	209.8	27S15.2	27S15.2	27S15.2	6033.7	6113.1	6235.4
S3	193.1	195.7	199.6	27S15.2	27S15.2	27S15.2	5741.2	5816.8	5933.1
S4	183.3	185.7	189.4	27S15.2	27S15.2	27S15.2	5448.9	5520.6	5631.0
S5	173.5	175.8	179.3	27S15.2	27S15.2	27S15.2	5156.9	5224.8	5329.3
S6	163.7	165.8	169.1	27S15.2	27S15.2	27S15.2	4865.3	4929.4	5028.0
S7	153.9	155.9	159.0	27S15.2	27S15.2	27S15.2	4574.2	4634.4	4727.1
S8	144.1	146.0	148.9	27S15.2	19S15.2	19S15.2	4283.5	3054.0	3115.1
S9	134.3	136.1	138.8	27S15.2	19S15.2	19S15.2	3993.4	2847.2	2904.1
S10	124.6	126.2	128.8	27S15.2	19S15.2	19S15.2	3704.1	2640.9	2693.8
S11	114.9	116.4	118.7	27S15.2	19S15.2	19S15.2	3415.8	2435.3	2484.1
S12	105.2	106.6	108.8	19S15.2	19S15.2	19S15.2	2201.6	2230.6	2275.2
S13	95.6	96.8	98.8	19S15.2	19S15.2	19S15.2	1999.3	2025.6	2066.1
S14	86.0	87.1	88.8	19S15.2	19S15.2	19S15.2	1798.1	1821.8	1858.2
TOTAL	2087.9	2115.4	2157.7			TOTAL	59542.2	55704.3	56818.4
						RATIO	1.069	1.000	1.020

Figure 5.2.5 Comparison Stay Cable Arrangement

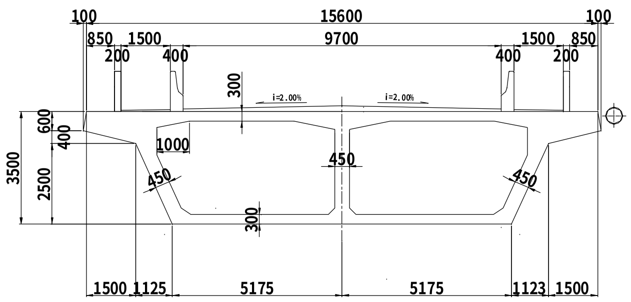
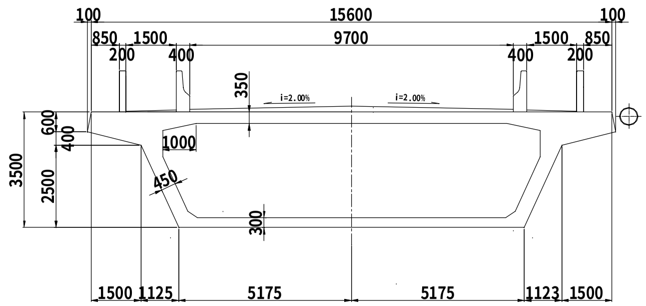
		Alternative-1		Alternative-2	
Cross Section					
General Feature		2 Cell in One Box Girder		One Cell in One Box Girder	
Slab	Thickness	t = 30cm		t = 35cm	
	Arrangement of PC Steel	4S15.2 (FLAT DUCT) c.t.c 500		4S15.2 (FLAT DUCT) c.t.c 375	
Quantity Per Unit Length (m)		CONCRETE 14.46 m3 / m TRANSVERSE PC STEEL 144.3 kg / m		CONCRETE 13.37 m3 / m TRANSVERSE PC STEEL 192.4 kg / m	
Construction Cost Ratio		1.000		1.323	
Construction Aspect		Because of two cell in one box girder, construction method is relatively complicated.	Poor	Because of wide space in one cell in one box, construction method is easier than the other.	Good
Structural Aspect		High rigidity and deformation performance of the section	Excellence	Suffering severe stress all the time because of long span of the deck	Bad
Construction Period		Slightly longer construction period than that of Alternative -2	Fair	Slightly shorter construction period than that of Alternative -1	Good
Durability of Deck Slab		Superior durability because of high rigidity and deformation performance of the section	Excellence	Inferior durability because of suffering severe stress all the time causing fatigue failure.	Bad
Technology Transfer Aspect		No difference	Fair	No difference	Fair
Overall Aspect		Selected because of advantageous cost, and structural aspect.		Discarded because of disadvantageous cost and structural aspect	

Figure 5.2.6 Comparison for Cross Section of Main Girder

Comparison for Stay Cable of Extra Dosed Bridge

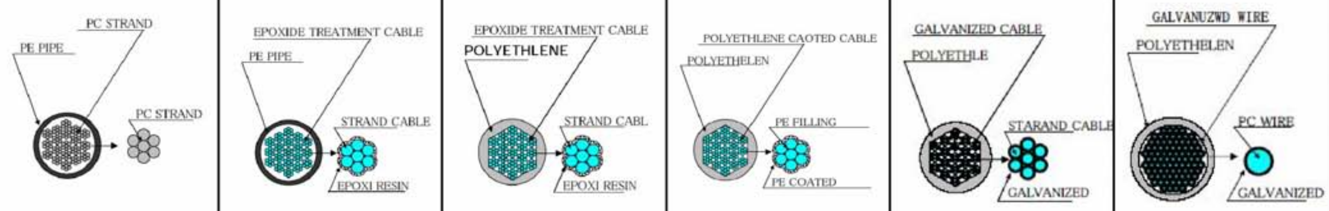
Cable Type		Site Assembly Cable		Semi-Fabricated Cable			Fabricated Cable
Cable Specifications		Normal Strand Cable	Epoxide treatment Strand Cable	Polyethylene Coated+ Epoxide Treatment	Polyethylene Coated Strand Cable	Galvanized Strand Cable	Galvanized Cable
Corrosion Prevention System		PE pipe+Grout	PE pipe+Grout+Epoxide Treatment	Polyethylene Coated+Epoxide Treatment	Polyethylene Coated+Polyethylene Coated	Galvanized + Polyethylene Coated	PE pipe+Galvanized + Polyethylene Coated
Corrosion of Anchorage		Cement milk grout	Cement milk grout	Cement milk grout	Cement milk grout	Grout materials other than cement flui	No Need
Section of Cable							
Construction Process		Arrangement of sheath tube ↓ Pull in Cable ↓ Assembly of Anchorage ↓ Prestressing ↓ Anchorage grout ↓ Sheath tube grout	Arrangement of sheath tube ↓ Pull in Cable ↓ Assembly of Anchorage ↓ Prestressing ↓ Anchorage grout ↓ Sheath tube grout	Arrangement of cable ↓ Assembly of Anchorage ↓ Prestressing ↓ Anchorage grout			Arrangement of cable ↓ Prestressing
Pylon Anchorage System		Saddle or Anchorage	Saddle or Anchorage	Saddle or Anchorage			Anchorage
Girder Anchorage System		Normal Wedge type	Wedge type for Epoxy cable	Wedge type for Epoxy cable	Normal Wedge type	Normal Wedge type	Nut Anchorage System
Evaluation	Construction Result	◎	○	△	◎	△	△
	Construction Cost	○	△	△	○	△	△
	Corrosion performance	△	○	○	○	○	○
	Construction Aspect	△	△	○	○	○	◎
	Overall Aspect	○	○	○	◎	△	△

Figure 5.2.7 Comparison of Stay Cable

5.3 Detailed Design of Naluchi Bridge

5.3.1 Design Method

For the bridge design, service load combinations are checked using allowable stress design method while limit state load combinations are checked using strength design method. For seismic design considerations, the main bridge and east side approach viaduct is also simulated through dynamic analysis due the complexities of their vibration responses. For the east side approach viaduct, static analysis is applied to check bridge seismic performance since its vibration response is relatively simpler.

(1) Allowable Design Method

Service load design by allowable stress method is in conformity with AASHTO-17th edition.

(2) Ultimate Design Method

For the limit state design, sectional forces on elements are checked against their yield resistance. The load combination and the calculation process are in conformity with AASHTO17th edition.

(3) Seismic Design

As mentioned above, dynamic analysis (time history response) was applied to the main bridge and west side approach viaduct for seismic design purpose. The El Centro earthquake wave in the United States (maximum acceleration g=0.35) is applied as an input to earthquake waveform.

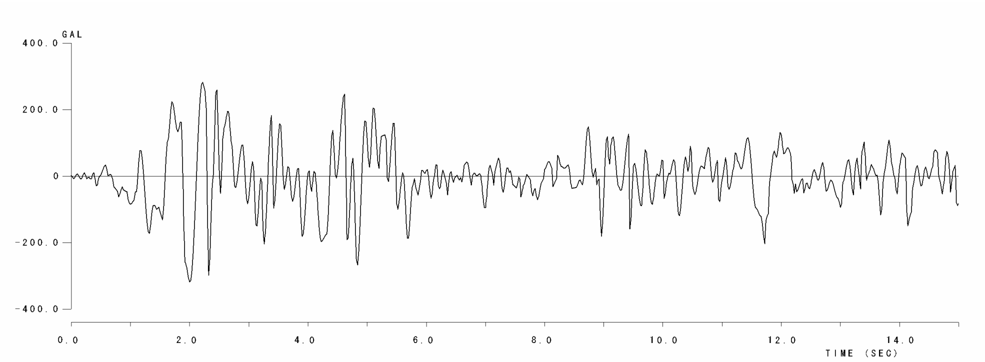


Figure 5.3.1 Applied Earthquake Wave (El Centro Wave 1940 USA)

5.3.2 Normal Condition Design

For checking stability and safety in both construction and service conditions, the stress check by static analysis is carried out in the design of the Naluchi Bridge. In the static analysis, two-dimensional frame model simulated, superimposed with dead load, live load and thermal load. For further checking of structural safety of a component, the Allowable Design Method is applied.

(1) Design Conditions

Table 5.3.1 Design Conditions for Naluchi Bridge

Superstructure	:	2 Spans Continuous Extra Dosed PC-Box Girder
Bridge Length	:	246.0m
Girder Length	:	245.5m
Span Length	:	2@121.9m
Bridge Width	:	15.6m
Effective Width	:	Carriage Way 9.7m
	:	Sidewalk 2 x 1.5m
Live Load	:	Class-A, Class –AA
Curvature	:	R=∞
Gradient	:	i = 2.0% ↙
Super Elevation	:	i = 2.0% ↙ ↘
Skew Angle	:	θ =90 °
Asphalt pavement	:	Carriageway 80mm～
	:	Foot way 30mm～

(Source : JICA Study Team)

(2) Material Properties

Table 5.3.2 Materials' Properties for Naluchi Bridge

(Concrete) (N/mm^2)

		PC-Box girder PC	Cross beam PC	Deck slab PC	Pylon RC
Class		D2-2	D2-2	D2-2	A3
28Days Cylinder Strength		40	40	40	40
Modulus of Elasticity		3.10×10^4	3.10×10^4	3.10×10^4	3.10×10^4
Allowable Compression Stress		16.00	16.00	16.00	16.00
Allowable Tensile Stress		-3.16	-3.16	-3.16	-
Temperature coefficient		10×10^{-6}	10×10^{-6}	10×10^{-6}	10×10^{-6}
Allowable Shear Stress		0.51	0.51	0.51	0.51
Maximum Average Shear Stress		2.50	2.50	2.50	2.50
Diagonal Stress	Permanent Load	1.00	1.00	1.00	-
	Design Combination Load	2.00	2.00	2.00	-

(PC steel)

	Unit	Longitudinal 12S15.2	Transverse 4S15.2	Cross Beam 12S15.2
Ultimate Strength	N/mm^2	1860	1860	1860
Minimum Breaking Strength	kN	3128.4	1042.8	3128.4
Minimum Yeild Strength	kN	2815.6	938.5	2815.6
Friction coefficient per 1 meter	1/m	0.0020	0.0020	0.0020
Friction coefficient per 1 radian	1/Rad	0.250	0.250	0.250
Set Losses	mm	9	9	9
Relaxation	%	1.5	1.5	1.5
Modulus of Elasticity	$x10^5N/mm^2$	2.0	2.0	2.0
Sectional area	mm^2	1664.4	554.8	1664.4
Diameter of sheath	mm	75	65	75

(Stay Cable)

	Unit	S1～S7 27S15.2	S8～S14 19S15.2
Ultimate Strength	N/mm^2	1860	1860
Minimum Breaking Strength	kN	7038.9	4953.3
Minimum Yeild Strength	kN	6335	4458.0
Modulus of Elasticity	$x10^5N/mm^2$	2.0	2.0
Sectional area	mm^2	3744.9	2635.3

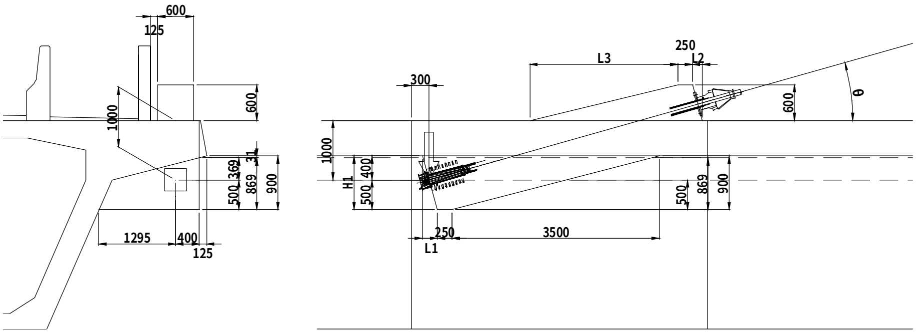
(Reinforcement Bar) (N/mm^2)

Yield strength	420
Modulus of Elasticity （$\times10^5$）	2.0
Allowable Tensile Stress	168

(JICA Study Team)

(3) Calculation of Ancillary Weight

a. Anchorage of Stay Cable

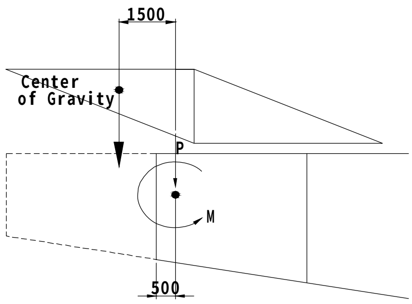


				S1	S2	S3	S4	S5	S6	S7	S8	S9	S10	S11	S12	S13	S14
Angle of Stay Cable			θ(°)	12.588	12.923	13.293	13.702	14.158	14.668	15.243	15.897	16.645	17.510	18.522	19.720	21.158	22.916
Lower side Anchorage	1-1 Section	L1(m)		0.890	0.892	0.893	0.894	0.896	0.898	0.901	0.904	0.907	0.911	0.916	0.923	0.932	0.943
		Area	$A1(m^2)$	2.072	2.073	2.073	2.073	2.074	2.075	2.076	2.078	2.079	2.081	2.083	2.086	2.090	2.095
	2-2 Section	L1(m)		0.512	0.513	0.514	0.515	0.516	0.517	0.518	0.520	0.522	0.524	0.527	0.531	0.536	0.543
		Area	$A2(m^2)$	0.739	0.739	0.740	0.740	0.740	0.740	0.741	0.741	0.742	0.742	0.743	0.744	0.745	0.747
	Concrete Volume		$Vl(m^3)$	2.154	2.155	2.156	2.156	2.157	2.157	2.159	2.161	2.162	2.164	2.166	2.169	2.173	2.178
	Weight of Anchorage		Wl(kN)	52.773	52.798	52.822	52.822	52.847	52.847	52.896	52.945	52.969	53.018	53.067	53.141	53.239	53.361
Upper side Anchorage	L2(m)			0.615	0.616	0.617	0.618	0.619	0.620	0.622	0.624	0.626	0.629	0.633	0.637	0.643	0.651
	L3(m)			2.500	2.500	2.500	2.500	2.500	2.500	2.500	2.500	2.500	2.000	2.000	2.000	2.000	2.000
	Area		$Au(m^2)$	1.085	1.085	1.085	1.085	1.086	1.086	1.087	1.087	1.088	0.939	0.940	0.941	0.943	0.945
	Concrete Volume		$Vu(m^3)$	0.651	0.651	0.651	0.651	0.652	0.652	0.652	0.652	0.653	0.563	0.564	0.565	0.566	0.567
	Weight of Anchorage		Wu(kN)	15.950	15.950	15.950	15.950	15.974	15.974	15.974	15.974	15.999	13.794	13.818	13.843	13.867	13.892
Total Weight of Anchorage			W(kN)	68.723	68.747	68.772	68.772	68.821	68.821	68.870	68.919	68.968	66.812	66.885	66.983	67.106	67.253

(Source: JICA Study Team)

Figure 5.3.2 Weight of Anchorage of Stay Cable

b. Traveller



Traveller Weight	P=		1300 kN
Moment	M=	1.5xP=	1950 kNm

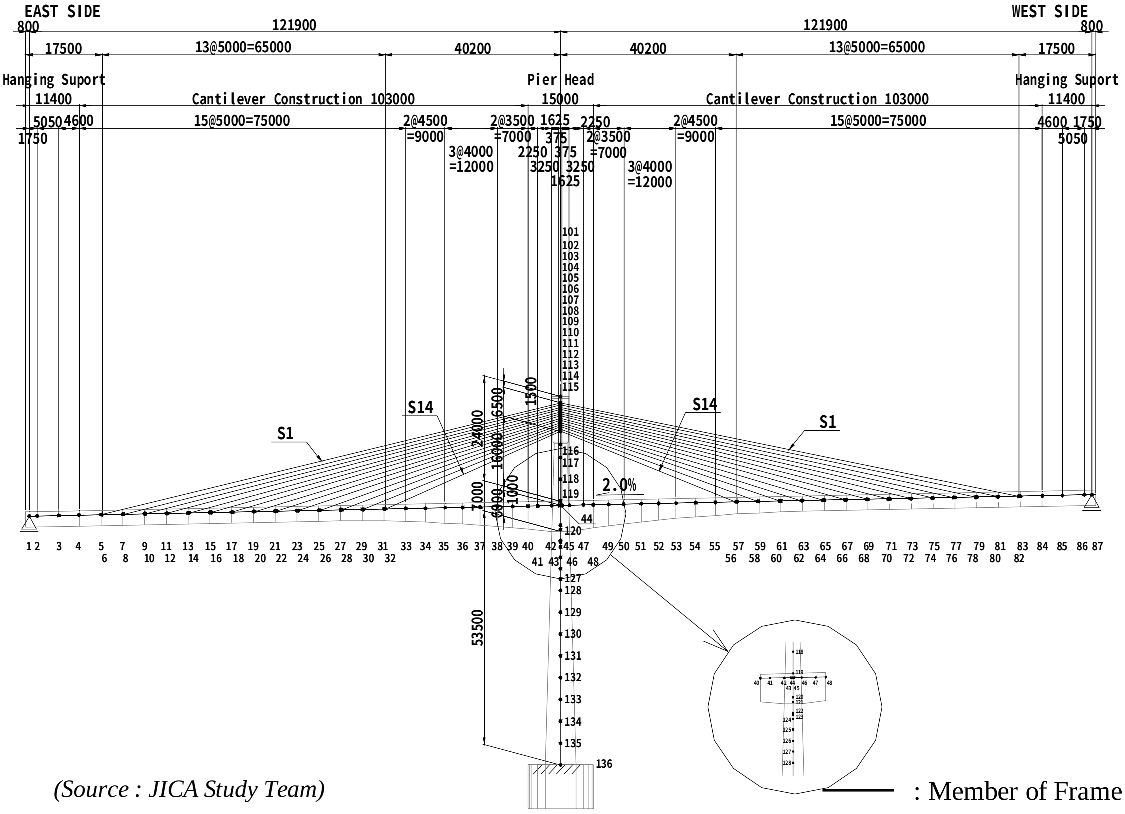
(Source: JICA Study Team)

Figure 5.3.3 Weight of Traveller

(4) Calculation Steps

The balanced cantilever method is assumed as the construction method of bridge. This method requires checking structural safety and stability of each step during erection, considering both thermal and creeping effect caused by concrete age differences between components, such as erection blocks, pier and pylon. The calculation steps and cumulative days from the start of construction of Pier P3 are utilized to calculate concrete ages.

(5) Skeleton of Bridge



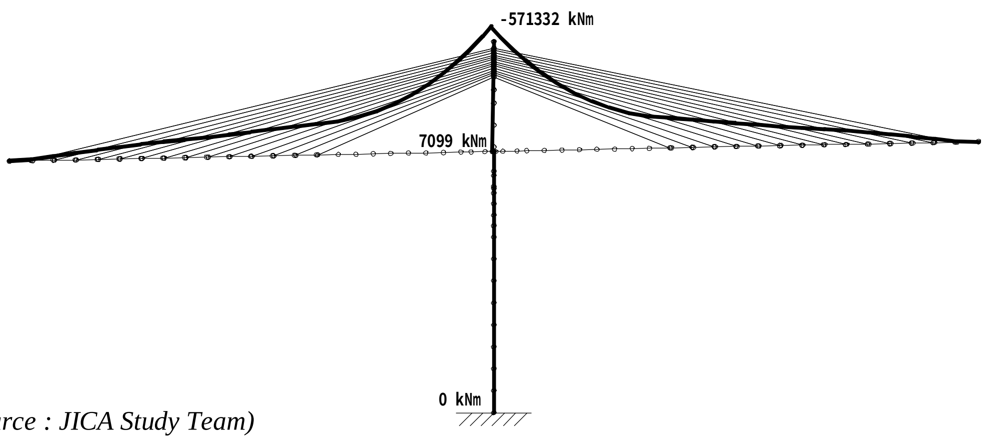
(Source : JICA Study Team)

: Member of Frame

Figure 5.3.4 Skeleton of Bridge

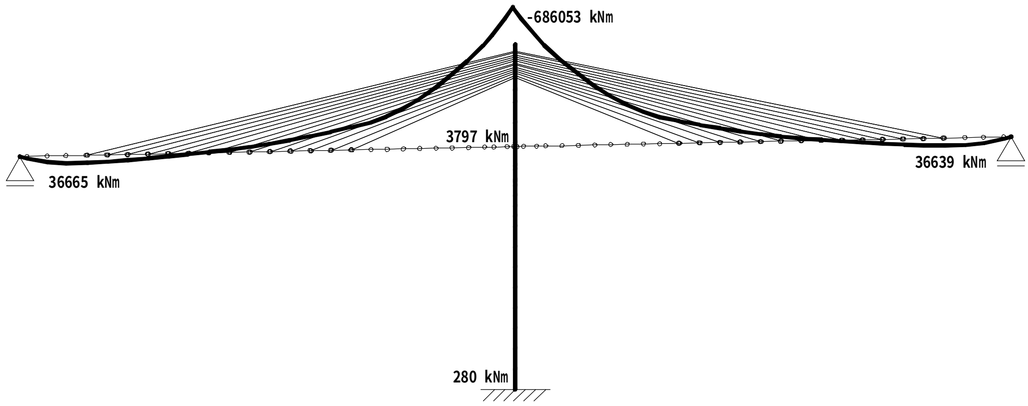
(6) Bending Moment Diagram

From the analysis, resulting bending moment on the main girder is shown in **Figures 5.3.5** to **5.3.7**.



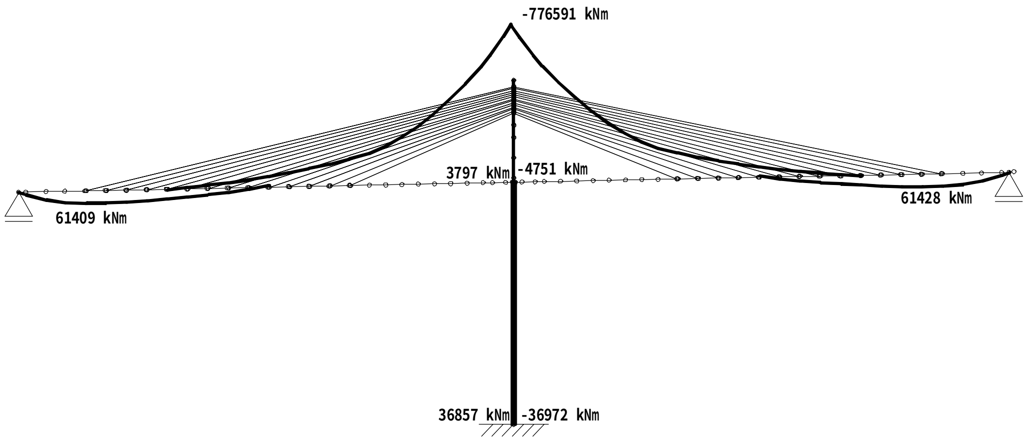
(Source : JICA Study Team)

Figure 5.3.5 Bending Moment during Cantilever Construction



(Source : JICA Study Team)

Figure 5.3.6 Bending Moment Diagram at Dead Load



(Source : JICA Study Team)

Figure 5.3.7 Bending Moment Diagram at Dead Load + Live Load

5.3.3 Seismic Design

(1) Purpose and Consideration of Seismic Design

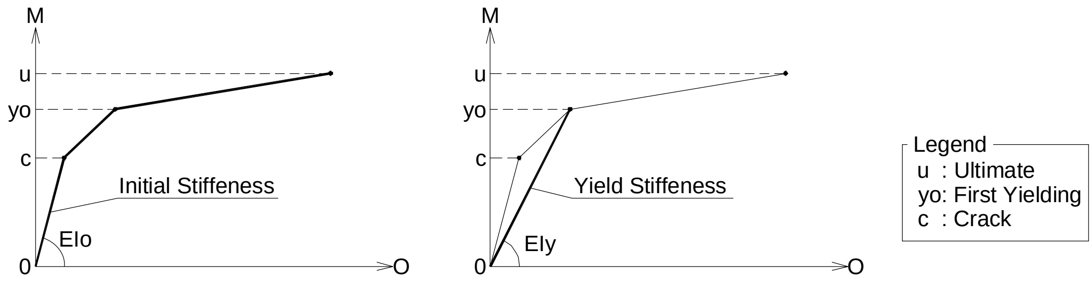
The Seismic design is generally classified into two main analytical methods: the static analysis and the dynamic analysis. In the static analysis, the effect of earthquake ground motion is converted to static inertia force on the structure and the structural responses such as stress and capacities are checked. This calculation method is simple, however, it produces less realistic results for bridges which have complicated seismic behaviour, i.e., multi-degree of redundancy and plural anticipated types of vibration modes. The Naluchi

Bridge, the extra-dosed bridge, is categorised as bridge with complicated seismic behaviour. In such case, the dynamic analysis is opted. The dynamic analysis can reproduce seismic behaviour dynamically by functioning actual earthquake wave and produce accurate time history responses.

(2) Analysis Method

The dynamic analysis is further subdivided into several methods: the response spectrum method and time-history response method. In the design, the time-history response analysis is utilized for the main bridge and to the west side approach viaduct due to their high degree of redundancy. Prior to execution of the dynamic analysis by the time-history response analysis, the vibration characteristics and the damping characteristics were required to be inputted to the model. The vibration characteristics are obtained from the result of the Eigenvalue Analysis. The damping characteristics are calculated from Rayleigh damping formula mentioned below, accomplished by inputting the calculated vibration characteristics such as frequency, and natural period.

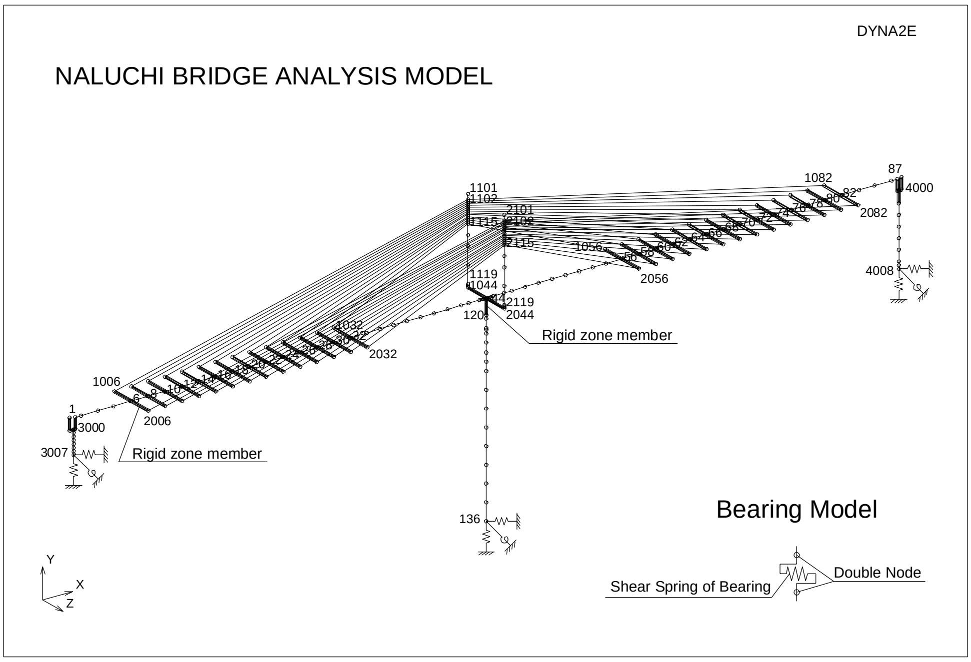
In the time-history response analysis, the El Centro seismic wave form is considered as the input earthquake ground motion. Although it is desirable to use observed seismic record at an actual bridge construction site for seismic wave input, obtaining such data quite complicated. Therefore, the El Centro seismic wave was considered in accordance with AASHTO, which is almost equivalent to maximum acceleration to the earthquake on 8th Oct, 2005. From view point of safety, Naluchi Bridge and its west side approach viaduct should have higher seismic performance. To keep such seismic performance, the response values as a result of the dynamic analysis are verified within yield point. The stiffness of members is defined as the bi-linear model as shown in the below.



(Source : JICA Study Team)

Figure 5.3.8 Applied stiffness model

(3) Analysis Model



(Source : JICA Study Team)

Figure 5.3.9 Analysis Model

Table 5.3.3 Summary of Modelling

Member	Modelling	Remarks
Girder	Beam element	A,Iz,Iy,J
Cross Beam for Stay Cable	Rigid element	A=Iz=Iy=J=infinity
Stay Cable	Beam element	
Substructure	Beam element	Connection of Girder to P3 Column and Pylon, Beam at top of P4 : Rigid zone element
Foundation	6-Direction Spring Coefficient	

(Source: JICA Study Team)

Table 5.3.4 Support Conditions

1 Static Analysis

Support	Horizontral Direction	Vertical Direction	Rotaion	Remarks
P2 Bearing	Free	Fixed	Free	
P3 Base of Column	Fixed	Fixed	Fixed	Ground Eo=300000kN/m^2
P4 Bearing	Free	Fixed	Free	

2 Dynamic Analysis for Longitudinal Direction

Support	Horizontral Direction (kN/m)	Vertical Direction (kN/m)	Rotaion (kNm/rad)	Remarks
P2 Bearing	1.65E+04	Fixed	Free	Bearing Kh=8250kN/m /One
P3 Base of Column	1.40E+07	1.02E+07	9.70E+08	Ground Eo=300000kN/m^2
P4 Bearing	1.65E+04	Fixed	Free	Bearing Kh=8250kN/m /One

3 Dynamic Analysis for Transverse Direction

Support	Horizontral Direction (kN/m)	Vertical Direction (kN/m)	Rotaion (kNm/rad)	Remarks
P2 Bearing	1.65E+04	Fixed	Fixed	Bearing Kh=8250kN/m /One
P3 Base of Column	1.40E+07	1.02E+07	9.70E+08	Ground Eo=300000kN/m^2
P4 Bearing	1.65E+04	Fixed	Fixed	Bearing Kh=8250kN/m /One

Difference in height between the center pier and piers at side span of Naluchi Bridge is large because of its topographic condition. Then, the difference in flexural rigidity of pier stud becomes large. Inertia force is expected to be concentrated at side span piers in case of earthquake. As a result of the comparison study of eigenvalue analysis between restrained and spring on support at the end of girder, it is found that there are significant differences in the natural period and the response acceleration. The result is shown in the table below. After examining the result, spring is selected for support on P2 and P4 in transverse direction since spring is considered to be closer to the actual behaviour.

Support Condition (Transverse direction)		Restrained	Spring
Natural Period (s)		2.84	3.49
Response Acceleration (gal)	P2	652	267
	P3	253	169
	P4	484	185

(4) Eigenvalue Analysis

a. Purpose and Method of Eigenvalue Analysis

From the Eigen value analysis outputs, vibration mode and viscosity damping characteristics are calculated. The dynamic analysis takes bridge damping effect calculated by viscosity damping matrix into consideration. The major viscosity damping matrix is as follows;

1) Viscosity damping matrix equivalent to "Mode damping coefficient" :Ceq
2) Viscosity damping matrix relative to mass :$CM=\alpha M$
3) Viscosity damping matrix relative to stiffness :$CK=\beta K$
4) Viscosity damping matrix in accordance with Reyleigh's formula :$C_R=\alpha M+\beta K$

From the above type of matrices, the matrix in accordance with Reyleigh's formula is applied to the dynamic analysis of Naluchi Bridge since Reyleigh equation gives proper viscosity damping matrix even in wide natural period.

Calculation formula of Reyliegh type viscosity damping coefficient is given as follows:

$$[C]=\alpha\cdot[Ko]+\beta\cdot[M]$$

$$\alpha=(fi\cdot hi-fj\cdot hj)/\{\pi\cdot(fi\text{^}2-fj^2)\}$$
$$\beta=4\pi\cdot fi\cdot(hi-a\cdot\pi\cdot fi)$$

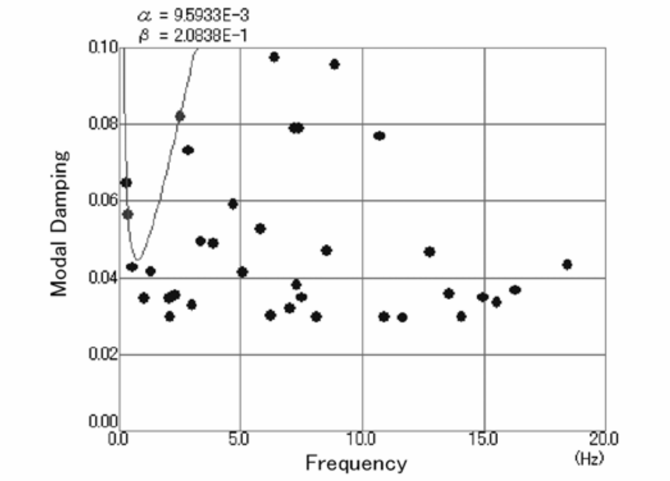
[C] : Damping Matrix
[Ko] : Elasticity Stiffeness Matrix
[M] : Mass Matrx
a : Coefficient of Stiffeness Matrix for Structural Syste
b : Coefficient of Mass Matrix for Structural System
f i : Freqency at "i"th Mode
f j : Freqency at "j"th Mode
h i : Damping Coefficient at "i"th Mode
h j : Damping Coefficient at "j"th Mode

b. Reyliegh Type Viscosity Damping Coefficient

Calculation result of Reyliegh type viscosity damping coefficient is presented below.

Longitudinal

Mode	Frequency fi (Hz)	Modal Damping hi	a	b
2	0.363	0.05665	9. 5933E-03	2. 0838E-01
9	2.499	0.08195		



Transverse

Mode	Frequency fi (Hz)	Modal Damping hi	a	b
1	0.287	0.06485	7. 5871E-03	2. 0919E-01
10	2.826	0.07326		

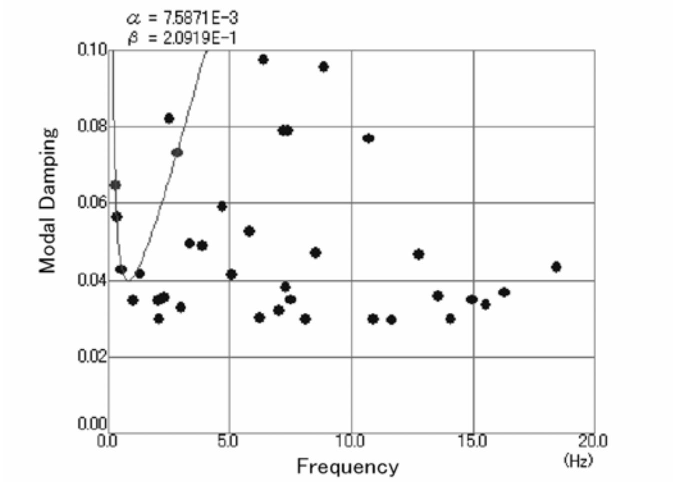
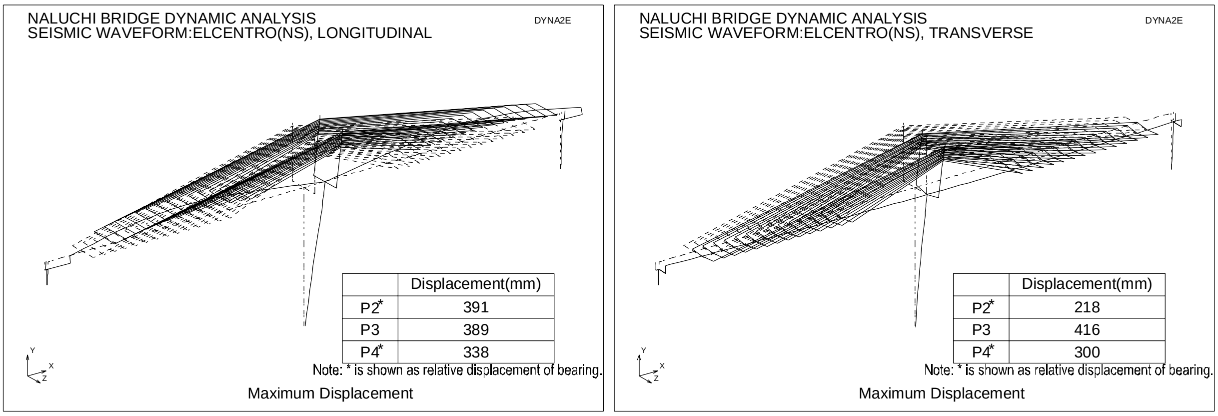


Figure 5.3.10 The relationship Between Frequency and Damping Coefficient

(5) Dynamic Analysis

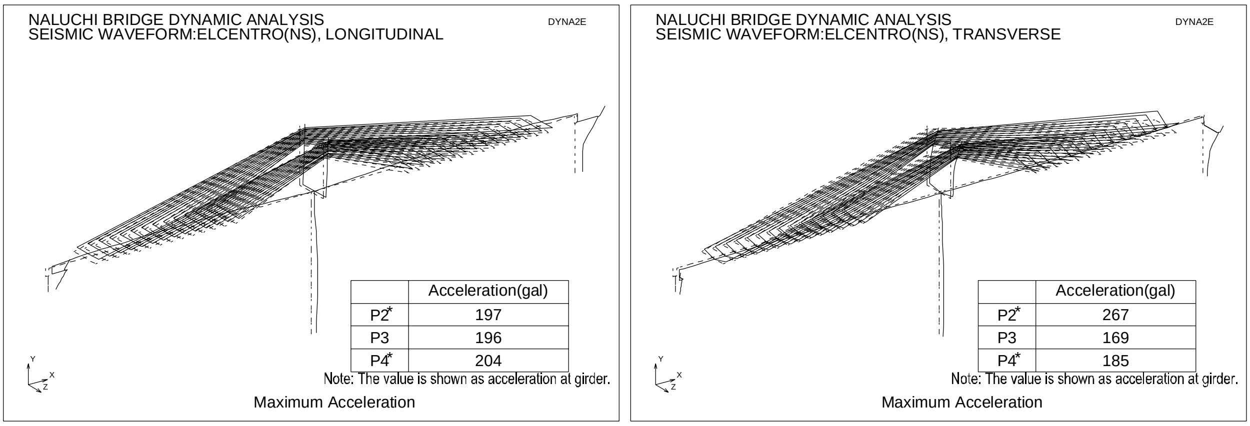
a. Maximum Displacement



(Source : JICA Study Team)

Figure 5.3.11 Maximum Displacement

b. Maximum Acceleration

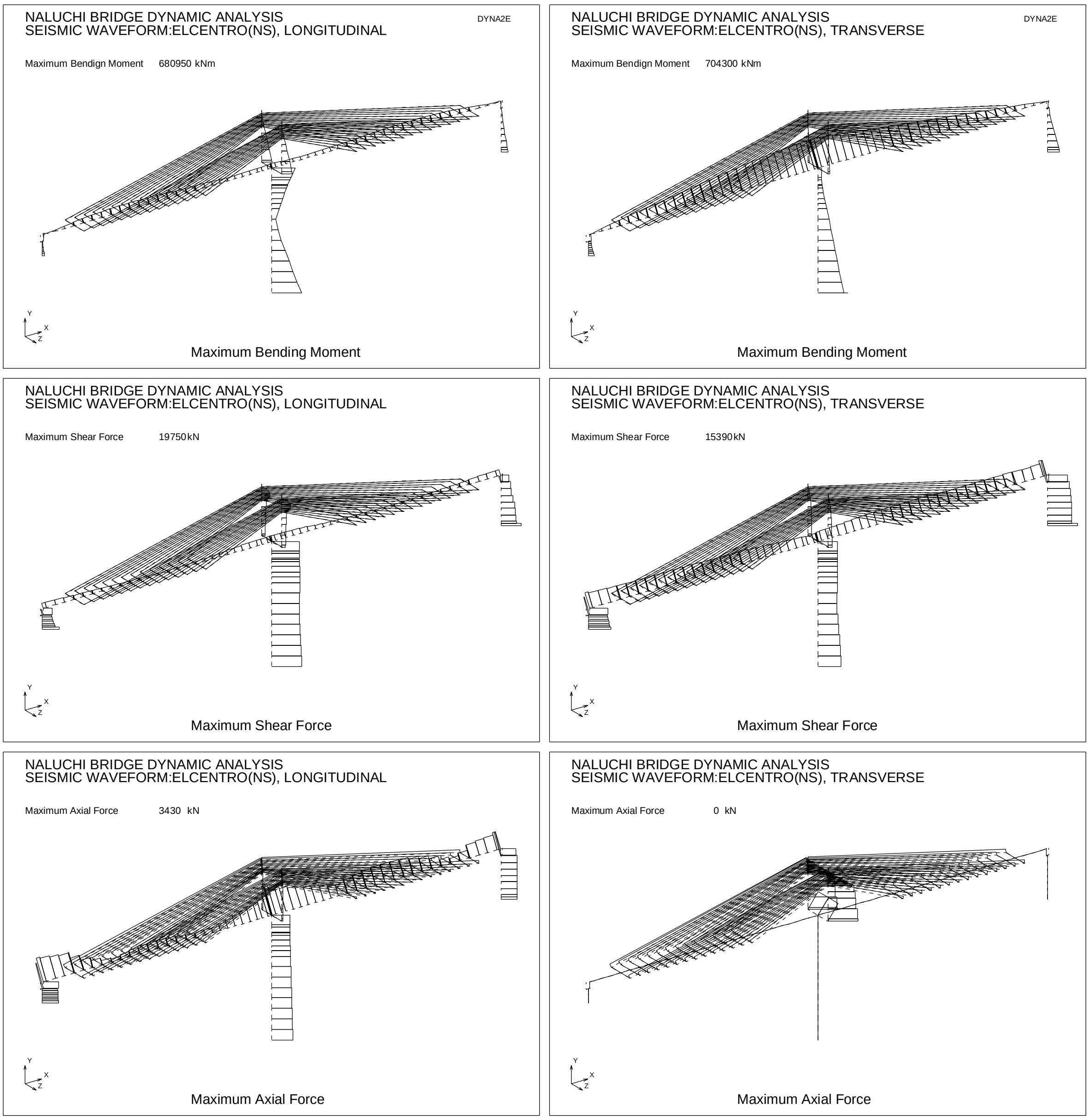


(Source : JICA Study Team)

Figure 5.3.12 Maximum Acceleration

From above analysis, it was considered that whole bridge has dynamic characteristics of long period and relatively large response displacement because of its structural configuration of two continuous long spans with high pier in the middle. On the other hand, end piers at both sides have short period and small response displacement but large acceleration because of the low pier with high rigidity.

c. Response Section Force



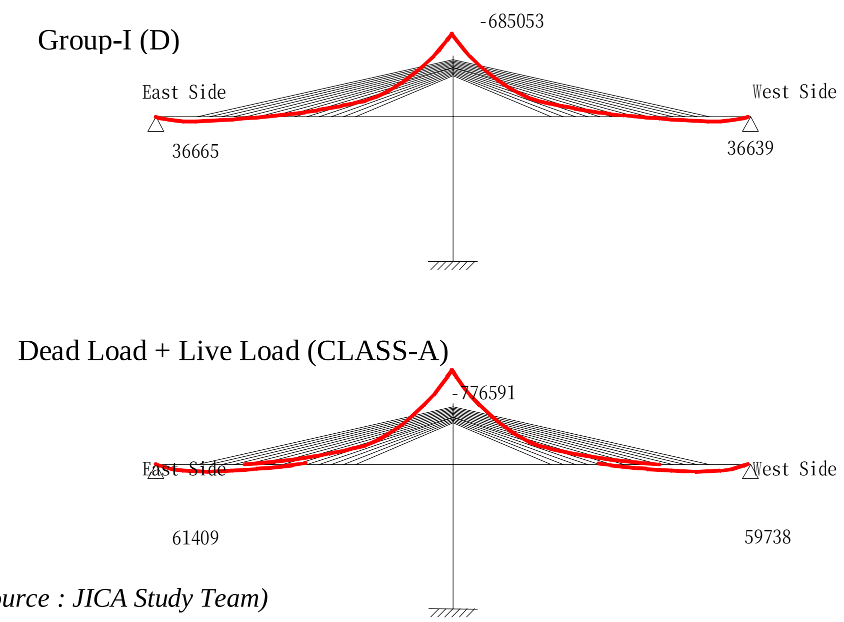
(Source : JICA Study Team)

Figure 5.3.13 Response Sectional Force Diagram

5.3.4 Summary of Structure Analysis and Cable Arrangement

The results of structure analysis are shown in the subsequent sub-chapters. In both the superstructure and substructure, it has been confirmed calculated results are within allowable values in each condition in accordance with applied standards.

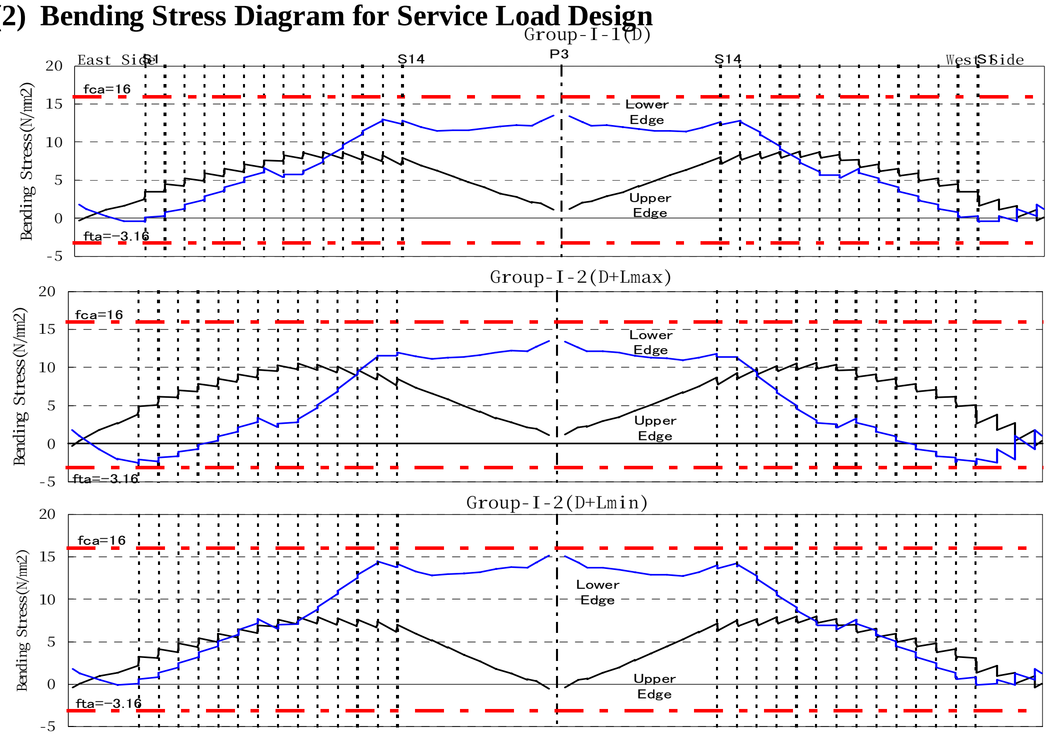
(1) Bending Moment Diagram for Service Load Design



(Source : JICA Study Team)

Figure 5.3.14 Bending Moment Diagram of Girder

(2) Bending Stress Diagram for Service Load Design



(Source : JICA Study Team)

Figure 5.3.15 Bending Stress Diagram of Girder for Service Load Design

Table 5.3.5 Design Summary for Service Load Design (1)

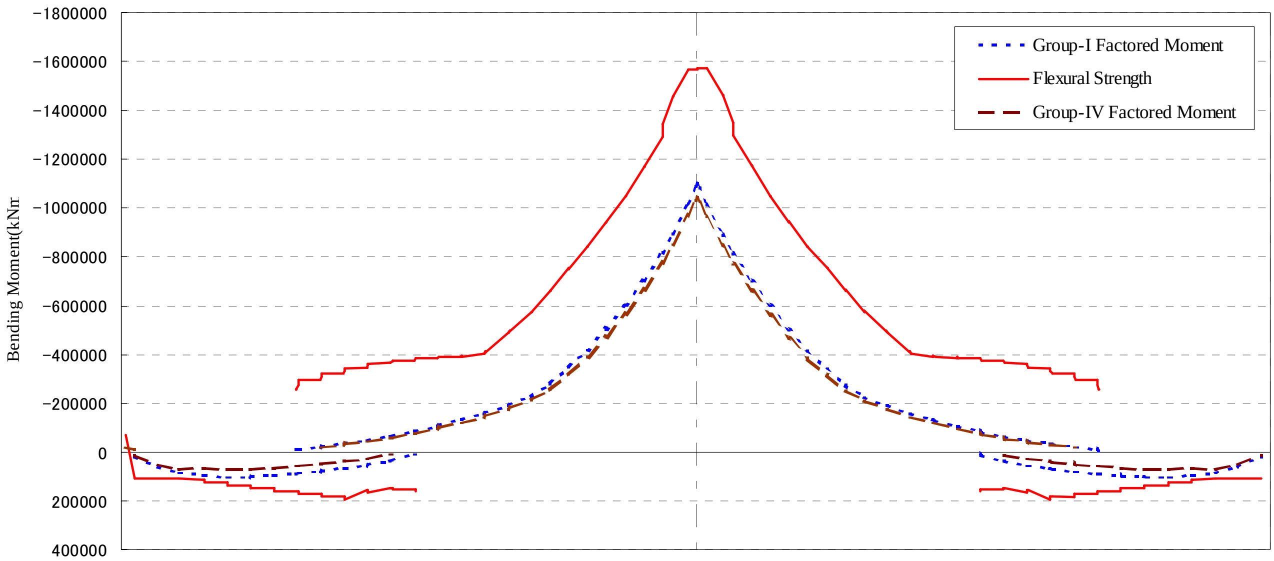
		S1-Block Joint No. 5				S2-Block Joint No. 7				S3-Block Joint No. 9				S4-Block Joint No. 11			
		Sectional Force		σcu	σcl	Sectional Force		σcu	σcl	Sectional Force		σcu	σcl	Sectional Force		σcu	σcl
		M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)
Self Weight + Stay Cable	D	-18776	-79	-1.11	1.73	-34319	8211	-1.42	3.79	-50874	16496	-1.78	5.94	-66527	24787	-2.10	8.01
Creep Effect of Self Weight	D	24782	30	1.45	-2.26	32497	160	1.91	-2.96	40368	298	2.38	-3.67	48424	457	2.86	-4.39
Surfacing	D	28573	24	1.67	-2.61	33962	83	1.99	-3.10	37849	164	2.22	-3.45	40263	268	2.37	-3.66
Creep Effect of Sufracing	D	-3330	-4	-0.20	0.30	-4277	64	-0.25	0.40	-5122	155	-0.29	0.48	-5835	268	-0.32	0.55
2ndary Force by Prestress	D	2796	3	0.17	-0.26	3481	-116	0.20	-0.33	4020	-277	0.22	-0.39	4327	-477	0.22	-0.44
Creep Effect of 2ndary Force	D	-7208	-9	-0.42	0.66	-9532	-192	-0.57	0.86	-12145	-398	-0.74	1.08	-15026	-628	-0.92	1.33
Effective Prestress	D	-16629	18439	0.39	2.91	-10938	21996	0.99	2.65	-6495	25574	1.51	2.50	-2023	29151	2.05	2.35
Losses of Prestress	D	1317	2	0.08	-0.12	1724	8	0.10	-0.16	2146	20	0.13	-0.20	2584	34	0.15	-0.23
CLASS-A Live Load Mmax	L	23623	26	1.39	-2.16	27920	-57	1.63	-2.56	31311	-114	1.82	-2.87	33346	-155	1.93	-3.06
CLASS-A Live Load Mmin	L	-5013	-6	-0.29	0.46	-6432	103	-0.37	0.60	-7733	206	-0.44	0.72	-8920	303	-0.50	0.84
CLASS-A Live Load Nmax	L	23158	27	1.36	-2.11	3335	106	0.20	-0.30	-7732	206	-0.44	0.72	-8919	303	-0.50	0.84
CLASS-A Live Load Nmin	L	406	-8	0.02	-0.04	25474	-62	1.49	-2.33	31253	-118	1.82	-2.86	33346	-155	1.93	-3.06
Shrinkage	S	7605	9	0.45	-0.70	9477	-425	0.52	-0.90	10860	-853	0.57	-1.06	11749	-1275	0.59	-1.17
Thermal Rise	T	-2317	-3	-0.14	0.21	-2889	128	-0.16	0.27	-3314	257	-0.18	0.32	-3591	382	-0.18	0.36
Temparature Difference	T	-8472	-10961	0.25	-0.03	-7386	-10945	0.31	-0.13	-6284	-10930	0.38	-0.23	-5165	-10916	0.45	-0.33
Group-I (L-Mmax)				3.41	-1.81			4.58	-1.41			5.47	-0.57			6.24	0.46
Group-I (L-Mmin)				1.73	0.81			2.59	1.74			3.22	3.02			3.81	4.35
Group-I (L-Nmax)				3.38	-1.76			3.16	0.85			3.22	3.02			3.81	4.35
Group-I (L-Nmin)				2.05	0.31			4.44	-1.19			5.47	-0.57			6.24	0.46
Allowable Stress for Group-I				-3.16<σ<16.00				-3.16<σ<16.00				-3.16<σ<16.00				-3.16<σ<16.00	
Group-IV (L-Mmax, T-Rise)				3.72	1.79			4.95	1.79			5.87	1.79			6.66	1.79
Group-IV (L-Mmin, T-Rise)				2.04	1.79			2.95	1.79			3.61	1.79			4.22	1.79
Group-IV (L-Nmax, T-Rise)				3.69	2.07			3.52	2.07			3.61	2.07			4.22	2.07
Group-IV (L-Nmin, T-Rise)				2.36	2.07			4.80	2.07			5.87	2.07			6.66	2.07
Group-IV (L-Mmax, T-Down)				3.99	-2.71			5.27	-2.58			6.22	-1.95			7.02	-1.06
Group-IV (L-Mmin, T-Down)				2.31	-0.10			3.27	0.57			3.96	1.64			4.59	2.83
Group-IV (L-Nmax, T-Down)				3.96	-2.67			3.84	-0.32			3.96	1.64			4.59	2.83
Group-IV (L-Nmin, T-Down)				2.63	-0.59			5.12	-2.36			6.22	-1.94			7.02	-1.06
Group-IV (L-Mmax, T-Rise, T-Diff.)				3.97	-2.32			5.26	-2.17			6.25	-1.54			7.10	-0.68
Group-IV (L-Mmin, T-Rise, T-Diff.)				2.29	0.29			3.26	0.98			3.99	2.06			4.67	3.21
Group-IV (L-Nmax, T-Rise, T-Diff.)				3.94	-2.28			3.84	0.09			3.99	2.06			4.67	3.21
Group-IV (L-Nmin, T-Rise, T-Diff.)				2.60	-0.20			5.12	-1.94			6.25	-1.53			7.10	-0.68
Group-IV (L-Mmax, T-Down, T-Diff.)				4.24	-2.75			5.58	-2.71			6.60	-2.18			7.46	-1.40
Group-IV (L-Mmin, T-Down, T-Diff.)				2.56	-0.13			3.58	0.44			4.34	1.41			5.03	2.50
Group-IV (L-Nmax, T-Down, T-Diff.)				4.21	-2.70			4.15	-0.46			4.34	1.41			5.03	2.50
Group-IV (L-Nmin, T-Down, T-Diff.)				2.88	-0.63			5.44	-2.49			6.60	-2.17			7.46	-1.40
Allowable Stress for Group-IV				-3.95<σ<20.00				-3.95<σ<20.00				-3.95<σ<20.00				-3.95<σ<20.00	

		S5-Block Joint No. 13				S6-Block Joint No. 15				S7-Block Joint No. 17				S8-Block Joint No. 19			
		Sectional Force		σcu	σcl	Sectional Force		σcu	σcl	Sectional Force		σcu	σcl	Sectional Force		σcu	σcl
		M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)
Self Weight + Stay Cable	D	-80905	33035	-2.34	9.96	-93633	41311	-2.48	11.76	-104442	49491	-2.52	13.37	-113056	57551	-2.43	14.72
Creep Effect of Self Weight	D	56661	605	3.35	-5.14	65064	737	3.85	-5.90	73614	849	4.36	-6.67	82293	943	4.87	-7.48
Surfacing	D	41235	393	2.43	-3.74	40796	539	2.42	-3.69	38980	706	2.32	-3.51	35819	889	2.15	-3.22
Creep Effect of Sufracing	D	-6387	403	-0.34	0.61	-6747	557	-0.35	0.66	-6884	730	-0.35	0.68	-6769	918	-0.33	0.69
2ndary Force by Prestress	D	4351	-713	0.21	-0.46	4041	-981	0.17	-0.45	3348	-1278	0.10	-0.41	2223	-1598	0.01	-0.32
Creep Effect of 2ndary Force	D	-18209	-879	-1.13	1.60	-21731	-1151	-1.35	1.90	-25628	-1440	-1.60	2.24	-29939	-1747	-1.88	2.62
Effective Prestress	D	2465	32730	2.58	2.20	6916	36278	3.11	2.06	11288	39753	3.63	1.91	29746	36127	4.45	-0.06
Losses of Prestress	D	3046	54	0.18	-0.27	3538	78	0.21	-0.32	4067	107	0.25	-0.36	4640	140	0.28	-0.42
CLASS-A Live Load Mmax	L	34208	-172	1.98	-3.14	34827	-157	2.02	-3.19	35238	-134	2.04	-3.23	34571	-99	2.01	-3.18
CLASS-A Live Load Mmin	L	-9998	392	-0.55	0.94	-10974	474	-0.60	1.04	-11854	547	-0.65	1.12	-12647	611	-0.69	1.21
CLASS-A Live Load Nmax	L	-9799	392	-0.54	0.92	-4403	477	-0.22	0.44	1160	568	0.11	-0.06	-588	660	0.01	0.10
CLASS-A Live Load Nmin	L	33548	-173	1.94	-3.08	29705	-173	1.72	-2.73	19676	-164	1.13	-1.81	18813	-151	1.08	-1.74
Shrinkage	S	12133	-1690	0.58	-1.23	12005	-2099	0.55	-1.25	11351	-2503	0.48	-1.22	10156	-2903	0.38	-1.15
Thermal Rise	T	-3719	506	-0.18	0.38	-3696	626	-0.17	0.38	-3520	744	-0.15	0.38	-3188	859	-0.12	0.36
Temparature Difference	T	-4031	-10904	0.51	-0.43	-2882	-10892	0.58	-0.54	-1718	-10880	0.65	-0.64	-536	-10867	0.72	-0.75
Group-I (L-Mmax)				6.93	1.63			7.59	2.83			8.23	4.02			9.14	3.35
Group-I (L-Mmin)				4.39	5.71			4.97	7.06			5.54	8.37			6.45	7.73
Group-I (L-Nmax)				4.40	5.69			5.35	6.46			6.30	7.19			7.15	6.63
Group-I (L-Nmin)				6.89	1.69			7.29	3.30			7.32	5.44			8.22	4.79
Allowable Stress for Group-I				-3.16<σ<16.00				-3.16<σ<16.00				-3.16<σ<16.00				-3.16<σ<16.00	
Group-IV (L-Mmax, T-Rise)				7.33	1.79			7.96	1.79			8.56	1.79			9.40	1.79
Group-IV (L-Mmin, T-Rise)				4.79	1.79			5.34	1.79			5.87	1.79			6.70	1.79
Group-IV (L-Nmax, T-Rise)				4.81	2.07			5.73	2.07			6.63	2.07			7.41	2.07
Group-IV (L-Nmin, T-Rise)				7.29	2.07			7.66	2.07			7.65	2.07			8.48	2.07
Group-IV (L-Mmax, T-Down)				7.69	0.02			8.30	1.20			8.86	2.43			9.64	1.85
Group-IV (L-Mmin, T-Down)				5.15	4.10			5.68	5.42			6.17	6.78			6.95	6.23
Group-IV (L-Nmax, T-Down)				5.17	4.08			6.06	4.82			6.93	5.59			7.65	5.13
Group-IV (L-Nmin, T-Down)				7.65	0.08			8.00	1.66			7.95	3.85			8.72	3.29
Group-IV (L-Mmax, T-Rise, T-Diff.)				7.84	0.34			8.55	1.43			9.21	2.54			10.12	1.81
Group-IV (L-Mmin, T-Rise, T-Diff.)				5.31	4.42			5.92	5.66			6.52	6.89			7.42	6.19
Group-IV (L-Nmax, T-Rise, T-Diff.)				5.32	4.40			6.31	5.06			7.28	5.70			8.13	5.09
Group-IV (L-Nmin, T-Rise, T-Diff.)				7.80	0.40			8.25	1.89			8.30	3.96			9.20	3.25
Group-IV (L-Mmax, T-Down, T-Diff.)				8.20	-0.41			8.88	0.66			9.51	1.79			10.36	1.10
Group-IV (L-Mmin, T-Down, T-Diff.)				5.67	3.67			6.26	4.89			6.82	6.14			7.67	5.48
Group-IV (L-Nmax, T-Down, T-Diff.)				5.68	3.65			6.65	4.29			7.58	4.95			8.37	4.38
Group-IV (L-Nmin, T-Down, T-Diff.)				8.16	-0.35			8.58	1.13			8.60	3.20			9.44	2.54
Allowable Stress for Group-IV				-3.95<σ<20.00				-3.95<σ<20.00				-3.95<σ<20.00				-3.95<σ<20.00	

Table 5.3.6 Design Summary for Service Load Design (2)

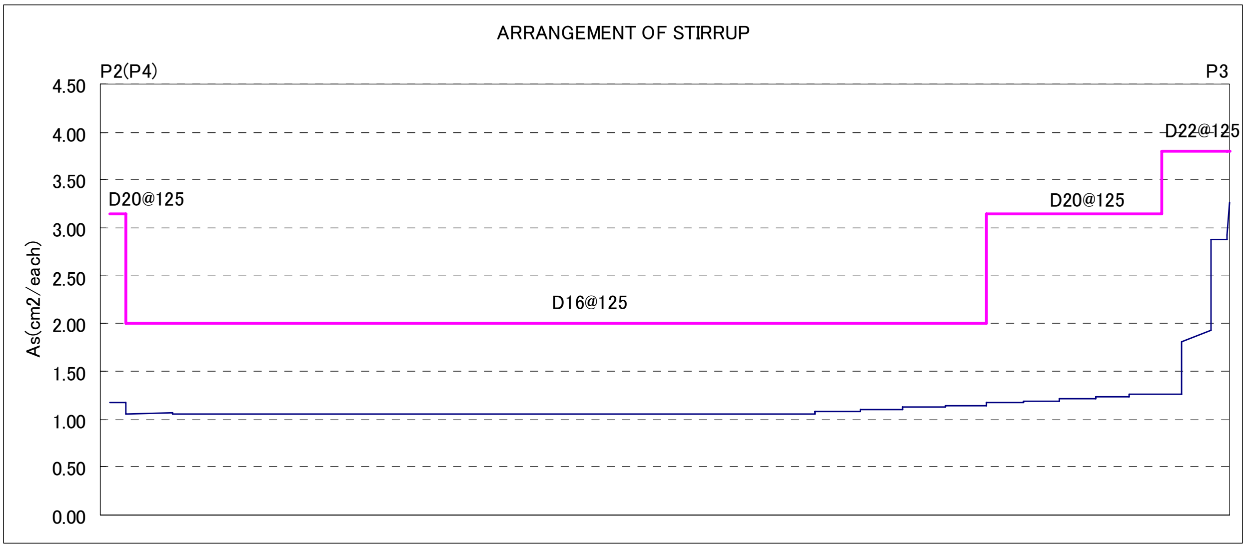
		S9-Block Joint No. 21				S10-Block Joint No. 23				S11-Block Joint No. 25				S12-Block Joint No. 27			
		Sectional Force		σcu	σcl	Sectional Force		σcu	σcl	Sectional Force		σcu	σcl	Sectional Force		σcu	σcl
		M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)
Self Weight + Stay Cable	D	-122158	63236	-2.55	15.96	-131591	68888	-2.69	17.25	-140977	74486	-2.84	18.54	-149893	80010	-2.87	19.61
Creep Effect of Self Weight	D	91063	1006	5.39	-8.30	99934	1075	5.91	-9.11	108915	1145	6.44	-9.93	118011	1214	6.89	-10.63
Surfacing	D	31267	1028	1.90	-2.80	25343	1176	1.56	-2.24	18068	1330	1.15	-1.56	9462	1486	0.65	-0.75
Creep Effect of Sufracing	D	-6449	1059	-0.30	0.67	-5910	1208	-0.26	0.63	-5133	1360	-0.20	0.57	-4102	1514	-0.13	0.48
2ndary Force by Prestress	D	752	-1839	-0.09	-0.21	-1090	-2089	-0.22	-0.06	-3334	-2347	-0.37	0.13	-6013	-2610	-0.55	0.35
Creep Effect of 2ndary Force	D	-34577	-1974	-2.16	3.03	-39566	-2212	-2.47	3.47	-44941	-2459	-2.80	3.95	-50741	-2716	-3.13	4.41
Effective Prestress	D	40869	36016	5.10	-1.09	44918	39558	5.61	-1.20	48967	43096	6.12	-1.31	53414	46605	6.61	-1.41
Losses of Prestress	D	5251	166	0.32	-0.47	5903	194	0.36	-0.53	6600	223	0.40	-0.59	7346	253	0.44	-0.65
CLASS-A Live Load Mmax	L	32749	-62	1.90	-3.01	29974	-6	1.74	-2.75	26456	52	1.54	-2.43	22052	117	1.27	-1.99
CLASS-A Live Load Mmin	L	-13382	650	-0.73	1.28	-14069	682	-0.77	1.34	-14717	707	-0.80	1.40	-15335	725	-0.83	1.45
CLASS-A Live Load Nmax	L	-3010	723	-0.12	0.33	-5105	784	-0.24	0.53	-3635	847	-0.15	0.40	-742	918	0.03	0.13
CLASS-A Live Load Nmin	L	20798	-135	1.20	-1.92	18392	-115	1.06	-1.70	15182	-94	0.88	-1.40	8418	-78	0.48	-0.77
Shrinkage	S	8556	-3183	0.26	-1.02	6543	-3462	0.13	-0.86	4096	-3739	-0.04	-0.65	1190	-4016	-0.23	-0.40
Thermal Rise	T	-2741	938	-0.09	0.32	-2179	1016	-0.05	0.28	-1497	1092	-0.01	0.22	-693	1166	0.05	0.15
Temparature Difference	T	659	-10858	0.79	-0.86	1869	-10847	0.86	-0.97	3097	-10836	0.94	-1.08	4219	-10824	1.00	-1.17
Group-I (L-Mmax)				9.51	3.79			9.54	5.48			9.43	7.38			9.19	9.42
Group-I (L-Mmin)				6.87	8.08			7.03	9.57			7.09	11.21			7.09	12.86
Group-I (L-Nmax)				7.48	7.13			7.56	8.76			7.74	10.21			7.94	11.55
Group-I (L-Nmin)				8.81	4.88			8.86	6.53			8.77	8.41			8.39	10.64
Allowable Stress for Group-I				-3.16<σ<16.00				-3.16<σ<16.00				-3.16<σ<16.00				-3.16<σ<16.00	
Group-IV (L-Mmax, T-Rise)				9.68	1.79			9.62	1.79			9.39	1.79			9.01	1.79
Group-IV (L-Mmin, T-Rise)				7.05	1.79			7.11	1.79			7.05	1.79			6.91	1.79
Group-IV (L-Nmax, T-Rise)				7.66	2.07			7.63	2.07			7.70	2.07			7.76	2.07
Group-IV (L-Nmin, T-Rise)				8.98	2.07			8.93	2.07			8.72	2.07			8.21	2.07
Group-IV (L-Mmax, T-Down)				9.86	2.45			9.72	4.35			9.40	6.51			8.92	8.87
Group-IV (L-Mmin, T-Down)				7.23	6.74			7.21	8.44			7.06	10.34			6.82	12.31
Group-IV (L-Nmax, T-Down)				7.84	5.79			7.74	7.63			7.71	9.34			7.67	11.00
Group-IV (L-Nmin, T-Down)				9.16	3.54			9.04	5.40			8.74	7.54			8.12	10.09
Group-IV (L-Mmax, T-Rise, T-Diff.)				10.47	2.23			10.48	3.93			10.32	5.87			10.01	7.99
Group-IV (L-Mmin, T-Rise, T-Diff.)				7.84	6.52			7.97	8.03			7.98	9.70			7.91	11.43
Group-IV (L-Nmax, T-Rise, T-Diff.)				8.45	5.58			8.50	7.21			8.63	8.69			8.76	10.12
Group-IV (L-Nmin, T-Rise, T-Diff.)				9.77	3.33			9.80	4.99			9.66	6.90			9.21	9.22
Group-IV (L-Mmax, T-Down, T-Diff.)				10.65	1.59			10.58	3.38			10.34	5.43			9.92	7.70
Group-IV (L-Mmin, T-Down, T-Diff.)				8.02	5.88			8.07	7.48			7.99	9.27			7.82	11.14
Group-IV (L-Nmax, T-Down, T-Diff.)				8.63	4.93			8.60	6.66			8.65	8.26			8.67	9.82
Group-IV (L-Nmin, T-Down, T-Diff.)				9.95	2.68			9.90	4.44			9.67	6.46			9.12	8.92
Allowable Stress for Group-IV				-3.95<σ<20.00				-3.95<σ<20.00				-3.95<σ<20.00				-3.95<σ<20.00	

		S13-Block Joint No. 29				S14-Block Joint No. 31				P3(L) Joint No. 42			
		Sectional Force		σcu	σcl	Sectional Force		σcu	σcl	Sectional Force		σcu	σcl
		M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)
Self Weight + Stay Cable	D	-157570	85634	-2.65	20.22	-165408	91137	-2.44	17.76	-521478	96156	-6.97	13.69
Creep Effect of Self Weight	D	127228	1282	7.18	-11.10	136578	1352	7.01	-9.47	209964	1435	4.18	-3.98
Surfacing	D	-456	1642	0.09	0.16	-11671	1794	-0.47	0.94	-149065	1896	-2.83	2.93
Creep Effect of Sufracing	D	-2803	1665	-0.04	0.37	-1225	1810	0.06	0.21	13255	1948	0.33	-0.18
2ndary Force by Prestress	D	-9157	-2872	-0.73	0.60	-12798	-3132	-0.88	0.69	-45097	-3384	-1.04	0.75
Creep Effect of 2ndary Force	D	-57010	-2981	-3.39	4.80	-63792	-3250	-3.45	4.25	-120602	-3521	-2.51	2.19
Effective Prestress	D	58785	50035	7.03	-1.50	72067	53493	7.34	-1.46	324638	107579	10.61	-2.26
Losses of Prestress	D	8147	283	0.47	-0.70	9008	315	0.48	-0.61	16168	346	0.33	-0.30
CLASS-A Live Load Mmax	L	16944	158	0.95	-1.48	12816	15	0.65	-0.90	165	-9	0.00	0.00
CLASS-A Live Load Mmin	L	-16000	814	-0.83	1.47	-18435	956	-0.87	1.35	-86066	1078	-1.63	1.69
CLASS-A Live Load Nmax	L	-7449	994	-0.34	0.73	-13931	1069	-0.63	1.05	-70839	1140	-1.33	1.40
CLASS-A Live Load Nmin	L	6465	-63	0.35	-0.57	5312	-49	0.26	-0.38	-13162	-63	-0.26	0.25
Shrinkage	S	-2209	-4294	-0.44	-0.12	-6140	-4572	-0.62	0.13	-41137	-4852	-0.98	0.61
Thermal Rise	T	241	1239	0.10	0.07	1309	1309	0.15	-0.01	10696	1378	0.26	-0.16
Temparature Difference	T	5120	-10810	1.05	-1.23	4376	-10841	1.05	-1.02	-7941	-11162	0.99	-0.26
Group-I (L-Mmax)				8.92	11.37			8.30	11.41			2.11	12.83
Group-I (L-Mmin)				7.14	14.31			6.79	13.65			0.47	14.53
Group-I (L-Nmax)				7.63	13.57			7.03	13.35			0.77	14.24
Group-I (L-Nmin)				8.32	12.27			7.92	11.93			1.85	13.09
Allowable Stress for Group-I				-3.16<σ<16.00				-3.16<σ<16.00				-3.16<σ<16.00	
Group-IV (L-Mmax, T-Rise)				8.59	1.79			7.84	1.79			1.39	1.79
Group-IV (L-Mmin, T-Rise)				6.81	1.79			6.33	1.79			-0.25	1.79
Group-IV (L-Nmax, T-Rise)				7.30	2.07			6.56	2.07			0.05	2.07
Group-IV (L-Nmin, T-Rise)				7.99	2.07			7.46	2.07			1.13	2.07
Group-IV (L-Mmax, T-Down)				8.38	11.18			7.54	11.54			0.87	13.60
Group-IV (L-Mmin, T-Down)				6.60	14.13			6.02	13.79			-0.77	15.29
Group-IV (L-Nmax, T-Down)				7.09	13.39			6.26	13.48			-0.47	15.00
Group-IV (L-Nmin, T-Down)				7.78	12.09			7.15	12.06			0.61	13.85
Group-IV (L-Mmax, T-Rise, T-Diff.)				9.64	10.09			8.89	10.50			2.37	13.03
Group-IV (L-Mmin, T-Rise, T-Diff.)				7.86	13.03			7.38	12.75			0.74	14.73
Group-IV (L-Nmax, T-Rise, T-Diff.)				8.35	12.29			7.62	12.44			1.04	14.44
Group-IV (L-Nmin, T-Rise, T-Diff.)				9.04	10.99			8.51	11.02			2.11	13.29
Group-IV (L-Mmax, T-Down, T-Diff.)				9.43	9.95			8.59	10.51			1.86	13.34
Group-IV (L-Mmin, T-Down, T-Diff.)				7.65	12.89			7.08	12.76			0.22	15.04
Group-IV (L-Nmax, T-Down, T-Diff.)				8.14	12.15			7.31	12.45			0.52	14.75
Group-IV (L-Nmin, T-Down, T-Diff.)				8.83	10.85			8.21	11.03			1.60	13.60
Allowable Stress for Group-IV				-3.95<σ<20.00				-3.95<σ<20.00				-3.95<σ<20.00	



(Source : JICA Study Team)

Figure 5.3.16 Flexural Strength Diagram of Girder for Load Factor Design

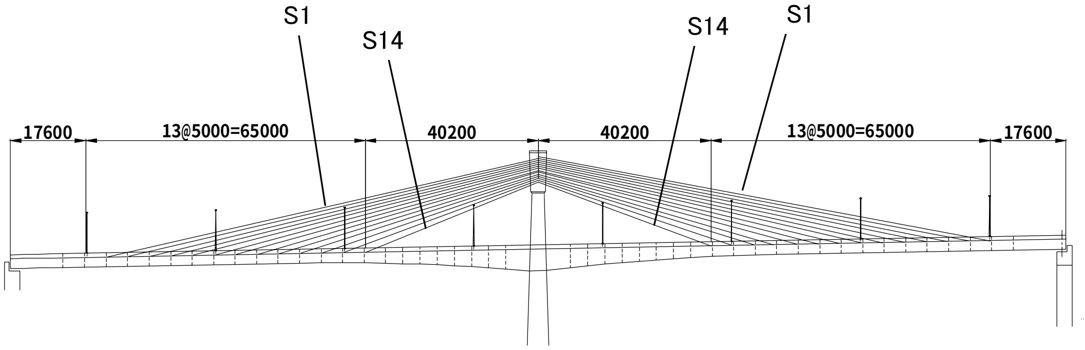


(Source : JICA Study Team)

Figure 5.3.17 Required Shear Reinforcement Diagram of Girder for Load Factor Design

(3) Summary of Stay Cable

Table 5.3.7 Summary of Tensile Force for Stay Cable



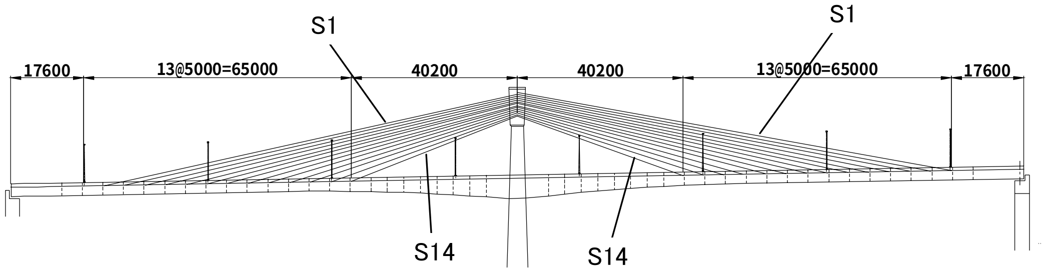
2.6.1 Summary of Tensile Force for Both Side Stay Cable Total (per both side)

No.	Initial Prestress **D** (kN)	Self Weight **D** (kN)	Surfacing **D** (kN)	Shrinkage **D** (kN)	2ndary Force **D** (kN)	Loss by Creep **D** (kN)	Live Load Nmax **L** (kN)	Live Load Nmin **L** (kN)	Temparature Rise **T** (kN)	Temparature Difference **T** (kN)
S1(E)	7500	1161	141	-442	-310	7	104	-68	134	17
S2(E)	7500	1183	189	-436	-377	12	98	-50	132	15
S3(E)	7400	1317	237	-430	-441	15	94	-34	129	14
S4(E)	7400	1276	285	-424	-501	20	94	-23	127	13
S5(E)	7500	1205	330	-418	-556	25	96	-13	124	12
S6(E)	7500	1104	373	-413	-606	30	101	-7	122	12
S7(E)	7500	985	411	-409	-650	34	108	-4	119	13
S8(E)	5400	610	315	-287	-486	27	80	-1	83	10
S9(E)	5400	604	334	-286	-509	29	85	0	81	11
S10(E)	5400	577	349	-285	-530	30	89	0	80	12
S11(E)	5400	533	358	-286	-547	32	91	0	78	13
S12(E)	5600	478	360	-288	-560	33	92	-1	77	14
S13(E)	5600	418	356	-292	-566	34	93	-4	76	15
S14(E)	5600	396	345	-296	-567	34	93	-7	74	16
S1(W)	7500	1143	136	-441	-307	7	105	-70	134	17
S2(W)	7500	1156	185	-434	-375	12	99	-52	131	15
S3(W)	7400	1282	233	-428	-439	15	94	-36	129	14
S4(W)	7400	1232	281	-421	-498	20	95	-24	126	13
S5(W)	7500	1153	328	-415	-554	25	96	-14	124	12
S6(W)	7500	1042	371	-410	-604	30	101	-8	121	12
S7(W)	7500	912	411	-405	-647	35	108	-4	119	13
S8(W)	5400	550	315	-284	-484	28	81	-1	82	10
S9(W)	5400	539	335	-283	-507	29	85	0	81	11
S10(W)	5400	504	350	-282	-528	31	89	0	79	12
S11(W)	5400	452	359	-283	-545	32	91	0	78	13
S12(W)	5600	389	362	-285	-557	33	93	-2	76	14
S13(W)	5600	318	358	-288	-564	34	94	-4	75	15
S14(W)	5600	318	358	-288	-564	34	94	-4	73	16

*) E : East Side Stay Cable
W : West Side Stay Cable

(Source : JICA Study Team)

Table 5.3.8 Service Load Design for Stay Cable



Tensile Force of Stay Cable per Both Side

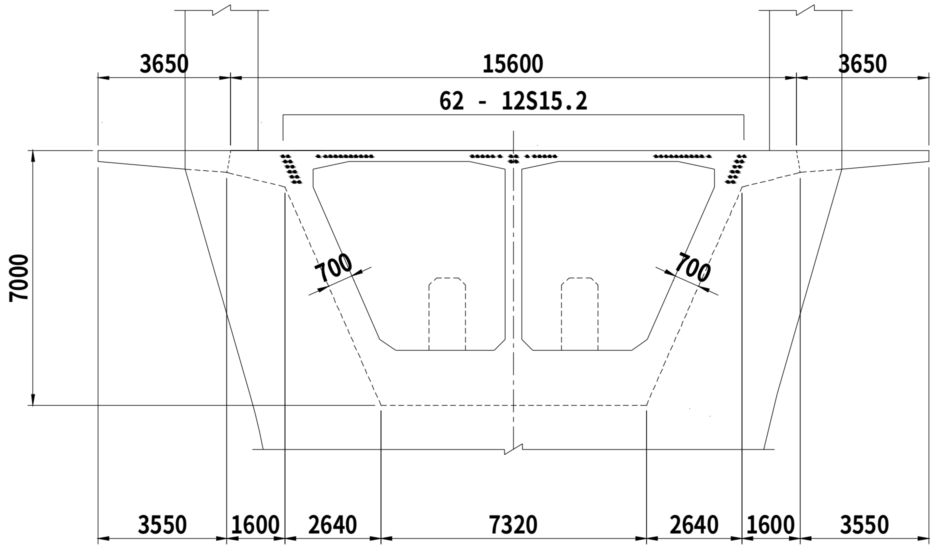
No..	Nos.of Strand	Tensile Force(kN)					
		Group-I			Group-IV		
		ΣD + Lmax (kN)	ΣD + Lmin (kN)	Allowable Tensile Force (kN)	ΣD+Lmax + T (kN)	ΣD+Lmin + T (kN)	Allowable Tensile Force (kN)
S1(E)	27S15.2	8,161	7,989	8,359	8,312	8,006	10,448
S2(E)	27S15.2	8,168	8,020	8,359	8,315	8,035	10,448
S3(E)	27S15.2	8,193	8,064	8,359	8,336	8,078	10,448
S4(E)	27S15.2	8,150	8,033	8,359	8,290	8,046	10,448
S5(E)	27S15.2	8,182	8,073	8,359	8,318	8,085	10,448
S6(E)	27S15.2	8,088	7,980	8,359	8,222	7,992	10,448
S7(E)	27S15.2	7,980	7,869	8,359	8,112	7,882	10,448
S8(E)	19S15.2	5,660	5,578	5,882	5,752	5,588	7,352
S9(E)	19S15.2	5,658	5,572	5,882	5,749	5,583	7,352
S10(E)	19S15.2	5,629	5,541	5,882	5,721	5,553	7,352
S11(E)	19S15.2	5,580	5,489	5,882	5,671	5,502	7,352
S12(E)	19S15.2	5,716	5,622	5,882	5,807	5,636	7,352
S13(E)	19S15.2	5,643	5,546	5,882	5,734	5,562	7,352
S14(E)	19S15.2	5,605	5,505	5,882	5,695	5,520	7,352
S1(W)	27S15.2	8,143	7,968	8,359	8,294	7,985	10,448
S2(W)	27S15.2	8,142	7,991	8,359	8,289	8,006	10,448
S3(W)	27S15.2	8,159	8,029	8,359	8,302	8,043	10,448
S4(W)	27S15.2	8,109	7,990	8,359	8,248	8,003	10,448
S5(W)	27S15.2	8,132	8,023	8,359	8,268	8,035	10,448
S6(W)	27S15.2	8,030	7,922	8,359	8,164	7,934	10,448
S7(W)	27S15.2	7,913	7,801	8,359	8,044	7,814	10,448
S8(W)	19S15.2	5,605	5,523	5,882	5,697	5,533	7,352
S9(W)	19S15.2	5,598	5,512	5,882	5,689	5,523	7,352
S10(W)	19S15.2	5,564	5,475	5,882	5,655	5,487	7,352
S11(W)	19S15.2	5,507	5,415	5,882	5,598	5,428	7,352
S12(W)	19S15.2	5,634	5,540	5,882	5,725	5,554	7,352
S13(W)	19S15.2	5,550	5,453	5,882	5,641	5,468	7,352
S14(W)	19S15.2	5,550	5,453	5,882	5,640	5,469	7,352

Allowable Tensile Force = 0.6 x fpu x Ap x 2

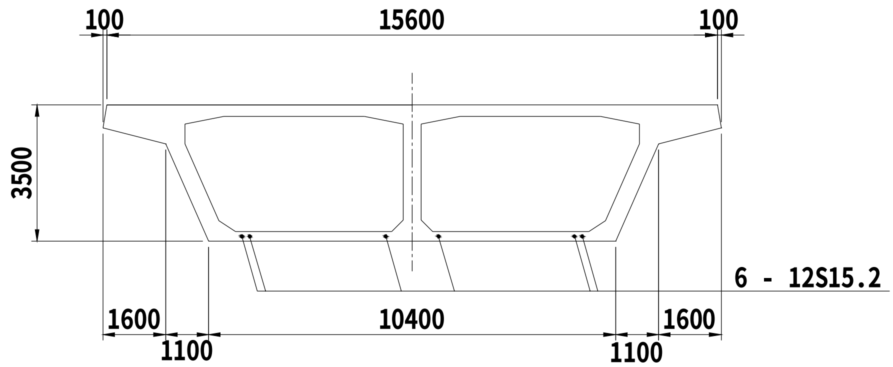
	Ap (mm^2)	fpu (N/mm^2)	Group-I Ta(kN) 2・0.6fpu・Ap	Group-IV Ta(kN) 2・0.6fpu・Ap・1.25
27S15.2	3744.9	1860	8,359	10,448
19S15.2	2635.3	1860	5,882	7,352

(Source : JICA Study Team)

(4) Internal PC-Steel Arrangement



Arrangement of Cantilever Cable



Arrangement of Side Span Cable

(Source : JICA Study Team)

Figure 5.3.18 Section of Internal PS-Steel Arrangement

(5) Detailed Design Result of Substructure

For some members, the determined stress is far less than the allowable stress with the following reasons:

For members having large section and requiring small reinforcing bar, steel are arranged for RC to ensure a tri-linear relationship between M and Φ.

For caisson type pile, the embedded depth is determined by the elastic domain, and reinforcing steel bars of cast-in-place pile are arranged complying with the minimum diameter and the maximum interval.

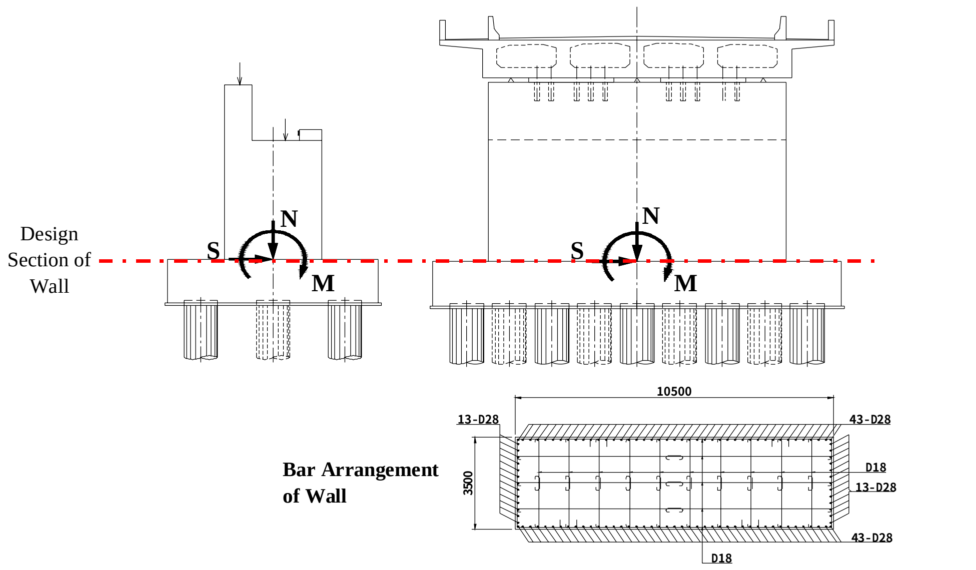
For piers, the stress resultant acts on the beam of piers by dead load, so the stress is controlled around 100N/mm2 for crack control.

Pier P2

Table 5.3.9 Design Summary for Pier P2 (1)

Summary of Design Calculation Result (1/3)

1. Wall



Size of Section			Longitudinal Direction	L_L=3.500m
			Transverse Direction	L_T=10.500m
Bar Arrangement			Longitudinal Direction	D28ctc250-1.0, d=3400mm
			Transverse Direction	D28ctc250-1.0, d=10400mm
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 3302 kNm, N= 21156 kN	
		Stress	σc= 0.5 < σca= 9.6 N/mm^2 (24 x 0.4)	
			σs= -4.6 < σsa= 168 N/mm^2	
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 0 kNm, N= 21156 kN	
		Stress	σc= 0.6 < σca= 9.6 N/mm^2 (24 x 0.4)	
			σs= -8.5 < σsa= 168 N/mm^2	
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 5083 kNm, Nu= 32302 kN	
			Su= 7263 kN	
		Strength	Flexural Strength φMn= 60929 kNm > Mu	
			Shear Strenth φVn= 24762 kN > Su	
	Transverse Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 0 kNm, Nu= 32302 kN	
			Su= 3815 kN	
		Strength	Flexural Strength φMn= 60929 kNm > Mu	
			Shear Strenth φVn= 24762 kN > Su	
Seismic Design	Longitudinal Direction	Sectional Force	M= 31577 kNm, N= 19729 kN	
		Stress	σc= 3.1 < σca= 20.4 N/mm^2 (24 x 0.85)	
			$\boxed{\text{σs= 73.4}}$ < σsa= 378 N/mm^2 (420 x 0.9)	
	Transverse Direction	Sectional Force	M= 76670 kNm, N= 16207 kN	
		Stress	σc= 3.1 < σca= 20.4 N/mm^2 (24 x 0.85)	
			$\boxed{\text{σs= 103.1}}$ < σsa= 378 N/mm^2 (420 x 0.9)	

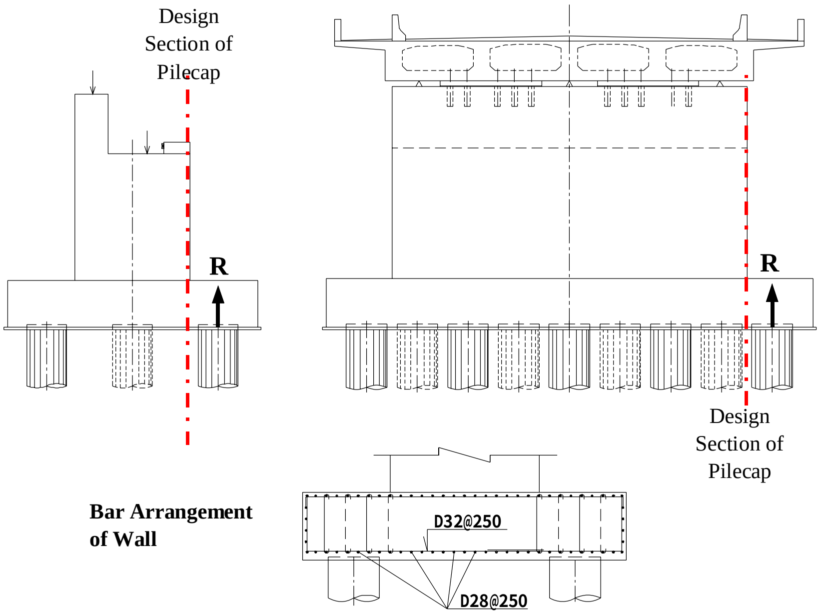
Note: The determined stress is far less than the allowable stress due to applying the minimum amount of steel material

(Source : JICA Study Team)

Table 5.3.10 Design Summary for Pier P2 (2)

Summary of Design Calculation Result (2/3)

2. Pilecap



Size of Section		Longitudinal Direction	L_L=7.600m T=1.800m
		Transverse Direction	L_T=14.400m T=1.800m
Bar Arrangement		Longitudinal Direction	D32ctc250-1.0, d=1550mm
		Transverse Direction	D28ctc250-1.0, d=1580mm
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 7631 kNm,
		Stress	σc= 1.9 < σca= 9.6 N/mm^2
			σs= 101.8 < σsa= 168 N/mm^2
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 2721 kNm,
		Stress	σc= 1.2 < σca= 9.6 N/mm^2
			σs= 82.8 < σsa= 168 N/mm^2
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 12135 kNm, Su= 14717 kN
		Strength	Flexural Strength φMn= 25737 kNm > Mu
			Shear Strenth φVn= 16480 kN > Su
	Transverse Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 4437 kNm, Su= 5792 kN
		Strength	Flexural Strength φMn= 10656 kNm > Mu
			Shear Strenth φVn= 8698 kN > Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 14063 kNm,
		Stress	σc= 3.8 < σca= 20.4 N/mm^2
			$\boxed{σs= 204.3}$ < σsa= 378 N/mm^2
	Transverse Direction	Sectional Force	M= 5598 kNm,
		Stress	σc= 2.7 < σca= 20.4 N/mm^2
			$\boxed{σs= 318.5}$ < σsa= 378 N/mm^2

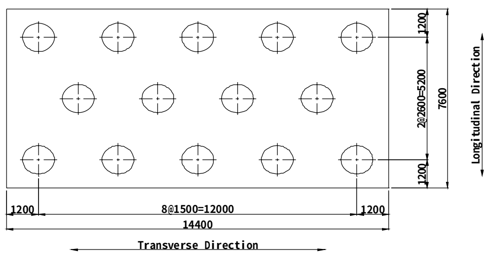
(Source : JICA Study Team)

Table 5.3.11 Design Summary for Pier P2 (3)

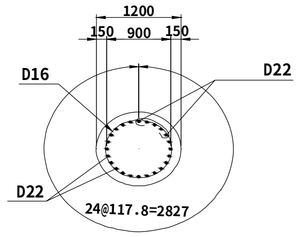
Summary of Design Calculation Result (3/3)

3. Cast in-situ Pile

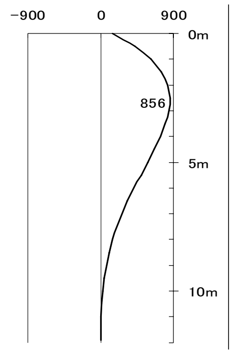
Pile Arrangement



Bar Arrangement



Bending Moment
(Seismic Design)



Pile Arranegement			Diameter of Pile	D= 1200 mm
			Length of Pile	L= 12.000 m
			Nos. of Pile	N= 14 Nos
Bar Arrangement			24-D22 (@147)	
Stability	Force at the Center of Pile Cap	Longitudinal Direction	Group-I	V=28054kN, H=0kN, M=3302kNm
			Group-III	V=28054kN, H=130kN, M=4961kNm
			Seismic Design	V=29625kN, H=9249kN, M=46447kNm
		Transverse Direction	Group-I	V=28054kN, H=0kN, M=0kNm
			Seismic Design	V=22909kN, H=5895kN, M=85414kNm
	Reaction at Pile Head	Longitudinal Direction	Group-I	Rmax=2312kN < Ra=2619kN, Rmin=2123kN > Pa=0kN
			Group-III	Rmax=2367kN < Ra=2619kN, Rmin=2068kN > Pa=0kN
			Seismic Design	Rmax=3826kN < Ra=3965kN, Rmin=406kN > Pa=-1917kN
		Transverse Direction	Group-I	Rmax=2218kN < Ra=2619kN, Rmin=2218kN > Pa=0kN
			Seismic Design	Rmax=3910kN < Ra=3965kN, Rmin=-637kN > Pa=-1917kN
	Displacement at Pile Head	Longitudinal Direction	Group-I	δx=0.2mm < 15mm
			Group-III	δx=0.38mm < 15mm
			Seismic Design	δx=5.65mm < 15mm
		Transverse Direction	Group-I	δx=0mm < 15mm
			Seismic Design	δx=3.4mm < 15mm
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 61 kNm,	N= 2312 kN
		Stress	σc= 2.1 <	σca= 9.6 N/mm^2
			σs= -30.8 <	σsa= 168 N/mm^2
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 93 kNm,	Nu= 3372 kN
			Su= 15 kN	
		Strength	Flexural Strength	φMn= 2037 kNm > Mu
			Shear Strenth	φVn= 692 kN > Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 857 kNm,	N= 406 kN
		Stress	σc= 9.1 <	σca= 20.4 N/mm^2
			σs= 245.8 <	σsa= 378 N/mm^2
	Transverse Direction	Sectional Force	M= 495 kNm,	N= -637 kN
		Stress	σc= 5.5 <	σca= 20.4 N/mm^2
			σs= 255.2 <	σsa= 378 N/mm^2

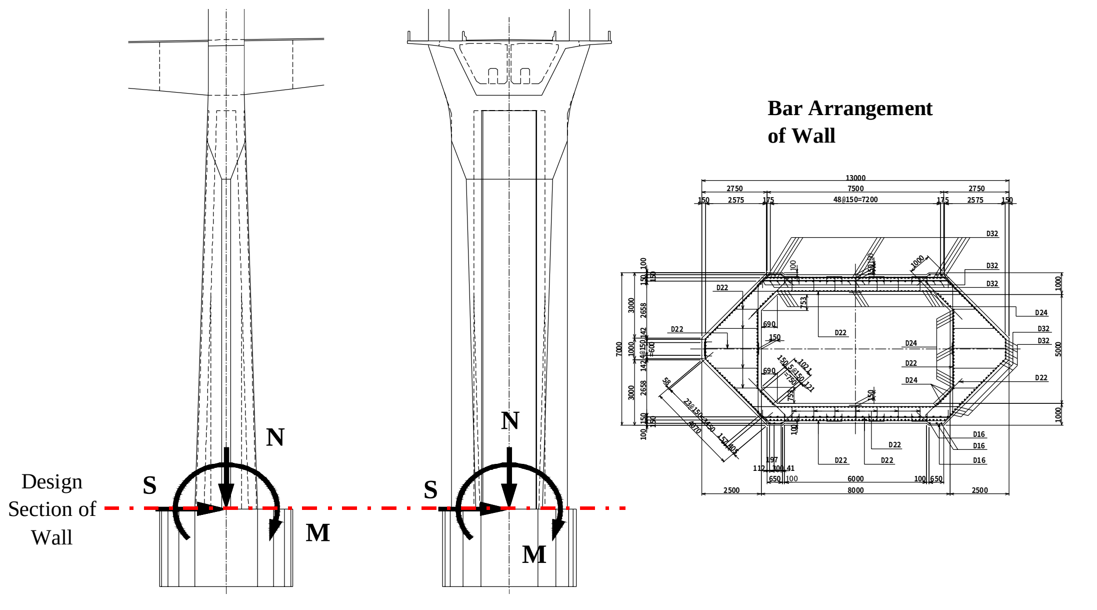
(Source : JICA Study Team)

Table 5.3.12 Design Summary for Pier P3 (1)

Pier P3

Summary of Design Calculation Result (1/3)

1. Wall



Size of Section			Longitudinal Direction LL=3.500m
			Transverse Direction LT=10.500m
Bar Arrangement			Longitudinal Direction D28ctc250-1.0, d=3400mm
			Transverse Direction D28ctc250-1.0, d=10400mm
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 37166 kNm, N= 176772 kN
		Stress	σc= 1.6 < σca= 9.6 N/mm² (24 x 0.4)
			σs= -13.5 < σsa= 168 N/mm²
	Transverse Direction	Load Case	Group-II : (D+SD+E+B)
		Sectional Force	M= 174827 kNm, N= 173932 kN
		Stress	σc= 1.0 < σca= 9.6 N/mm² (24 x 0.4)
			σs= -14.6 < σsa= 168 N/mm²
Load Factor Design	Longitudinal Direction	Load Case	Group-I : 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 80174 kNm, Nu= 232277 kN
			Su= 19750 kN (Seicmic)
		Strength	Flexural Strength φMn= 889101 kNm > Mu
			Shear Strenth φVn= 42665 kN > Su
	Transverse Direction	Load Case	Group-II : 1.3(D+SD+E+B)
		Sectional Force	Mu= 227274 kNm, Nu= 226111 kN
			Su= 15386 kN (Seicmic)
		Strength	Flexural Strength φMn= 1500579 kNm > Mu
			Shear Strenth φVn= 59234 kN > Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 681228 kNm, N= 176460 kN
		Stress	σc= 11.8 < σca= 20.4 N/mm² (24 x 0.85)
			$\boxed{\sigma s= 362.2}$ < σsa= 378 N/mm² (420 x 0.9)
	Transverse Direction	Sectional Force	M= 704291 kNm, N= 172987 kN
		Stress	σc= 7.8 < σca= 20.4 N/mm² (24 x 0.85)
			σs= 213.7 < σsa= 378 N/mm² (420 x 0.9)

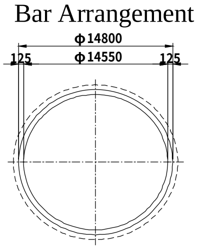
(Source : JICA Study Team)

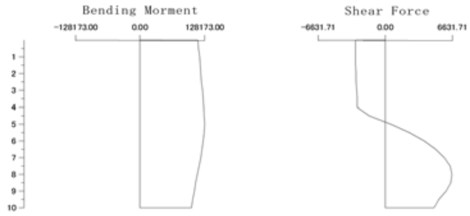
Table 5.3.13 Design Summary for Pier P3 (2)

Summary of Design Calculation Result (2/2)

2. Shinso Pile

Bar Arrangement



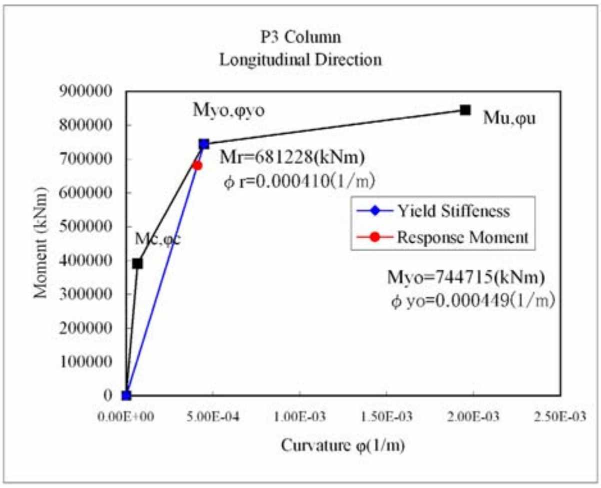


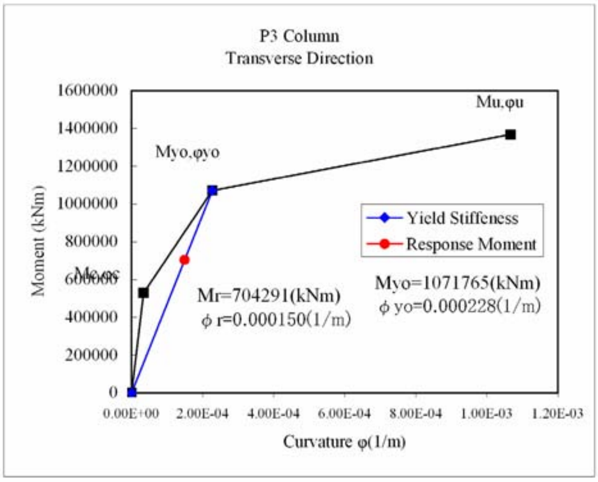
(Seismic Design)

Pile Arranegement			Diameter of Pile	D= 15000 mm
			Length of Pile	L= 10.000 m
Bar Arrangement			156-D22 (@223)	
Stability	Force at the Top of Shinso Pile	Longitudinal Direction	Group-I	V=176780kN, H=0kN, M=37170kNm
			Seismic Design	V=176460kN, H=19750kN, M=681230kNm
		Transverse Direction	Group-I	V=176780kN, H=960kN, M=4350kNm
			Seismic Design	V=172990kN, H=15390kN, M=704300kNm
	Ground Reaction	Longitudinal Direction	Group-I	qmax=1329kN/㎡ < qa=6500kN/㎡
			Seismic Design	qmax=4245kN/㎡ < qa=9750kN/㎡
		Transverse Direction	Group-I	qmax=1272kN/㎡ < qa=6500kN/㎡
			Seismic Design	$\boxed{\text{qmax=4340kN/㎡}}$ < qa=9750kN/㎡
	Displacement at Pile Head	Longitudinal Direction	Group-I	δx=0.43mm < 50mm
			Seismic Design	δx=5.56mm < 50mm
		Transverse Direction	Group-I	δx=0.16mm < 50mm
			Seismic Design	δx=7.83mm < 50mm
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 37170 kNm, N= 176780 kN	
		Stress	σc= 1.1 < σca= 9.6 N/mm²	
			σs= -13.5 < σsa= 168 N/mm²	
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 61 kNm, N= 2312 kN	
		Stress	σc= 2.1 < σca= 9.6 N/mm²	
			σs= -30.8 < σsa= 168 N/mm²	
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 70113 kNm, Nu= 274493 kN Su= 4733 kN	
		Strength	Flexural Strength φMn= 3169565 kNm > Mu Shear Strenth φVn= 10179 kN > Su	
	Transverse Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 12229 kNm, Nu= 261805 kN Su= 1240 kN	
		Strength	Flexural Strength φMn= 3033934 kNm > Mu Shear Strenth φVn= 10179 kN > Su	
Seismic Design	Longitudinal Direction	Sectional Force	M= 769779 kNm, N= 198107 kN	
		Stress	σc= 4.3 < σca= 20.4 N/mm²	
			σs= 42.6 < σsa= 378 N/mm²	
	Transverse Direction	Sectional Force	M= 651299 kNm, N= 132470 kN	
		Stress	σc= 4.4 < σca= 20.4 N/mm²	
			$\boxed{\text{σs= 85.7}}$ < σsa= 378 N/mm²	

Note: The determined stress is far less than the allowable stress due to applying the minimum amount of steel material.

(Source : JICA Study Team)





(Source : JICA Study Team)

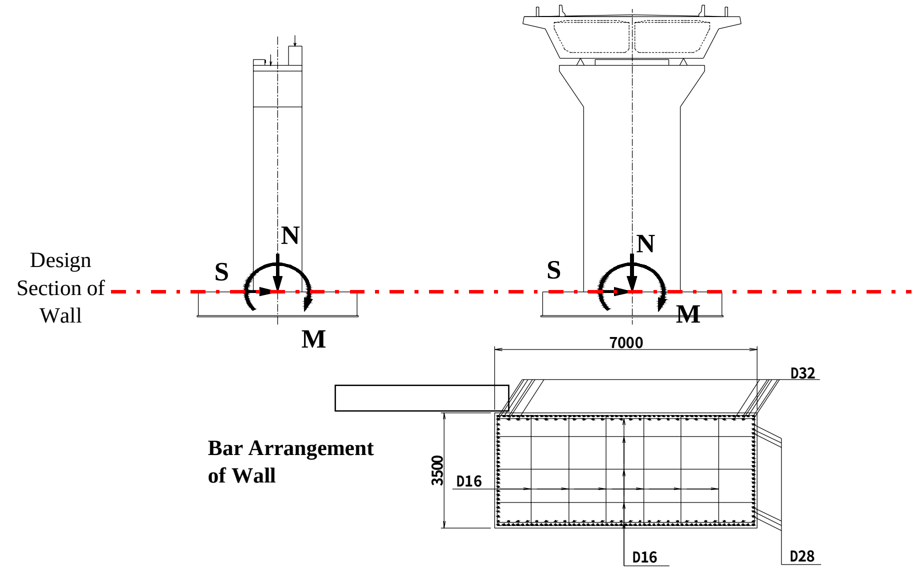
Figure 5.3.19 Check of Response Force of Pier P3

Table 5.3.14 Design Summary for P4 Pier (1)

<u>P4 Pier</u>

Summary of Design Calculation Result (1/3)

1. Wall



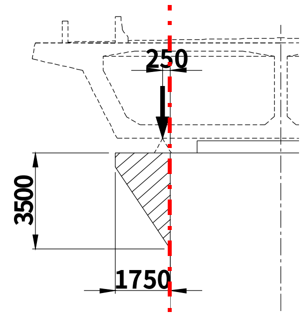
Size of Section		Longitudinal Direction	L_L=3.500m
		Transverse Direction	L_T=7.000m
Bar Arrangement		Longitudinal Direction	D32ctc125-1.5, d=3400mm
		Transverse Direction	D28ctc125-1.0, d=6900mm
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 3645 kNm, N= 30681 kN
		Stress	σc= 1.6 < σca= 9.6 N/mm^2 (24 x 0.4)
			σs= -13.5 < σsa= 168 N/mm^2
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 0 kNm, N= 30681 kN
		Stress	σc= 1.3 < σca= 9.6 N/mm^2 (24 x 0.4)
			σs= -18.2 < σsa= 168 N/mm^2
Load Factor Design	Longitudinal Direction	Load Case	Group-I : 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 5113 kNm, Nu= 45129 kN
			Su= 182 kN (Group-III)
		Strength	Flexural Strength φMn= 151198 kNm > Mu
			Shear Strenth φVn= 16507 kN > Su
	Transverse Direction	Load Case	Group-I : 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 0 kNm, Nu= 45129 kN
			Su= 0 kN
		Strength	Flexural Strength φMn= 230740 kNm > Mu
			Shear Strenth φVn= 16750 kN > Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 127514 kNm, N= 28918 kN
		Stress	σc= 11.8 < σca= 20.4 N/mm^2 (24 x 0.85)
			$\boxed{\text{σs= 362.2}}$ < σsa= 378 N/mm^2 (420 x 0.9)
	Transverse Direction	Sectional Force	M= 130314 kNm, N= 25310 kN
		Stress	σc= 7.8 < σca= 20.4 N/mm^2 (24 x 0.85)
			$\boxed{\text{σs= 213.7}}$ < σsa= 378 N/mm^2 (420 x 0.9)

(Source : JICA Study Team)

Table 5.3.15 Design Summary for P4 Pier (2)

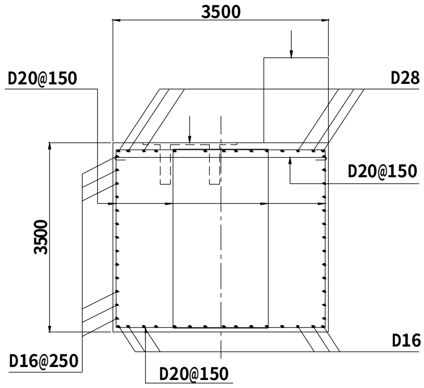
Summary of Design Calculation Result 2/3)

2. Beam



Design Section of Beam

Bar Arrangement of Beam



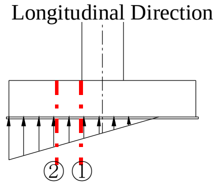
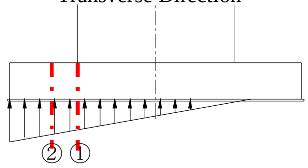
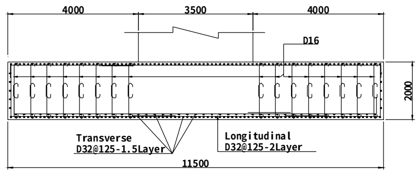
Size of Section			Vertical Direction	L_V=3.500m			
			Longitudinal Direction	L_L=3.500m			
Bar Arrangement			Upper Edge Side	D28ctc250-1.0, d=100mm			
			Side Horizontal Bar	D16ctc250-1.0, d=100mm			
Service Load Design	Vertical Direction	Load Case	Group-I : (D+SD+L+CF+E+B)				
		Sectional Force	M= 1599 kNm,				
		Stress	σc= 0.64	<	σca=	9.6 N/mm^2	
			$\boxed{\sigma s= 66.4}$	<	σsa=	168 N/mm^2	
	Longitudinal Direction	Load Case	Group-VII : (D+SD+E+B+EQ)				
		Sectional Force	M= 389 kNm,				
		Stress	σc= 0.27	<	σca=	12.8 N/mm^2	
			σs= 55.2	<	σsa=	223 N/mm^2	
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)				
		Sectional Force	Mu= 2079 kNm,				
			Su= 7689 kN				
		Strength	Flexural Strength		φMn=	10414 kNm	> Mu
			Shear Strenth		φVn=	8132 kN	> Su
	Transverse Direction	Load Case	Group - VII 1.3(D+SD+E+B+EQ)				
		Sectional Force	Mu= 506 kNm,				
			Su= 1805 kN				
		Strength	Flexural Strength		φMn=	2774 kNm	> Mu
			Shear Strenth		φVn=	8327 kN	> Su

Note: The determined stress is far less than the allowable stress due to crack control.

(Source : JICA Study Team)

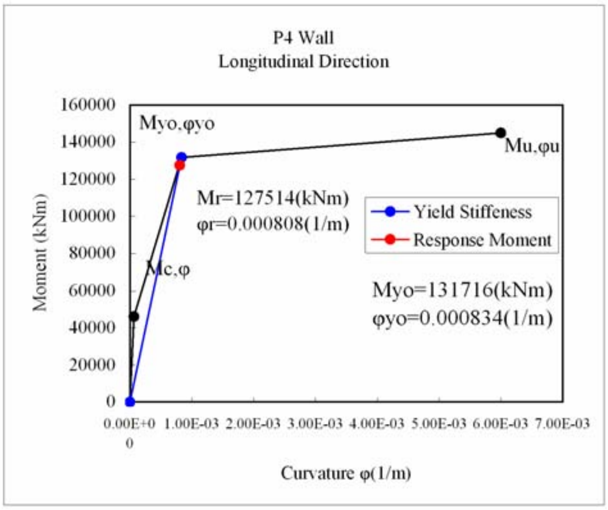
Table 5.3.16 Design Summary for P4 Pier (3)

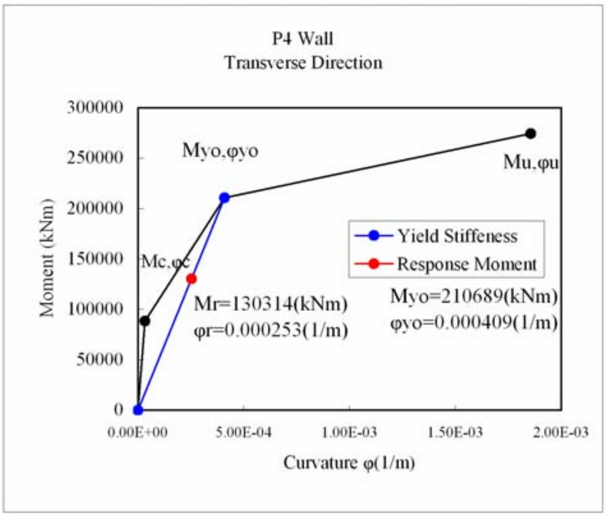
Summary of Design Calculation Result (3/3)

3. Spread Footing				
Longitudinal Direction 		Transverse Direction 		Bar Arrangement of Footing 
Size of Section			Longitudinal Direction	LL=11.500m T=2.000m
			Transverse Direction	LT=13.000m T=2.000m
Bar Arrangement			Longitudinal Direction	D32ctc125-2.0, d=1890mm
			Transverse Direction	D32ctc125-1.5, d=1922mm
Stability	Force at the Center of Footing	Longitudinal Direction	Group-III	V=42472kN, H=0kN, M=3645kNm
			Seismic Design	V=41871kN, H=11117kN, M=146457kNm
		Transverse Direction	Group-I	V=42472kN, H=0kN, M=0kNm
			Seismic Design	V=38261kN, H=12070kN, M=151310kNm
	Safety Factor for Horizontal Force	Longitudinal Direction	Group-I	Safety Factor=182 > 1.5 (Allowable Factor)
			Seismic Design	Safety Factor=2.26 > 1.2 (Allowable Factor)
		Transverse Direction	Group-I	-
			Seismic Design	Safety Factor=1.902 > 1.2 (Allowable Factor)
	Ground Reaction	Longitudinal Direction	Group-I	qmax=297kN/m^2 < qa=700kN/m^2
			Seismic Design	qmax=953kN/m^2 < qa=1050kN/m^2
		Transverse Direction	Group-I	qmax=284kN/m^2 < qa=700kN/m^2
			Seismic Design	qmax=871kN/m^2 < qa=1050kN/m^2
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 1977 kNm, ①	
		Stress	σc= 3.0 < σca= 9.6 N/mm^2	
			σs= 93.0 < σsa= 168 N/mm^2	
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 1381 kNm, ①	
		Stress	σc= 2.1 < σca= 12.8 N/mm^2	
			σs= 64.3 < σsa= 223 N/mm^2	
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 2922 kNm, ①	
			Su= 1094 kN ②	
		Strength	Flexural Strength φMn= 7549 kNm	> Mu
			Shear Strenth φVn= 1311 kN	> Su
	Transverse Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 2045 kNm, ①	
			Su= 909 kN ②	
		Strength	Flexural Strength φMn= 7693 kNm	> Mu
			Shear Strenth φVn= 1333 kN	> Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 6568 kNm,	
		Stress	σc= 10.1 < σca= 20.4 N/mm^2	
			σs= 308.9 < σsa= 378 N/mm^2	
	Transverse Direction	Sectional Force	M= 4761 kNm,	
		Stress	σc= 7.1 < σca= 20.4 N/mm^2	
			σs= 221.7 < σsa= 378 N/mm^2	

(Source : JICA Study Team)

(Source : JICA Study Team)





(Source : JICA Study Team)

Figure 5.3.20 Check of response force of P4 wall

(6) Determinant Load Combination

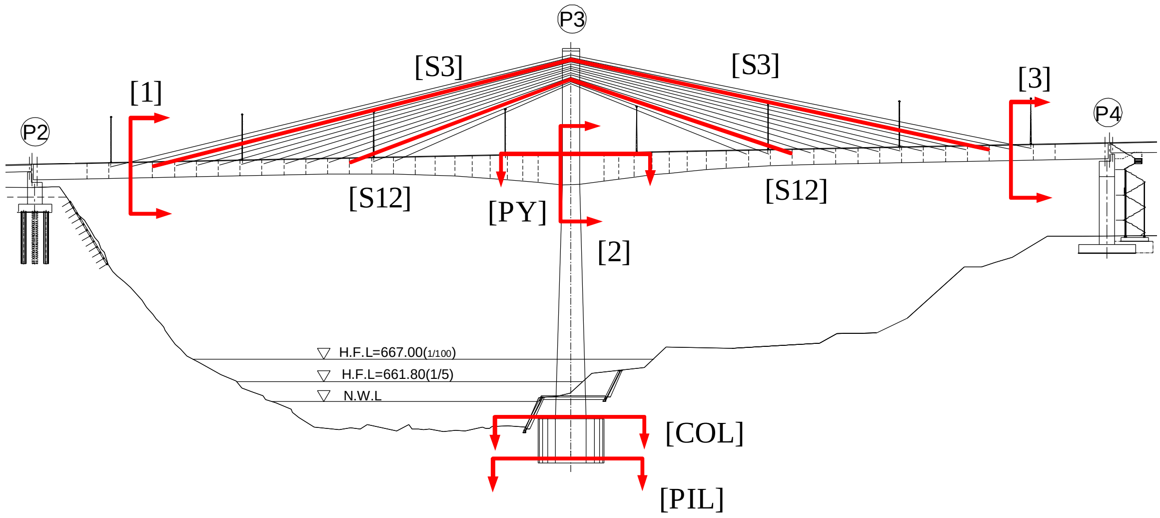


Table 5.3.17 The Most Severe Load Combination

Structural Member		Analysis Method	Design Method	Load Combination	Most Severe Case
PC Box Girder	[1]	Static Analysis	Service Load Design	GROUP-**I** GROUP-**IV**	GROUP-**I**
			Load Factor Design	GROUP-**I** GROUP-**IV**	-
	[2]	Static Analysis	Service Load Design	GROUP-**I** GROUP-**IV**	GROUP-**I**
			Load Factor Design	GROUP-**I** GROUP-**IV**	-
	[3]	Static Analysis	Service Load Design	GROUP-**I** GROUP-**IV**	GROUP-**I**
			Load Factor Design	GROUP-**I** GROUP-**IV**	-
Stay Cable	[S3]	Static Analysis	Service Load Design	GROUP-**I** GROUP-**IV**	GROUP-**I**
	[S12]	Static Analysis	Service Load Design	GROUP-**I** GROUP-**IV**	GROUP-**I**
Pylon		Static Analysis	Service Load Design	GROUP-**I** GROUP-**III** GROUP-**IV**	Seismic Condition
		Dynamic Analysis	Seismic Design	-	
Column		Static Analysis	Service Load Design	GROUP-**I** GROUP-**III** GROUP-**IV**	Seismic Condition
		Dynamic Analysis	Seismic Design	-	
Shinso-PIle		Static Analysis	Service Load Design	GROUP-**I** GROUP-**III** GROUP-**IV**	Seismic Condition
		Dynamic Analysis	Seismic Design	-	

Note) GROUP-**I** : D+(L+I)+SH+CR+PS
GROUP-**III** : D+(L+I)+SH+CR+PS+W
GROUP-**IV** : D+(L+I)+SH+CR+PS+T
D : Dead Load (Self Weight, Surfacing)
(L+I): Live Load with Impact by CLASS-A
SH : Shrinkage
CR : Creep Effect
PS : Pre-Stress within 2ndary Force by Pre-stress
W : Wind Load
T : Thermal Effect

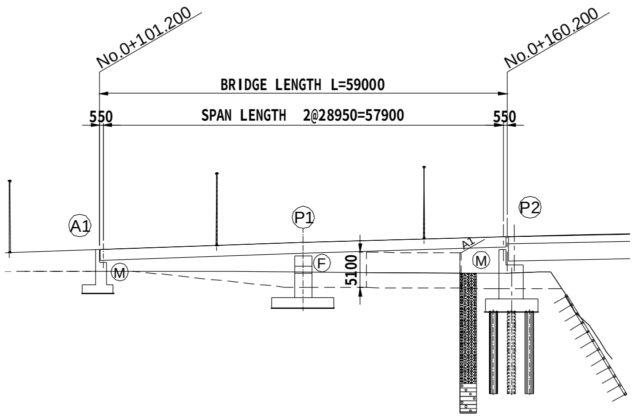
5.4 Design of Viaducts

5.4.1 East Side Approach Viaduct

(1) Conceivable Span Arrangement

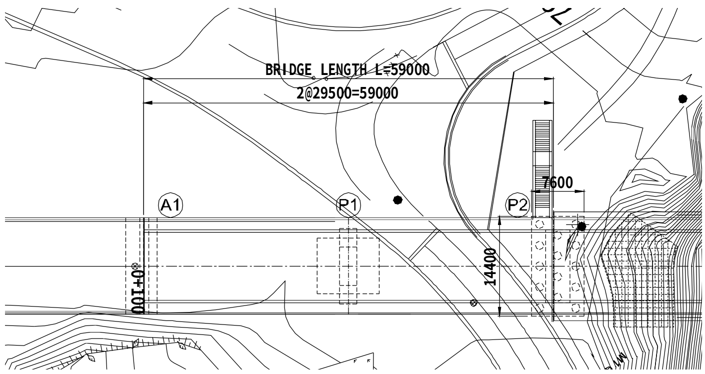
East side approach viaduct is a grade separated bridge, which has underpass for traffic to/from Muzaffarabad city centre. The bridge length of 59m is determined by the intersection improvement plan. The vertical clearance of underpass should be 5.1m high from the new road surface to the girder soffit. The structural type of this viaduct needs to consider lower/shallower height of girder to minimise the road excavation and to lower the elevation of the viaduct.

Arrangement of a pier can be installed in the intermediate position of bridge length, and with clear span lengths of 29.5m.



(Source : JICA Study Team)

Figure 5.4.1 Profile of East Side Approach Viaduct



(Source: JICA Study Team)

Figure 5.4.2 Plan of East Side Approach

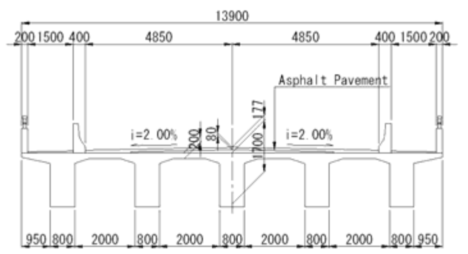
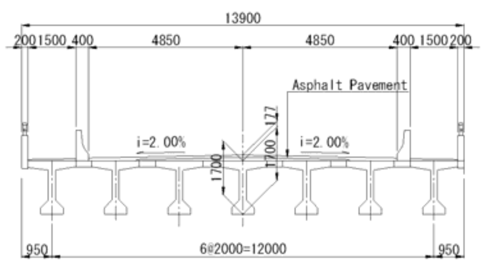
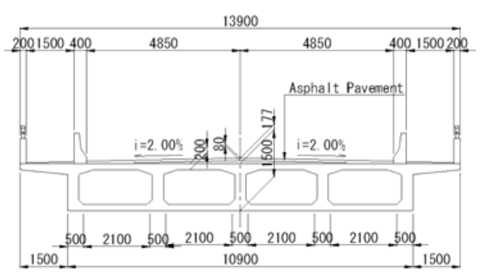
(2) Alternative Study of Superstructure

The following three alternatives were considered in the comparative study of east side viaduct.

- Alternative 1 : PC slab girder bridge: (girder height =1.8m)
- Alternative 2 : PC Post-Tension T section girder bridge: (girder height =1.7m)
- Alternative 3: PC box girder bridge: (girder height =1.5m)

The lower structural height of the girder is one of the most important factors in selecting the bridge type in consideration of underpass vertical clearance. Since PC slab girder bridge has RC-deck slab cast in situ, its self weight is heavy and is comparatively deep. The PC Post-tension T-section girder also has deeper sections. Moreover, a large size of crane or special erection equipment, which is difficult to mobilize into Muzaffarabad, is required for erection.

PC box girder can be designed with 1.5 m girder height, which is the shallowest among the three alternatives. Consequently Alternative-3 (PC box girder) with four cells was selected as an optimum bridge type shown in **Figure 5.4.3.**

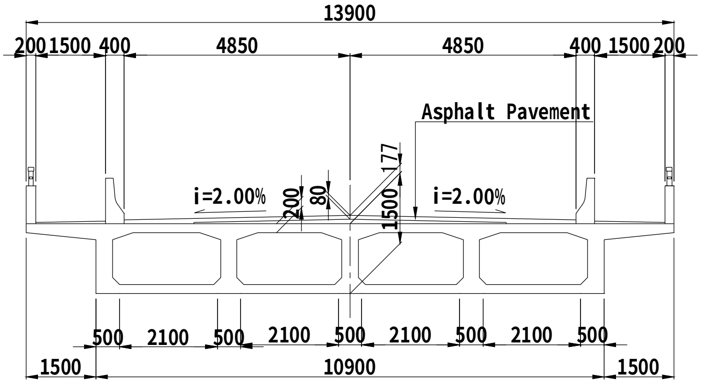
	Girder Type	Evaluation Item	Evaluation	Rating Result
Alternative 1	PC Slab Girder Cast in Situ	Construction Cost	Most economical alternative (1.00)	Good
		Construction Aspect	The structure is simple and construction is easy.	Fair
		Structural Aspect	Structural height is large.(h=1.8m)	Bad
		Constt Period	Construction period.(about 6months)	Fair
		Maintenance	minimal workload expect it for bearings and drainnage pipe required periodocal inspection and maintenance requisite.	Fair
		Technology Transfer	less technology transfer elements expect construction of PC girder cast in situ.	Poor
		Overall	It is difficult to obtain the launching equipment in the site.	Discarded
Alternative 2	PC Post-Tension T Girder (Pre-Cast)	Construction Cost	Moderate Cost (1.15)	Poor
		Construction Aspect	A large-capacity crane or the special construction equipment is necessary for construction, and procurement in the locale is difficult.	Bad
		Structural Aspect	Structural height is large.(h=1.7m)	Poor
		Constt Period	Shortest construction period.(about 5months)	Fair
		Maintenance	minimal workload expect it for bearings and drainnage pipe required periodocal inspection and maintenance requisite.	Fair
		Technology Transfer	less technology transfer elements expect construction of PC girder cast in situ.	
		Overall	A large-capacity crane or the special construction equipment is necessary for construction, and procurement in the locale is difficult.	Discarded
Alternative 3	PC Box Girder Cast in Situ	Construction Cost	Most economical alternative (1.00)	
		Construction Aspect	It is a minimum structural height of the BOX girder, and it is necessary to devise the upper slab construction.	
		Structural Aspect	The height that crossing roads control rises because structural height is lo	
		Constt Period	Construction period.(about 6months)	Fair
		Maintenance	minimal workload expect it for bearings and drainnage pipe required periodocal inspection and maintenance requisite.	Fair
		Technology Transfer	less technology transfer elements expect construction of PC girder cast in situ.	Poor
		Overall	No negative evaluation items and relatively advantageous alternative especially structural aspect among the others.	Selected

(Source : JICA Study Team)

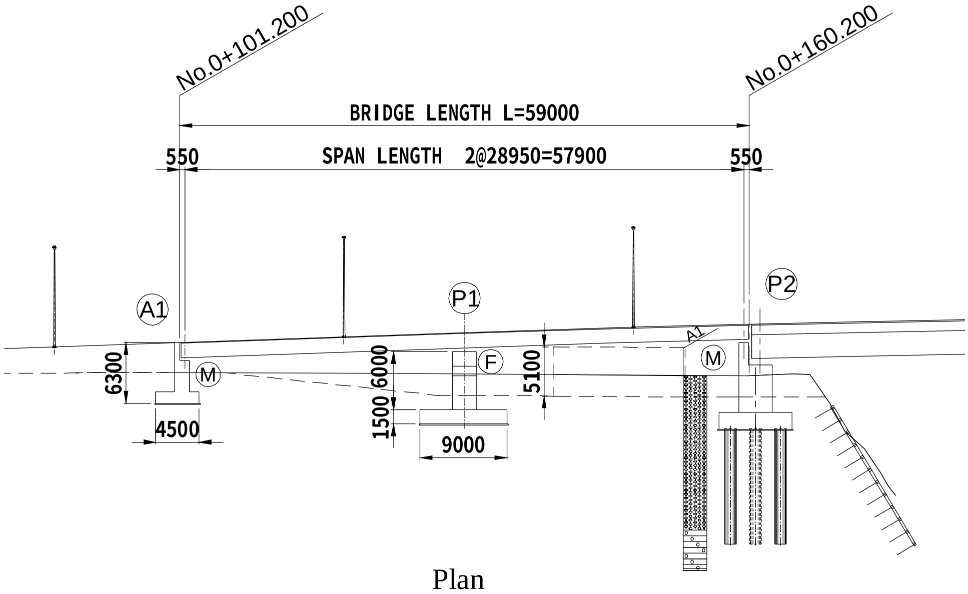
Figure 5.4.3 Comparison for Superstructure of the Viaduct

(3) Basic Design Result

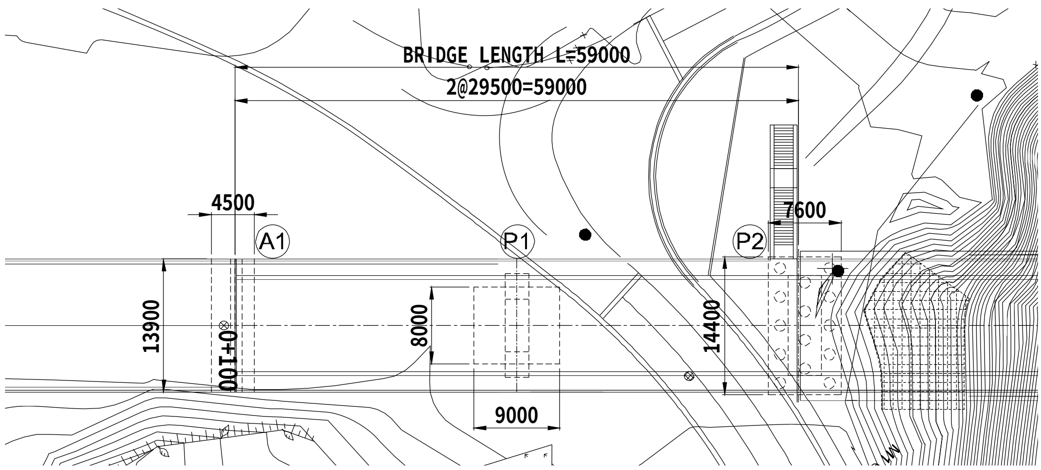
Typical Cross Section



Profile



Plan



(Source : JICA Study Team)

Figure 5.4.4 General View of East Side Approach Viaduct

(4) Detailed Design Result

a. Design Criteria

Table 5.4.1 Design Conditions for East Side Viaduct

Superstructure	:	2 spans Continuous PC-Slab girder
Bridge Length	:	59.0m
Girder Length	:	58.8m
Span Length	:	2@28.95m
Bridge Width	:	13.9m
Effective Width	:	Carriage Way 9.7m
	:	Foot Way 2 x 1.5m
Live Load	:	Class-A, Class –AA
Curvature	:	R=∞
Gradient	:	i = 3.5% ↙ 2.0% ↙
Super Elevation	:	i = 2.0% ↙ ↘
Skew Angle	:	θ =90 °
Asphalt pavement	:	Carriageway 80mm～
	:	Foot way 30mm～

(Source : JICA Study Team)

b. Materials Properties

Table 5.4.2 Material Properties

Materials

Concrete (N/mm^2)

		PC-Slab girder PC	Cross beam RC	Deck slab RC
Class		D1	D1	D1
28Days Cylinder Strength		35	35	35
Modulus of Elasticity		2.95×10^4	2.95×10^4	2.95×10^4
Allowable Compression Stress		14.00	14.00	14.00
Allowable Tensile Stress		-2.96	-	-
Temperature coefficient		10×10^{-6}	10×10^{-6}	10×10^{-6}
Allowable Shear Stress		0.47	0.47	0.47
Maximum Average Shear Stress		2.50	2.50	2.50
Diagonal Stress	Permanent Load	0.90	-	-
	Design Combination Load	1.85	-	-

PC steel

	Unit	Longitudinal 12S15.2
Ultimate Strength	N/mm^2	1860
Minimum Breaking Strength	kN	3128.4
Minimum Yeild Strength	kN	2815.6
Friction coefficient per 1 meter	1/m	0.0020
Friction coefficient per 1 radian	1/Rad	0.250
Set Losses	mm	9
Relaxation	%	1.5
Modulus of Elasticity	x10^{5}N/mm^2	2.0
Sectional area	mm^2	1664.4
Diameter of sheath	mm	75

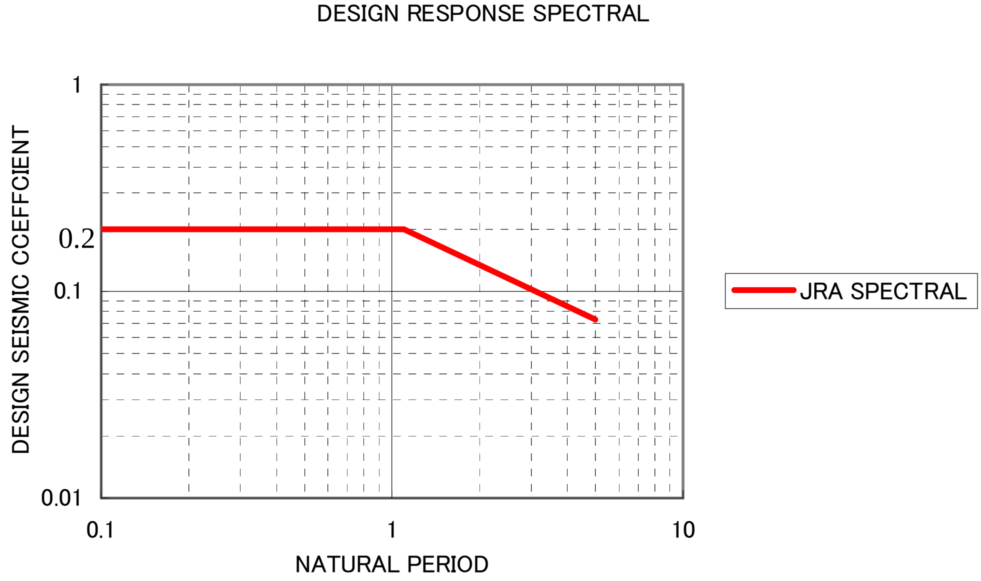
Reinforcement Bar (N/mm^2)

Yield strength	420
Modulus of Elasticity（$\times10^5$）	2.0
Allowable Tensile Stress	168

(Source: JICA Study Team)

Since Abutment has the attenuation effect of an earthquake motion with soil on the back in case of an earthquake, it assumes a middle-scale earthquake (LEVEL1).

A design seismic coefficient is set to kh = 0.20 by referring to the Japanese standard.



(Source : JICA Study Team)

Figure 5.4.5 Design Response Spectral

Table 5.4.3 Design Seismic Coefficient for P1 Pier

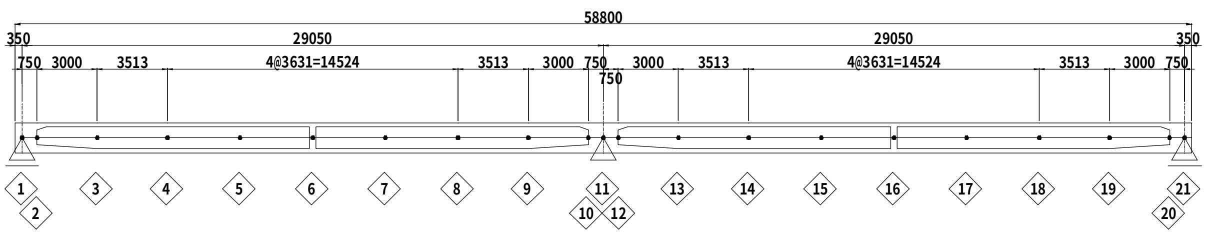
Design Seismic Coefficient : Cs

$Cs=1.2AS / Tm^{2/3}$ Design Seismic Coefficient kh=Cs/R (=2)

Structure	Longitudinal	Transverse
Natural Period	0.47	0.34
Design Seismic Coefficient	0.35	0.43

(Source : JICA Study Team)

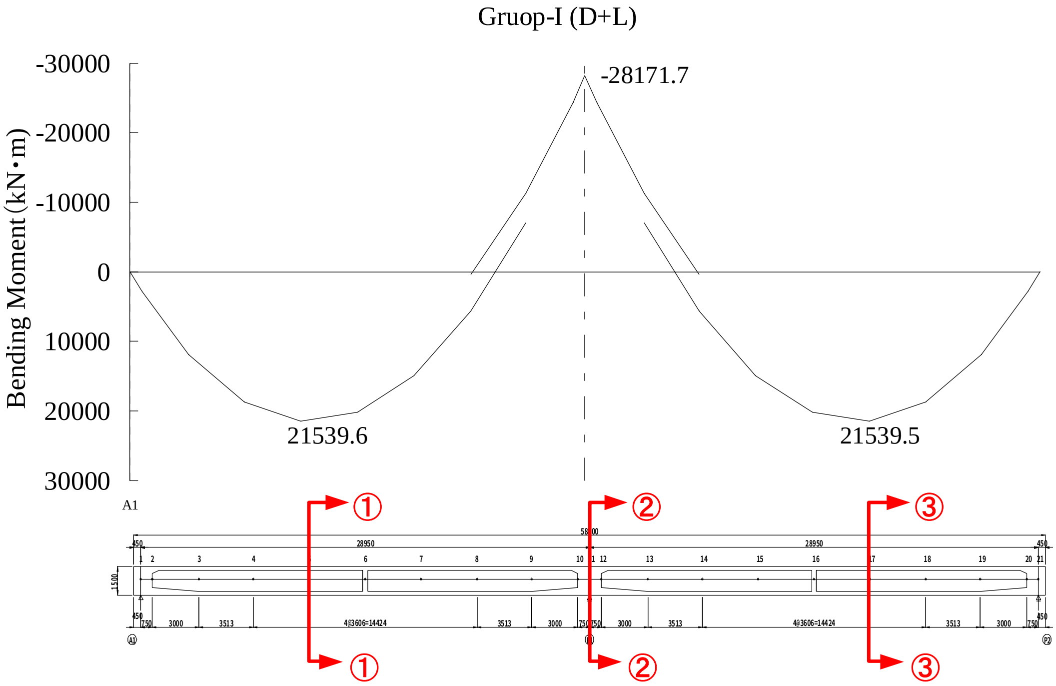
c. Analysis Model



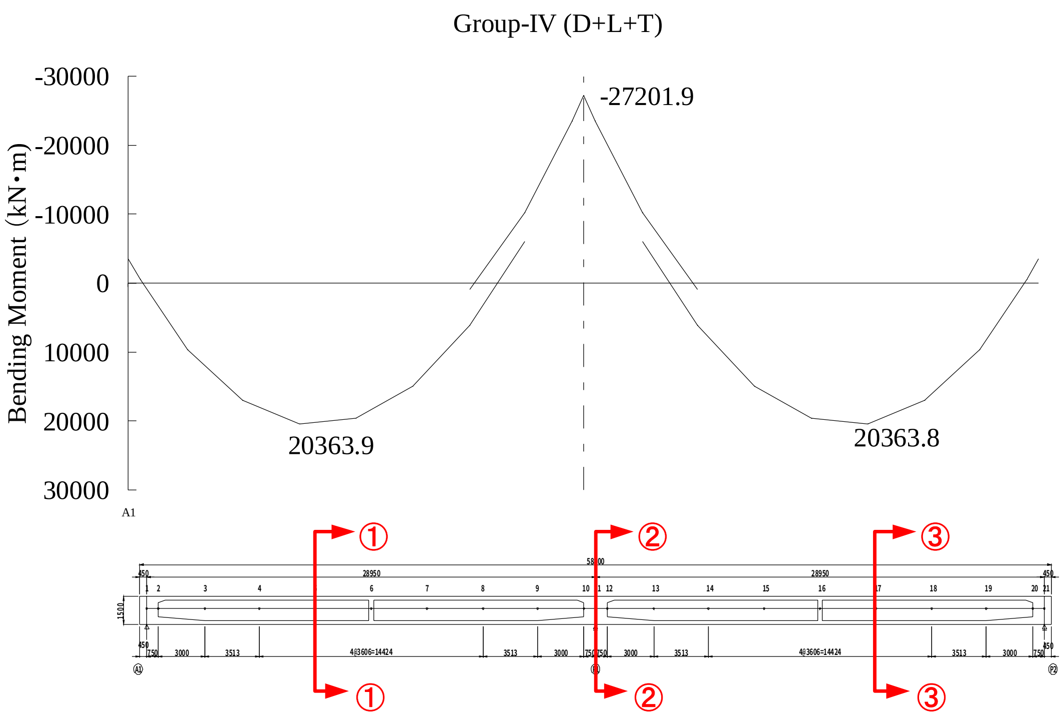
(Source : JICA Study Team)

Figure 5.4.6 Analysis Frame Model

d. Summary of Structure Analysis and PC Cable Arrangement



*）Marking is Maximum or Miminum Moment Section

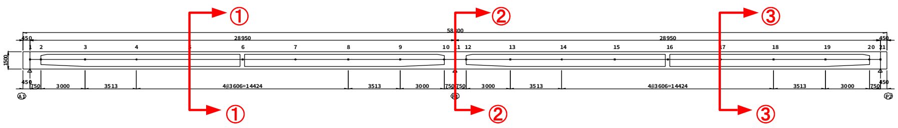


*）Marking is Maximum or Miminum Moment Section

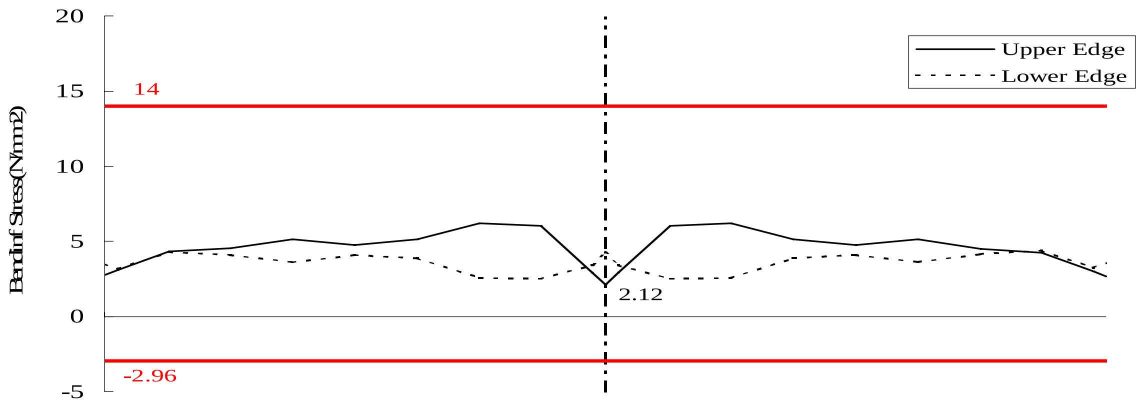
(Source : JICA Study Team)

Figure 5.4.7 Bending Moment Diagram

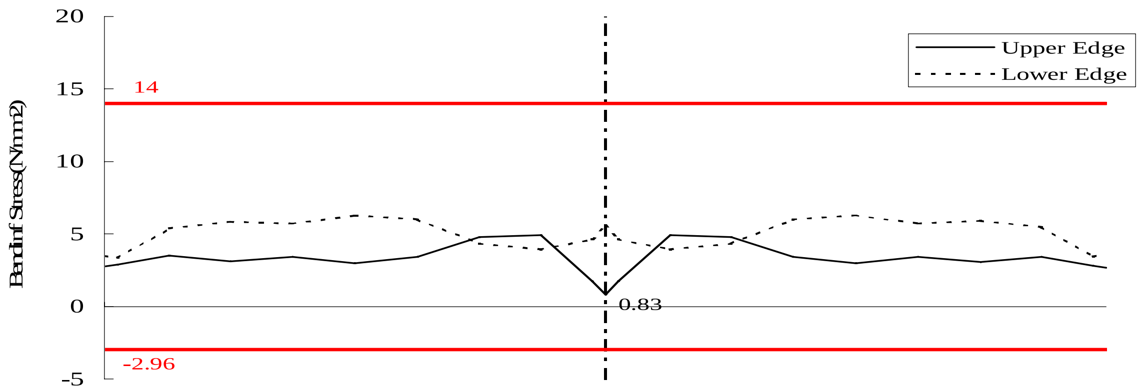
Loadings	Type of Load	SECTION-①				SECTION-②				SECTION-③			
		Sectional Force		Upper Egde σcu	Lower Egde σcl	Sectional Force		Upper Egde σcu	Lower Egde σcl	Sectional Force		Upper Egde σcu	Lower Egde σcl
		M(kNm)	N(kN)	(N/mm^2)	(N/mm^2)	M(kNm)	N(kN)	(N/mm^2)	(N/mm^2)	M(kNm)	N(kN)	(N/mm^2)	(N/mm^2)
Self-Weight	D	11046	0	3.01	-3.87	-20775	0	-5.35	5.27	11046	0	3.01	-3.87
Surfacing	D	3689	0	1.00	-1.23	-6908	0	-1.71	1.74	3689	0	1.00	-1.23
Secondary Force by Pre-Stress	D	1730	0	0.47	-0.61	4814	0	1.24	-1.22	1730	0	0.47	-0.61
Effective Pre-Stress	D	-18914	32968	-0.75	11.03	18491	33897	7.94	-1.51	-18926	32990	-0.75	11.03
Temparature Difference	T	-1176	-5380	0.46	-0.31	970	-5712	1.19	-0.77	-1176	-5380	0.46	-0.31
Live Load with Impact Mmax	L1	5130	0	1.39	-1.71	0	0	0.00	0.00	5130	0	1.39	-1.71
Live Load with Impact Mmin	L2	-1184	0	-0.32	0.40	-5165	0	-1.28	1.30	-1184	0	-0.32	0.40
Losd Combination Group													
Group-I (Mmax) ΣD+L1				5.12	3.61			2.12	4.27			5.12	3.62
Group-I (Mmin) ΣD+L2				3.41	5.72			0.83	5.57			3.41	5.72
Allowable Stress for Group-I		-2.96<σ<14.00				-2.96<σ<14.00				-2.96<σ<14.00			
Group-IV (Mmax) ΣD+L1+T				5.58	3.31			3.31	3.51			5.58	3.31
Group-IV (Mmin) ΣD+L2+T				3.87	5.41			2.03	4.81			3.87	5.42
Allowable Stress for Group-IV		-3.70<σ<17.50				-3.70<σ<17.50				-3.70<σ<17.50			

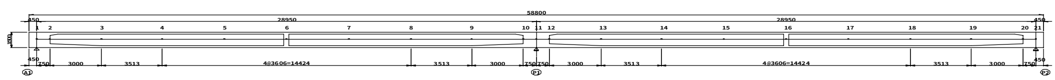


Group-I (D+Lmax)



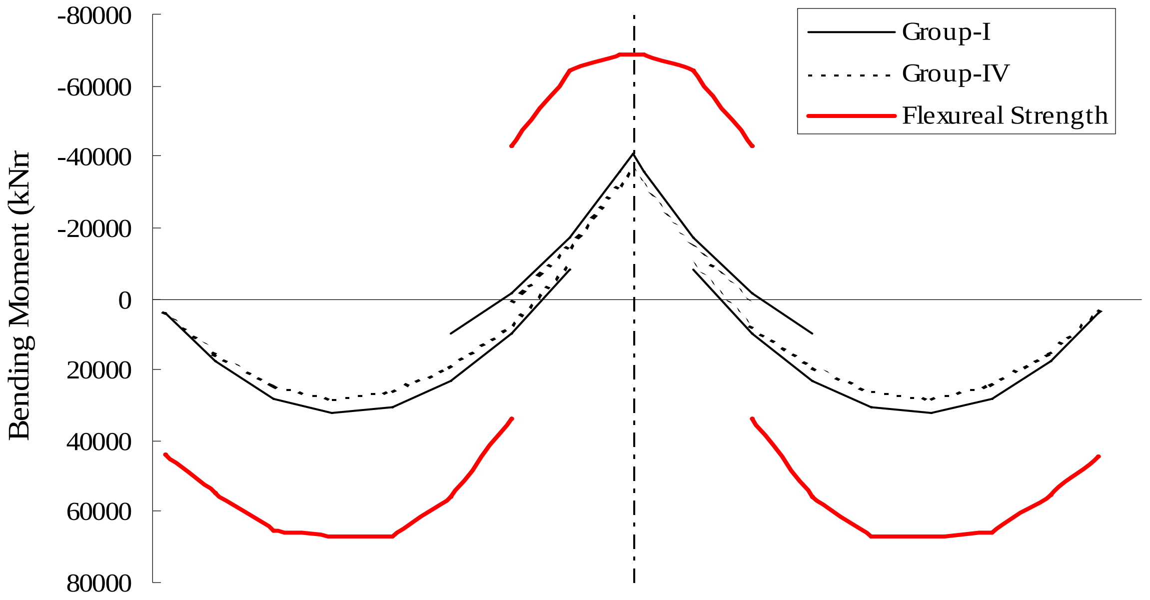
Group-I (D+Lmin)





(Source : JICA Study Team)

Figure 5.4.8 Bending Stress Diagram for Service Load Design



(Source : JICA Study Team)

Figure 5.4.9 Flexural Strength Diagram for Load Factor Design

e. Cable Arrangement

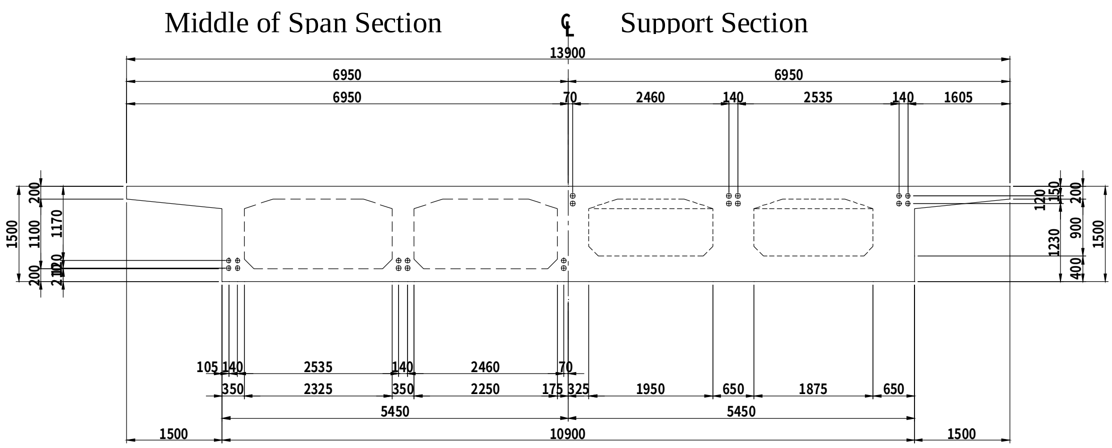


Figure 5.4.10 Cable Arrangement (Section)

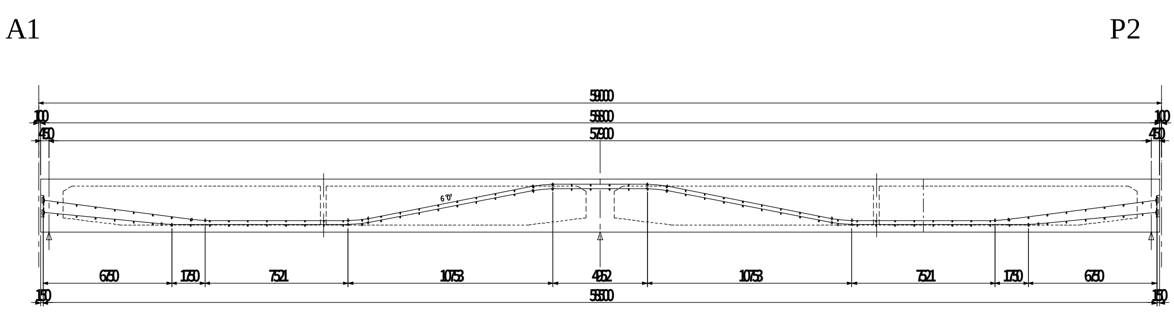


Figure 5.4.11 Cable Arrangement Elevation

f. Detail Design Result of Substructure

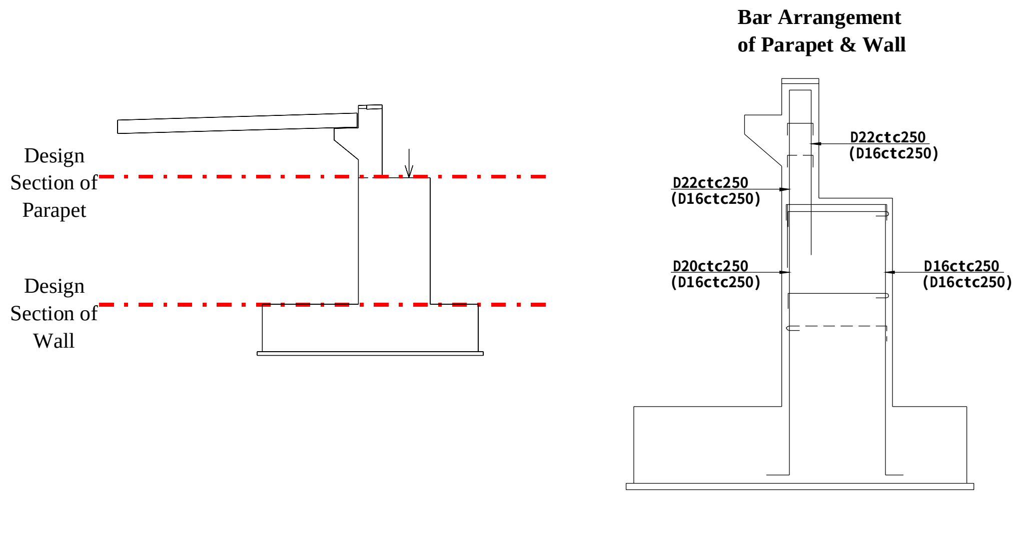
For beam of piers, the determined stress is far less than the allowable stress because stress resultant is constantly acting on the members by dead load, so the stress is controlled around 100N/mm2 for crack control.

<u>A1 Abutment</u>

Table 5.4.4 Design Summary of A1 Abutment (1)

Summary of Design Calculation Result (1/3)

1. Wall and Parapet



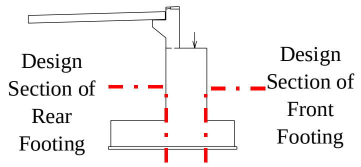
Size of Section			Thickness of Parapet	t=0.500m					
			Thickness of Wall	t=1.500m					
Bar Arrangement			Parapet	D22ctc250-1.0, d=400mm					
			Wall	D20ctc250-1.0, d=1400mm					
Wall	Service Load Design	Load Case	Group-I : (D+SD+L+CF+E+B)						
		Sectional Force	M=	241	kNm,		N=	446 kN	
		Stress	σc=	1.2	<	σca=	9.6 N/mm²	(24 x 0.4)	
			σs=	15.2	<	σsa=	168 N/mm²		
		Load Case	Group-VII : (D+SD+E+B+EQ)						
		Sectional Force	M=	411	kNm,		N=	390 kN	
		Stress	σc=	2.8	<	σca=	12.8 N/mm²	(9.6 x 1.33)	
			$\boxed{σs= 114.5}$		<	σsa=	223 N/mm²	(168 x 1.33)	
	Load Factor Design	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)						
		Sectional Force	Mu=	313	kNm,		Nu=	580 kN	
		Strength	Flexural Strength			φMn=	966 kNm	> Mu	
		Load Case	Group - VII 1.3(D+SD+E+B+EQ)						
		Sectional Force	Mu=	534	kNm,		Nu=	507 kN	
		Strength	Flexural Strength			φMn=	924 kNm	> Mu	
Parapet	Service Load Design	Load Case	Group-I : (D+SD+L+CF+E+B)						
		Sectional Force	M=	97	kNm,		N=	0 kN	
		Stress	σc=	3.4	<	σca=	9.6 N/mm²	(24 x 0.4)	
			$\boxed{σs= 127.9}$		<	σsa=	168 N/mm²		

(Source: JICA Study Team)

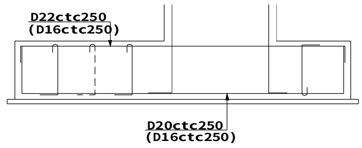
Table 5.4.5 Design Summary of A1 Abutment (2)

Summary of Design Calculation Result (2/3)

2. Spread Footing



Bar Arrangement of Footing



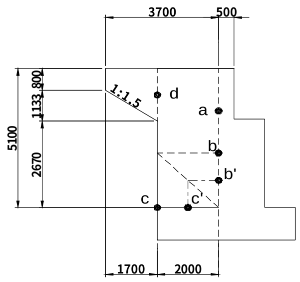
Size of Section			Longitudinal Direction	L_L=4.500m T=1.200m
			Transverse Direction	L_T=13.900m T=1.200m
Stability	Force at the Center of Footing		Group-I	V=11533kN, H=1551kN, M=2742kNm
			Group-VII	V=10153kN, H=3774kN, M=9506kNm
	Safety Factor for Horizontal Force		Group-I	Safety Factor=3.739 > 1.5 (Allowable Factor)
			Group-VII	Safety Factor=1.351 > 1.2 (Allowable Factor)
	Ground Reaction		Group-I	q_{max}=243kN/m² < q_a=400kN/m²
			Group-VII	q_{max}=371kN/m² < q_a=600kN/m²
Service Load Design	Front Footing	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 92 kNm, ①	
		Stress	σc= 1.0 < σca= 9.6 N/mm^2	
			σs= 71.4 < σsa= 168 N/mm^2	
		Load Case	Group-VII : (D+SD+E+B+EQ)	
		Sectional Force	M= 160 kNm, ①	
		Stress	σc= 1.7 < σca= 12.8 N/mm^2	
			$\boxed{σs= 123.4}$ < σsa= 223 N/mm^2	
	Rear Footing	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 47 kNm, ①	
		Stress	σc= 0.4 < σca= 9.6 N/mm^2	
			σs= 30.5 < σsa= 168 N/mm^2	
		Load Case	Group-VII : (D+SD+E+B+EQ)	
		Sectional Force	M= 257 kNm, ①	
		Stress	σc= 2.5 < σca= 12.8 N/mm^2	
			$\boxed{σs= 165.4}$ < σsa= 223 N/mm^2	
Load Factor Design	Front Footing	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 119 kNm, ①	
			Su= 234 kN ②	
		Strength	Flexural Strength φMn= 489 kNm	> Mu
			Shear Strenth φVn= 425 kN	> Su
		Load Case	Group - VII 1.3(D+SD+E+B+EQ)	
		Sectional Force	Mu= 208 kNm, ①	
			Su= 394 kN ②	
		Strength	Flexural Strength φMn= 489 kNm	> Mu
			Shear Strenth φVn= 425 kN	> Su
	Rear Footing	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= -82 kNm, ①	
			Su= 179 kN ②	
		Strength	Flexural Strength φMn= -587 kNm	> Mu
			Shear Strenth φVn= 425 kN	> Su
		Load Case	Group - VII 1.3(D+SD+E+B+EQ)	
		Sectional Force	Mu= 63 kNm, ①	
			Su= 1599 kN ②	
		Strength	Flexural Strength φMn= 587 kNm	> Mu
			Shear Strenth φVn= 425 kN	> Su

(Source : JICA Study Team)

Table 5.4.6 Design Summary of A1 Abutment (3)

Summary of Design Calculation Result (3/3)

3. Wing



Bar Arranegement

Section	Reinforcement-bar Arranngement	
	Inside (mm^2)	Outside (mm2)
a	D16-4Nos.=804.25	D16-4Nos.=804.25
b	D16-4Nos.=804.25	D16-4Nos.=804.25
b'	D16-4Nos.=804.25	D16-4Nos.=804.25
c	D20-4Nos.=804.25	D16-4Nos.=1256.6
c'	D16-4Nos.=804.25	D16-4Nos.=804.25
d	D16-4Nos.=804.25	D16-4Nos.=804.25

Size of Section			Thickness of Wing t=0.700m
Service Load Design	Section-"a"	Load Case	Group-VI : (D+L+E)
		Sectional Force	M= 45.4 kNm,
		Stress	σc= 2.2 < σca= 9.6 N/mm^2
			σs= 123.1 < σsa= 168 N/mm^2
	Section-"b"	Load Case	Group-VI : (D+L+E)
		Sectional Force	M= 53.5 kNm,
		Stress	σc= 3.4 < σca= 9.6 N/mm^2
			σs= 113.7 < σsa= 168 N/mm^2
	Section-"c"	Load Case	Group-VI : (D+L+E)
		Sectional Force	M= 64.0 kNm,
		Stress	σc= 3.4 < σca= 9.6 N/mm^2
			σs= 108.1 < σsa= 168 N/mm^2
	Section-"d"	Load Case	Group-VI : (D+L+E)
		Sectional Force	M= 6.4 kNm,
		Stress	σc= 0.3 < σca= 9.6 N/mm^2
			σs= 18.7 < σsa= 168 N/mm^2
Load Factor Design	Section-"a"	Load Case	Group-VII : 1.3(D+E+EQ)
		Sectional Force	Mu= 59.0 kNm,
		Strength	Flexural Strength φMn= 118 kNm > Mu
	Section-"b"	Load Case	Group-VII : 1.3(D+E+EQ)
		Sectional Force	Mu= 69.6 kNm,
		Strength	Flexural Strength φMn= 118 kNm > Mu
	Section-"c"	Load Case	Group-VII : 1.3(D+E+EQ)
		Sectional Force	Mu= 83.2 kNm,
		Strength	Flexural Strength φMn= 154 kNm > Mu
	Section-"d"	Load Case	Group-VII : 1.3(D+E+EQ)
		Sectional Force	Mu= 8.4 kNm,
		Strength	Flexural Strength φMn= 118 kNm > Mu

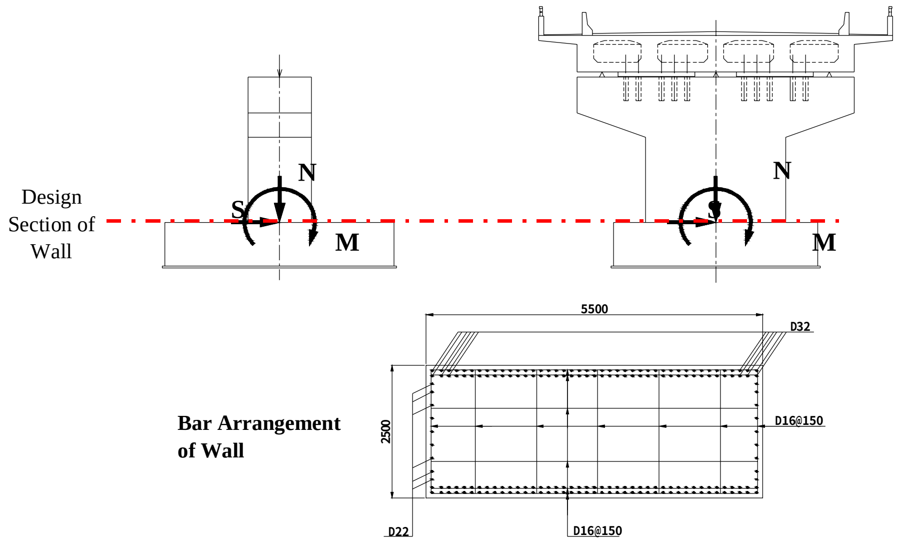
(Source : JICA Study Team)

Table 5.4.7 Design Summary for P1 Pier (1)

P1 Pier

Summary of Design Calculation Result (1/3)

1. Wall



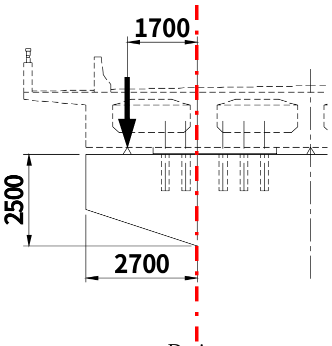
Size of Section			Longitudinal Direction L_L=2.500m
			Transverse Direction L_T=5.500m
Bar Arrangement			Longitudinal Direction D32ctc125-1.0, d=2400mm
			Transverse Direction D32ctc250-1.0, d=5400mm
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 0 kNm, N= 13203 kN
		Stress	σc= 0.84 < σca= 9.6 N/mm^2 (24 x 0.4)
			σs= -12.5 < σsa= 168 N/mm^2
		Load Case	Group-VII : (D+SD+E+B+EQ)
		Sectional Force	M= 37355 kNm, N= 12163 kN
		Stress	σc= 7.3 < σca= 12.8 N/mm^2 (9.6 x 1.33)
			σs= 183.3 < σsa= 223 N/mm^2 (168 x 1.33)
	Transverse Direction	Load Case	Group-VII : (D+SD+E+B+EQ)
		Sectional Force	M= 28504 kNm, N= 12163 kN
		Stress	σc= 5.83 < σca= 12.8 N/mm^2 (9.6 x 1.33)
			σs= 180.9 < σsa= 223 N/mm^2 (168 x 1.33)
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 0 kNm, Nu= 18069 kN
			Su= 0 kN
		Strength	Flexural Strength φMn= 67991 kNm > Mu
			Shear Strenth φVn= 9156 kN > Su
		Load Case	Group - VII 1.3(D+SD+E+B+EQ)
		Sectional Force	Mu= 44304 kNm, Nu= 15812 kN
			Su= 7384 kN
		Strength	Flexural Strength φMn= 45715 kNm > Mu
			Shear Strenth φVn= 9364 kN > Su
	Transverse Direction	Load Case	Group - VII 1.3(D+SD+E+B+EQ)
		Sectional Force	Mu= 31824 kNm, Nu= 15812 kN
			Su= 5304 kN
		Strength	Flexural Strength φMn= 66128 kNm > Mu
			Shear Strenth φVn= 9156 kN > Su

(Source :JICA Study Team)

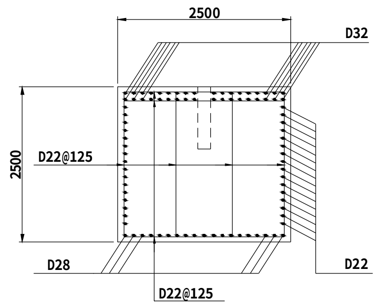
Table 5.4.8 Design Summary for P1 Pier (2)

Summary of Design Calculation Result (2/3)

2. Beam



Bar Arrangement of Beam



Size of Section			Vertical Direction	L_V=2.500m	
			Longitudinal Direction	L_L=2.500m	
Bar Arrangement			Upper Edge Side	D32ctc125-1.0, d=100mm	
			Side Horizontal Bar	D32ctc133-1.0, d=100mm	
Service Load Design	Vertical Direction	Load Case	Group-I : (D+SD+L+CF+E+B)		
		Sectional Force	M= 6371 kNm,		
		Stress	σc= 3.4 <	σca= 9.6 N/mm^2	
			σs= 113.7 <	σsa= 168 N/mm^2	
	Longitudinal Direction	Load Case	Group-VII : (D+SD+E+B+EQ)		
		Sectional Force	M= 3362 kNm,		
		Stress	σc= 2.14 <	σca= 12.8 N/mm^2	
			σs= 174.1 <	σsa= 223 N/mm^2	
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)		
		Sectional Force	Mu= 8282 kNm, Su= 4989 kN		
		Strength	Flexural Strength	φMn= 22317 kNm	> Mu
			Shear Strenth	φVn= 16422 kN	> Su
	Transverse Direction	Load Case	Group - VII 1.3(D+SD+E+B+EQ)		
		Sectional Force	Mu= 4371 kNm, Su= 2612 kN		
		Strength	Flexural Strength	φMn= 4905 kNm	> Mu
			Shear Strenth	φVn= 4161 kN	> Su

Note: The determined stress is far less than the allowable stress due to crack control.

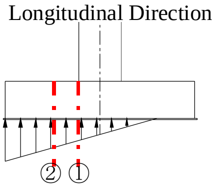
(Source :JICA Study Team)

Table 5.4.9 Design Summary for P1 Pier (3)

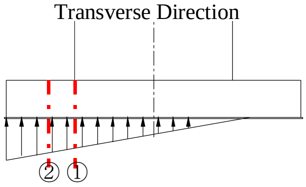
Summary of Design Calculation Result (3/3)

3. Spread Footing

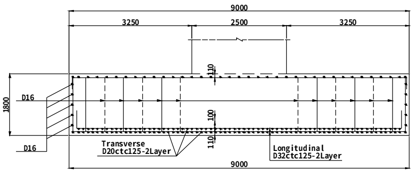
Longitudinal Direction



Transverse Direction



Bar Arrangement of Footing



Size of Section			Longitudinal Direction	L_L=9.000m T=1.800m
			Transverse Direction	L_T=8.000m T=1.800m
Bar Arrangement			Longitudinal Direction	D32ctc125-2.0, d=1690mm
			Transverse Direction	D20ctc125-2.0, d=1716mm
Stability	Force at the Center of Footing	Longitudinal Direction	Group-III	V=19145kN, H=40kN, M=460kNm
			Group-VII	V=18105kN, H=6619kN, M=53101kNm
		Transverse Direction	Group-I	V=19145kN, H=0kN, M=0kNm
			Group-VII	V=18105kN, H=6599kN, M=39153kNm
	Safety Factor for Horizontal Force	Longitudinal Direction	Group-I	Safety Factor=287 > 1.5 (Allowable Factor)
			Group-VII	Safety Factor=1.641 > 1.2 (Allowable Factor)
		Transverse Direction	Group-I	-
			Group-VII	Safety Factor=1.646 > 1.2 (Allowable Factor)
	Ground Reaction	Longitudinal Direction	Group-I	q_{max}=270kN/m^2 < qa=700kN/m^2
			Group-VII	q_{max}=963kN/m^2 < qa=1050kN/m^2
		Transverse Direction	Group-I	q_{max}=266kN/m^2 < qa=700kN/m^2
			Group-VII	q_{max}=730kN/m^2 < qa=1050kN/m^2
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 921 kNm, ①	
		Stress	σc= 2.1 < σca= 9.6 N/mm^2	
			σs= 54.9 < σsa= 168 N/mm^2	
		Load Case	Group-VII : (D+SD+E+B+EQ)	
		Sectional Force	M= 3429 kNm, ①	
		Stress	σc= 8.0 < σca= 12.8 N/mm^2	
			σs= 207.5 < σsa= 223 N/mm^2	
	Transverse Direction	Load Case	Group-VII : (D+SD+E+B+EQ)	
		Sectional Force	M= 769 kNm, ①	
		Stress	σc= 2.9 < σca= 12.8 N/mm^2	
			σs= 136.2 < σsa= 223 N/mm^2	
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 1263 kNm, ①	
			Su= 562 kN ②	
		Strength	Flexural Strength φMn= 6647 kNm	> Mu
			Shear Strenth φVn= 1743 kN	> Su
		Load Case	Group - VII 1.3(D+SD+E+B+EQ)	
		Sectional Force	Mu= 3674 kNm, ①	
			Su= 1599 kN ②	
		Strength	Flexural Strength φMn= 6647 kNm	> Mu
			Shear Strenth φVn= 1743 kN	> Su
	Transverse Direction	Load Case	Group - VII 1.3(D+SD+E+B+EQ)	
		Sectional Force	Mu= 848 kNm, ①	
			Su= 395 kN ②	
		Strength	Flexural Strength φMn= 2783 kNm	> Mu
			Shear Strenth φVn= 1190 kN	> Su

(Source :JICA Study Team)

5.4.2 West Side Approach Viaduct

(1) Conceivable Span Arrangement

West side approach viaduct connected to the man bridge links an approach point on the right bank terrace as shown in **Figure 5.4.11**. Elevation difference between top of proposed road and the ground level is about 20m high, with the 5.6% gradient. The horizontal alignment is 150 m in radius. Considering such conditions four (4) clear span lengths approx. 40 m per span, is required.

Viaduct Section L=168.0m

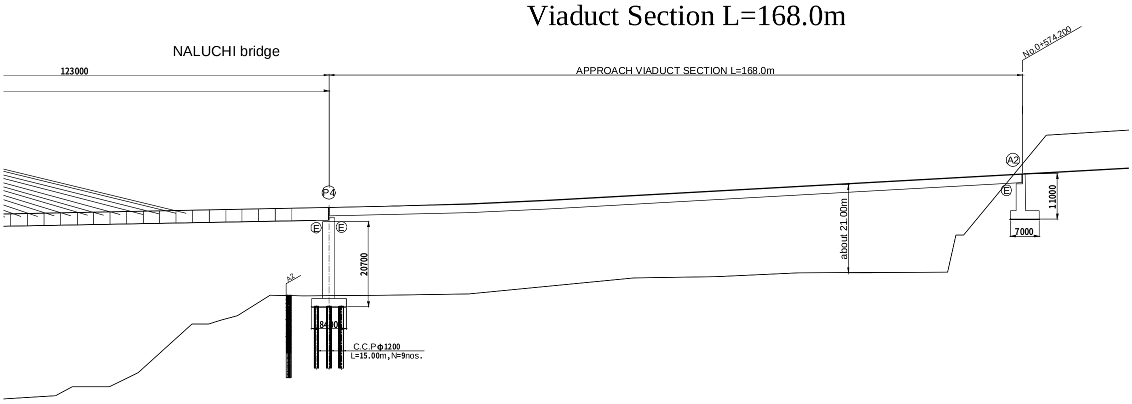


Figure 5.4.12 Profile of the Viaduct

Horizontal road alignment R=150m

(Source : JICA Study Team)

Figure 5.4.13 Plan of the Viaduct

(2) Alternative Study of Superstructure

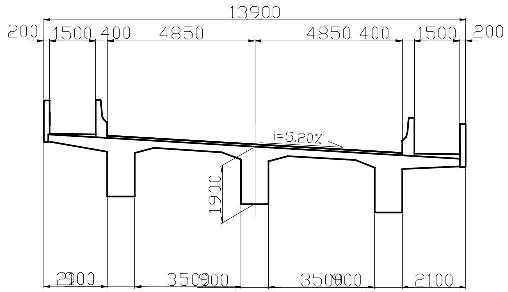
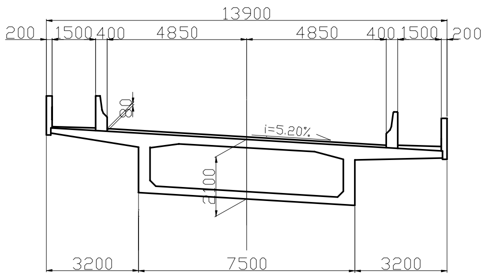
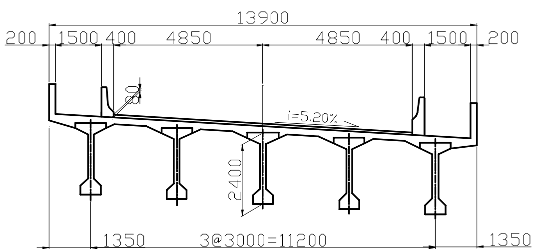
The following three alternatives were formulated in consideration of the curve alignment with 150 m radius, and required clear span lengths. The comparative study is shown in **Figure 5.4.14**.

- Alternative 1: PC Slab girder
- Alternative 2 : PC Box girder
- Alternative 3 : PC I girder

For the PC-I girder, all of the girder length varies due to the curved radius of 150 meter. Moreover, five piers are required due to limitaion of applicable span length for PC-I girder. PC Slab girder also has a limited clear span of 35 m, which will also require more than five piers.

PC Box girder meanwhile can accommodate clear spans of more than 35m.

Based on comparative study of above alternatives shown in **Figure 5.4.13**, PC-Box girder is selected as most suitable considering construction cost, construction aspect, structural aspect, maintenance aspect and technology transfer.

	General View	Evaluation Item	Evaluation	Rating Results
Alternative-1	**Five span continuous PC Girder Cast in Situ** 	Construction Cost	Most economical alternative (1.00)	Excellent
		Construction Aspect	No technical difficulty because of all staging construction method.	Fair
		Structural Aspect	Fair seismicity because of five span continuous girder	Fair
		Const. Period	Longer construction period (about 20 months)	Fair
		Maintenance	Minimal workload except it for bearings and drainage pipes required periodical inspection and maintenance requisite	Poor
		Technology transfer	Less technology transfer elements except construction of PC girder cast in suit	Poor
		Overall	Due to longer construction period and less seismicity with high pier, Alt-1 is discarded	Discarded
Alternative-2	**Four span continuous PC Box girder cast in situ** 	Construction Cost	Moderate cost (1.03)	Good
		Construction Aspect	Both balanced cantilever erection or all staging construction method are suited.	Good
		Structural Aspect	High ductility and seismicity and high monolithic structure. Moreover high torsional rigidity	Excellent
		Const. Period	Relatively shorter construction period (about 18 months)	Good
		Maintenance	Almost maintenance free except cleaning of drainage pipes.	Excellent
		Technology transfer	Much technology transfer elements because of a few construction opportunity of curved box girder bridge in Pakistan	Good
		Overall	No negative evaluation items and relatively advantageous alternative especially structural aspect among the others	Selected
Alternative-3	**Five span continuous PC I section Pre cast Girder** 	Construction Cost	Because of difficulty of erection method this alternative become expensive alternative (1.06)	Poor
		Construction Aspect	Because of difficulty of mobilization of large capacity of truck crane with 100 ton due to poor alignments in access road, erection of pre cast girder by application of erection girder will be costly item.	Bad
		Structural Aspect	Less seismicity / torsional rigidity and low ductility, due to application of I section girder in curved alignment.	Bad
		Const. Period	Shortest construction period (about 16 .months)	Poor
		Maintenance	Minimal workload except it for bearings and drainage pipes required periodical inspection and maintenance requisite	Poor
		Technology transfer	No element of technology transfer because of common practice in Pakistan.	Poor
		Overall	Because of various disadvantageous items, alt-3 is discarded.	Discarded

Note: Figure in ( ) means construction cost ratio against that of Alternative-1.

Figure 5.4.14 Comparison for West side Approach Viaduct

(3) Result of Basic Design

From the resulting of basic design mentioned above, general view of west side viaduct is presented in **Figure.5.4.15**.

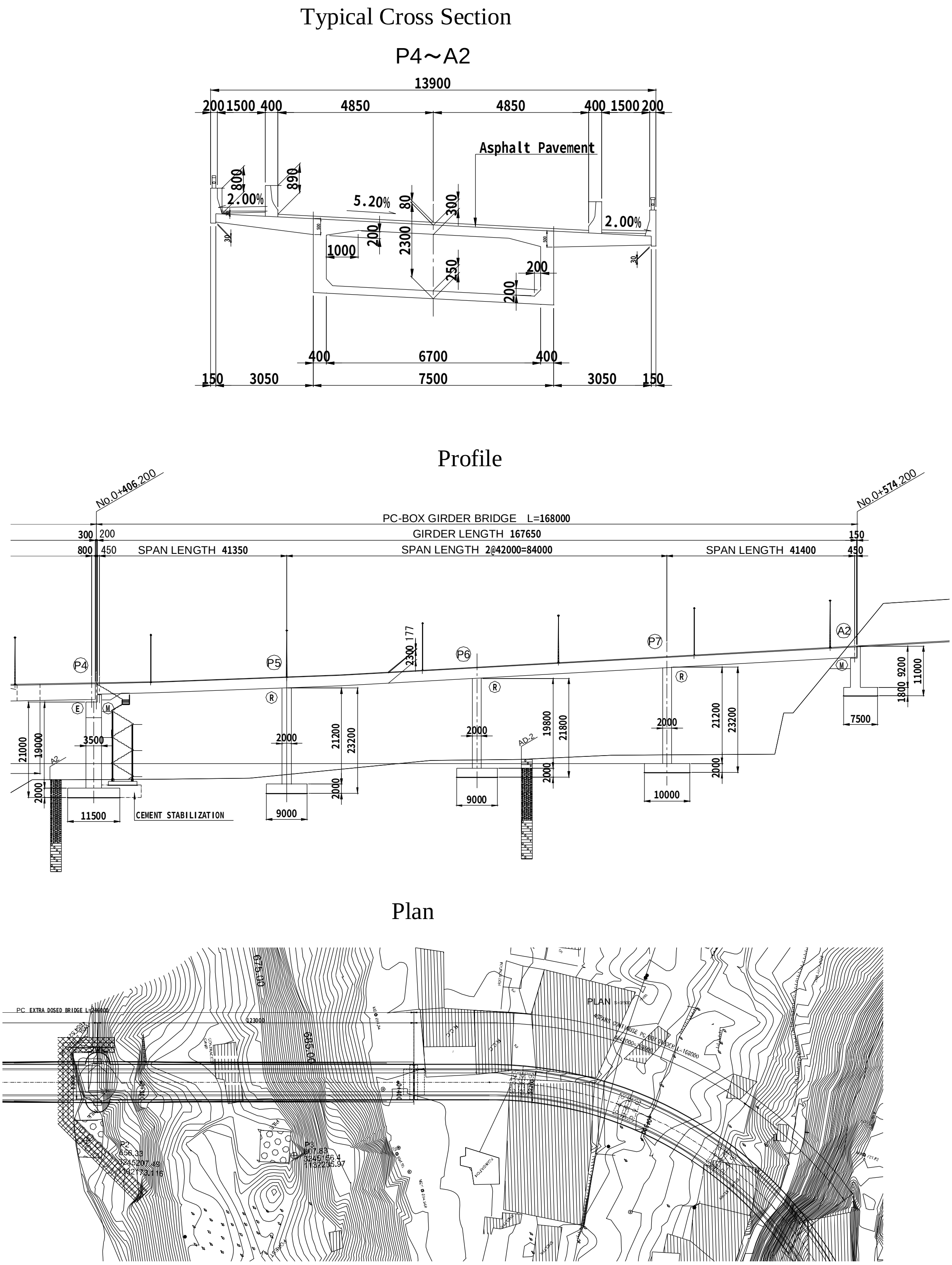


Figure 5.4.15 General View of Viaduct

(4) Detail Design Result

a. Design Conditions

Table 5.4.10 Design Conditions for West Side Viaduct

Superstructure	:	4 spans Continuous PC-Box girder
Bridge Length	:	168.0m
Girder Length	:	167.65m
Span Length	:	41.35m + 2@42.0m + 41.4m
Bridge Width	:	13.9m
Effective Width	:	Carriage Way 9.7m
	:	Foot Way 2 x 1.5m
Live Load	:	Class-A, Class –AA
Curvature	:	R=150m
Gradient	:	i = 2.0% ↙ ～ 5.564%↙
Super Elevation	:	i = 2.0% ↙ ↘ ～ 5.2% ↘
Skew Angle	:	P4 – P7: θ =90 ° A2: θ =65 °
Asphalt pavement	:	Carriageway 80mm～
	:	Foot way 30mm～

(Source : JICA Study Team)

b. Materials Properties

Table 5.4.11 Material Properties

Materials
Concrete (N/mm^2)

		PC-Box girder PC	Cross beam PC	Deck slab PC
Class		D1	D1	D1
28Days Cylinder Strength		35	35	35
Modulus of Elasticity		2.95×10^4	2.95×10^4	2.95×10^4
Allowable Compression Stress		14.00	14.00	14.00
Allowable Tensile Stress		-2.96	-2.96	-2.96
Temperature coefficient		10×10^{-6}	10×10^{-6}	10×10^{-6}
Allowable Shear Stress		0.47	0.47	0.47
Maximum Average Shear Stress		2.50	2.50	2.50
Diagonal Stress	Permanent Load	0.90	0.90	0.90
	Design Combination Load	1.85	1.85	1.85

PC steel

	Unit	Longitudinal 12S15.2	Transverse 3S15.2	Cross Beam 12S15.2
Ultimate Strength	N/mm^2	1860	1860	1860
Minimum Breaking Strength	kN	3128.4	782.1	3128.4
Minimum Yeild Strength	kN	2815.6	703.9	2815.6
Friction coefficient per 1 meter	1/m	0.0020	0.0020	0.0020
Friction coefficient per 1 radian	1/Rad	0.250	0.250	0.250
Set Losses	mm	9	9	9
Relaxation	%	1.5	1.5	1.5
Modulus of Elasticity	x10^{5}N/mm^2	2.0	2.0	2.0
Sectional area	mm^2	1664.4	416.1	1664.4
Diameter of sheath	mm	75	65	75

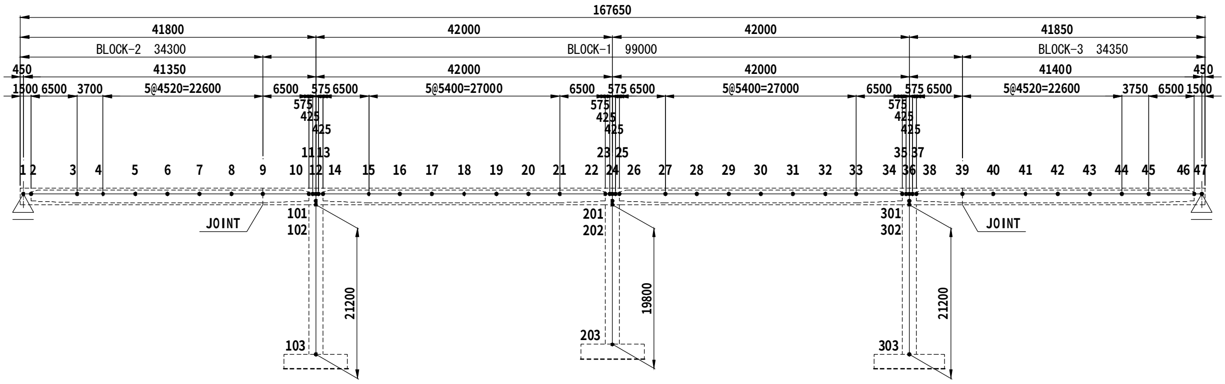
Reinforcement Bar (N/mm^2)

Yield strength	420
Modulus of Elasticity（$\times10^5$）	2.0
Allowable Tensile Stress	168

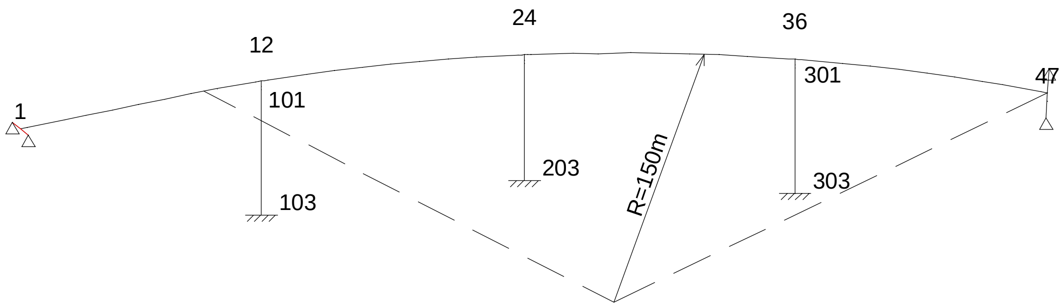
(Source : JICA Study Team)

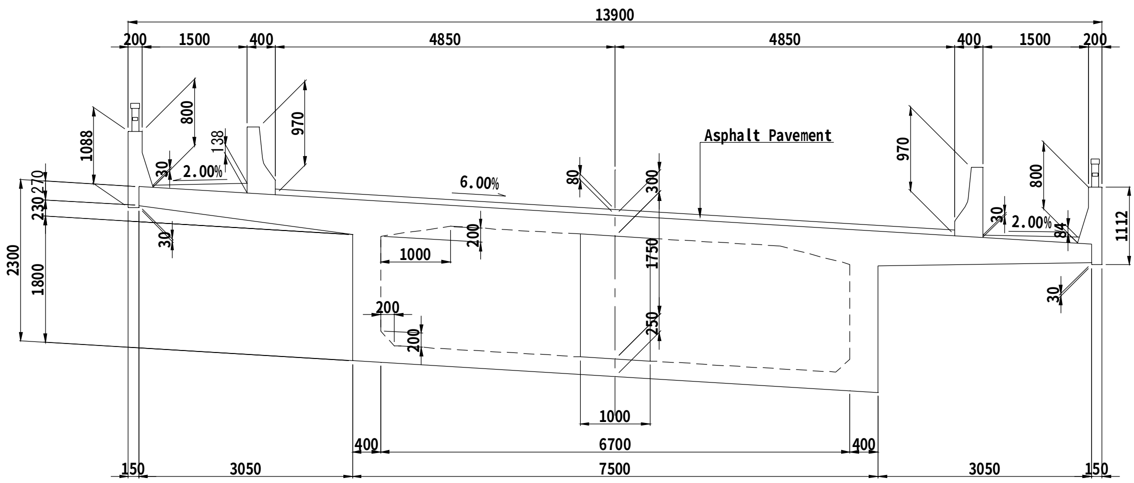
c. Analysis Model

Analysis Model for Bending Moment, Shear Force and Axial Force



3D-Analysis Model for Torsional Moment





(Source : JICA Study Team)

Figure 5.4.16 Analysis Frame Model

d. Summary of Structural Analysis

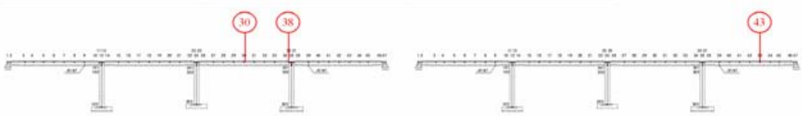
Table 5.4.12 Design Summary of Service Load Design

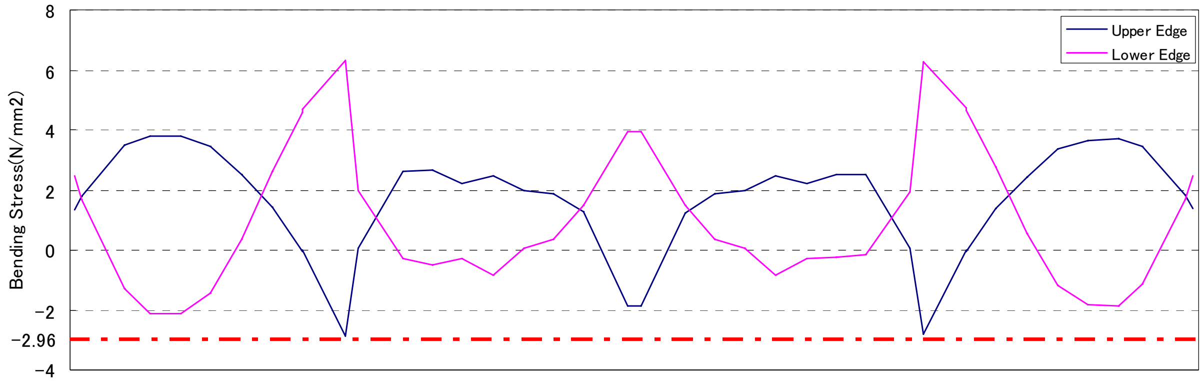
		5-i Joint No. 5				9-j Joint No. 10				18-i Joint No. 18				21-j Joint No. 22			
		Sectional Force		σcu	σcl	Sectional Force		σcu	σcl	Sectional Force		σcu	σcl	Sectional Force		σcu	σcl
		M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)
Self Weight + Stay Cable	D	26151	0	3.57	-6.50	-37363	0	-4.87	6.14	12629	-324	1.68	-3.18	-25052	-324	-3.29	4.08
Creep Effect of Self Weight	D	-3946	0	-0.54	0.97	-10057	0	-1.31	1.65	-262	-514	-0.10	0.01	1770	-514	0.18	-0.34
Surfacing	D	7418	0	1.01	-1.84	-11086	0	-1.44	1.82	3387	-146	0.44	-0.86	-6769	-146	-0.90	1.10
Creep Effect of Sufracing	D	-318	0	-0.04	0.08	-810	0	-0.10	0.13	177	-97	0.01	-0.05	-294	-97	-0.05	0.04
2ndary Force by Prestress	D	8689	0	1.19	-2.16	22148	0	2.89	-3.64	11861	189	1.64	-2.92	6622	189	0.88	-1.07
Creep Effect of 2ndary Force	D	298	0	0.04	-0.08	759	0	0.11	-0.13	-593	-87	-0.09	0.13	418	-87	0.04	-0.08
Effective Prestress	D	-25864	21651	-0.84	9.11	15536	19707	3.93	-0.65	-21771	18224	-0.71	7.67	11489	15884	3.03	-0.35
Losses of Prestress	D	-907	0	-0.12	0.22	-2311	0	-0.30	0.38	-1641	0	-0.22	0.40	-1225	0	-0.16	0.20
CLASS-A Live Load Mmax	L	8199	0	1.11	-1.97	385	0	0.05	-0.06	5797	165	0.81	-1.37	1872	-266	0.22	-0.33
CLASS-A Live Load Mmin	L	-988	0	-0.13	0.24	-9978	0	-1.28	1.64	-1762	-321	-0.28	0.38	-8299	134	-1.05	1.37
CLASS-A Live Load Nmax	L	0	0	0.00	0.00	0	0	0.00	0.00	5311	268	0.75	-1.24	-5813	268	-0.72	0.98
CLASS-A Live Load Nmin	L	0	0	0.00	0.00	0	0	0.00	0.00	-1531	-399	-0.26	0.32	-498	-399	-0.10	0.04
Shrinkage	S	-1729	0	-0.23	0.41	-4407	0	-0.56	0.72	3220	-1991	0.19	-1.03	-7433	-1991	-1.15	1.03
Thermal Rise	T	860	0	0.12	-0.21	2193	0	0.28	-0.36	-714	533	-0.03	0.24	1527	533	0.25	-0.20
Temparature Difference	T	-1781	-7541	0.31	-0.50	1221	-7659	0.90	-0.93	284	-7332	0.62	-0.97	-1315	-7450	0.59	-0.50
Group-I (L-Mmax)				5.37	-2.17			-1.05	5.65			3.46	-0.18			-0.04	3.25
Group-I (L-Mmin)				4.13	0.04			-2.38	7.35			2.38	1.57			-1.31	4.96
Group-I (L-Nmax)				4.26	-0.20			-1.10	5.71			3.41	-0.05			-0.97	4.57
Group-I (L-Nmin)				4.26	-0.20			-1.10	5.71			2.40	1.51			-0.36	3.63
Allowable Stress for Group-I				-3.16<σ<16.00				-3.16<σ<16.00				-3.16<σ<16.00				-3.16<σ<16.00	
Group-IV (L-Mmax, T-Rise)				5.26	1.79			-1.33	1.79			3.63	1.79			-0.94	1.79
Group-IV (L-Mmin, T-Rise)				4.01	1.79			-2.66	1.79			2.54	1.79			-2.21	1.79
Group-IV (L-Nmax, T-Rise)				4.14	2.07			-1.38	2.07			3.57	2.07			-1.88	2.07
Group-IV (L-Nmin, T-Rise)				4.14	2.07			-1.38	2.07			2.56	2.07			-1.26	2.07
Group-IV (L-Mmax, T-Down)				5.02	-1.55			-1.89	6.73			3.69	-1.44			-1.44	4.48
Group-IV (L-Mmin, T-Down)				3.78	0.65			-3.22	8.43			2.60	0.31			-2.70	6.19
Group-IV (L-Nmax, T-Down)				3.91	0.41			-1.94	6.80			3.64	-1.31			-2.37	5.79
Group-IV (L-Nmin, T-Down)				3.91	0.41			-1.94	6.80			2.63	0.25			-1.75	4.86
Group-IV (L-Mmax, T-Rise, T-Diff.)				5.57	-2.46			-0.43	5.08			4.24	-1.94			-0.35	3.59
Group-IV (L-Mmin, T-Rise, T-Diff.)				4.32	-0.26			-1.76	6.78			3.16	-0.18			-1.62	5.29
Group-IV (L-Nmax, T-Rise, T-Diff.)				4.45	-0.50			-0.48	5.14			4.19	-1.81			-1.28	4.90
Group-IV (L-Nmin, T-Rise, T-Diff.)				4.45	-0.50			-0.48	5.14			3.18	-0.25			-0.67	3.96
Group-IV (L-Mmax, T-Down, T-Diff.)				5.33	-2.05			-0.99	5.80			4.31	-2.41			-0.84	3.98
Group-IV (L-Mmin, T-Down, T-Diff.)				4.09	0.15			-2.32	7.50			3.22	-0.66			-2.11	5.69
Group-IV (L-Nmax, T-Down, T-Diff.)				4.22	-0.08			-1.04	5.86			4.25	-2.28			-1.78	5.30
Group-IV (L-Nmin, T-Down, T-Diff.)				4.22	-0.08			-1.04	5.86			3.24	-0.72			-1.16	4.36
Allowable Stress for Group-IV				-3.95<σ<20.00				-3.95<σ<20.00				-3.95<σ<20.00				-3.95<σ<20.00	

*) Group-I : ΣD + L
Group-I : ΣD + L + S + T

		30-i Joint No. 30				38-i Joint No. 38				43-i Joint No. 43			
		Sectional Force		σcu	σcl	Sectional Force		σcu	σcl	Sectional Force		σcu	σcl
		M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)	M (kN.m)	N(kN)	(N/mm²)	(N/mm²)
Self Weight + Stay Cable	D	12814	-282	1.65	-3.21	-35719	0	-4.53	5.84	26400	0	3.49	-6.54
Creep Effect of Self Weight	D	-968	-537	-0.19	0.18	-11941	0	-1.51	1.95	-4649	0	-0.62	1.14
Surfacing	D	3467	-134	0.45	-0.88	-10591	0	-1.34	1.73	6748	0	0.90	-1.67
Creep Effect of Sufracing	D	99	-99	0.00	-0.04	-1018	0	-0.13	0.17	-396	0	-0.05	0.10
2ndary Force by Prestress	D	12272	156	1.64	-3.02	21626	0	2.74	-3.54	8419	0	1.11	-2.09
Creep Effect of 2ndary Force	D	-84	-49	-0.02	0.01	2705	0	0.35	-0.44	1053	0	0.14	-0.27
Effective Prestress	D	-22049	18216	-0.71	7.67	15252	19755	3.79	-0.62	-26230	21670	-0.84	9.12
Losses of Prestress	D	-1679	4	-0.22	0.41	-2206	0	-0.28	0.36	-859	0	-0.11	0.21
CLASS-A Live Load Mmax	L	5798	164	0.79	-1.37	386	0	0.05	-0.06	8141	0	1.08	-1.95
CLASS-A Live Load Mmin	L	-1757	-319	-0.27	0.38	-9938	0	-1.24	1.62	-982	0	-0.13	0.24
CLASS-A Live Load Nmax	L	5310	268	0.74	-1.24	0	0	0.00	0.00	0	0	0.00	0.00
CLASS-A Live Load Nmin	L	-1525	-398	-0.25	0.32	0	0	0.00	0.00	0	0	0.00	0.00
Shrinkage	S	3241	-1988	0.19	-1.02	-4255	0	-0.53	0.69	-1657	0	-0.22	0.39
Thermal Rise	T	-713	533	-0.03	0.24	2196	0	0.27	-0.36	855	0	0.11	-0.20
Temparature Difference	T	285	-7332	0.63	-0.95	1224	-7659	0.91	-0.92	-1802	-7541	0.33	-0.48
Group-I (L-Mmax)				3.39	-0.25			-0.85	5.39			5.09	-1.95
Group-I (L-Mmin)				2.33	1.50			-2.14	7.08			3.88	0.23
Group-I (L-Nmax)				3.34	-0.12			-0.90	5.46			4.01	-0.01
Group-I (L-Nmin)				2.35	1.44			-0.90	5.46			4.01	-0.01
Allowable Stress for Group-I				-3.16<σ<16.00				-3.16<σ<16.00				-3.16<σ<16.00	
Group-IV (L-Mmax, T-Rise)				3.55	1.79			-1.10	1.79			4.98	1.79
Group-IV (L-Mmin, T-Rise)				2.49	1.79			-2.39	1.79			3.77	1.79
Group-IV (L-Nmax, T-Rise)				3.50	2.07			-1.15	2.07			3.90	2.07
Group-IV (L-Nmin, T-Rise)				2.51	2.07			-1.15	2.07			3.90	2.07
Group-IV (L-Mmax, T-Down)				3.61	-1.51			-1.65	6.45			4.75	-1.36
Group-IV (L-Mmin, T-Down)				2.55	0.24			-2.94	8.13			3.55	0.82
Group-IV (L-Nmax, T-Down)				3.56	-1.38			-1.70	6.51			3.68	0.59
Group-IV (L-Nmin, T-Down)				2.57	0.18			-1.70	6.51			3.68	0.59
Group-IV (L-Mmax, T-Rise, T-Diff.)				4.18	-1.99			-0.20	4.81			5.31	-2.24
Group-IV (L-Mmin, T-Rise, T-Diff.)				3.12	-0.24			-1.49	6.49			4.10	-0.06
Group-IV (L-Nmax, T-Rise, T-Diff.)				4.13	-1.86			-0.25	4.87			4.23	-0.30
Group-IV (L-Nmin, T-Rise, T-Diff.)				3.15	-0.30			-0.25	4.87			4.23	-0.30
Group-IV (L-Mmax, T-Down, T-Diff.)				4.24	-2.46			-0.75	5.52			5.08	-1.84
Group-IV (L-Mmin, T-Down, T-Diff.)				3.18	-0.71			-2.04	7.21			3.88	0.35
Group-IV (L-Nmax, T-Down, T-Diff.)				4.19	-2.33			-0.79	5.59			4.01	0.11
Group-IV (L-Nmin, T-Down, T-Diff.)				3.21	-0.77			-0.79	5.59			4.01	0.11
Allowable Stress for Group-IV				-3.95<σ<20.00				-3.95<σ<20.00				-3.95<σ<20.00	

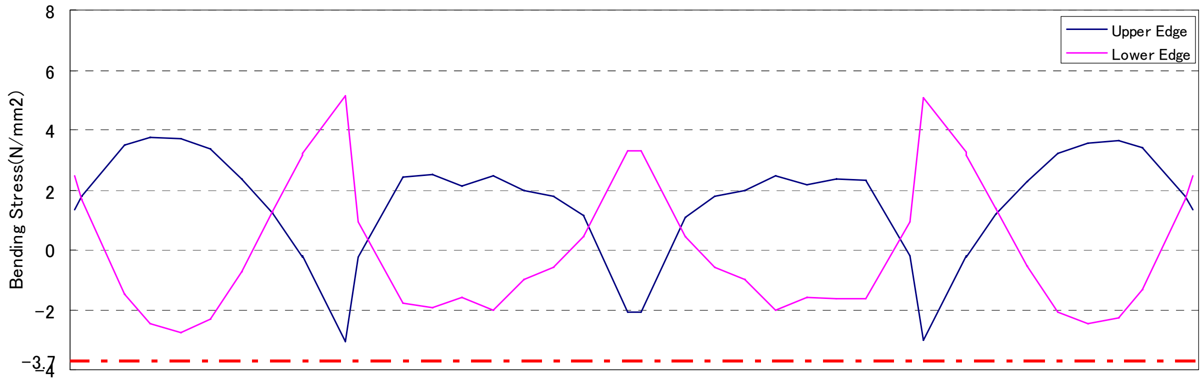
*) Group-I : ΣD + L
Group-I : ΣD + L + S + T





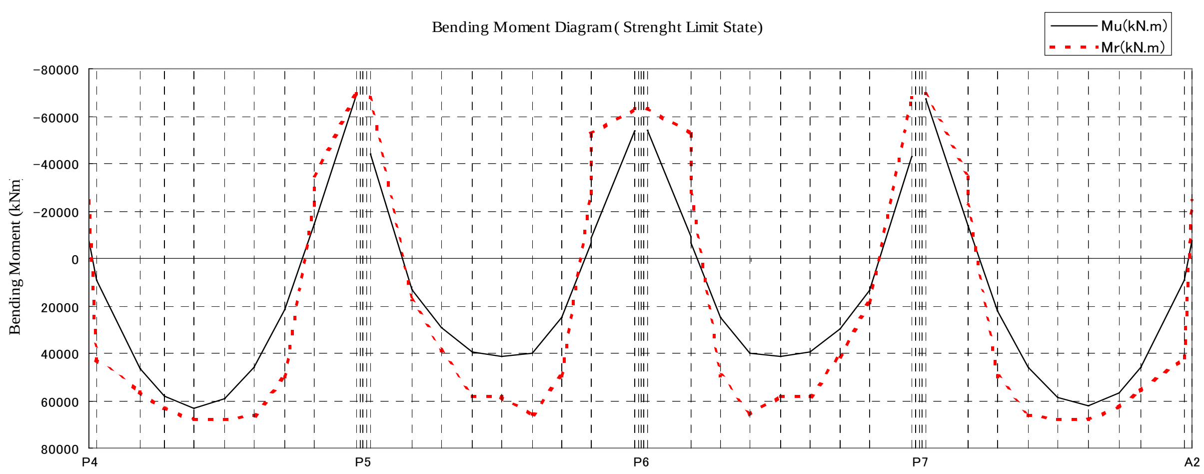
(Source : JICA Study Team)

Figure 5.4.17 Bending Stress Diagram for Service Load Design Group-I



(Source : JICA Study Team)

Figure 5.4.18 Bending Stress Diagram for Service Load Design Group-IV



(Source : JICA Study Team)

Figure 5.4.19 Flexural Strength Diagram for Load Factor Design

e. Cable Arrangement

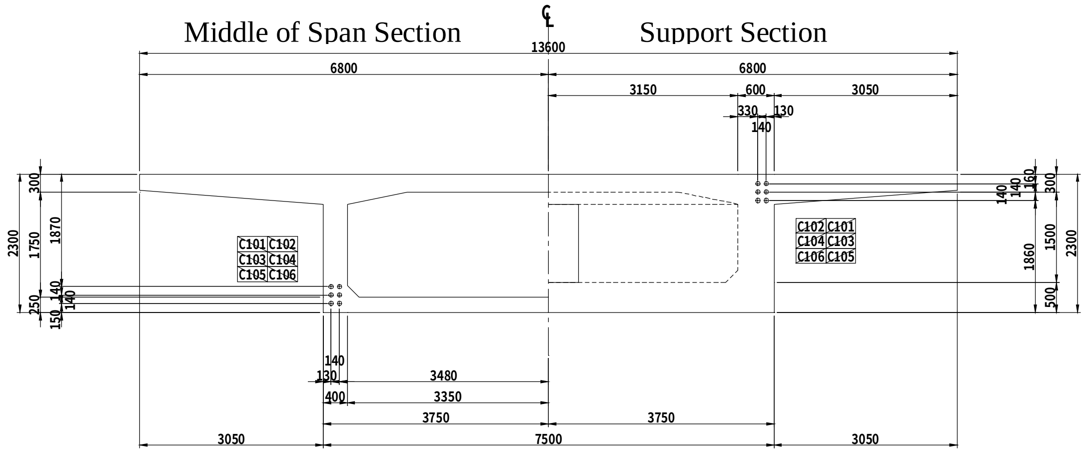
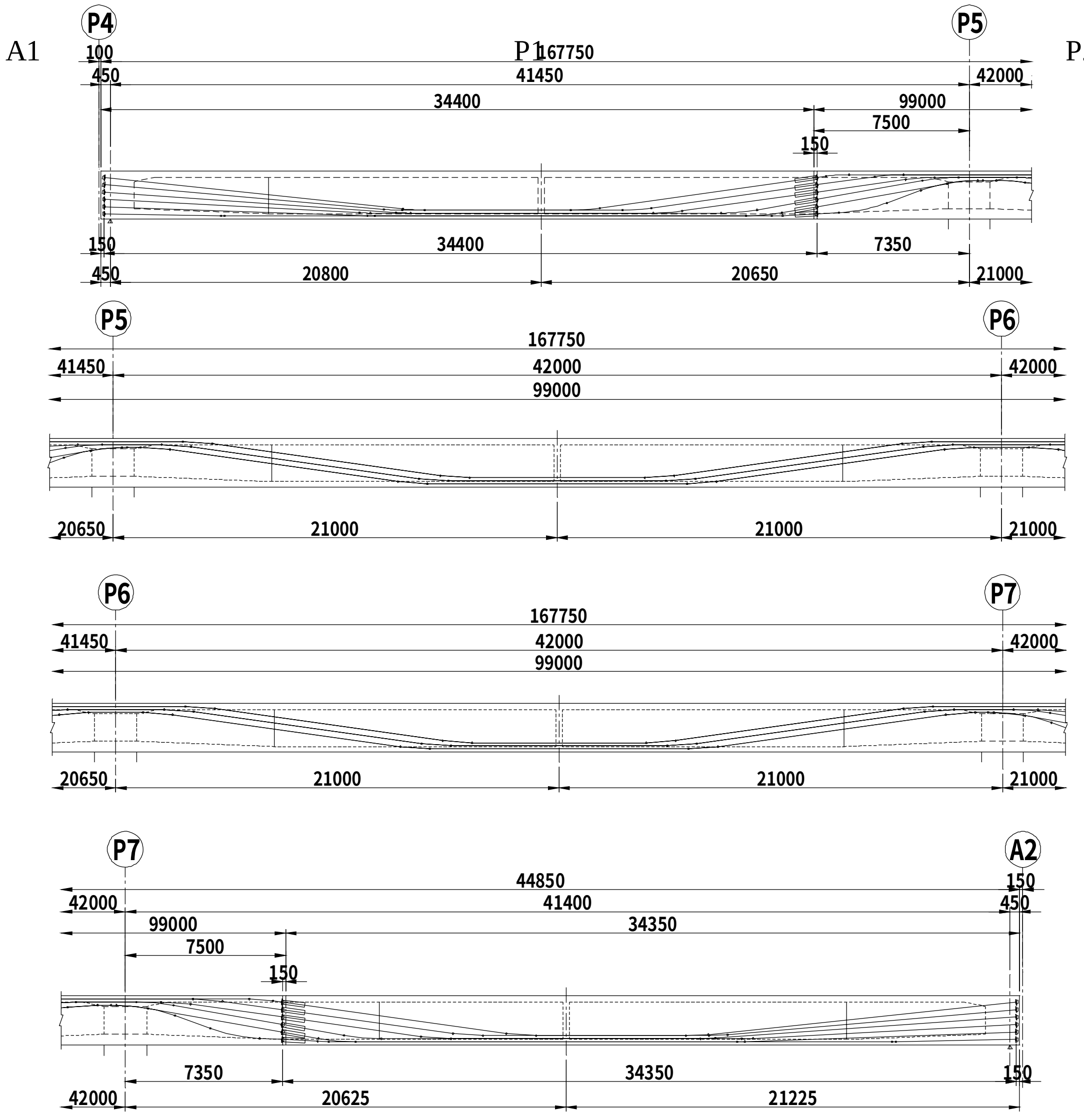


Figure 5.4.20 Cable Arrangement (section)



(Source : JICA Study Team)

Figure 5.4.21 Cable Arrangement (elevation)

f. Detailed Design Result of Substructure

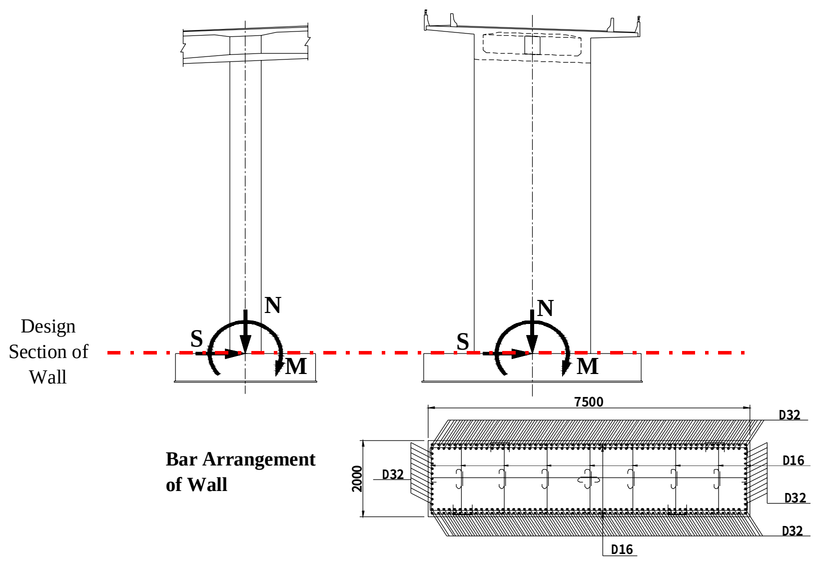
For some members, the determined stress is far less than the allowable stress because required reinforced steel bars in the bridge axis direction are arranged on the side, so the allowance becomes large in transverse direction.

Table 5.4.13 Design Summary for P5 Pier (1)

<u>Pier P5</u>

Summary of Design Calculation Result (1/2)

1. Wall



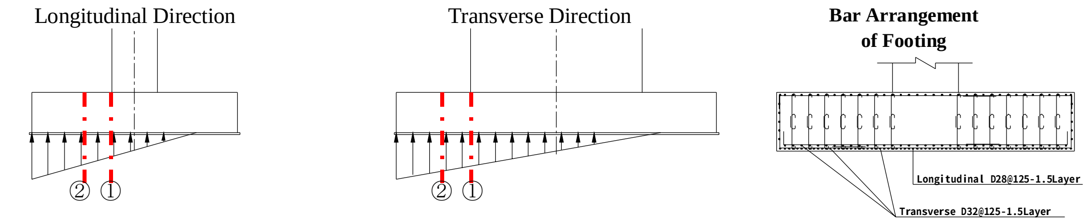
Size of Section		Longitudinal Direction	L_L=2.000m
		Transverse Direction	L_T=7.500m
Bar Arrangement		Longitudinal Direction	D32ctc125-1.0, d=1900mm
		Transverse Direction	D32ctc133-1.0, d=7400mm
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 13175 kNm, N= 20034 kN
		Stress	σc= 4.1 < σca= 9.6 N/mm^2 (24 x 0.4)
			σs= 21.6 < σsa= 168 N/mm^2
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 8710 kNm, N= 18991 kN
		Stress	σc= 1.7 < σca= 9.6 N/mm^2 (24 x 0.4)
			σs= -11.8 < σsa= 168 N/mm^2
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 19041 kNm, Nu= 26873 kN Su= 1791 kN
		Strength	Flexural Strength φMn= 68314 kNm > Mu Shear Strenth φVn= 9752 kN > Su
	Transverse Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 11323 kNm, Nu= 25706 kN Su= 446 kN
		Strength	Flexural Strength φMn= 230093 kNm > Mu Shear Strenth φVn= 10265 kN > Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 69320 kNm, N= 22518 kN
		Stress	σc= 18.4 < σca= 20.4 N/mm^2 (24 x 0.95)
			$\boxed{σs= 369.4}$ < σsa= 378 N/mm^2 (420 x 0.9)
	Transverse Direction	Sectional Force	M= 114645 kNm, N= 20486 kN
		Stress	σc= 8.4 < σca= 20.4 N/mm^2 (24 x 0.95)
			$\boxed{σs= 175.3}$ < σsa= 378 N/mm^2 (420 x 0.9)

(Source : JICA Study Team)

Table 5.4.14 Design Summary for P5 Pier (2)

Summary of Design Calculation Result (2/2)

2. Spread Footing



Size of Section			Longitudinal Direction	LL=9.000m T=2.000m
			Transverse Direction	LT=14.000m T=2.000m
Bar Arrangement			Longitudinal Direction	D28ctc125-1.5, d=1890mm
			Transverse Direction	D32ctc125-1.5, d=1920mm
Stability	Force at the Center of Footing	Longitudinal Direction	Group-I	V=30848kN, H=1184kN, M=15543kNm
			Seicmic Design	V=32690kN, H=9351kN, M=85861kNm
		Transverse Direction	Group-I	V=29805kN, H=343kN, M=9396kNm
			Seicmic Design	V=30660kN, H=8161kN, M=128811kNm
	Safety Factor for Horizontal Force	Longitudinal Direction	Group-I	Safety Factor=15.633 > 1.5 (Allowable Factor)
			Seicmic Design	Safety Factor=2.098 > 1.2 (Allowable Factor)
		Transverse Direction	Group-I	Safety Factor=52.168 > 1.5 (Allowable Factor)
			Seicmic Design	Safety Factor=2.254 > 1.2 (Allowable Factor)
	Ground Reaction	Longitudinal Direction	Group-I	qmax=327kN/m² < qa=700kN/m²
			Seicmic Design	qmax=831kN/m² < qa=1050kN/m²
		Transverse Direction	Group-I	qmax=269kN/m² < qa=700kN/m²
			Seicmic Design	qmax=811kN/m² < qa=1050kN/m²
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 1657 kNm, ①	
		Stress	σc= 3.7 < σca= 9.6 N/mm²	
			σs= 137.0 < σsa= 168 N/mm²	
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 1422 kNm, ①	
		Stress	σc= 2.8 < σca= 9.6 N/mm²	
			σs= 89.7 < σsa= 168 N/mm²	
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 2278 kNm, ①	
			Su= 923 kN ②	
		Strength	Flexural Strength φMn= 4572 kNm	> Mu
			Shear Strenth φVn= 1242 kN	> Su
	Transverse Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 1916 kNm, ①	
			Su= 815 kN ②	
		Strength	Flexural Strength φMn= 5993 kNm	> Mu
			Shear Strenth φVn= 1262 kN	> Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 4376 kNm, ①	
		Stress	σc= 9.7 < σca= 20.4 N/mm²	
			σs= 361.8 < σsa= 378 N/mm²	
	Transverse Direction	Sectional Force	M= 5072 kNm, ①	
		Stress	σc= 9.9 < σca= 20.4 N/mm²	
			σs= 319.9 < σsa= 378 N/mm²	

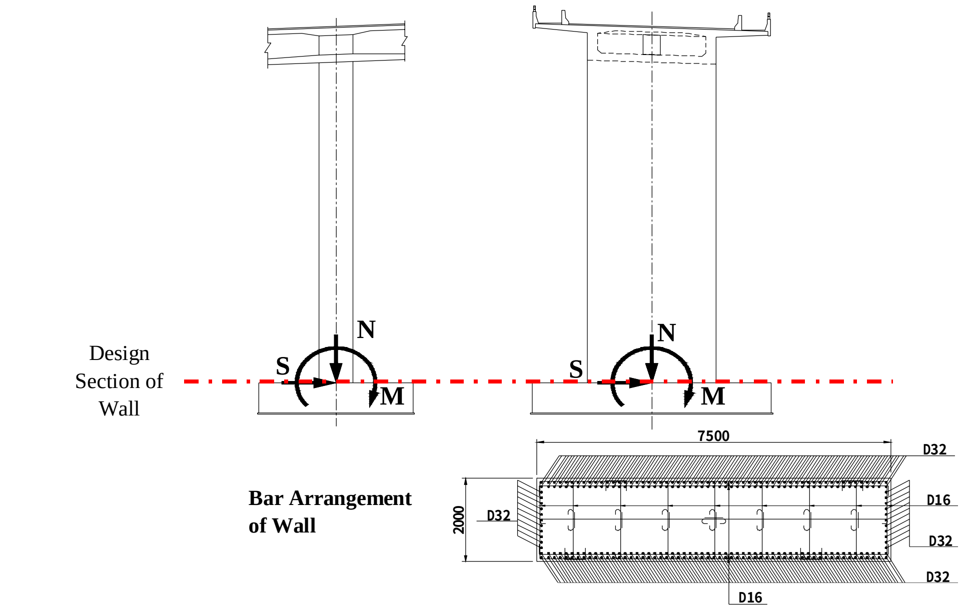
(Source : JICA Study Team)

Table 5.4.15 Design Summary for P6 Pier (1)

Pier P6

Summary of Design Calculation Result (1/2)

1. Wall



Size of Section			Longitudinal Direction L_L=2.000m
			Transverse Direction L_T=7.500m
Bar Arrangement			Longitudinal Direction D32ctc125-1.0, d=1900mm
			Transverse Direction D32ctc133-1.0, d=7400mm
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 2324 kNm, N= 17675 kN
		Stress	σc= 1.8 < σca= 9.6 N/mm²
			σs= -7.8 < σsa= 168 N/mm²
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 7500 kNm, N= 16733 kN
		Stress	σc= 1.3 < σca= 9.6 N/mm²
			σs= -9.1 < σsa= 168 N/mm²
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 6033 kNm, Nu= 23579 kN
			Su= 759 kN
		Strength	Flexural Strength φMn= 71234 kNm > Mu
			Shear Strenth φVn= 9752 kN > Su
	Transverse Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 9750 kNm, Nu= 22977 kN
			Su= 406 kN
		Strength	Flexural Strength φMn= 255145 kNm > Mu
			Shear Strenth φVn= 10265 kN > Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 59081 kNm, N= 16733 kN
		Stress	σc= 15.7 < σca= 20.4 N/mm²
			$\boxed{\sigma s= 311.8}$ < σsa= 378 N/mm²
	Transverse Direction	Sectional Force	M= 112382 kNm, N= 19916 kN
		Stress	σc= 8.24 < σca= 20.4 N/mm²
			$\boxed{\sigma s= 172.7}$ < σsa= 378 N/mm²

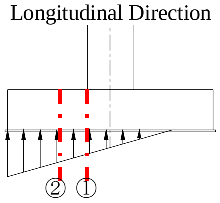
(Source : JICA Study Team)

Table 5.4.16 Design Summary for P6 Pier (2)

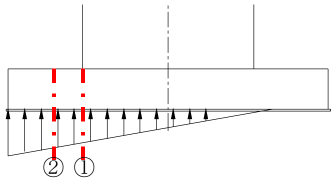
Summary of Design Calculation Result (2/2)

2. Spread Footing

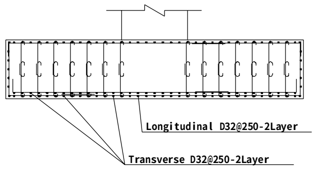
Longitudinal Direction



Transverse Direction



Bar Arrangement of Footing



Size of Section			Longitudinal Direction	L_L=9.000m T=2.000m
			Transverse Direction	L_T=14.000m T=2.000m
Bar Arrangement			Longitudinal Direction	D32ctc250-2.0, d=1890mm
			Transverse Direction	D32ctc250-2.0, d=1920mm
Stability	Force at the Center of Footing	Longitudinal Direction	Group-I	V=28489kN, H=370kN, M=3743kNm
			Seicmic Design	V=30100kN, H=408kN, M=74701kNm
		Transverse Direction	Group-I	V=27547kN, H=312kN, M=8125kNm
			Seicmic Design	V=30090kN, H=8891kN, M=127731kNm
	Safety Factor for Horizontal Force	Longitudinal Direction	Group-I	Safety Factor=46.233 > 1.5 (Allowable Factor)
			Seicmic Design	Safety Factor=44.265 > 1.2 (Allowable Factor)
		Transverse Direction	Group-I	Safety Factor=52.913 > 1.5 (Allowable Factor)
			Seicmic Design	Safety Factor=2.031 > 1.2 (Allowable Factor)
	Ground Reaction	Longitudinal Direction	Group-I	qmax=246kN/m² < qa=700kN/m²
			Seicmic Design	qmax=710kN/m² < qa=1050kN/m²
		Transverse Direction	Group-I	qmax=246kN/m² < qa=700kN/m²
			Seicmic Design	qmax=809kN/m² < qa=1050kN/m²
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 1150 kNm, ①	
		Stress	σc= 2.7 < σca= 9.6 N/mm²	
			σs= 111.5 < σsa= 168 N/mm²	
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 1244 kNm, ①	
		Stress	σc= 2.2 < σca= 9.6 N/mm²	
			σs= 77.3 < σsa= 168 N/mm²	
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 1626 kNm, ①	
			Su= 661 kN ②	
		Strength	Flexural Strength φMn= 3921 kNm	> Mu
			Shear Strenth φVn= 1242 kN	> Su
	Transverse Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 1697 kNm, ①	
			Su= 722 kN ②	
		Strength	Flexural Strength φMn= 6080 kNm	> Mu
			Shear Strenth φVn= 1262 kN	> Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 3701 kNm, ①	
		Stress	σc= 8.8 < σca= 20.4 N/mm²	
			σs= 358.6 < σsa= 378 N/mm²	
	Transverse Direction	Sectional Force	M= 5037 kNm, ①	
		Stress	σc= 9.0 < σca= 20.4 N/mm²	
			σs= 313.1 < σsa= 378 N/mm²	

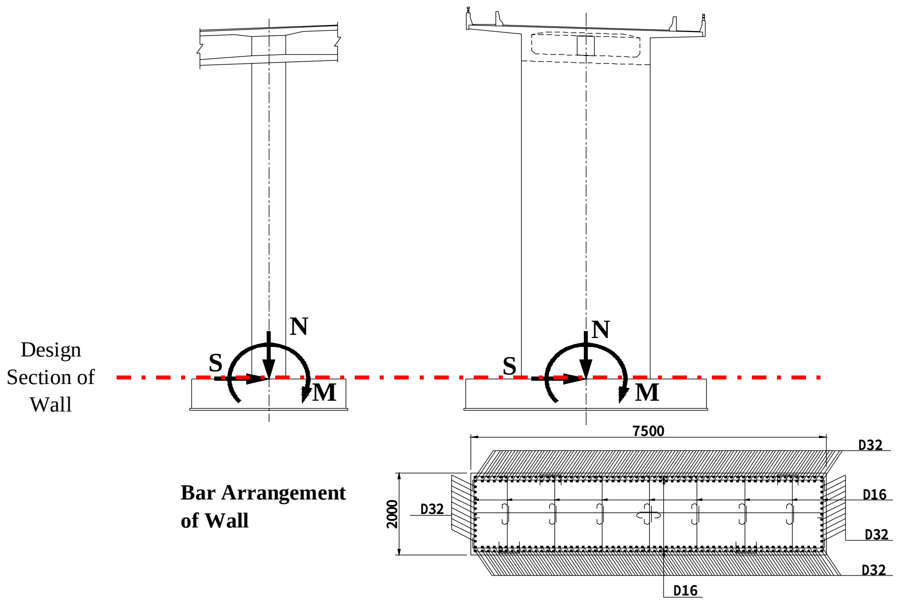
(Source : JICA Study Team)

Table 5.4.17 Design Summary for P7 Pier (1)

Pier P7

ımary of Design Calculation Result (1/2)

1. Wall



Size of Section			Longitudinal Direction LL=2.000m
			Transverse Direction LT=7.500m
Bar Arrangement			Longitudinal Direction D32ctc125-1.0, d=1900mm
			Transverse Direction D32ctc133-1.0, d=7400mm
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 9511 kNm, N= 19708 kN
		Stress	σc= 3.3 < σca= 9.6 N/mm^2 (24 x 0.4)
			σs= 7.1 < σsa= 168 N/mm^2
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 8691 kNm, N= 18925 kN
		Stress	σc= 1.4 < σca= 9.6 N/mm^2 (24 x 0.4)
			σs= -10.2 < σsa= 168 N/mm^2
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 11092 kNm, Nu= 26223 kN Su= 728 kN
		Strength	Flexural Strength φMn= 72158 kNm > Mu Shear Strenth φVn= 9752 kN > Su
	Transverse Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 11298 kNm, Nu= 25954 kN Su= 445 kN
		Strength	Flexural Strength φMn= 259494 kNm > Mu Shear Strenth φVn= 10265 kN > Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 66558 kNm, N= 22198 kN
		Stress	σc= 17.7 < σca= 20.4 N/mm^2 (24 x 0.85)
			σs= 352.3 < σsa= 378 N/mm^2 (420 x 0.9)
	Transverse Direction	Sectional Force	M= 70922 kNm, N= 20366 kN
		Stress	σc= 5.18 < σca= 20.4 N/mm^2
			σs= 71.5 < σsa= 378 N/mm^2

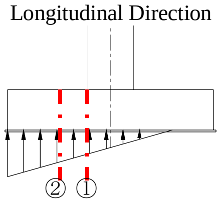
(Source : JICA Study Team)

Table 5.4.18 Design Summary for P7 Pier (2)

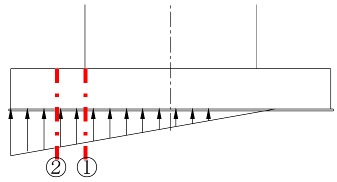
Summary of Design Calculation Result (2/2)

2. Spread Footing

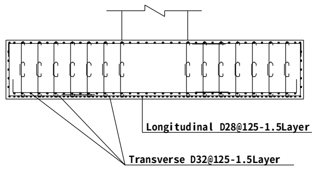
Longitudinal Direction



Transverse Direction



Bar Arrangement of Footing



Size of Section			Longitudinal Direction	L_L=10.000m T=2.000m
			Transverse Direction	L_T=10.000m T=2.000m
Bar Arrangement			Longitudinal Direction	D28ctc125-1.5, d=1890mm
			Transverse Direction	D32ctc125-1.5, d=1920mm
Stability	Force at the Center of Footing	Longitudinal Direction	Group-I	V=28161kN, H=689kN, M=10890kNm
			Seicmic Design	V=30160kN, H=8095kN, M=81035kNm
		Transverse Direction	Group-I	V=27378kN, H=342kN, M=9375kNm
			Seicmic Design	V=26330kN, H=6185kN, M=81585kNm
	Safety Factor for Horizontal Force	Longitudinal Direction	Group-I	Safety Factor=24.511 > 1.5 (Allowable Factor)
			Seicmic Design	Safety Factor=2.235 > 1.2 (Allowable Factor)
		Transverse Direction	Group-I	Safety Factor=48.029 > 1.5 (Allowable Factor)
			Seicmic Design	Safety Factor=2.554 > 1.2 (Allowable Factor)
	Ground Reaction	Longitudinal Direction	Group-I	qmax=347kN/m² < qa=700kN/m²
			Seicmic Design	$\boxed{\text{qmax=869kN/m²}}$ < qa=1050kN/m²
		Transverse Direction	Group-I	qmax=330kN/m² < qa=700kN/m²
			Seicmic Design	$\boxed{\text{qmax=923kN/m²}}$ < qa=1050kN/m²
Service Load Design	Longitudinal Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 1476 kNm, ①	
		Stress	σc= 3.5 < σca= 9.6 N/mm²	
			σs= 142.2 < σsa= 168 N/mm²	
	Transverse Direction	Load Case	Group-I : (D+SD+L+CF+E+B)	
		Sectional Force	M= 2077 kNm, ①	
		Stress	σc= 3.7 < σca= 9.6 N/mm²	
			σs= 129.1 < σsa= 168 N/mm²	
Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 1910 kNm, ①	
			Su= 776 kN ②	
		Strength	Flexural Strength	φMn= 4572 kNm > Mu
			Shear Strenth	φVn= 1242 kN > Su
	Transverse Direction	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)	
		Sectional Force	Mu= 2824 kNm, ①	
			Su= 1201 kN ②	
		Strength	Flexural Strength	φMn= 5993 kNm > Mu
			Shear Strenth	φVn= 1262 kN > Su
Seismic Design	Longitudinal Direction	Sectional Force	M= 3873 kNm, ①	
		Stress	σc= 9.2 < σca= 20.4 N/mm²	
			$\boxed{\text{σs= 373.0}}$ < σsa= 378 N/mm²	
	Transverse Direction	Sectional Force	M= 6011 kNm, ①	
		Stress	σc= 10.7 < σca= 20.4 N/mm²	
			$\boxed{\text{σs= 374.0}}$ < σsa= 378 N/mm²	

(Source : JICA Study Team)

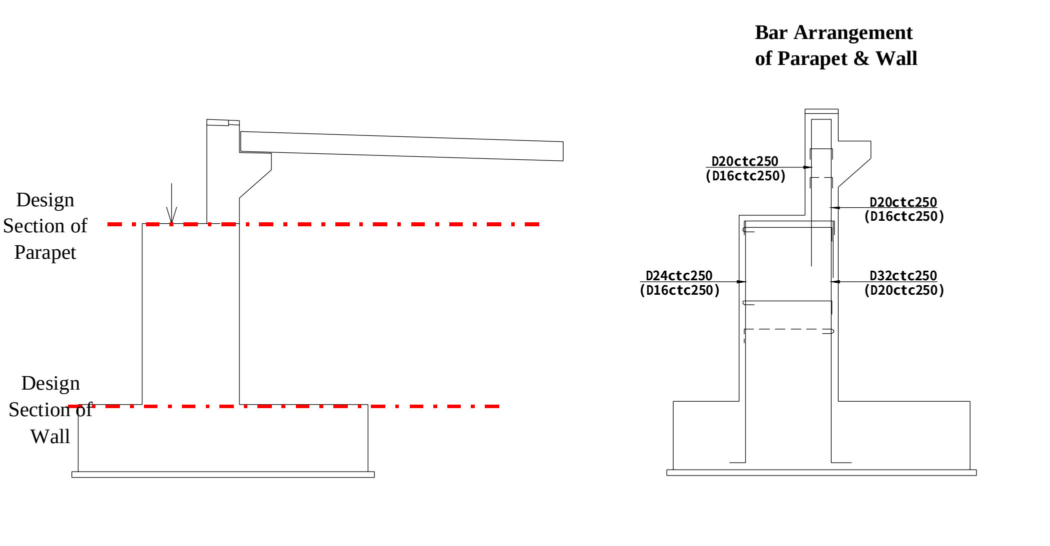
Abutment A2

Table 5.4.19 Design Summary of A2 Abutment (1/3)

Summary of Design Calculation Result (1/3)

Stability	Force at the Center of Footing	Group-I	V=34989kN, H=5510kN, M=7597kNm
		Group-VII	V=32436kN, H=12605kN, M=45403kNm
	Safety Factor for Horizontal Force	Group-I	Safety Factor=3.188 > 1.5 (Allowable Factor)
		Group-VII	Safety Factor=1.292 > 1.2 (Allowable Factor)
	Subgrade Reaction	Group-I	q_{max}=322kN/m^2 < q_a=700kN/m^2
		Group-VII	q_{max}=541kN/m^2 < q_a=1050kN/m^2

1. Wall and Parapet



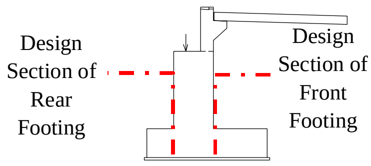
Size of Section			Thickness of Parapet t=0.700m
			Thickness of Wall t=2.200m
Bar Arrangement			Parapet D20ctc250-1.0, d=600mm
			Wall D32ctc250-1.0, d=2100mm
Wall	Service Load Design	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 892 kNm, N= 786 kN
		Stress	σ_c= 2.1 < σ_{ca}= 9.6 N/mm^2 (24 x 0.4)
			σ_s= 44.2 < σ_{sa}= 168 N/mm^2
		Load Case	Group-VII : (D+SD+E+B+EQ)
		Sectional Force	M= 1744 kNm, N= 735 kN
		Stress	σ_c= 4.5 < σ_{ca}= 12.8 N/mm^2 (9.6 x 1.33)
			$\boxed{\sigma_s= 175.7}$ < σ_{sa}= 223 N/mm^2 (168 x 1.33)
	Load Factor Design	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 1278 kNm, Nu= 1067 kN
		Strength	Flexural Strength φMn= 8641 kNm > Mu
		Load Case	Group - VII 1.3(D+SD+E+B+EQ)
		Sectional Force	Mu= 2349 kNm, Nu= 956 kN
		Strength	Flexural Strength φMn= 8627 kNm > Mu
Parapet	Service Load Design	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 78 kNm, N= 0 kN
		Stress	σ_c= 2.19 < σ_{ca}= 9.6 N/mm^2 (24 x 0.4)
			$\boxed{\sigma_s= 121.9}$ < σ_{sa}= 168 N/mm^2

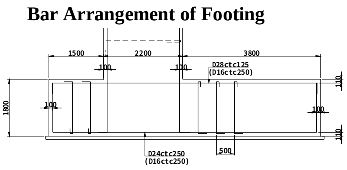
(Source : JICA Study Team)

Table 5.4.20 Design Summary of A2 Abutment (2/3)

Summary of Design Calculation Result (2/3)

2. Spread Footing





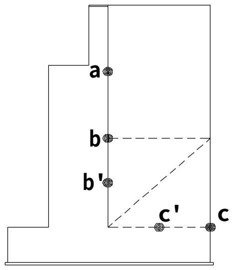
Size of Section			Longitudinal Direction LL=7.500m T=1.800m
			Transverse Direction LT=17.000m T=1.800m
Service Load Design	Front Footing	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 244 kNm, ①
		Stress	σc= 1.0 < σca= 9.6 N/mm^2
			σs= 62.4 < σsa= 168 N/mm^2
		Load Case	Group-VII : (D+SD+E+B+EQ)
		Sectional Force	M= 476 kNm, ①
		Stress	σc= 1.9 < σca= 12.8 N/mm^2
			σs= 122.0 < σsa= 223 N/mm^2
	Rear Footing	Load Case	Group-I : (D+SD+L+CF+E+B)
		Sectional Force	M= 399 kNm, ①
		Stress	σc= 1.2 < σca= 9.6 N/mm^2
			σs= 52.4 < σsa= 168 N/mm^2
		Load Case	Group-VII : (D+SD+E+B+EQ)
		Sectional Force	M= 1510 kNm, ①
		Stress	σc= 4.5 < σca= 12.8 N/mm^2
			σs= 198.2 < σsa= 223 N/mm^2
Load Factor Design	Front Footing	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 341 kNm, ①
			Su= kN ②
		Strength	Flexural Strength φMn= 1474 kNm > Mu
			Shear Strenth φVn= 542 kN > Su
		Load Case	Group - VII 1.3(D+SD+E+B+EQ)
		Sectional Force	Mu= 702 kNm, ①
			Su= kN ②
		Strength	Flexural Strength φMn= 1474 kNm > Mu
			Shear Strenth φVn= 542 kN > Su
	Rear Footing	Load Case	Group - I 1.3(D+SD+1.67L+CF+E+B)
		Sectional Force	Mu= 1749 kNm, ①
			Su= 450.4 kN ②
		Strength	Flexural Strength φMn= 2668 kNm > Mu
			Shear Strenth φVn= 659 kN > Su
		Load Case	Group - VII 1.3(D+SD+E+B+EQ)
		Sectional Force	Mu= 2040 kNm, ①
			Su= 320.5 kN ②
		Strength	Flexural Strength φMn= 2668 kNm > Mu
			Shear Strenth φVn= 659 kN > Su

(Source : JICA Study Team)

Table 5.4.21 Design Summary of A2 Abutment (3/3)

Summary of Design Calculation Result (3/3)

3. Wing



Bar Arranegement

Section	Reinforcement-bar Arranngement	
	Inside (mm^2)	Outside (mm2)
a	D28-4Nos.=2463.2	D20-4Nos.=1256.8
b	D32-4Nos.=3216.8	D20-4Nos.=1256.8
c	D28-8Nos.=4926.4	D28-4Nos.=2463.2
c'	D28-4Nos.=2463.2	D20-4Nos.=1256.8

Size of Section			Thickness of Wing t=0.700m
Service Load Design	Section-"a"	Load Case	Group-VI : (D+L+E)
		Sectional Force	M= 134.4 kNm,
		Stress	σc= 2.7 < σca= 9.6 N/mm^2
			σs= 101.1 < σsa= 168 N/mm^2
	Section-"b"	Load Case	Group-VI : (D+L+E)
		Sectional Force	M= 247.0 kNm,
		Stress	σc= 3.5 < σca= 9.6 N/mm^2
			σs= 143.9 < σsa= 168 N/mm^2
	Section-"c"	Load Case	Group-VI : (D+L+E)
		Sectional Force	M= 299.7 kNm,
		Stress	σc= 5.0 < σca= 9.6 N/mm^2
			σs= 123.7 < σsa= 168 N/mm^2
	Section-"c'"	Load Case	Group-VI : (D+L+E)
		Sectional Force	M= 88.1 kNm,
		Stress	σc= 1.9 < σca= 9.6 N/mm^2
			σs= 70.4 < σsa= 168 N/mm^2
Load Factor Design	Section-"a"	Load Case	Group-VII : 1.3(D+E+EQ)
		Sectional Force	Mu= 231.0 kNm,
		Strength	Flexural Strength φMn= 1134 kNm > Mu
	Section-"b"	Load Case	Group-VII : 1.3(D+E+EQ)
		Sectional Force	Mu= 429.7 kNm,
		Strength	Flexural Strength φMn= 444 kNm > Mu
	Section-"c"	Load Case	Group-VII : 1.3(D+E+EQ)
		Sectional Force	Mu= 523.0 kNm,
		Strength	Flexural Strength φMn= 541 kNm > Mu
	Section-"c'"	Load Case	Group-VII : 1.3(D+E+EQ)
		Sectional Force	Mu= 154.1 kNm,
		Strength	Flexural Strength φMn= 373 kNm > Mu

(Source : JICA Study Team)

5.5 Minor Bridge and Culvert Design

5.5.1 Design of Minor Bridges

(1) General

The Project road crosses four major streams (S3, S9, S12 and S14 as shown in **Table 5.5.4**). The estimated discharge except that of S14 is over $40m^3/s$ for 50-year return period. A river crossing structure should be constructed to provide enough capacity/opening to accommodate run off discharge at each site. Specifically at S12 however, 3-cell box culvert is planned due to limited available vertical clearance between proposed road height and existing ground level. Nevertheless, this will be designed to accommodate required freeboard and high water levels. Finally, the following three locations are decided to plan bridge structures.

Table 5.5.1 Culverts Dimension according to Discharge

No.*	Station No.	Structure	Discharge Estimated $Q_{50}(m^3/s)$	Remarks
S3	1+745	Bridge	46.7	Bridge No.1
S9	3+515	Bridge	45.1	Bridge No.2
S12	4+100	Box culvert	57.4	
S14	4+730	Bridge	197.4	Bridge No.3

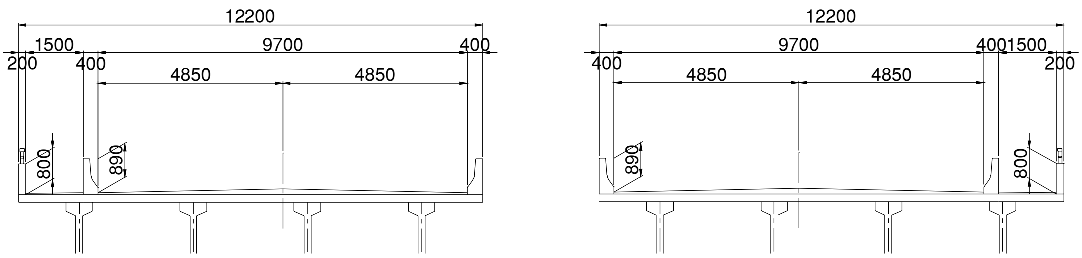
Note: See number of stream point in conjunction with* ***table 5.5.4*
Source: JICA Study Team

(2) Applicable Design Standard

The applicable standard for these bridges are similar to that mentioned in section 5.1 for Naluchi Bridge. The other specific criteria applicable solely to these minor bridges are mentioned in the following sections.

(3) Cross section

The specification of cross section on the bridge deck is in accordance with "Standardization of Bridge Superstructure (January, 2005)" published by NHA. This standard recommends that a walkway should be provided in populated areas. The standard cross section for the 2-lane carriageway with walkway is shown in **Figure 5.5.1**.



(Bridge No.1 and2) (Bridge No.3)

Source: Standardization of Bridge Superstructures, NHA (January, 2005)

Figure 5.5.1 Standard Bridge Cross Section for 2 Lane Carriageway

(4) Determination of Span Length and Location of Abutment

The location of the abutment was decided from the following considerations:

- In order to secure safe capacity of the run off for the estimated discharge, the position of abutment should be placed at locations where it should ensure more than the standard span length, considering desirable clear span length derived from the following equation in River Structural Guideline in Japan.

  Standard Span Length = 20+0.005Q (m)

- The bridge opening should avoid not cause an obstacle to stream flow, considering the existing topographical conditions.

- At locations of Bridge No.1 and No.2, stream flow could be obstructed. In this regard, vertical clearance of 4.3 m should be considered.

- At location of Bridge No.3, risk for debris flow is possible. From this reason, a revetment structures on both riverbank and riverbed are necessary to provide protection against erosion and scouring. Additionally, the opening requires 150% of original estimated discharge in consideration of debris.

(5) Bridge Type Selection

The bridge types to be designed should meet the following conditions:

- The relationship between bridge type and applicable span length is considered in addition to consideration of reduced construction time; ease of construction and life cycle cost etc.

- Cast-in-situ type construction is advisable in consideration of restriction for transportation of heavy equipment into Muzaffarabad City.

- Due to the difficulty of transportation of equipment, the application of different structure types will not be economical. Standardization of structural type for all the three bridge locations should be considered.
- Local contractors' experience in AJK region is a significant consideration

From the above considerations, the PC-I Girder type is finally recommended as the most viable economical alternative. The length of span should be standardized to 30 m (Girder length: 29.9 m) for all bridge locations for practicality of construction. Moreover, have experience in constructing PC-I girders in Muzaffarabad. The PC girder section is designed based on relevant provisions of Standardization of Bridge Superstructures, NHA, January, 2005 and Designing of Bridge on National Highways (DBNH).

(6) Seismic Factor

Seismic design method is the same as that of East Side Approach Viaduct.

(7) General View of Bridge

The typical cross section for Bridge No.1, 2 & 3 are shown in **Figure 5.5.1**.

(8) Design Result

The main calculation results of superstructure and substructure for Bridge No.1, 2 & 3 are shown in the following tables.

For some members, the determined stress is far less than the allowable stress due to applying the minimum amount of steel, and for members having large section and requiring small reinforcing bar, steel are arranged for RC to ensure a tri-linear relationship between M and Φ.

Table 5.5.2 Calculation Result of Superstructure

Bridge No.1 and 2		Unit	G1				G2			
			at x=H/2 (102)		at Middle (107)		at x=H/2 (102)		at Middle (107)	
			Top	Botto	Top	Botto	Top	Botto	Top	Botto
Bending Moment	Dead Load	kNm	609.5		4,674.1		623.4		4,858.9	
	Live Load - Max	kNm	533.9		3,299.2		461.8		2,592.3	
	Live Load - Min	kNm	-216.8		-116.8		-91.6		0.0	
	Total - Max	kNm	1,143.3		7,973.2		1,085.2		7,451.2	
	Total - Min	kNm	392.7		4,557.3		531.9		4,858.9	
Stress in Girder	Transfer	N/mm^2	4.30	8.98	-0.20	16.12	4.30	8.98	-0.20	16.12
	- Allowable	N/mm^2	-3.1<<24.0				-3.1<<24.0			
	Dead Load	N/mm^2	3.43	6.52	4.36	5.06	3.45	6.49	4.55	4.72
	- Allowable	N/mm^2	0.0<<16.0				0.0<<16.0			
	Service Load - Max	N/mm^2	3.83	5.46	6.75	-1.47	3.81	5.57	6.48	-0.43
	Service Load - Min	N/mm^2	3.27	6.95	4.27	5.30	3.38	6.67	4.55	4.72
	- Allowable	N/mm^2	-3.1<<16.0				-3.1<<16.0			
Stress in PC Tendon	Service Load	N/mm^2	834.6		931.5		834.2		927.9	
	- Allowable	N/mm^2	1,110.0				1,110.0			
Flexural Capacity	Moment	kNm	2,127.0		14,324.2		1,694.9		12,797.4	
	Flexural Resistance	kNm	11,106.6		16,344.5		11,092.1		16,330.0	
Shear Stress	Mean Shear	N/mm^2	1.64		0.66		1.40		0.61	
	- Allowable	N/mm^2	2.50				2.50			
	Diagonal Tensile	N/mm^2	-0.84		-0.20		-0.63		-0.16	
	- Allowable	N/mm^2	-2.00				-2.00			
Shear Capacity	Shearing Force	kN	2,079.7		-597.3		1,848.6		-583.3	
	Shearing Resistance	kN	3,038.8		2,048.7		3,038.8		2,048.7	

Bridge No.3		Unit	G1				G2			
			at x=H/2 (102)		at Middle (107)		at x=H/2 (102)		at Middle (107)	
			Top	Botto	Top	Botto	Top	Botto	Top	Botto
Bending Moment	Dead Load	kNm	626.5		4,901.8		617.7		4,718.1	
	Live Load - Max	kNm	554.9		3,530.8		479.5		2,537.9	
	Live Load - Min	kNm	-243.8		-115.2		-88.3		0.0	
	Total - Max	kNm	1,181.4		8,432.6		1,097.1		7,256.1	
	Total - Min	kNm	382.7		4,786.6		529.4		4,718.1	
Stress in Girder	Transfer	N/mm^2	4.66	8.64	-0.26	16.18	4.66	8.64	-0.24	16.14
	- Allowable	N/mm^2	-3.1<<24.0				-3.1<<24.0			
	Dead Load	N/mm^2	3.66	6.25	4.39	4.73	3.68	6.26	4.47	4.92
	- Allowable	N/mm^2	0.0<<16.0				0.0<<16.0			
	Service Load - Max	N/mm^2	4.07	5.16	6.70	-2.17	4.07	5.30	6.46	-0.16
	Service Load - Min	N/mm^2	3.48	6.73	4.31	4.95	3.61	6.44	4.47	4.92
	- Allowable	N/mm^2	-3.1<<16.0				-3.1<<16.0			
Stress in PC Tendon	Service Load	N/mm^2	837.8		940.4		837.1		921.5	
	- Allowable	N/mm^2	1,110.0				1,110.0			
Flexural Capacity	Moment	kNm	2,201.7		15,199.4		2,001.6		12,478.4	
	Flexural Resistance	kNm	10,909.6		16,395.4		10,855.2		16,297.8	
Shear Stress	Mean Shear	N/mm^2	1.82		0.82		1.29		0.92	
	- Allowable	N/mm^2	2.50				2.50			
	Diagonal Tensile	N/mm^2	-0.98		-0.22		-0.53		-0.33	
	- Allowable	N/mm^2	-2.00				-2.00			
Shear Capacity	Shearing Force	kN	2,262.1		-732.8		1,748.0		-765.3	
	Shearing Resistance	kN	3,039.9		2,048.7		3,039.9		2,048.7	

Source: JICA Study Team

Table 5.5.3 Calculation Result of Bridge No.1 Substructure

					A1 Abutment		A2 Abutment	
					G-I	G-VII	G-I	G-VII
WALL	Stability	Force at the Center of Footing		V kN	23,297	20,818	21404	18986
				H (kN)	4,097	9,144	3424	8587
				M (kN・m)	20,818	34,153	3244	29269
		Safety Factor for Horizontal			3.415 > 1.5	1.412 > 1.2	3.751 > 1.5	1.327 > 1.2
		Ground Reaction		qmax(kN/m²)	397 < 600	707 < 900	283 < 700	530 < 1050
	Size of Section		Parapet	Thickness (m)	0.5		0.5	
			Wall	Thickness (m)	1.5		1.5	
	Bar Arrangement		Parapet		D20ctc250-1.0, d=400mm		D20ctc250-1.0, d=500mm	
			Wall		D32ctc125-1.0, d=1400mm		D32ctc125-1.0, d=1400mm	
	Parapet	Service Load Design	Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force		M= 68.8 kNm, N= 0 kN		M= 69 kNm, N= 0 kN	
			Stress	σc (kN/m2)	3.7 < 9.6		3.7 < 9.6	
				σs (kN/m2)	149.0 < 168		128.0 < 168	
	Wall	Service Load Design	Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force	M (kN・m)	1021		748	
				N (kN)	446		611	
			Stress	σc (kN/m2)	3.9 < 9.6		2.3 < 9.6	
				σs (kN/m2)	80.5 < 168		50.9 < 168	
			Load Case		*Group-VII : (D+SD+E+B+EQ)*		Group-VII : (D+SD+E+B+EQ)	
			Sectional Force	M (kN・m)	1707		1660	
				N (kN)	390		735	
			Stress	σc (kN/m2)	6.4 < 12.8		4.9 < 12.8	
				σs (kN/m2)	169.0 < 223		166.2 < 223	
		Load Factor Design	Load Case		Group - I 1.3(D+SD+1.67L+CF+E+B)		Group - I 1.3(D+SD+1.67L+CF+E+B)	
			Sectional Force	M (kN・m)	1451		1073	
				N (kN)	934		886	
			Strength	Mn (kN・m)	3517 > Mu		2476 > Mu	
			Load Case		Group - VII 1.3(D+SD+E+B+EQ)		Group - VII 1.3(D+SD+E+B+EQ)	
			Sectional Force	M (kN・m)	2791		2159	
				N (kN)	635		661	
			Strength	Mn (kN・m)	3517 > Mu		2476 > Mu	
FOOTING	Size of Section		Longtudinal		LL=6.500m	T=1.500m	LL=7.000m	T=1.500m
			Transverse		LT=12.200m	T=1.500m	LT=12.200m	T=1.500m
	Service Load Design	Front Footing	Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force	M (kN・m)	92		252	
			Stress	σc (kN/m2)	1.0 < 9.6		1.7 < 9.6	
				σs (kN/m2)	106.7 < 168		106.7 < 168	
			Load Case		Group-VII : (D+SD+E+B+EQ)		Group-VII : (D+SD+E+B+EQ)	
			Sectional Force	M (kN・m)	160		484	
			Stress	σc (kN/m2)	3.3 < 12.8		3.2 < 12.8	
				σs (kN/m2)	214.0 < 223		205.0 < 223	
		Rear Footing	Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force	M (kN・m)	473		285	
			Stress	σc (kN/m2)	1.8 < 9.6		1.1 < 9.6	
				σs (kN/m2)	59.7 < 168		35.9 < 168	
			Load Case		Group-VII : (D+SD+E+B+EQ)		Group-VII : (D+SD+E+B+EQ)	
			Sectional Force	M (kN・m)	1504		1543	
			Stress	σc (kN/m2)	5.6 < 12.8		5.8 < 12.8	
				σs (kN/m2)	189.8 < 223		194.7 < 223	
	Load Factor Design	Front Footing	Load Case		Group - I 1.3(D+SD+1.67L+CF+E+B)		Group - I 1.3(D+SD+1.67L+CF+E+B)	
			Sectional Force	M (kN・m)	436		359	
				Su (kN)	293.8		240.9	
			Strength	φMn (kNm)	998 > Mu		890 > Mu	
				φVn (kN)	1133 > Su		542 > Su	
			Load Case		Group - VII 1.3(D+SD+E+B+EQ)		Group - VII 1.3(D+SD+E+B+EQ)	
			Sectional Force	M (kN・m)	865		622	
				Su (kN)	598.9		425.9	
			Strength	φMn (kNm)	998 > Mu		890 > Mu	
				φVn (kN)	1133 > Su		1132 > Su	
		Rear Footing	Load Case		Group - I 1.3(D+SD+1.67L+CF+E+B)		Group - I 1.3(D+SD+1.67L+CF+E+B)	
			Sectional Force	M (kN・m)	602		425	
				Su (kN)	226.6		126.4	
			Strength	φMn (kNm)	2446 > Mu		2668 > Mu	
				φVn (kN)	> Su		659 > Su	
			Load Case		Group - VII 1.3(D+SD+E+B+EQ)		Group - VII 1.3(D+SD+E+B+EQ)	
			Sectional Force	M (kN・m)	2093		2018	
				Su (kN)	932.8		727.6	
			Strength	φMn (kNm)	2446 > Mu		2994 > Mu	
				φVn (kN)	1193 > Su		1133 > Su	

Source: JICA Study Team

Table 5.5.4 Calculation Result of Bridge No.2 Substructure

					A1 Abutment		A2 Abutment	
					G-I	G-VII	G-I	G-VII
WALL	Stability		Force at the Center of Footing	V kN	18,349	16,077	181924	17854
				H (kN)	3,264	7,197	2392	6995
				M (kN･m)	4,173	20,910	3244	20545
			Safety Factor for Horizontal		3.415 > 1.5	1.412 > 1.2	4.748 > 1.5	1.532 > 1.2
			Ground Reaction	qmax(kN/m²)	397 < 600	707 < 900	207 < 700	365 < 900
	Size of Section		Parapet	Thickness (m)	0.5		0.5	
			Wall	Thickness (m)	1.7		1.7	
	Bar Arrangement		Parapet		D20ctc250-1.0, d=400mm		D20ctc250-1.0, d=400mm	
			Wall		D20ctc125-1.0, d=1600mm		D20ctc125-1.0, d=1600mm	
	Parapet	Service Load Design	Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force		M= 70.2 kNm, N= 0 kN		M= 69 kNm, N= 0 kN	
			Stress	σc (kN/m2)	3.7 < 9.6		3.4 < 9.6	
				σs (kN/m2)	152.0 < 168		149.5 < 168	
	Wall	Service Load Design	Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force	M (kN･m)	665		496	
				N (kN)	616		597	
			Stress	σc (kN/m2)	1.6 < 9.6		1.8 < 9.6	
				σs (kN/m2)	43.2 < 168		31.1 < 168	
			Load Case		*Group-VII : (D+SD+E+B+EQ)*		Group-VII : (D+SD+E+B+EQ)	
			Sectional Force	M (kN･m)	1176		1660	
				N (kN)	514		735	
			Stress	σc (kN/m2)	3 < 12.8		5.8 < 12.8	
				σs (kN/m2)	132.0 < 223		188.0 < 223	
		Load Factor Design	Load Case		Group - I 1.3(D+SD+1.67L+CF+E+B)		Group - I 1.3(D+SD+1.67L+CF+E+B)	
			Sectional Force	M (kN･m)	957		719	
				N (kN)	668		793	
			Strength	Mn (kN･m)	2146 > Mu		1716 > Mu	
			Load Case		Group - VII 1.3(D+SD+E+B+EQ)		Group - VII 1.3(D+SD+E+B+EQ)	
			Sectional Force	M (kN･m)	1970		1514	
				N (kN)	635		568	
			Strength	Mn (kN･m)	2146 > Mu		1716 > Mu	
FOOTING	Size of Section		Longtudinal		LL=6.000m T=1.500m		LL=6.000m T=1.500m	
			Transverse		LT=12.200m T=1.500m		LT=12.200m T=1.500m	
	Service Load Design	Front Footing	Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force	M (kN･m)	178		189	
			Stress	σc (kN/m2)	1.0 < 9.6		1.6 < 9.6	
				σs (kN/m2)	106.7 < 168		108.8 < 168	
			Load Case		Group-VII : (D+SD+E+B+EQ)		Group-VII : (D+SD+E+B+EQ)	
			Sectional Force	M (kN･m)	305		327	
			Stress	σc (kN/m2)	3.3 < 12.8		2.8 < 12.8	
				σs (kN/m2)	214.0 < 223		188.9 < 223	
		Rear Footing	Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force	M (kN･m)	473		59	
			Stress	σc (kN/m2)	1.8 < 9.6		0.3 < 9.6	
				σs (kN/m2)	59.7 < 168		11.3 < 168	
			Load Case		Group-VII : (D+SD+E+B+EQ)		Group-VII : (D+SD+E+B+EQ)	
			Sectional Force	M (kN･m)	1504		1048	
			Stress	σc (kN/m2)	5.6 < 12.8		5.7 < 12.8	
				σs (kN/m2)	189.8 < 223		198.6 < 223	
	Load Factor Design	Front Footing	Load Case		Group - I 1.3(D+SD+1.67L+CF+E+B)		Group - I 1.3(D+SD+1.67L+CF+E+B)	
			Sectional Force	M (kN･m)	261		258	
				Su (kN)	201.8		194.7	
			Strength	φMn (kNm)	654 > Mu		654 > Mu	
				φVn (kN)	979 > Su		979 > Su	
			Load Case		Group - VII 1.3(D+SD+E+B+EQ)		Group - VII 1.3(D+SD+E+B+EQ)	
			Sectional Force	M (kN･m)	487		397	
				Su (kN)	385.6		303.8	
			Strength	φMn (kNm)	654 > Mu		654 > Mu	
				φVn (kN)	1133 > Su		979 > Su	
		Rear Footing	Load Case		Group - I 1.3(D+SD+1.67L+CF+E+B)		Group - I 1.3(D+SD+1.67L+CF+E+B)	
			Sectional Force	M (kN･m)	365		120	
				Su (kN)	149.8		20.8	
			Strength	φMn (kNm)	1684 > Mu		1529 > Mu	
				φVn (kN)	979 > Su		979 > Su	
			Load Case		Group - VII 1.3(D+SD+E+B+EQ)		Group - VII 1.3(D+SD+E+B+EQ)	
			Sectional Force	M (kN･m)	1397		1347	
				Su (kN)	695.6		438.8	
			Strength	φMn (kNm)	1684 > Mu		1529 > Mu	
				φVn (kN)	979 > Su		979 > Su	

Source: JICA Study Team

Table 5.5.5 Calculation Result of Bridge No.3 Substructure(A1&A2) (1/2)

					A1 Abutment		A2 Abutment	
PILE	Pile Arrangement			Dia. D	1200mm		1200mm	
				L	8m		9.5m	
				Nos.	8nos.		8nos.	
	Bar Arrangement				24-D24 (@147)		24-D24 (@147)	
	Stability				G-I	G-VII	G-I	G-VII
		Force at the Center of Footing		V kN	12,846	11,942	16,506	14,409
				H (kN)	1,550	4,827	2,587	5,970
				M (kN･m)	2,169	13,384	1,142	13,444
		Safety Factor for Horizontal Force	Longitudinal	Rmax kN)	1385 < 2138	2379 < 3317	2336 < 2377	2775 < 3615
				Rmin (kN)	1377 > 0	606 > -1284	1791 > 0	828 > -1521
		Displacement at Pile Head		longitudinal	1.8 < 15	4.3 < 15	2.6 < 15	5.0 < 15
	Service Load Design		Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force	M (kN･m)	233		392	
				N (kN)	1377		1647	
			Stress	σc (N/mm2)	2.2 < 9.6		3.3 < 9.6	
				σs (N/mm2)	-24.9 < 168		-42.0 < 168	
			Load Case		Group-VII : (D+SD+E+B+EQ)		Group-VII : (D+SD+E+B+EQ)	
			Sectional Force	M (kN･m)	864		914	
				N (kN)	607		828	
			Stress	σc (N/mm2)	8.1 < 12.8		8.5 < 12.8	
				σs (N/mm2)	178.3 < 223		172.0 < 223	
	Load Factor Design		Load Case		Group-I : 1.3(D+SD+1.67L+CF+E+B)		Group-I : 1.3(D+SD+1.67L+CF+E+B)	
			Sectional Force	Mu (kN･m)	333		557	
				Nu (kN)	1806		2255	
				Su (kN)	276		452	
			Strength	φMn (kNm)	1879 > Mu		1493 > Mu	
				φVn (kN)	1744 > Su		1744 > Su	
			Load Case		Group-VII : 1.3(D+SD+E+B+EQ)		Group-VII : 1.3(D+SD+E+B+EQ)	
			Sectional Force	Mu (kN･m)	1062		1117	
				Nu (kN)	673		932	
				Su (kN)	749		927	
			Strength	φMn (kNm)	1493 > Mu		1583 > Mu	
				φVn (kN)	1744 > Su		1744 > Su	
WALL	Size of Section		Parapet	Thickness (m)	0.5		0.5	
			Wall	Thickness (m)	1.7		1.7	
	Bar Arrangement		Parapet		D20ctc250-1.0, d=400mm		D20ctc250-1.0, d=400mm	
			Wall		D20ctc125-1.0, d=1600mm		D20ctc125-1.0, d=1600mm	
	Parapet	Service Load Design	Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force		M= 70.2 kNm, N= 0 kN		M= 70.2 kNm, N= 0 kN	
			Stress	σc (kN/m2)	3.4 < 9.6		3.4 < 9.6	
				σs (kN/m2)	151.2 < 168		152.6 < 168	
	Wall	Service Load Design	Load Case		Group-I : (D+SD+L+CF+E+B)		Group-I : (D+SD+L+CF+E+B)	
			Sectional Force	M (kN･m)	193		352	
				N (kN)	507		524	
			Stress	σc (kN/m2)	0.7 < 9.6		1.3 < 9.6	
				σs (kN/m2)	1.2 < 168		17.1 < 168	
			Load Case		*Group-VII : (D+SD+E+B+EQ)*		*Group-VII : (D+SD+E+B+EQ)*	
			Sectional Force	M (kN･m)	578		578	
				N (kN)	406		406	
			Stress	σc (kN/m2)	2.8 < 12.8		2.8 < 12.8	
				σs (kN/m2)	123.4 < 223		103.0 < 223	
		Load Factor Design	Load Case		Group - I 1.3(D+SD+1.67L+CF+E+B)		Group - I 1.3(D+SD+1.67L+CF+E+B)	
			Sectional Force	M (kN･m)	288		517	
				N (kN)	699		702	
			Strength	Mn (kN･m)	928 > Mu		1811 > Mu	
			Load Case		Group - VII 1.3(D+SD+E+B+EQ)		Group - VII 1.3(D+SD+E+B+EQ)	
			Sectional Force	M (kN･m)	730		806	
				N (kN)	479		498	
			Strength	Mn (kN･m)	928 > Mu		1753 > Mu	

Note: The determined stress is far less than the allowable stress due to applying the minimum amount of steel material.
Source: JICA Study Team

Table 5.5.6 Calculation Result of Bridge No.3 Substructure(A1&A2) (2/2)

FOOTING	Service Load Design	Front Footing	Load Case		Group-I : (D+SD+L+CF+E+B)	Group-I : (D+SD+L+CF+E+B)
			Sectional Force	M (kN・m)	51	28.7
			Stress	σc (kN/m2)	0.5 < 9.6	0.3 < 9.6
				σs (kN/m2)	48.6 < 168	27.6 < 168
			Load Case		Group-VII : (D+SD+E+B+EQ)	Group-VII : (D+SD+E+B+EQ)
			Sectional Force	M (kN・m)	62	30.5
			Stress	σc (kN/m2)	0.5 < 12.8	0.3 < 12.8
				σs (kN/m2)	58.6 < 223	31.6 < 223
		Rear Footing	Load Case		Group-I : (D+SD+L+CF+E+B)	Group-I : (D+SD+L+CF+E+B)
			Sectional Force	M (kN・m)	38	81.6
			Stress	σc (kN/m2)	0.3 < 9.6	0.6 < 9.6
				σs (kN/m2)	35.5 < 168	53.8 < 168
			Load Case		Group-VII : (D+SD+E+B+EQ)	Group-VII : (D+SD+E+B+EQ)
			Sectional Force	M (kN・m)	255	310.2
			Stress	σc (kN/m2)	1.9 < 12.8	2.1 < 12.8
				σs (kN/m2)	167.9 < 223	186.6 < 223
	Load Factor Design	Front Footing	Load Case		Group - I 1.3(D+SD+1.67L+CF+E+B)	Group - I 1.3(D+SD+1.67L+CF+E+B)
			Sectional	Mu (kN・m)	70	43.2
			Strength	φMn (kNm)	460 > Mu	400 > Mu
			Load Case		Group - VII 1.3(D+SD+E+B+EQ)	Group - VII 1.3(D+SD+E+B+EQ)
			d	M (kN・m)	70	41.9
			Strength	φMn (kNm)	460 > Mu	400 > Mu
		Rear Footing	Load Case		Group - I 1.3(D+SD+1.67L+CF+E+B)	Group - I 1.3(D+SD+1.67L+CF+E+B)
			Sectional Force	Mu (kN・m)	34	137.1
				Su (kN)	316.3	381.9
			Strength	φMn (kNm)	727 > Mu	633 > Mu
				φVn (kN)	1260 > Su	1098 > Su
			Load Case		Group - VII 1.3(D+SD+E+B+EQ)	Group - VII 1.3(D+SD+E+B+EQ)
			Sectional Force	M (kN・m)	345	417.9
				Su (kN)	245.3	311.5
			Strength	φMn (kNm)	727 > Mu	633 > Mu
				φVn (kN)	1260 > Su	1098 > Su

Note: The determined stress is far less than the allowable stress due to applying the minimum amount of steel material.

Source: JICA Study Team

Table 5.5.7 Calculation Result of Bridge No.3 Substructure(P1) (1/2)

PILE	Service Load Design	Longitudinal Direction	Load Case	Group-I	(D+SD+L+B)		
			Sectional Force	M= 0 kNm,	N= 2112 kN		
			Stress	σc= 1.5 <	σca= 9.6 N/mm^2	(24 x 0.4)	
				σs= -22.4 <	σsa= 168 N/mm^2		
			Load Case	Group-VII	(D+SD+B+EQ)		
			Sectional Force	M= 1240 kNm,	N= 99 kN		
			Stress	σc= 9.4 <	σca= 12.8 N/mm^2	(9.6 x 1.33)	
				σs= 220.2 <	σsa= 223 N/mm^2	(168 x 1.33)	
		Transverse Direction	Load Case	Group-VII	(D+SD+B+EQ)		
			Sectional Force	M= 755 kNm,	N= -130 kN		
			Stress	σc= 5.7 <	σca= 12.8 N/mm^2	(9.6 x 1.33)	
				σs= 145.5 <	σsa= 223 N/mm^2	(168 x 1.33)	
	Load Factor Design	Longitudinal Direction	Load Case	Group-I	1.3(D+SD+1.67L+B)		
			Sectional Force	Mu= 0 kNm,	Nu= 2991 kN		
				Su= 0 kN			
			Strength	Flexural Strength	φMn= 2697 kNm	> Mu	
				Shear Strenth	φVn= 1744 kN	> Su	
			Load Case	Group - VII	1.3(D+SD+B+EQ)		
			Sectional Force	Mu= 1820 kNm,	Nu= 449 kN		
				Su= 612 kN			
			Strength	Flexural Strength	φMn= 2697 kNm	> Mu	
				Shear Strenth	φVn= 1744 kN	> Su	
		Transverse Direction	Load Case	Group - VII	1.3(D+SD+B+EQ)		
			Sectional Force	Mu= 853 kNm,	Nu= 74 kN		
				Su= 845 kN			
			Strength	Flexural Strength	φMn= 2697 kNm	> Mu	
				Shear Strenth	φVn= 1744 kN	> Su	

Source: JICA Study Team

Table 5.5.8 Calculation Result of Bridge No.3 Substructure(P1) (2/2)

WALL	Service Load Design	Longitudinal Direction	Load Case	Group-I (D+SD+L+B)
			Sectional Force	M= 0 kNm, N= 12744 kN
			Stress	σc= 0.4 < σca= 9.6 N/mm^2
				σs= -6.2 < σsa= 168 N/mm^2
			Load Case	Group-VII (D+SD+EQ)
			Sectional Force	M= 19134 kNm, N= 10494 kN
			Stress	σc= 2.8 < σca= 12.8 N/mm^2
				$\boxed{\sigma s= 71.5}$ < σsa= 223 N/mm^2
		Transverse Direction	Load Case	Group-VII (D+SD+EQ)
			Sectional Force	M= 26371 kNm, N= 10494 kN
			Stress	σc= 0.8 < σca= 12.8 N/mm^2
				σs= 2.1 < σsa= 223 N/mm^2
	Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+B)
			Sectional Force	Mu= 0 kNm, Nu= 18532 kN Su= 0 kN
			Strength	Flexural Strength φMn= 66698 kNm > Mu Shear Strenth φVn= 21971 kN > Su
			Load Case	Group - VII 1.3(D+SD+EQ)
			Sectional Force	Mu= 24822 kNm, Nu= 13642 kN Su= 3924 kN
			Strength	Flexural Strength φMn= 66698 kNm > Mu Shear Strenth φVn= 21971 kN > Su
		Transverse Direction	Load Case	Group - VII 1.3(D+SD+EQ)
			Sectional Force	Mu= 34274 kNm, Nu= 13642 kN Su= 5417 kN
			Strength	Flexural Strength φMn= 218638 kNm > Mu Shear Strenth φVn= 20325 kN > Su
PILE CAP	Service Load Design	Longitudinal Direction	Load Case	Group-I (D+SD+L+B)
			Sectional Force	M= 85 kNm, ①
			Stress	σc= 0.8 < σca= 9.6 N/mm^2
				σs= 82.0 < σsa= 168 N/mm^2
			Load Case	Group-VII (D+SD+B+EQ)
			Sectional Force	M= 185 kNm, ①
			Stress	σc= 1.7 < σca= 12.8 N/mm^2
				$\boxed{\sigma s= 180.0}$ < σsa= 223 N/mm^2
		Transverse Direction	Load Case	Group-VII (D+SD+B+EQ)
			Sectional Force	M= 29 kNm, ①
			Stress	σc= 0.3 < σca= 12.8 N/mm^2
				σs= 27.6 < σsa= 223 N/mm^2
	Load Factor Design	Longitudinal Direction	Load Case	Group - I 1.3(D+SD+1.67L+B)
			Sectional Force	Mu= 129 kNm, ① Su= 68 kN ②
			Strength	Flexural Strength φMn= 400 kNm > Mu Shear Strenth φVn= 1097 kN > Su
			Load Case	Group - VII 1.3(D+SD+1.67L+B)
			Sectional Force	Mu= 219 kNm, ① Su= 68 kN ②
			Strength	Flexural Strength φMn= 400 kNm > Mu Shear Strenth φVn= 1097 kN > Su
		Transverse Direction	Load Case	Group - VII 1.3(D+SD+1.67L+B)
			Sectional Force	Mu= 38 kNm, ①
			Strength	Flexural Strength φMn= 393 kNm > Mu

Note: The determined stress is far less than the allowable stress due to applying the minimum amount of steel material.

Source: JICA Study Team

Culvert Design

(1) Design Criteria

The project road crosses fairly large natural streams at fifteen locations (S1-S15) along the project route. The discharge for each location of culvert is calculated in Section 3.5. The considerations for design are summarized as follows:

- Concrete box culvert is adopted where sediment load is large and abrasive. Pipe culvert is applicable at locations where the discharge is below $1m^3/s$
- Existing culvert structures which are mostly insufficient capacity are removed when it obstructs the new culverts.
- New culvert structure (S2') is necessary at Sta.1+160, in consideration of volume of discharge.
- Vertical clearance for passenger car (minimum 2.5 m) is considered for S2 and S8, as consultation with AJK
- The portal type culvert is utilized to S2 and S8 due to its landform dimension
- Size of culverts are categorized according to four dimensional types for construction practicality, except at locations where dimension is defined by the available vertical clearance
- Dimension is basically determined by the uniform flow calculation. For a 1%slope, 70% of effective height is required with roughness coefficient of 0.22.
- For all these crossings the 1:20 year peak floods have been adopted.
- Maximum Dual Axle Load of Class A loading (111 kN with 1.22m distance) is adopted as Design Live Load

The dimension and type of culverts are as follows:

Table 5.5.9 Culverts Dimension according to Discharge

No.	No.	Station (km)		Structure	Discharge Estimated (m3/s)	Design Opening (B x H) (m)	Discharge Capacity** (m^3/s)	Type	Remarks
D/D	B/D	B/D	D/D						
1			0+620	Pipe culvert		φ910mm	1.5		add in D/D
2	S1	0+725	0+723	Box culvert	8.0	2.5 x 2.0	11.8	II	
3	S2	0+990	0+992	Portal culvert	4.1	5.5 x 3.0	45.8	V	Underpass
4	S2'	1+330	1+160	Pipe culvert	15.0	2.5 x 2.5	16.1		
5			1+460	Pipe culvert		φ910mm	1.5		add in D/D
---	S3	1+745		Bridge	46.7*	---			PC-I Girder
6	S4	2+060	2+063	Box culvert	4.5	2.7 x 2.0	27.8	III	
7	S5	2+670	2+680	Box culvert	1.8	1.5 x 2.0	5.9	I	
8	S6	2+920	2+925	Box culvert	3.0	1.5 x 2.0	5.9	I	
9	S7	3+045	3+053	Box culvert	3.7	1.5 x 2.0	5.9	I	
10			3+200	Pipe culvert		φ910mm	1.5		add in D/D
11	S8	3+300	3+297	Portal culvert	1.6	5.5 x 3.0	45.8	V	Underpass
---	S9	3+515		Bridge	45.1*	---			PC-I Girder
12	S10	3+790	3+794	Box culvert	1.6	1.5 x 2.0	5.9	I	
13	S11	3+865	3+869	Box culvert	7.3	2.0 x 2.5	11.8	II	
14	S12	4+100	4+100	Box culvert	57.4	3 @ 3.5 x 3.0	129.6	IV	
15	S13-1	4.1-4.7	4+289	Pipe culvert	Less than 1.0	φ910mm	1.5		
16	S13-2		4+435	Pipe culvert		φ910mm	1.5		
17	S13-3		4+511	Pipe culvert		φ910mm	1.5		
18	S13-4		4+544	Pipe culvert		φ910mm	1.5		
19	S13-5		4+581	Pipe culvert		φ910mm	1.5		
20	S13-6		4+661	Pipe culvert		φ910mm	1.5		
---	S14		4+730	Bridge	197.4*	---			PC-I Girder
21	S15	4+860	4+870	Box culvert	3.7	2.0 x 1.5	5.9	I	

Note* : The return period for these discharge is 50 years because of bridge structures.
Note** : The discharge capacity is calculated based on uniform flow (slope 1%, 70% effective height, n=0.022).
Source: JICA Study Team

(2) Design

The concrete box culverts can be classified into two groups: for waterways and for trafficable roads. The latter allows passing vehicles as shown in **Figure 5.5.3**.

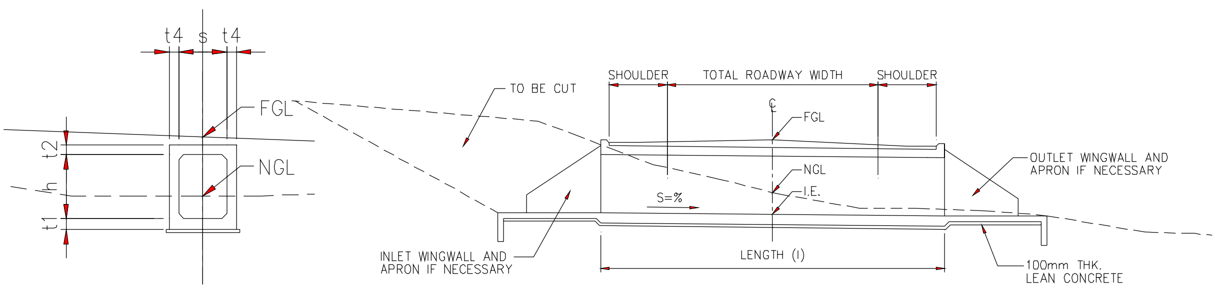


Figure 5.5.2 Typical Structural Detail for 1 cell Box Culvert

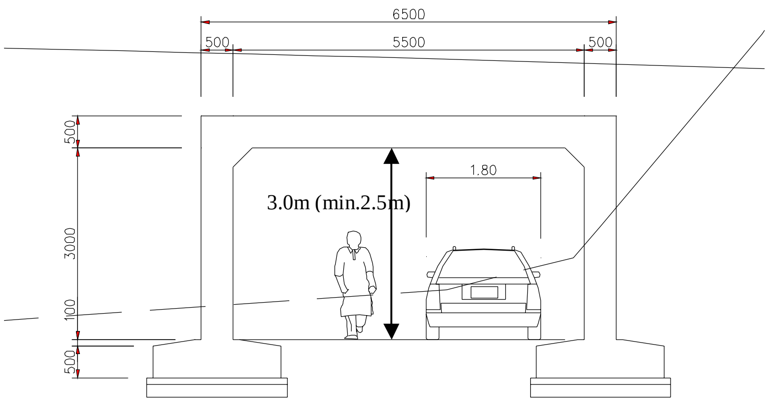


Figure 5.5.3 Vertical Clearance for S8 culvert

(3) Design Calculation

a. Design Calculation for box and portal culverts

i. Design Conditions

Table 5.5.10 Design Conditions

Material Unit Weight	Pavement		γa	22.5	kN/m^3
	Embankment	at wet condition	γt	18	kN/m^3
		at saturated condition	γsat	18.8	kN/m^3
	Concrete		γc	24.5	kN/m^3
	water		γw	9.8	kN/m^3
Soil Coefficient	Vertical		α	1	
	Horizontal		Ko	0.5	
Concrete	Design Strength		Σck	24	N/mm^2
	Allowable Bending Stress		Σca	8	N/mm^2
	Allowable Bearing Pressure		Σca	7.2	N/mm^2
	Allowable Shear Stress		τa1	0.39	N/mm^2
	Allowable Bending Stress		τa2	1.7	N/mm^2
	Allowable Punching Shear Pressure		τa	0.9	N/mm^2
	Allowable Bond Stress		τoa	1.6	N/mm^2
	Young's module		Ec	2.50×10^4	N/mm^2
Reinforcing Steel	Design Strength		―	SD295	―
	Allowable Tensile Stress		σsa	180	N/mm^2
	Allowable Compressive Stress		σsa	180	N/mm^2
Young's module ratio (Es / Ec)			n	15	―
Concrete Cover for Reinforcement		Bottom Slab		11	cm
		Others		10	cm

Source : JICA Stud Team

ii. Loading

- Dead Load

Dead Load to be considered includes culvert's self weight, lateral soil pressure, pavement reaction force from the ground and water pressure inside culvert (except portal culverts)..

- Live Load

Both Type "A" and "AA" loadings are considered for live load. When Type "A" is applied to culverts, the heaviest double axle of truck (111 kN x 2 axles) are applied to its center span. The impact coefficient for Type "A", and Type "AA: is 0.3 and 0.15, respectively.

iii. Calculation Case

The load cases are as follows;

Table 5.5.11 Loading Cases for Culverts

No	Loading cases for Single cell (Type I, II, III, V)
1	Dead Load-1
2	Dead Load-2 (with inside water)
3	Dead Load-1＋Live Load-1(A-Loading)
4	Dead Load -1＋Live Load -2 (AA-Loading)
5	Dead Load -2＋Live Load -1 (A-Loading)
6	Dead Load -2＋Live Load -2 (AA-Loading)

No.	Loading Cases for Triple cells (Type IV)
1	Dead Load-1
2	Dead Load-2 (with inside water)
3	Dead Load-1＋Live Load-1(A-Loading at left cell)
4	Dead Load -1＋Live Load -2 (AA-Loading at left cell)
5	Dead Load -1＋Live Load -1 (A-Loading at centre cell)
6	Dead Load -1＋Live Load -2 (AA-Loading at centre cell)
7	Dead Load-2＋Live Load-1(A-Loading at left cell)
8	Dead Load -2＋Live Load -2 (AA-Loading at left cell)
9	Dead Load -2＋Live Load -1 (A-Loading at centre cell)
10	Dead Load -2＋Live Load -2 (AA-Loading at centre cell)

Source: JICA Study Team

iv. Design Result

- Required Bearing Capacity

Required Bearing Capacity is calculated for each load case. The maximum reaction force is summarized in the following table. From the maximum reactions on bottom slab, no considerable value on the geology of the west bank can be identified as gravel layer (600k N/m^2).

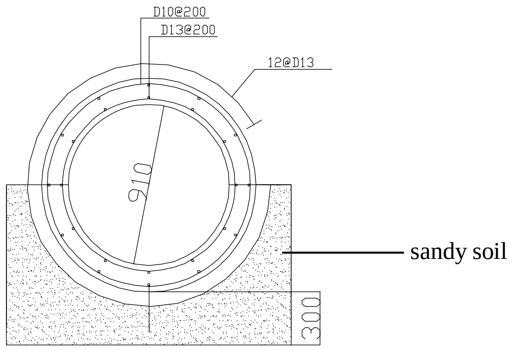
Table 5.5.12 Required Bearing Capacity

Type	Loading case	Maximum reaction force on Bottom Slab (kN/m^2)
Type I	Dead Load -2＋Live Load -2 (AA-Loading)	112.351
Type II	Dead Load -2＋Live Load -2 (AA-Loading)	114.830
Type III	Dead Load -2＋Live Load -2 (AA-Loading)	109.679
Type IV	Dead Load -2＋Live Load -2 (AA-Loading at centre cell)	84.694
Type V	Dead Load -2＋Live Load -2 (AA-Loading)	175.119

Source: JICA Study Team

v. Design calculation for pipe culverts

General Specification of Pakistan specifies that reinforced concrete pipe culvert shall meet the requirements of AASHTO M170, class II and IV. It is assumed to utilize factory fabricated precast products in Pakistan, which meets these AASHTO requirements for this project. The type and dimension of haunch for pipe culvert is determined by applying live load (Type “A”).



Source: JICA Study Team

Figure 5.5.4 Typical Structural Detail for Pipe Culvert